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Design Calculations

For

NOTE
This Plan must be read together with the written comments contained within the Project Information Memorandum and Building Consent.

ZONE - J
QUEENSGATE MALL
REDEVELOPMENT

APPROVED
BUILDING CONSENT MAY BE ISSUED
NAME 241059
DATE 13 SEP 2004
HUTT CITY COUNCIL

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■ draft report

Queensgate Shopping Mall, Zone J, Structural Engineering Design Statements

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3-1

■ draft report

Queensgate Shopping Mall, Zone J, Structural Engineering Design Statements

Prepared for
Westfield (New Zealand) Limited

By
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June 2004

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Revision History

Revision Nº	Prepared By	Description	Date
A	Jamil Khan	Draft report for building conscience submission	17/6/04

Document Acceptance

Action	Name	Signed	Date
Prepared by	Jamil Khan		
Reviewed by	Hamish Brookie		
Approved by	David Whittaker		

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1 Introduction

This report summarizes the structural design criteria, assumptions and approaches that has been established as being the most appropriate for the analysis and design of this project, giving all constraints associated with projects in Lower Hutt, New Zealand. The primary object of this report is to explain clearly the design approach with all well-defined parameters of design.

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2 Structural Design Criteria

2.1 Design Codes and Other Design Documents

Structural design shall comply with the NZ Building Code. As a means of compliance, the following standards are adopted, together with any other code as appropriate.

- NZS 4203:1992 NZ Loadings Code
- NZS 3101:1995 Design of Concrete Structures
- NZS 3404:1997 Design of Steel Structures

2.1.1 Design Loadings

Typical design loadings are shown below. Non-typical areas are considered on a case-by-case basis.

A Live Loads

Carpark general	2.50 kPa
Carparking on Main Mall	2.50 kPa
General Retail Floor	4.00 kPa
Department Store general	5.00 kPa
Department Store storage area	10.0 kPa.
Plant Rooms	5.00 kPa
Roof	0.25 kPa (no access)

Any areas not noted above shall be in accordance with NZS 4203:1992
Loadings different from above will be shown as on the drawings.

B Superimposed Dead Loads

Carpark	Level 3	0.20 kPa
	Level 2	0.20 kPa
Main Mall*	Level 2 (Retail with retail under)	1.00 kPa
Department Store**	Roof level	0.50 kPa
	Level 2 (Retail)	0.50 kPa

*Main Mall superimposed dead loads are made up as below

Level 2 (with retail under)	
ceilings under	0.20
services under	0.30
partitions	0.50
	1.00

**Department Store superimposed dead loads are made up as below

Level 2 (with retail under)	
ceilings under	0.20
services under	0.30
	0.50

C Wind Load

In accordance with the applicable provisions of NZS 4203:1992.

Category IV Buildings (Mr = 1.0)

Terrain Category 3

Wind Region V, v=48 m/s

Shielding Multiplier Ms = 1.0

Topographic Multiplier Mt = 1.00

D Earthquake Load

New buildings in accordance with the applicable provisions of NZS 4203:1992

All buildings

Category IV building

Risk Factor R = 1.0

E Vehicle Barrier Loads

Barriers shall be designed in accordance with AS 1170.1-1989 Clause 4.5 (40 kN force).

F Pedestrian Barrier Loads

Barriers shall be designed in accordance with NZS 4203 Clause 3.5 for commercial buildings. Panic loadings will be considered as required by Westfield.

2.2 Materials

A Concrete Reinforcement

- Bars: Generally deformed, Grade 300 and 500 complying with AS/NZS 4671 Hot Rolled Steel Bars for Reinforcement of Concrete for both deformed and plain bar. 241059

- Welded Wire Fabric: Grade 485 complying with NZS 3422:1985 Welded Fabric of Drawn Steel Wire for Concrete Reinforcement. Use of wire fabric in floors would be minimised, as the industry does not consider its performance in fire conditions as entirely satisfactory.

B Concrete

■ Footings, beams, columns	25 MPa
■ Slab on grade	30 MPa
■ suspended slabs (type A aggregate for suspended slabs)	20 <Pa
■ Zone J, Level 2, Floor Slab	40 MPa
■ Filling to SHS / CHS steel columns (For 1 hour fire rating)	Fibre Reinforced Concrete with 60 MPa Cube Strength, fibre reinforcement 5% of dry mass of the constitutes of concrete.
■ Pre-cast Concrete	30 MPa
■ Proprietary pre-cast concrete units.	As supplied by manufacturer but not less than 30 MPa

C Structural Steel

■ Flat bar and plates		Grade 250
■ Rolled or welded beams and columns		Grade 300
■ Channels and angles		Grade 300
■ RHS, SHS, CHS sections		Grade 350
■ Bolts	M10, M12, M16	Grade 4.6
	M20 and larger	Grade 8.8
	Holding down bolts	Grade 4.6 or 8.8

D Cold Rolled Steel Purlins — Dimond DHS or Steel & Tube HST

E Flooring Systems

■ Metal Decking 0.75 Hibond	Hibond or equivalent -refer Dimond catalogue. Nelson studs welded to primary and secondary beams to provide composite action. Crack control by reinforcing mesh and saddle bars over primary beams and around columns.
■ Hollow Core Flooring	Minimum 65mm toping, end support core broken out and reinforced.

2.3 Deflection Criteria

As NZS 4203:1992 unless noted otherwise. Generally the estimated long term deflections will be kept within the following limits maintain reasonable appearance and to avoid damage to finishes and secondary elements:

- Deflection of underside of beams limited to span/250
- Deflection of top surface of floors limited to span/360

Hibond floors and supporting steel beams will be designed to not require propping during construction.

2.4 Floor Vibration Criteria

Floors are designed to the acceptability criteria given for Shopping Centres and Carparks in the publication "Floor Vibrations Due to Human Activity" AISC Steel Design Guide Series Murray, Allen, Ungar.

2.5 Concrete Finishes

The concrete finishes are:

- Formed Concrete

Visible to view on completion	F5 (smooth surfaces where appearance is important)
Not visible on completion	F1 (surfaces where roughness is acceptable)
To receive hardwall plaster finish	F2 (surface to provide a good key for plaster or other applied finish)

- Floors — U3 (power float finish)
- Ramps — U3 (lightly broomed)

Alkali-Aggregate Reactions: To reduce the risk of adverse alkali-aggregate reaction in the concrete the requirements in Cement and concrete Association of NZ Report N° TR3, Alkali-Aggregate Reaction: Minimising the Risk of Damage to Concrete shall be followed.

2.6 Concrete Covers

Concrete covers are based on an exposure classification of A2 to all exposed concrete. Concrete in interior environments is considered with an exposure classification of A1 where appropriate and so as not to complicate the design.

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2.7 Fire Protection

The assumed approach for providing fire resistance rating to the structure is in accordance with the Fire Report.

- Provide rating to all columns and beam/column joints, as required by the Fire Report. Square /circular hollow section columns filled with fibre reinforced concrete, are designed according to BS Code, as there is no such provision in NZ standards.
- The steel fibre reinforcement in fibre reinforced concrete shall be spread uniformly throughout the concrete and forming approximately 5% by dry mass of the constituents, before addition of water. The Fibre shape and dimension shall be such as to provide adequate pullout strength. Typically, fibres shall be 0.5mm diameter, not longer than 38mm, and have crimped flats or hooked ends to ensure adequate pullout resistance.
- The required design cube strength of Fibre Reinforced Concrete is 60 MPa.
- Where required, other steel columns will be provided with applied passive fire protection.
- No passive fire protection is provided to steel beams supporting floors. It is assumed that areas with high fire load will be sprinkler protected and therefore the risk of exposure to temperatures, which would result in loss of structural capacity of the beams, is small and assumed to be acceptable.
- It is assumed that no passive fire protection is required to the following elements:
  - Roof steelwork (including columns for roof)
  - Earthquake bracing

The Fire Engineer shall review and confirm that these allowances are adequate or any additional passive protection requirements to the structure.

2.8 Corrosion Protection

Enclosed and unexposed structural steelwork shall be primed. Exposed structural steelwork shall receive a primer and finish coats to the Architect's specification.

All bolts shall be custom hot-dip galvanised. Steel to be buried shall be painted with 3 coats of Mulseal and protected from damage during backfilling.

Steel members embedded in concrete shall not be painted.

3 Seismic Design Philosophy

Design for earthquake resistance of the buildings in the Queensgate Mall being carried out in accordance with the Building Act, the requirements of the New Zealand Building Code and its embodied structural design codes.

The philosophy embodied in the current earthquake design codes for new buildings is to ensure a high level of life-safety protection to building occupants. Protection of buildings and other property against earthquake-induced damage is treated as a secondary, less critical objective. Although the requirements of the code provide for considerable resistance to sustain earthquake shaking, buildings are not "earthquake proof" and significant damage is likely to be caused by severe shaking. The building is, however, unlikely to collapse, and in many instances could be repaired.

The current Loadings Code for buildings (NZS 4203:1992) provides for five different Risk Factor categories of buildings depending on their usage, ranging from essential facilities to buildings of a secondary nature. All buildings are designated by the code in the fourth category, as normal usage building, and are required to be designed for earthquake loadings of ordinary buildings, i.e. the Risk Factor = 1.0.

The Hutt Valley lies in the highly active seismic zone in New Zealand. The Queensgate Mall site is located over a layer of relatively weak surficial soil. The geotechnical engineer has advised that these ground conditions designate the site as "flexible or deep" in terms of the Loadings Code, and earthquake loadings are specified accordingly. Foundation movements and settlements are likely under severe shaking.

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4 Foundations

The following information is summarised from the "Westfield Centre Geotechnical Investigation Report"" prepared by Connell Wagner Ltd, dated November 2000 (Reference 5627 35 CN – Revision 1).

4.1 Foundation Design Criteria

For the purposes of 4.6.2.2 from NZS 4203:1992 Site Subsoil Categories the site shall be classified as site subsoil category (c) (Flexible or deep soil site).

Construction of the new buildings generally requires piled foundations. Pile lengths are likely to vary across the site, depending of the thickness of the soft soil layer, but should not be less than 4 m deep.

In design to NZS 4203:1992 load combinations, the ultimate soil bearing capacities shall be reduced by a strength reduction factor $\phi = 0.65$. In capacity design (including over strength) a strength reduction factor of $\phi = 0.9$ shall be used. In case of driven piles, strength reduction factors could be improved if piles are tested during construction (e.g. by Pile Driving Analyser).

4.2 Soil Profile

Ground conditions comprise a sequence of alluvial silts, sands and gravels. Thickness of various soil layers varies from 2.5 to 5 metres.

Ground water level is at the depths of 1.6 m to 2.5 m measured from the ground surface.

In order to protect the Hutt Aquifer, which lies below the area, drilling and piling is strictly controlled. Foundations, extending 8 m below the mean sea level, require special design and considerable costs. This requirement shall be checked with Wellington Regional Council.

4.3 Liquefiable Sands

The geotechnical engineer has recommended that no significant liquefaction effects be expected at the site.

In extreme seismic shaking liquefaction is unlikely to be a significant cause of damage at this site. The surficial layer of silt is only susceptible to liquefaction in few localised pockets and the underlying layers of sands and gravels are of dense nature, thus resulting in minimal potential for significant liquefaction.

4.4 Piles

The piles for the project are being procured on a design and built basis, based on the schedule pile loads included in the performance design specifications.

Performance design specifications for supply of pile, will provide a design and construction solution, which will meet the load and other performance requirements.

The design documentation will be lodge as an amendment to the building consent applications.

The piles caps for 2, 3 & 4 piles groups are designed to transfer the column loads to individual piles with the provision of maximum allowable pile eccentricity, specified in pile specifications. The pile caps are designed using the beam theory and strut-tie mechanism, with compliances of code's provisions for deep-flexural members. Pile caps for single piles are designed for unbalance moments induced due to maximum allowable pile eccentricity.

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5 Outline Descriptions of Building Structures

The following gives outline descriptions of the structural system, design philosophy and construction materials to be used in this building.

5.1 Ground Floor

The ground floor structural system consists of cast-in-situ concrete ground beams and concrete slab on grade floor. The ground beams are designed to tie the pile caps and transfer the seismic base shear to foundation piles. All ground beams are designed for bending moment induced by maximum allowable eccentricity of piles.

The ground beams types GB1 are also designed to transfer the over strength seismic shear from K-brace system to the foundation system. The foundation lateral resistance consists of, frictional resistance between the lowest ground slab and soil, pile lateral load resistance, and passive resistance against pile caps and ground beams.

The friction resistance is calculated by multiplying the slab weight with friction coefficient, which is taken as 0.25. Therefore, the friction resistance for 150mm thick ground slab is 0.25*(0.150*24) = 0.9 KPa.

It is specified in performance design specifications for pilling work, that the pile to be designed to resist an ultimate lateral load of 120KN. For our design we are assuming that each pile will be able to resist, at least, 60KN lateral load under seismic over strength combination.

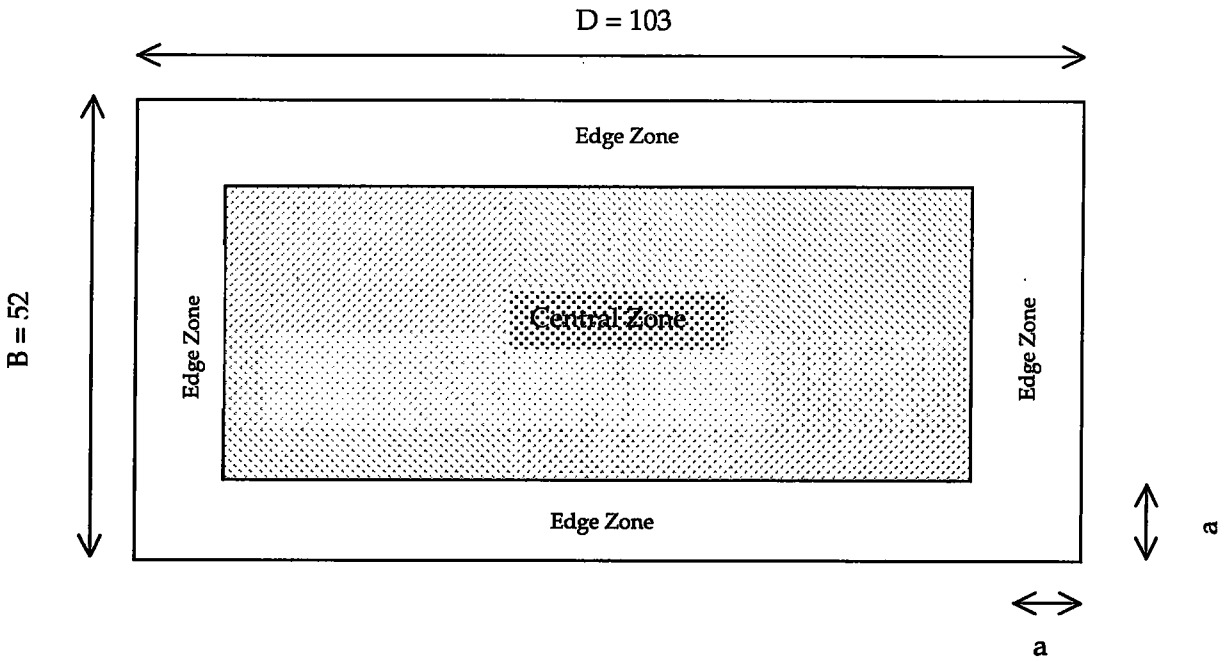
The ground slab is designed as slab on grade and also provide foundation tie action. The interface shear transfer connection between ground beam and slab is designed using Shear Friction.

5.2 First Floor

The first floor structure is designed using steel structural beams with Hibond composite slab. The secondary beams are of 8.2/10.5m spans typically and are spaced at 2.4m centre to centre. The primary beams are of 10.5/12.4m spans typically. Both primary and secondary beams are designed as composite beams. In zone J, the primary beams are designed for a live load of 6.50 KPa, and the secondary beams are design for higher load to account the local seismic effects due from the racking system. The main beams at grid line (SA) to (SG) from (S1) to (S3) are also designed to support the portal frame near grid lines (S2) & (S3).

5.3 Roof

The roof decking will be supported on purlines spaced at 2-m centre to centre. These purlines will be supported on portal frame spaced @ about 10.5m centre to centre. The purlies and portal are designed for ultimate gravity and ultimate wind load combinations.



The roof purlines are divided into two zones edge zone and central zone as per the code's recommendations. The edge zone is designed for higher wind pressure compare to the central zone. Following loads are considered in purlin design

1) Gravity loads
   a) Dead Load — 0.15 KPa, including purline self weight.
   b) Live Load — 0.25 KPa,

2) Wind load for edge zone
   a) Outward Ultimate load — -1.35 KPa,
   b) Outward Service Load — -0.81 KPa,

3) Wind load for central zone
   a) Outward Ultimate load — -0.50 KPa,
   b) Outward Service Load — -0.30 KPa,

Ultimate Limit State Combinations loads for 2m spacing of purlines

a) 1.2 G + 1.6 Q — 1.16 KPa,
b) 0.9 G + Wu (for edge zone) — -2.43 KPa
c) 0.9 G + Wu (for central zone) — -0.73 KPa,

The span of typical purlin is 10.5m and for central portion we are using Diamond DHS 300/15-1B and for edge zone DHS 300/18-2B Lapped Purlins.

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The mezzanine floor is under the edge zone, where ultimate wind outward load governs the design of roof purlies, therefore, the additional loading of 0.5 KPa for services and ceiling over the mezzanine floor will be beneficial for purlins.

The portal frames are analysed as individual braced frame for different combinations of dead, live and wind load. The rafters are designed as continuous beams for most critical load combination of dead, live and wind load. Restraint to lateral buckling of compression flanges is provided with Flybracing at appropriate locations.

Roof bracing is provided to transfer the seismic loads of the roof structure to the perimeter wall bracing at the first floor and then to the ground via the K-brace frame system at the ground floor.

5.4 K-Braces

The seismic loads as determined in accordance with NZS4203: 1992 are resisted at the ground floor by six 10.5m K-brace frames in the longitudinal direction, three along each external grid, and eight 8.2m K-brace frames in the transverse direction, four each along grid S4 and S11. The K-brace frames are designed for a ductility of six ($\mu = 6$) with an active link region of length 700mm. The design follows the procedures and recommendations outlined in sections 9-11 of Hera Report R4-76: Seismic Design Procedures for Steel Structures. This includes the capacity design philosophy to ensure the ductile response remains within the specified active link region.

5.5 Seismic Separations

The only adjacent building to zone J is Zone G & F building. A seismic separation gap of 185mm is provided between adjacent buildings.

5.6 Supplementary Systems

5.6.1 Seismic Restraint of Building Services and Equipment

The building services engineers should specify that all building services shall be restrained in accordance with NZS 4219. All mechanical equipment shall be positively anchored in place by anchor bolts, isolation mounts or other effective means. Ductwork piping shall have sway bracing and expansion capabilities in the transverse and longitudinal directions.

5.6.2 Ceiling Grid

The Architect should specify that all Ceiling systems shall be braced to resist earthquake-induced movements in accordance with the NZ Building Code.

5.6.3 Steel Frame Connection Types

Beams are typically simply connected to columns or primary beams using standard HERA connection details (eg flexible end plate or welded plate type connections). Connections, which are highly visible, may have additional architectural feature detailing.

5.6.4 Precast Wall Panels

Precast walls shall be 125 mm or 150 mm reinforced concrete panels fixed to the slab or to other structural elements. Steel framing will be provided to connect the panels and to support roofs.

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Estimation
Of
Wind Loads

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CALCULATION SHEET

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PART 5 PROVISION FOR WIND FORCES

Eaves Height	he =	12.00 m
Roof pitch	α =	5.00 °
Height to apex	ht =	14.34 m
Building length	l =	103.00 m
Building width	b =	53.40 m
Portal frame spacing (or tributary width to portal)	S =	8.20 m
Purlin spacing	P =	2.00 m
Girt spacing	G =	3.00 m

5.4 SITE WIND SPEED

Basic directional wind speed, V — V = 48 m/s
Wind region (Fig. 5.4.1) — V
Wind direction — Non-dir

Limit state multiplier (ULS), M_{ls} — M_{ls} (ULS) = 0.93
Limit state multiplier (SLS), M_{ls} — M_{ls} (SLS) = 0.75

Site terrain/height multiplier, $M_{(z,cat)}$
Terrain catagory (Cl. 5.4.4.2) — Catagory 3
Height above ground, z (= ht) — z = 14.34 m
$M_{(z,cat)}$ = 0.882

Sheilding mulitiplier, M_s — M_s = 1.0
Shielding parameter, s

$$s = \frac{l_s}{\sqrt{(h_s b_s)}}$$

s = 13.0 ie. s > 12
Number of upwind buildings within 45° sector of radius 2 and with height > H, n_s — n_s = 0
Average height of shielding buildings, h_s — h_s = 0.0 m
Average breadth of shielding buildings, b_s — b_s = 0.0 m

Topographic multiplier, M_t (for sites below elevation 500 m) — M_t = 1.0
Hill shape multiplier, M_h — M_h = 1.0
Lee effect multiplier, M_L — M_L = 1.0
Channeling multiplier, M_c — M_c = 1.0

Structure category — IV
Structure risk multiplier — M_r = 1.0

Site wind speed, $V_{(z)}$ (ULS)
$V_{(z)} = V M_{ls} M_{(z,cat)} M_s M_t M_r$ — $V_{(z)}$ = 39 m/s *

Site wind speed, $V_{(z)}$ (SLS)
$V_{(z)} = V M_{ls} M_{(z,cat)} M_s M_t M_r$ — $V_{(z)}$ = 32 m/s

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5.5 DESIGN WIND PRESSURE

Design wind pressure, q_z (ULS)

$q_{(z)} = 0.6\ V_{d(z)}^2 \times 10^{-3}$ $\quad q_{(z)} =$ 0.9 kPa

Design wind pressure, q_z (SLS)

$'q_{(z)} = 0.6\ V_{d(z)}^2 \times 10^{-3}$ $\quad q_{(z)} =$ 0.6 kPa

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5.6 FORCES AND PRESSURE ON ENCLOSED BUILDINGS

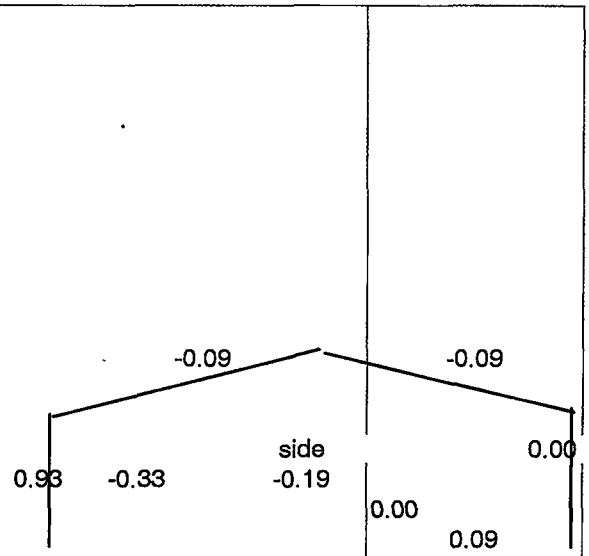
5.6.1.1 Design Wind Pressure pz = pe - pi

Ultimate Limit State Loads

Wind at θ = 0°

	pe	pi +ve	pz
Windward wall, W	0.65	-0.28	0.93
Leeward wall, L	-0.28	-0.28	0.00
Side walls, S			
0 to 1 h	-0.60	-0.28	-0.33
1 h to 2 h	-0.47	-0.28	-0.19
2 h to 3 h	-0.28	-0.28	0.00
> 3 h	-0.19	-0.28	0.09
Up wind slope, U	-0.37	-0.28	-0.09
Down wind slope, D	-0.37	-0.28	-0.09

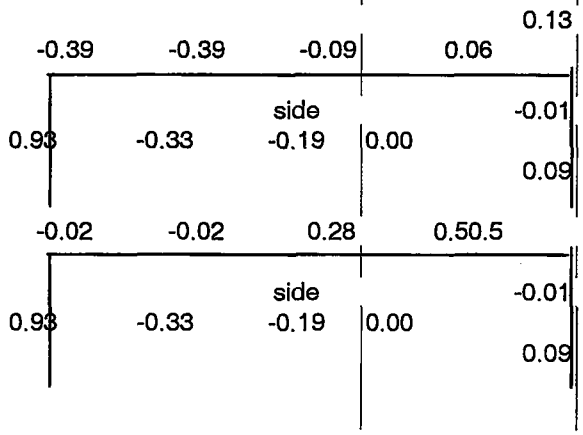
	pe	pi -ve	pz
Windward wall, W	0.65	0.00	0.65
Leeward wall, L	-0.28	0.00	-0.28
Side walls, S			
0 to 1 h	-0.60	0.00	-0.60
1 h to 2 h	-0.47	0.00	-0.47
2 h to 3 h	-0.28	0.00	-0.28
> 3 h	-0.19	0.00	-0.19
Up wind slope, U	-0.37	0.00	-0.37
Down wind slope, D	-0.37	0.00	-0.37





Wind at θ = 90°

	pe	pi +ve	pz
Windward wall, W	0.65	-0.28	0.93
Leeward wall, L	-0.29	-0.28	-0.01
Side walls, S			
0 to 1 h	-0.60	-0.28	-0.33
1 h to 2 h	-0.47	-0.28	-0.19
2 h to 3 h	-0.28	-0.28	0.00
> 3 h	-0.19	-0.28	0.09
windward edge, -ve			
0 to 0.5h	-0.67	-0.28	-0.39
0.5h to 1h	-0.67	-0.28	-0.39
1h to 2h	-0.37	-0.28	-0.09
2h to 3h	-0.22	-0.28	0.06
> 3h	-0.15	-0.28	0.13
windward edge, +ve			
0 to 0.5h	-0.30	-0.28	-0.02
0.5h to 1h	-0.30	-0.28	-0.02
1h to 2h	0.00	-0.28	0.28
2h to 3h	0.22	-0.28	0.50
> 3h	0.22	-0.28	0.50



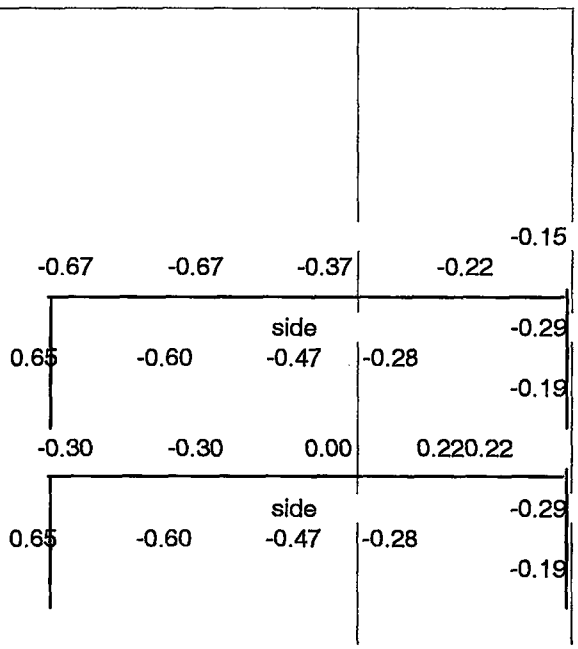
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	pe	pi -ve	pz
Windward wall, W	0.65	0.00	0.65
Leeward wall, L	-0.29	0.00	-0.29
Side walls, S			
0 to 1 h	-0.60	0.00	-0.60
1 h to 2 h	-0.47	0.00	-0.47
2 h to 3 h	-0.28	0.00	-0.28
> 3 h	-0.19	0.00	-0.19
windward edge, -ve			
0 to 0.5h	-0.67	0.00	-0.67
0.5h to 1h	-0.67	0.00	-0.67
1h to 2h	-0.37	0.00	-0.37
2h to 3h	-0.22	0.00	-0.22
> 3h	-0.15	0.00	-0.15
windward edge, +ve			
0 to 0.5h	-0.30	0.00	-0.30
0.5h to 1h	-0.30	0.00	-0.30
1h to 2h	0.00	0.00	0.00
2h to 3h	0.22	0.00	0.22
> 3h	0.22	0.00	0.22



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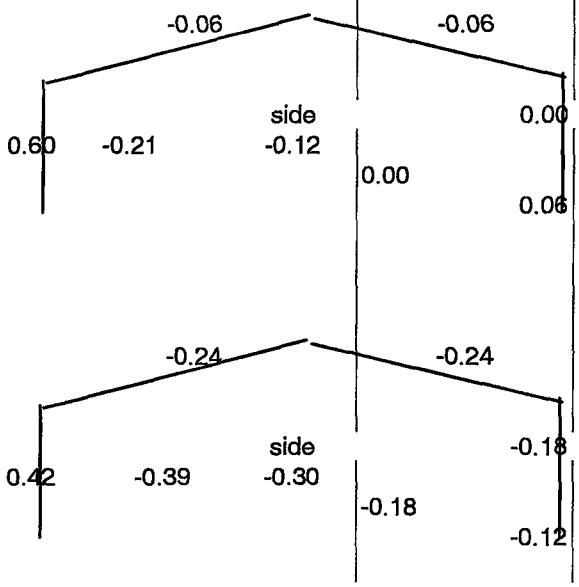
DATE: 29/4/04

Servicability Limit State Loads

Wind at θ = 0°

	pe	pi +ve	pz
Windward wall, W	0.42	-0.18	0.60
Leeward wall, L	-0.18	-0.18	0.00
Side walls, S			
0 to 1 h	-0.39	-0.18	-0.21
1 h to 2 h	-0.30	-0.18	-0.12
2 h to 3 h	-0.18	-0.18	0.00
> 3 h	-0.12	-0.18	0.06
Up wind slope, U	-0.24	-0.18	-0.06
Down wind slope, D	-0.24	-0.18	-0.06

	pe	pi -ve	pz
Windward wall, W	0.42	0.00	0.42
Leeward wall, L	-0.18	0.00	-0.18
Side walls, S			
0 to 1 h	-0.39	0.00	-0.39
1 h to 2 h	-0.30	0.00	-0.30
2 h to 3 h	-0.18	0.00	-0.18
> 3 h	-0.12	0.00	-0.12
Up wind slope, U	-0.24	0.00	-0.24
Down wind slope, D	-0.24	0.00	-0.24



241059

Beca Carter Hollings & Ferner **CALCULATION SHEET**

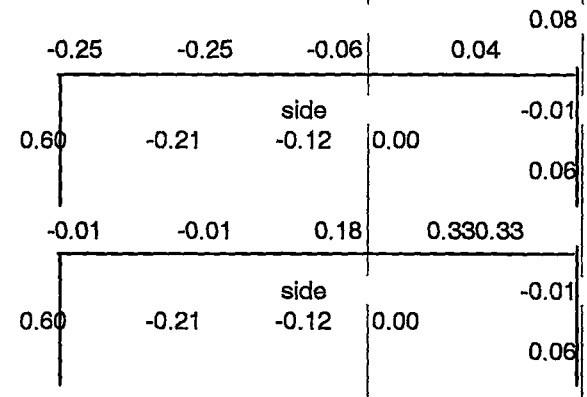
JOB NAME: Queesgate Mall Redevelopment JOB NO: 2711265

SUBJECT: Warehouse Building PAGE NO ..... OF .....

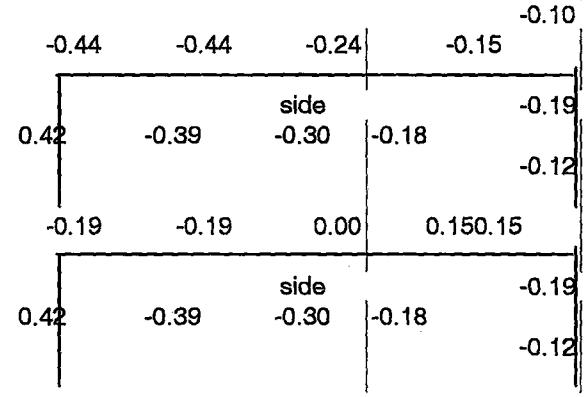
BY: Jamil Khan DATE: 29/4/04

Wind at $\theta = 90^{o}$

	pe	pi +ve	pz
Windward wall, W	0.42	-0.18	0.60
Leeward wall, L	-0.19	-0.18	-0.01
Side walls, S			
0 to 1 h	-0.39	-0.18	-0.21
1 h to 2 h	-0.30	-0.18	-0.12
2 h to 3 h	-0.18	-0.18	0.00
> 3 h	-0.12	-0.18	0.06
windward edge, -ve			
0 to 0.5h	-0.44	-0.18	-0.25
0.5h to 1h	-0.44	-0.18	-0.25
1h to 2h	-0.24	-0.18	-0.06
2h to 3h	-0.15	-0.18	0.04
> 3h	-0.10	-0.18	0.08
windward edge, +ve			
0 to 0.5h	-0.19	-0.18	-0.01
0.5h to 1h	-0.19	-0.18	-0.01
1h to 2h	0.00	-0.18	0.18
2h to 3h	0.15	-0.18	0.33
> 3h	0.15	-0.18	0.33



	pe	pi -ve	pz
Windward wall, W	0.42	0.00	0.42
Leeward wall, L	-0.19	0.00	-0.19
Side walls, S			
0 to 1 h	-0.39	0.00	-0.39
1 h to 2 h	-0.30	0.00	-0.30
2 h to 3 h	-0.18	0.00	-0.18
> 3 h	-0.12	0.00	-0.12
windward edge, -ve			
0 to 0.5h	-0.44	0.00	-0.44
0.5h to 1h	-0.44	0.00	-0.44
1h to 2h	-0.24	0.00	-0.24
2h to 3h	-0.15	0.00	-0.15
> 3h	-0.10	0.00	-0.10
windward edge, +ve			
0 to 0.5h	-0.19	0.00	-0.19
0.5h to 1h	-0.19	0.00	-0.19
1h to 2h	0.00	0.00	0.00
2h to 3h	0.15	0.00	0.15
> 3h	0.15	0.00	0.15



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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
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5.6 FORCES AND PRESSURE ON ENCLOSED BUILDINGS

5.6.2 Internal Pressure pi

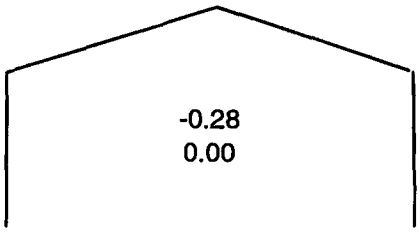
Ultimate Limit State Loads

Average internal pressure coefficient, Cpi

+ve Cpi = -0.3
-ve Cpi = 0.0

Internal pressure, pi

+ve pi = -0.28 kPa
-ve pi = 0.00 kPa



pi

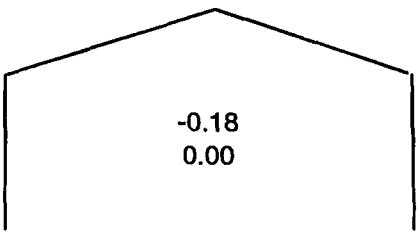
Servicability Limit State Loads

Average internal pressure coefficient, Cpi

+ve Cpi = -0.30
-ve Cpi = 0.00

Internal pressure, pi

+ve pi = -0.18 kPa
-ve pi = 0.00 kPa



pi

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

SUBJECT: Warehouse Building — PAGE NO: OF

BY: Jamil Khan — DATE: 29/4/04

5.6 FORCES AND PRESSURE ON ENCLOSED BUILDINGS

5.6.3 External Pressure, pe

Area reduction factor, Ka	Roof	Ka =	0.8
	Side walls		1.00
	Windward & leeward walls		1.0
Local pressure factor, Kl (main frame)		Kl =	1.0
Porous cladding reduction factor, Kp		Kp =	1.0

Average external pressure coefficient, Cpe

Wind at θ = 0°

Design height, h		h =	12 m
Building breadth, b		b =	103.00 m
Building depth, d		d =	53.40 m
Windward wall, W		Cpe =	0.7
Leeward wall, L		Cpe =	-0.3
Side walls, S	0 to 1 h	Cpe =	-0.65
	1 h to 2 h		-0.5
	2 h to 3 h		-0.3
	> 3 h		-0.2
Up wind slope, U		Cpe =	-0.5
Down wind slope, D		Cpe =	-0.5

Wind at θ = 90°

Design height, h		h =	14.34 m
Building breadth, b		b =	53.40 m
Building depth, d		d =	103.00 m
		d/b =	1.93
		h/d =	0.14
Windward wall, W		Cpe =	0.70
Leeward wall, L		Cpe =	-0.31
Side walls, S	0 to 1 h	Cpe =	-0.65
	1 h to 2 h		-0.50
	2 h to 3 h		-0.30
	> 3 h		-0.20
Horiz. distance from windward edge, -ve	0 to 0.5h	Cpe =	-0.90
	0.5h to 1h		-0.90
	1h to 2h		-0.50
	2h to 3h		-0.30
	> 3h		-0.20
Horiz. distance from windward edge, +ve	0 to 0.5h	Cpe =	-0.40
	0.5h to 1h		-0.40
	1h to 2h		0.00
	2h to 3h		0.30
	> 3h		0.30

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
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Ultimate Limit State Loads

Wind at θ = 0°

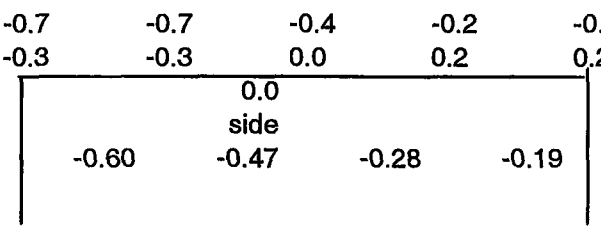
Windward wall, W		pe =	0.65 kPa
Leeward wall, L		pe =	-0.28 kPa
Side walls, S	0 to 1 h	pe =	-0.60 kPa
	1 h to 2 h		-0.47 kPa
	2 h to 3 h		-0.28 kPa
	> 3 h		-0.19 kPa
Up wind slope, U		pe =	-0.37 kPa
Down wind slope, D		pe =	-0.37 kPa



pe (ULS, θ = 0°)

Wind at θ = 90°

Windward wall, W		Cpe =	0.65
Leeward wall, L		Cpe =	-0.29
Side walls, S	0 to 1 h	Cpe =	-0.60
	1 h to 2 h		-0.47
	2 h to 3 h		-0.28
	> 3 h		-0.19
Horiz. distance from windward edge, -ve	0 to 0.5h	Cpe =	-0.67
	0.5h to 1h		-0.67
	1h to 2h		-0.37
	2h to 3h		-0.22
	> 3h		-0.15
Horiz. distance from windward edge, +ve	0 to 0.5h	Cpe =	-0.30
	0.5h to 1h		-0.30
	1h to 2h		0.00
	2h to 3h		0.22
	> 3h		0.22



pe (ULS, θ = 90°)

241059

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Warehouse Building
PAGE NO: OF
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Servicability Limit State Loads

Wind at θ = 0°

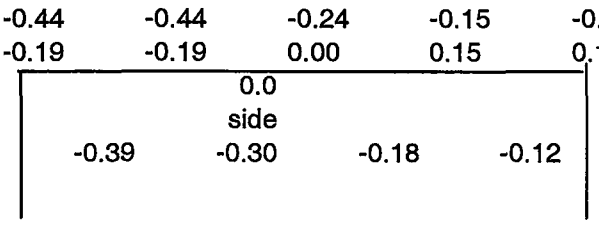
Windward wall, W		pe =	0.42 kPa
Leeward wall, L		pe =	-0.18 kPa
Side walls, S	0 to 1 h	pe =	-0.39 kPa
	1 h to 2 h		-0.30 kPa
	2 h to 3 h		-0.18 kPa
	> 3 h		-0.12 kPa
Up wind slope, U		pe =	-0.24 kPa
Down wind slope, D		pe =	-0.24 kPa



pe (SLS, θ = 0°)

Wind at θ = 90°

Windward wall, W		Cpe =	0.42
Leeward wall, L		Cpe =	-0.19
Side walls, S	0 to 1 h	Cpe =	-0.39
	1 h to 2 h		-0.30
	2 h to 3 h		-0.18
	> 3 h		-0.12
Horiz. distance from windward edge, -ve	0 to 0.5h	Cpe =	-0.44
	0.5h to 1h		-0.44
	1h to 2h		-0.24
	2h to 3h		-0.15
	> 3h		-0.10
Horiz. distance from windward edge, +ve	0 to 0.5h	Cpe =	-0.19
	0.5h to 1h		-0.19
	1h to 2h		0.00
	2h to 3h		0.15
	> 3h		0.15



pe (SLS, θ = 90°)

Estimation of Earthquake Loads

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Loadings/Zone 25
By: Dean Saunders (DJS)

Job No: 2711265/109
Page No: 1 of 3
Date: 3-5-04

Wall loadings

① Knights road 150 Precast panels
All load to Ground or 1st floor.

1st floor under 5.4/2 = 2.7m.
Length. S4 to S8 + S4 to S7
= 7.5m + 31.6m
= 39.1m.

1st floor over = 4.8m
length S4 to S8 + S4 to S6 + 2.8m
= 7.5m + 21.1m + 2.8m
= 31.4m.

② Bloom finial Terrace 150 precast panels
1st floor under. = 2.7m
length = S6 to S3 + 8.4m
= 49.37m.

③ Blockwalls 200 series. under first floor
1st floor under = 2.7m
length = along S4 + Core + S5 to S11 + stairs.
= 43.97m + 39.2m + 62.665 + 10.8
= 156.64m

④ Tin wall Cladding. on 3 sides above panels & carpark
1st floor over 6.5/2 = 3.25m.
roof 1/2 LVL 2 + parapet height = 7.59m.
Length. = 2453.7m + 14.81m + 7.31m + 79.07m
= 208.59m.
* + roof level 6.04m
Length = 31.4m.

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Loadings/Zone 2J
By: Dean Saunders.

Job No: 2711265/109
Page No: 2 of 3
Date: 3-5-04

Roof loading:

Plan area = 6119 m²

DL @ 0.25 kPa = 6119 m² × 0.25
= 1530 kN

walkway loading = (7.59 m × 208.59 m + 6.04 m × 31.4 m) × 0.35 kPa
= 620.5 kN

DL Plant 3 units @ 1.5 tonne = 1500 kg × 9.81 × 3
= 44.1 kN

TOTAL = 1530 + 620.5 + 44.1 kN
= 2195 kN

Roof
2195 kN

LVL 2 Loading:

Plan Area = 6119 m²

160 Hibond = 6119 m² × 0.135 × 23.5 kN/m³
= 19,413 kN

Structure = 6119 m² × 50 kg/m² × $\frac{9.81 \text{ m s}^{-2}}{1000}$
= 3001 kN

SDL = 6119 m² × 0.5 kPa
= 3060 kN

wall loading Tin = 3.25 m × 208.59 m × 0.35 kPa
= 237 kN

P/C = [2.7 m × (39.1 + 49.37) m + 4.8 × 31.4 m] × 0.15 × 23.5 kN/m³
= 1373 kN

Blockwall = 2.7 m × 156.64 m × 0.2 m × 23.5 kN/m³
= 1988 kN

Live load = 6119 m² × 6.5 kPa × R = 1.0
= 39,774 kN

TOTAL:
= 68,846 kN

LVL 2
68,846 kN.
unfactored

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Loadings / Zone ZJ
By: Dean Saunders

Job No: 2711265/109
Page No: 3 of 3
Date: 3-5-04

Mezzanine. S8 - S11 by SA - SB

Plan Area = 386.4 m²

130 H.board = 386.4 m² × 0.105 m × 23.5 kN/m³
= 954 kN

Structure = 386.4 m² × 50 kg/m² × 9.81/1000
= 189 kN.

SDL = 386.4 m² × 0.5 kPa
= 193 kN

Live Load = 386.4 m² × 3.0 kPa × R = 1.0 (one way slab)
= 1159 kN

TOTAL:
= 2495 kN

Mezzanine.
2495 kN
unfactored.
add to LVL2.

Summary of Masses.

Roof: = 2195 kN

LVL2: = 29,072 kN + 0.4 × 39,774 + 1336 kN + 0.4 × 1159
= 46,781 kN

Roof 2195 kN

LVL2 46,781 kN

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Lateral System Zone ZJ
By: Dean Saunders

Job No: 2711265/109
Page No: 1 of 9
Date: 3-5-04

Lump all mass at the first floor

M_i = 2095 + 46,781 kN
= 48,976 kN

M_i = 48,976 kN

Distribution factors for lateral load

μ = 6.0 , Soft Soils

Assume T = 0.1 seconds.

$C = C_h(T,\mu) S_p R Z L_u$

L_u = 1.0
R = 1.0
Z = 1.2
S_p = 0.67

soft soil $C_h(0.1, 6) = 0.19$

$C = 0.19 \times 0.67 \times 1 \times 1.2 \times 1$
= 0.153

μ = 6
Soft soils
T ≈ 0.1 sec.

$V_b = C W_t$
= 0.153 × 48,976 kN
= 7493 kN

V_b = 7493 kN

Design Capacity Base shear for K-braces

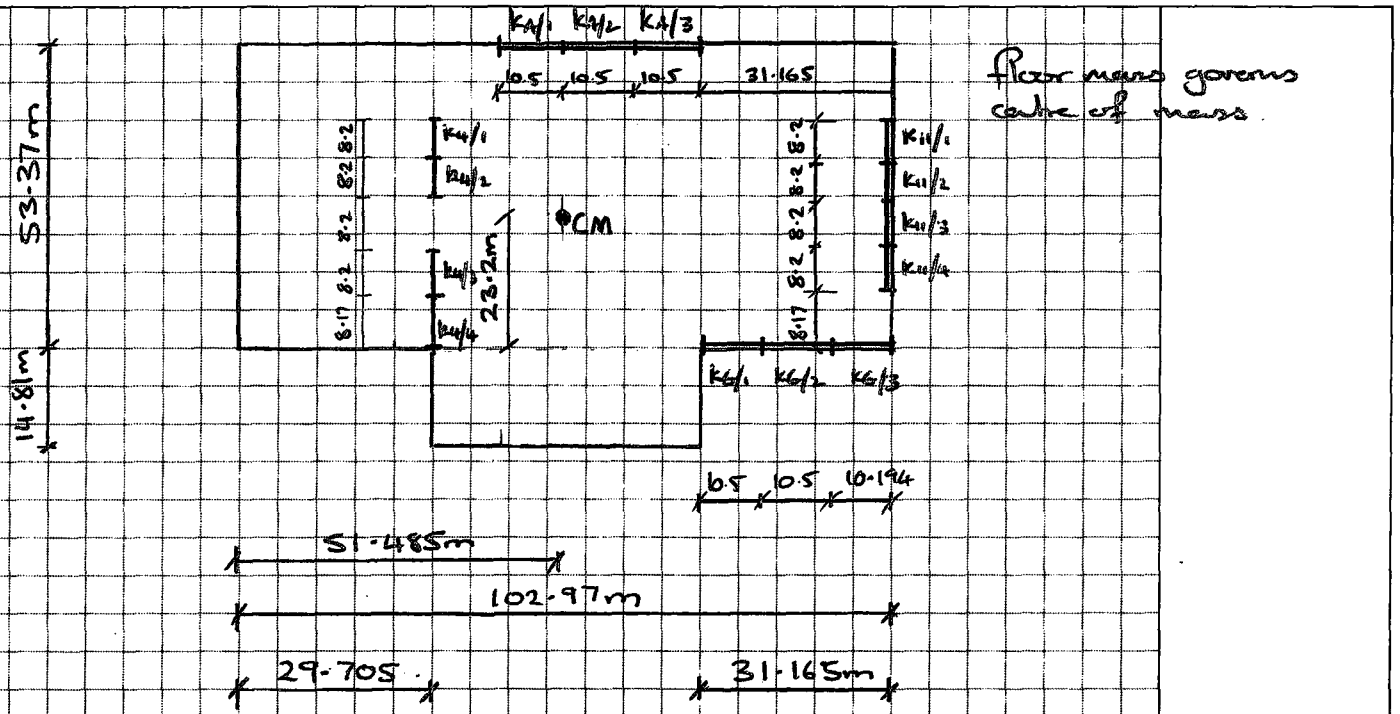
2 × 0.6 V_b = 1.2 V_b + Overstrength
1.2 × 1.6 × 0.9 = 1.73

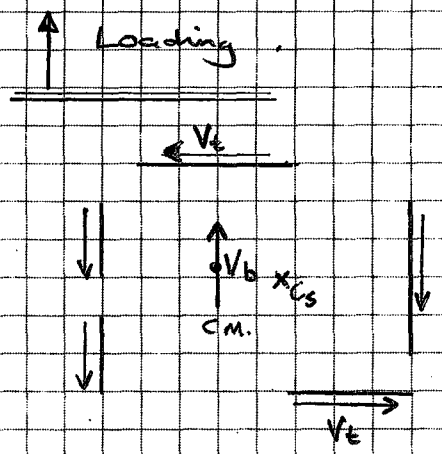
$V_{b\,o/s}$ = 7493 × 1.73
≈ 13,000 kN

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Lateral System Zone 2J
By: Dean Saunders

Job No: 2711265/109
Page No: 2 of
Date: 3-5-04





assuming equivalent stiffness for K-braced frames

$K_{11/1-4}$ and $K_{4/1-4}$

$K_{A/1-3}$ and $K_{6/1-3}$

K-brace $K_{11/1-4}$ max = $0.460V_b$

K-brace $K_{4/1-4}$ max = $0.728V_b$

K-brace $K_{A/1-3}$ or $K_{6/1-3}$ max = $0.053V_b$

redistribution of up to 30% allowed

241059

Design $K_{11/1-4}$ and $K_{4/1-4}$ for $0.6V_b$

check torsion demands on K_A & K_6

$$V_b \times 25.15m - \left(0.6V_b \times 36.63 - 0.4V_b \times 36.63\right) = V_t \times 53.37$$

$$25.15V_b - 7.33V_b = 53.37V_t$$

$$V_t = 0.334V_b$$

241059

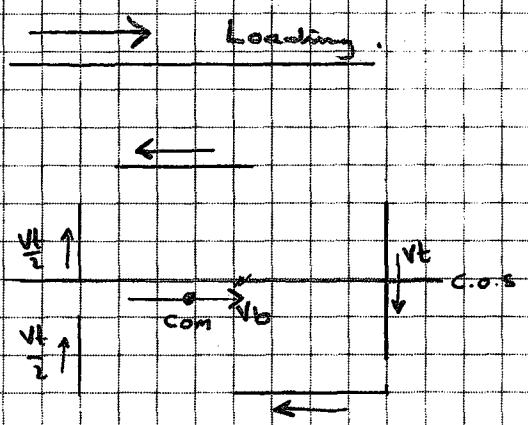
Design $K_{11/1-4}$ & $K_{4/1-4}$ $0.6V_b$

o.k. ✓

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Lateral System Zone ZJ
By: Dean Saunders

Job No: 2711265/109
Page No: 3 of
Date: 4-5-04.



K-brace $K_{A/1-3}$ max $0.522V_b$

K-brace $K_{G/1-3}$ max $0.564V_b$

K-brace $K_4 = K_{11}$ max $0.023V_b$.

Redistribute and design $K_{A/1-3} = K_{G/1-3}$ for $0.5V_b$.

Checks demands on K_4 & K_{11}

$V_b \times 10.3 - (2 \times 0.5V_b \times 26.69) = -V_t \times 73.27$

$V_t = 0.224 V_b$ ok ✓

Design K_4 & K_6 for $0.5V_b$

LOAD DISTRIBUTION

EARTHQUAKE IN X DIRECTION

ECCENTRICITY -10.3

EARTHQUAKE IN Y DIRECTION

ECCENTRICITY = -25.15

	RELATIVE TO COS														
	X	Y	Kiy	Kix	X*X*Kiy	Y*Y*Kix	Six'	Six"	Six	Siy		Siy'	Siy"	Siy	Six
1	31.533	-26.690	0	10.5	0	7479.739	0.167	0.022	0.188	0.000		0.000	0.000	0.000	-0.053
2	21.193	-26.690	0	10.5	0	7479.739	0.167	0.022	0.188	0.000	0.564	0.000	0.000	0.000	-0.053
3	10.693	-26.690	0	10.5	0	7479.739	0.167	0.022	0.188	0.000		0.000	0.000	0.000	-0.053
4	0.223	26.690	0	10.5	0	7479.739	0.167	-0.022	0.145	0.000		0.000	0.000	0.000	0.053
5	-10.278	26.690	0	10.5	0	7479.739	0.167	-0.022	0.145	0.000		0.000	0.000	0.000	0.053
6	-20.778	26.690	0	10.5	0	7479.739	0.167	-0.022	0.145	0.000		0.000	0.000	0.000	0.053
7	-36.633	-22.590	8.2	0	11003.91	0	0.000	0.000	0.000	-0.023		0.125	0.057	0.182	0.000
8	-36.633	-14.410	8.2	0	11003.91	0	0.000	0.000	0.000	-0.023		0.125	0.057	0.182	0.000 0.728
9	-36.633	1.990	8.2	0	11003.91	0	0.000	0.000	0.000	-0.023		0.125	0.057	0.182	0.000
10	-36.633	10.190	8.2	0	11003.91	0	0.000	0.000	0.000	-0.023		0.125	0.057	0.182	0.000
11	36.633	-14.410	8.2	0	11003.91	0	0.000	0.000	0.000	0.023		0.125	-0.057	0.068	0.000
12	36.633	-6.210	8.2	0	11003.91	0	0.000	0.000	0.000	0.023		0.125	-0.057	0.068	0.000
13	36.633	1.990	8.2	0	11003.91	0	0.000	0.000	0.000	0.023		0.125	-0.057	0.068	0.000
14	36.633	10.190	8.2	0	11003.91	0	0.000	0.000	0.000	0.023		0.125	-0.057	0.068	0.000
15	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
16	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
17	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
18	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
19	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
20	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
21	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
22	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
0	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
0	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
			65.6	63	I polar	132909.7	1.000	0.000	1.000	0.000		1.000	0.000	1.000	0.000

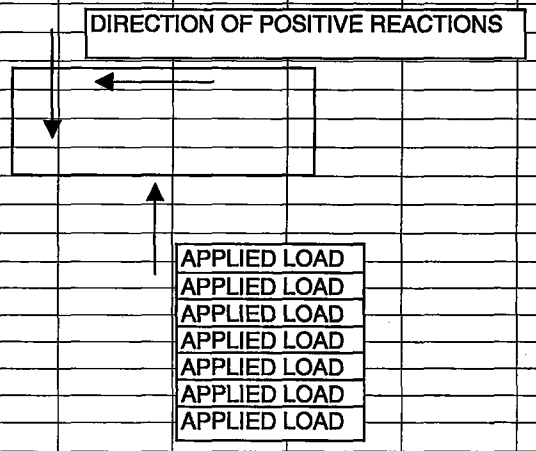
241059

DETERMINATION OF CENTRE OF STIFFNESS

	X(m)	Y(m)	Ky	Kx	Ky * X	Kx * Y
1	97.87	14.8	0	10.5	0	155.4
2	87.53	14.8	0	10.5	0	155.4
3	77.03	14.8	0	10.5	0	155.4
4	66.56	68.18	0	10.5	0	715.89
5	56.06	68.18	0	10.5	0	715.89
6	45.56	68.18	0	10.5	0	715.89
7	29.705	18.9	8.2	0	243.581	0
8	29.705	27.08	8.2	0	243.581	0
9	29.705	43.48	8.2	0	243.581	0
10	29.705	51.68	8.2	0	243.581	0
11	102.97	27.08	8.2	0	844.354	0
12	102.97	35.28	8.2	0	844.354	0
13	102.97	43.48	8.2	0	844.354	0
14	102.97	51.68	8.2	0	844.354	0
15						
16						
17						
18						
19						
20						
21						
22						
	TOTALS		65.6	63	4351.74	2613.87

X= 66.338

Y= 41.490



LOAD DISTRIBUTION

EARTHQUAKE IN X DIRECTION

ECCENTRICITY 3.33

EARTHQUAKE IN Y DIRECTION

ECCENTRICITY = -4.56

	RELATIVE TO COS																
	X	Y	Kiy	Kix	X*X*Kiy	Y*Y*Kix	Six'	Six''	Six	Siy		Siy'	Siy''	Siy	Six		
1	31.533	-26.690	0	10.5	0	7479.739	0.167	-0.007	0.160	0.000		0.000	0.000	0.000	-0.010		
2	21.193	-26.690	0	10.5	0	7479.739	0.167	-0.007	0.160	0.000		0.000	0.000	0.000	-0.010		
3	10.693	-26.690	0	10.5	0	7479.739	0.167	-0.007	0.160	0.000		0.000	0.000	0.000	-0.010		
4	0.223	26.690	0	10.5	0	7479.739	0.167	0.007	0.174	0.000		0.000	0.000	0.000	0.010		
5	-10.278	26.690	0	10.5	0	7479.739	0.167	0.007	0.174	0.000	0.522	0.000	0.000	0.000	0.010		
6	-20.778	26.690	0	10.5	0	7479.739	0.167	0.007	0.174	0.000		0.000	0.000	0.000	0.010		
7	-36.633	-22.590	8.2	0	11003.91	0	0.000	0.000	0.000	0.008		0.125	0.010	0.135	0.000		
8	-36.633	-14.410	8.2	0	11003.91	0	0.000	0.000	0.000	0.008		0.125	0.010	0.135	0.000		
9	-36.633	1.990	8.2	0	11003.91	0	0.000	0.000	0.000	0.008		0.125	0.010	0.135	0.000		
10	-36.633	10.190	8.2	0	11003.91	0	0.000	0.000	0.000	0.008		0.125	0.010	0.135	0.000		
11	36.633	-14.410	8.2	0	11003.91	0	0.000	0.000	0.000	-0.008		0.125	-0.010	0.115	0.000		
12	36.633	-6.210	8.2	0	11003.91	0	0.000	0.000	0.000	-0.008		0.125	-0.010	0.115	0.000	0.460	
13	36.633	1.990	8.2	0	11003.91	0	0.000	0.000	0.000	-0.008		0.125	-0.010	0.115	0.000		
14	36.633	10.190	8.2	0	11003.91	0	0.000	0.000	0.000	-0.008		0.125	-0.010	0.115	0.000		
15	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
16	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
17	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
18	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
19	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
20	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
21	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
22	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
0	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
0	-66.338	-41.490	0	0	0	0	0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000		
			65.6	63	I polar	132909.7	1.000	0.000	1.000	0.000		1.000	0.000	1.000	0.000		

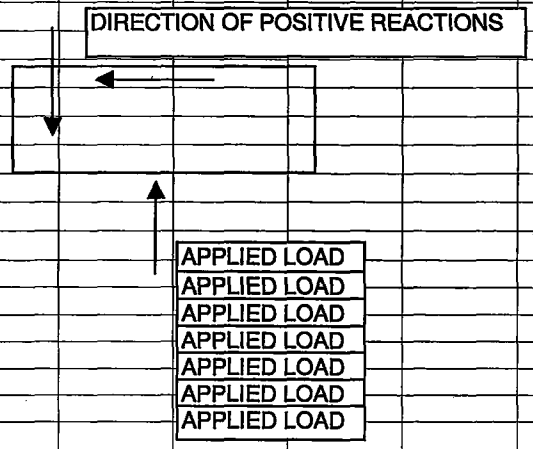
241059

DETERMINATION OF CENTRE OF STIFFNESS

	X(m)	Y(m)	Ky	Kx	Ky * X	Kx * Y
1	97.87	14.8	0	10.5	0	155.4
2	87.53	14.8	0	10.5	0	155.4
3	77.03	14.8	0	10.5	0	155.4
4	66.56	68.18	0	10.5	0	715.89
5	56.06	68.18	0	10.5	0	715.89
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12	102.97	35.28	8.2	0	844.354	0
13	102.97	43.48	8.2	0	844.354	0
14	102.97	51.68	8.2	0	844.354	0
15						
16						
17						
18						
19						
20						
21						
22						
	TOTALS		65.6	63	4351.74	2613.87

X= 66.338

Y= 41.490



LOAD DISTRIBUTION								EARTHQUAKE IN X DIRECTION					EARTHQUAKE IN Y DIRECTION			
									ECCENTRICITY		-3.49			ECCENTRICITY =		-14.85
	RELATIVE TO COS															
	X	Y	Kiy	Kix	X*X*Kiy	Y*Y*Kix		Six'	Six"	Six	Siy		Siy'	Siy"	Siy	Six
1	31.533	-26.690	0	10.5	0	7479.739		0.167	0.007	0.174	0.000		0.000	0.000	0.000	-0.031
2	21.193	-26.690	0	10.5	0	7479.739		0.167	0.007	0.174	0.000		0.000	0.000	0.000	-0.031
3	10.693	-26.690	0	10.5	0	7479.739		0.167	0.007	0.174	0.000		0.000	0.000	0.000	-0.031
4	0.223	26.690	0	10.5	0	7479.739		0.167	-0.007	0.159	0.000		0.000	0.000	0.000	0.031
5	-10.278	26.690	0	10.5	0	7479.739		0.167	-0.007	0.159	0.000		0.000	0.000	0.000	0.031
6	-20.778	26.690	0	10.5	0	7479.739		0.167	-0.007	0.159	0.000		0.000	0.000	0.000	0.031
7	-36.633	-22.590	8.2	0	11003.91	0		0.000	0.000	0.000	-0.008		0.125	0.034	0.159	0.000
8	-36.633	-14.410	8.2	0	11003.91	0		0.000	0.000	0.000	-0.008		0.125	0.034	0.159	0.000
9	-36.633	1.990	8.2	0	11003.91	0		0.000	0.000	0.000	-0.008		0.125	0.034	0.159	0.000
10	-36.633	10.190	8.2	0	11003.91	0		0.000	0.000	0.000	-0.008		0.125	0.034	0.159	0.000
11	36.633	-14.410	8.2	0	11003.91	0		0.000	0.000	0.000	0.008		0.125	-0.034	0.091	0.000
12	36.633	-6.210	8.2	0	11003.91	0		0.000	0.000	0.000	0.008		0.125	-0.034	0.091	0.000
13	36.633	1.990	8.2	0	11003.91	0		0.000	0.000	0.000	0.008		0.125	-0.034	0.091	0.000
14	36.633	10.190	8.2	0	11003.91	0		0.000	0.000	0.000	0.008		0.125	-0.034	0.091	0.000
15	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
16	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
17	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
18	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
19	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
20	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
21	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
22	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
0	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
0	-66.338	-41.490	0	0	0	0		0.000	0.000	0.000	0.000		0.000	0.000	0.000	0.000
			65.6	63	I polar	132909.7		1.000	0.000	1.000	0.000		1.000	0.000	1.000	0.000

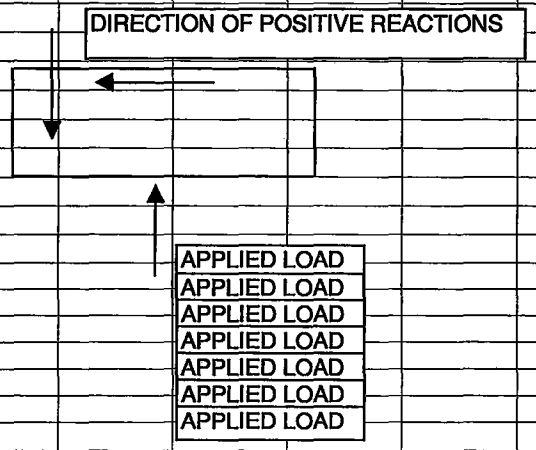
241059

DETERMINATION OF CENTRE OF STIFFNESS

	X(m)	Y(m)	Ky	Kx	Ky * X	Kx * Y
1	97.87	14.8	0	10.5	0	155.4
2	87.53	14.8	0	10.5	0	155.4
3	77.03	14.8	0	10.5	0	155.4
4	66.56	68.18	0	10.5	0	715.89
5	56.06	68.18	0	10.5	0	715.89
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8	29.705	27.08	8.2	0	243.581	0
9	29.705	43.48	8.2	0	243.581	0
10	29.705	51.68	8.2	0	243.581	0
11	102.97	27.08	8.2	0	844.354	0
12	102.97	35.28	8.2	0	844.354	0
13	102.97	43.48	8.2	0	844.354	0
14	102.97	51.68	8.2	0	844.354	0
15						
16						
17						
18						
19						
20						
21						
22						
	TOTALS		65.6	63	4351.74	2613.87

X= 66.338

Y= 41.490



Design
Of
K-Braces

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design
By: Dean Saunders

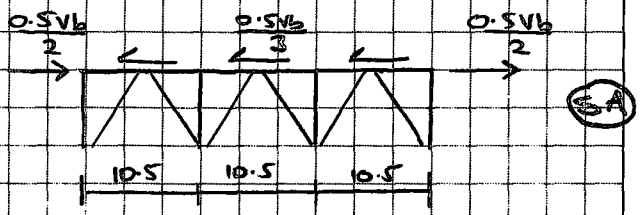
Job No: 2711265/109
Page No: 1 of 61
Date: 5-5-04

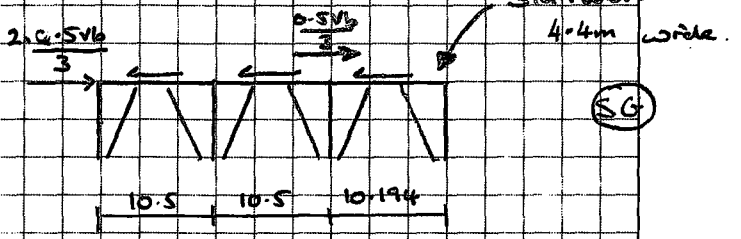
K-brace frames on (5A) and (5G)

3 bays with a loading of $0.5V_b$.

from lateral load analysis $\mu = 6$, soft soil, $T = 0.1$ sec

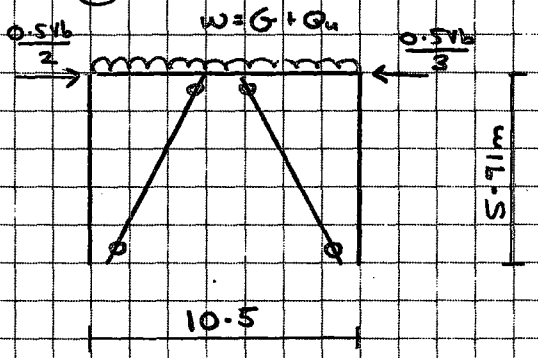
$V_b = 7493$ kN





* ensure midspan secondary beam connects to stairwell wall.

Grid (5A)



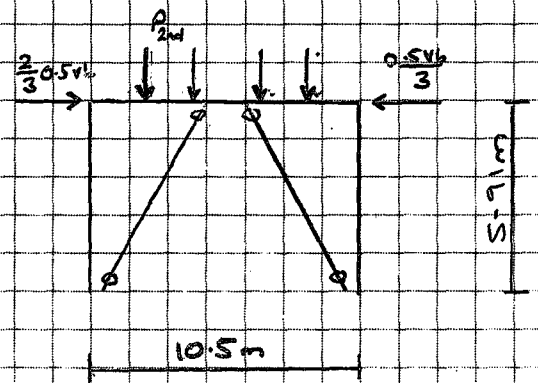
$w = G + Q_u$

$G = 2.07m^2 \times (0.135 \times 23.5 + 0.5)$
$= 7.59$ kN/m

$Q_u = 0.4 \times 6.5$ kPa $\times 2.07$ m
$= 5.37$ kN/m

$w = 7.59 + 5.37$
$= 13.0$ kN/m

Grid (5G)



$P_{2nd} = 8.61m^2 \times (0.135 \times 23.5 + 0.5 + 0.49 + 6.5 \times 0.4)$

$= 58.2$ kN

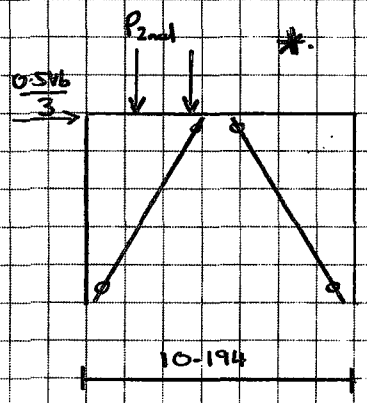
241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: lc-brace frame design
By: Dean Saunders.

Job No: 2711265/109
Page No: 2 of 61
Date: 5-5-04

or



NOTE: Midspan secondary beam must be supported by stairwell wall structure as concrete with active link location.

when designing columns must include axial loads from structure above.

Space-gass modeling for loads. 3 load cases.

Beam 700WB130.
active link 700mm clear.
Column 310UB40
Brace 310UC137

Interstorey Drift check

max = 5.125mm = 0.09%
max allowable = 20mm = 0.4% ok ✓

link Size:

$V^*_{link} \leq \phi V_v$

753kN $\leq \phi V_v$

1100 for 700WB130 to lyr

use 530UB82.0 ϕV_v = 876kN

753kN $\leq$ 876kN ✓

$\frac{502}{9.6} \leq \frac{82}{\sqrt{\frac{320}{250}}}$

52.3 $\leq$ 72.5 ✓

530UB82.0 link Size.

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace design
By: Dean Saunders

Job No: 2711265/109
Page No: 3 of
Date: 5-5-04

Section 12.4 check ok ✓

section 12.5 check Double symetry ✓

web $52.3 \leq 82$ ✓ Must add web stiffeners

flange $7.55 \leq 9$ ✓

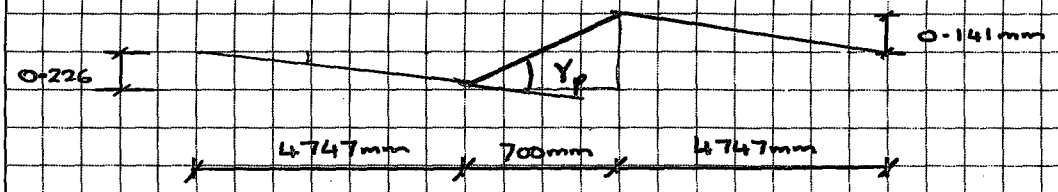
Section 12.11

$$d_b \leq e \leq 1.6\frac{M_s}{V_w}$$

$$528 \leq 700 \leq 1.6 \times \frac{558}{876} \quad \left(\frac{\phi M_s}{\phi V_w}\right)$$

$528 \leq 700 \leq 1019\text{mm}$ ✓

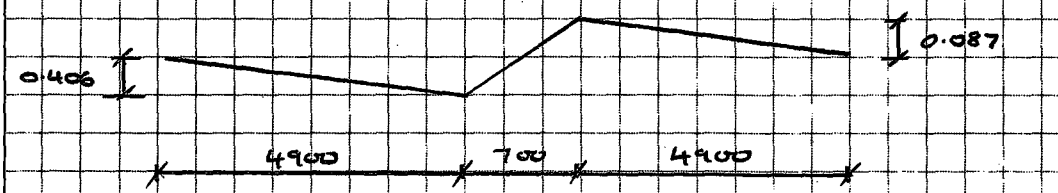
12.11.3.3



$$\gamma_p = \text{ATAN}\left(\frac{0.226}{4747}\right) + \text{ATAN}\left(\frac{0.226+0.141}{700}\right)$$

$= 0.00057$ radians ≤ 0.09 rad ✓

OR



$$\gamma_p = \text{ATAN}\left(\frac{0.406}{4900}\right) + \text{ATAN}\left(\frac{0.493}{700}\right)$$

$= 0.00079$ radians ≤ 0.09 rad

12.11.3.3 ✓

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design.
By: Dean Saunders

Job No: 2711265/109
Page No: 4 of
Date: 5-5-04

$V_{val} = 973\,kN$.

$M_{sal} = 2759mm^2 \times 514.8mm \times 300MPa \times \lambda_e \div 7.55$

$= 426\,kNm$

$N_{sal} = 2759mm^2 \times 2 \times 300MPa$

$= 1655\,kN$.

$b_e = 99.7 \times \frac{16}{7.55}$

$= 99.7mm$

$A_e = 209 \times 13.2$

$= 2759mm^2$

$M^* \leq \phi M_{sal}$

$\leq 0.9 \times 426\,kNm$

$307 \leq 383\,kNm$ ✓

$N^* \leq \phi N_{sal}$ must transfer $\frac{2}{3} 0.5V_b = 2498kN$

$\leq 0.9 \times 1655\,kN$

$2498kN \leq 1490\,kN$ ✗ Axial load excessive

530 UB NO GOOD.

No UB big enough

Try 700WB130.

$\phi N_{sal} = 2700\,kN$.

$\phi M_{sal} = 1020\,kNm$

quick

Combined action $\phi M_r = 1020\left(1 - \frac{2498}{2700}\right)$

$= 76.3\,kNm$ ✗ no good.

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace design
By: Dean Saunders

Job No: 2711265/109
Page No: 5 of
Date: 5-5-04

use special section.

try custom WB web 475 min 10 tw.

$\phi V_v \geq 0.9 \times 0.6 \times 300 \times 475 \times 10$.

$\geq 769.5\ kN$

$\phi V_v \geq V^*$

$769.5 \geq 753$ ✓

$\frac{475}{10} \leq 72.5$

$47.5 \leq 72.5$ ✓ must add web stiffeners.

Check compactness

web $\lambda_e = \frac{475}{10} \times \sqrt{\frac{300}{250}} \leq 82$

$52 \leq 82$ ✓ web o.k

flange 295 × 25 mm

$\frac{\left(\frac{295}{2} \div 10\right)}{25} \times \sqrt{\frac{300}{250}} \leq 8$

$5.7 \leq 8$ flange ok Compact section.

Section 12-11

$d_b \leq R \leq 1.6 \frac{M_s}{V_w}$

$475 \leq 700 \leq 1.6 \times \frac{295 \times 25 \times 500 \times 300}{769,500}$ ✓ flanges only. so conservative.

$475 \leq 700 \leq 2300$ ✓

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Job No: 2711265
Subject: K-brace frame design Zone 25
Page No: 6 of
By: Dean Saunders.
Date: 6-5-04

check 12.11.3.3 at final design.

$V_{val} = 855 kN$

$M_{sal} = 295 \times 25 \times 500 \times 300$

$= 1106 kNm$

$N_{sal} = 295 \times 25 \times 2 \times 300$.

$= 4425 kN$

$N^* \leq \phi N_{sal}$ $\quad N^* = \frac{2}{3} \times 0.5 V_b$.

$2498 kN \leq 0.9 \times 4425 \quad = 2498 kN$.

$2498 kN \leq 3982.5 kN$ ✓

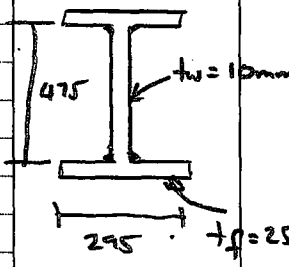
combined act'n

$M^* \leq \phi M_{sal} \left(1 - \frac{N^*}{\phi N_{sal}}\right)$.

$307 kNm \leq 0.9 \times 1106 \left(1 - \frac{2498}{3982.5}\right)$

$307 kNm \leq 371 kNm$ ✓

use custom WB 525 overall



web stiffening requirements.

full width of WB.

$t_{stiff} \geq 0.75 t_w$.

$\geq 7.5mm$ ∴ use 8mm plate.

stiffeners full width 8mm t2.2k.

each end.

Calculation Sheet

Job name: Queensgate Redevelopment. Job No: 2711265/109

Subject: K-brace frame Zone ZJ. Page No: 7 of

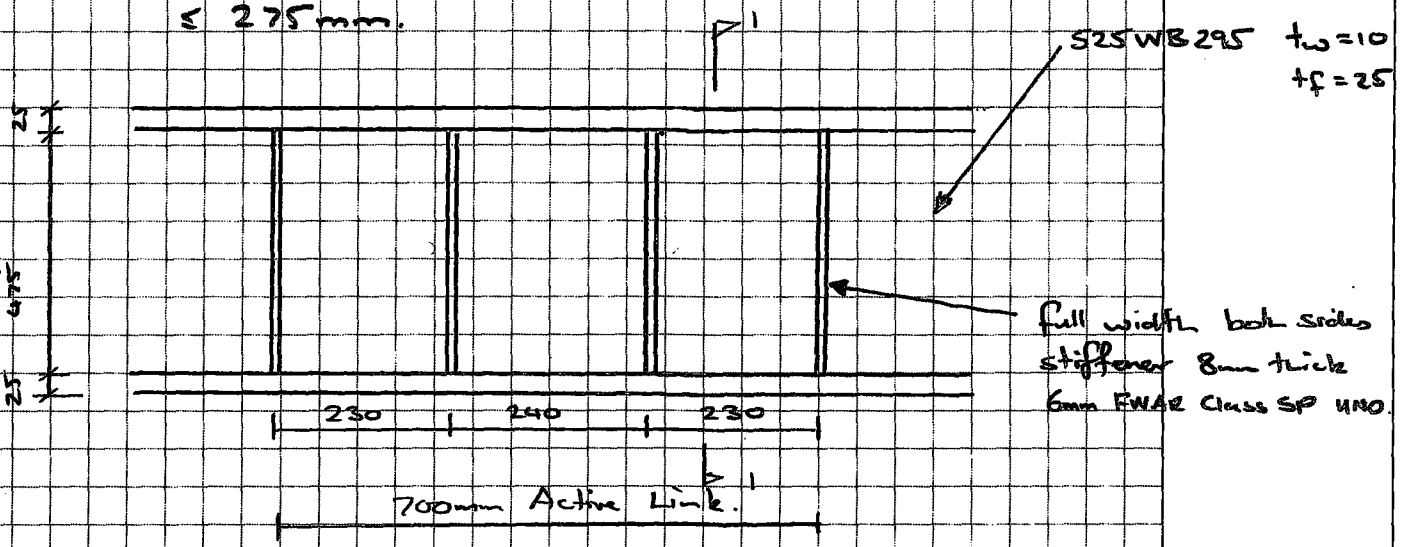
By: Dean Saunders. Date: 6-5-04

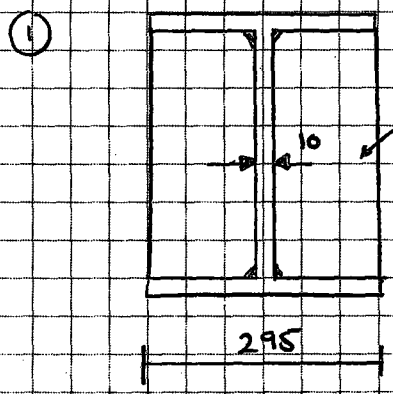
Spacing of I-t stiffener

$S \leq 38 t_{wb} - d_b/5$

$\leq 38 \times 10 - 525/5$

≤ 275 mm.





weld check $\phi A_s f_{ys}$ = 0.9 × 8 × 142.5 × 340

= 307.8 kN

$\phi V_w = 0.8 \times 0.6 \times f_{uw} \times t_t \times k_r$

$= 0.8 \times 0.6 \times 410 \times \frac{6}{\sqrt{2}} \times 1$

= 835 N/mm

Web. $V^* = \phi A_s f_{ys}$ = 307.8kN $\leq \phi V_w \times L$

$\leq$ 835N/mm L.

L $\geq$ 369mm

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame Zone ZJ
By: Dean Saunders

Job No: 2711265
Page No: 8 of
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flange $V^* = \frac{\phi A_s f_{ys}}{4} = 77\,kN \leq \phi v_w L$

$\leq 635\,N/mm \times L$.

$L \geq 92\,mm$

use full width.

Lateral Restraint Requirements at ends

2.5% of $\phi f_y A_{fb} = 0.025 \times 0.9 \times 300 \times 295 \times 25$

$= 49.8\,kN$.

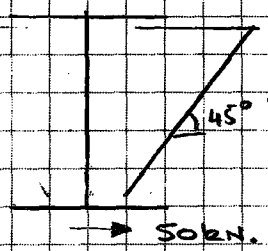
12.6.2.3.1

$\frac{L}{r_y} \leq 30\sqrt{\frac{250}{300}}$.

$L \leq 27.4 r_y$.

$L \leq 27.4 \times 74$

$\leq 2.0m$ no additional restraint needed.



or handcal.

$N_t^* = 71\,kN$.

use 75×5EA.

$N_t^* \leq \phi N_t$.

$71 \leq 0.9 \times 300 \times 672$.

$71 \leq 181\,kN$ ✓

fly brace.
75×5EA

Bolts.

M20 needed for strip plates from tables.

$71\,kN \leq 92.6$ ✓

$\phi V_b = 0.9 \times 3.2 \times 5 \times 20 \times 440$ or $0.9 \times 9e \times 5 \times 440$.

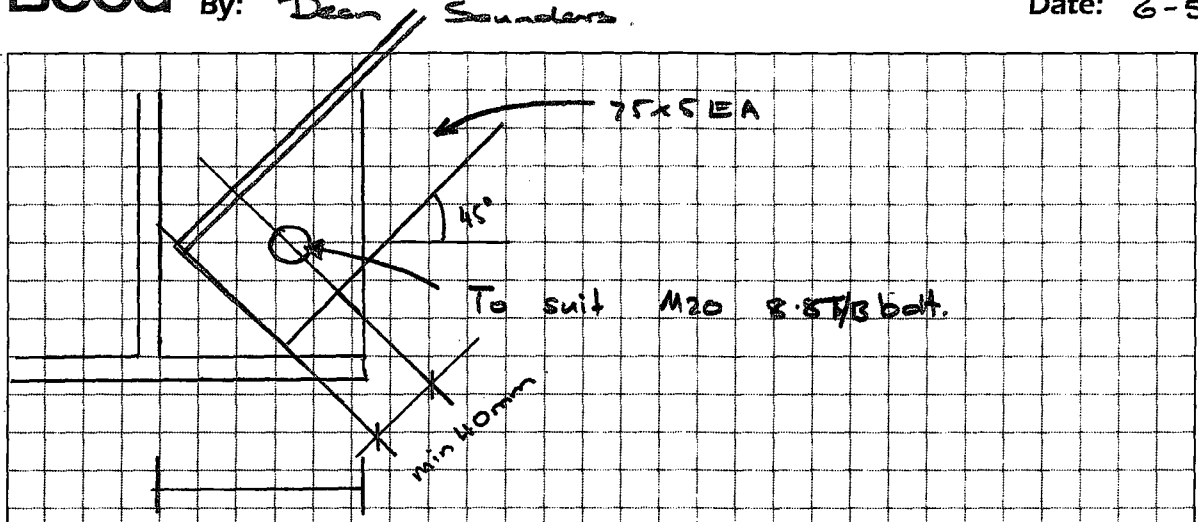
$= 126\,kN$ or $1.98e$.

$e \geq \frac{71}{1.98} = 36mm$

Calculation Sheet

Job name: Queensgate Redevelopment
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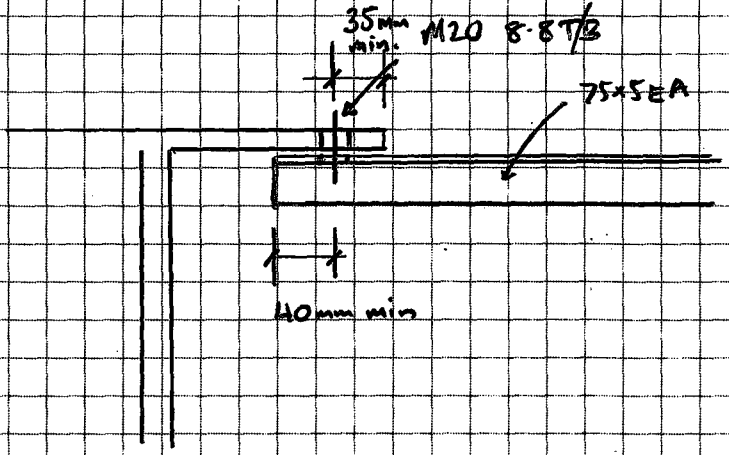
75×5 ⊑A.
M20 8.8T/B
edge to ℄
min 40mm.

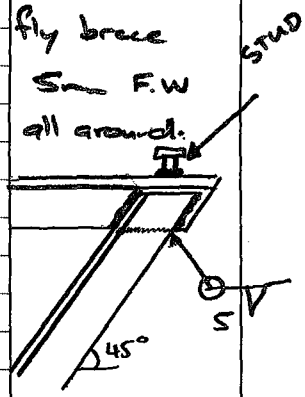
welded.

$\phi V_{wn} = 0.6 \times 0.6 \times \frac{5}{\sqrt{2}} \times 410 \times 2 \times 80mm$

$= 83kN$ for 2 sides 5mm FW

71kN ≤ 83kN 5mm FW all around ok.





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Calculation Sheet

Job name: Queenagate Redevelopment
Subject: K-brace frame design
By: Dean Saunders

Job No: 2711265/109
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$V^{\circ}_{1.1k} = \phi_{1.1} V_w$

$= 1.40 \times 855\,kN$

$= 1197\,kN$

$\phi^{\circ} = 1.40$

$V^{\circ}_{1.1k} = 1197\,kN$

Brace Capacity design:

$N^{*}_{brace} = \phi^{\circ} N_{brace\,E} + N_{brace\,G,Qu}$

$= 1.40 \times 993\,kN + 113\,kN$

$= 1503\,kN$

$N^{*}_{brace} = 1277\,kN$ Tension.

$N^{*}_{brace} = 1503\,kN$ Compression.

Beam Capacity Design

$N^{*}_{beam} = 0.8 \times 1.40 \times \frac{2}{3} \times 0.5 V_b$ from floor loadings

$= 0.8 \times 1.4 \times \frac{2}{3} \times 0.5 \times 7493.$

$= 2797\,kN$

$N^{c}_{beam} = 2797\,kN$ C & T

$M^{c}_{beam} = 0.8 \times \phi^{\circ} \cdot M_{beam\,E}$

$= 0.8 \times 1.4 \times 252\,kNm$

$= 282\,kNm$

$M^{*}_{beam} = 282\,kNm + M_{G,Qu}$

$= 282\,kNm + 61.9\,kNm$

$= 344\,kNm$

$M^{*}_{beam} = 344\,kNm$

$V^{c}_{beam} = 1.4 \times 52.1\,kN$

$= 73\,kN$

$V^{*}_{beam} = 73\,kN + V_{G,Qu}$

$= 73\,kN + 86.8\,kN$

$= 160\,kN$

$V^{*}_{beam} = 160\,kN$

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design
By: Dean Saunders.

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Collector beam design:

$N^* = 2797\ kN$ Compression or Tension.

$M^* = 344\ kNm$

$V^* = 160\ kN.$

$M^* \leq \phi M_{s,x}$
$\leq \phi M_{bx}$

Compact section

$\leq \phi M_s$
$\leq \phi f_y Z_e$
$\leq 1150\ kNm$

$\phi M_{bx} = 1591\ kNm$ fully restrained

$344 \leq 1150\ kNm$ ✓ M^* ok.

$V^* \leq \phi V_w$.

$160 \leq 770\ kN$ ✓ V^* ok.

$N^* \leq \phi N_c$.
$\leq \phi N_s$.

$\phi N_s = 0.9\ k_f A_n f_y$.
$= 0.9 \times 0.92 \times 18,400 \times 300$
$= 4571\ kN$

web $b_e = 475 \times \frac{35}{52}$
$= 319.5$

$k_f = \frac{295 \times 25 \times 2 + 319.5 \times 10}{295 \times 25 \times 2 + 475 \times 10}$.
$= 0.92$

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Calculation Sheet

Job name: Queensgate Redevelopment. Job No: 2711265/109
Subject: K-brace frame Zone 7J Page No: 12 of
By: Dean Saunders. Date: 6-5-04

$\phi N_c = \phi \alpha_c N_s$

$= 0.9 \times 0.598 \times 4571$

$= 2461\ kN$

$\lambda_n = \frac{4908}{72} \times \sqrt{k_f} \times \sqrt{\frac{200}{250}}$

71.6

$\alpha_b = 1$

$\alpha_c = 0.598$

$N^* \leq \phi N_c$

$2797\ kN \not\leq 2461\ kN$ must reduce L_e ∴ place 75×50A fly brace at midspan of segment.

$\lambda_n = \frac{2454}{72} \times \sqrt{k_f} \times \sqrt{\frac{300}{250}}$

$= 35.8$

$\phi N_c = 0.9 \times 0.847 \times 4571$ $\alpha_b = 1$

$= 3484\ kN$ $\alpha_c = 0.847$

$2797\ kN \leq 3484\ kN$ ✓ N^* ok.

Combined Actin. $\delta_b = 1.0$ by calcs $= \frac{0.8}{1 - \frac{2797}{83,198}} \geq 1.0$

$M^* \leq \phi M_{cx}$

$\leq \phi M_{sx} \left(1 - \frac{N^*}{\phi N_s}\right)$

$\leq 0.9 \times 1150 \left(1 - \frac{2797}{4571}\right)$

$344\ kNm \leq 401\ kNm$ ✓ Beam ok. with midspan flybraces

Check $\lambda_{nx} = 22.5$ ok ✓

Brace Design.

N^*_{brace} = 1277 kN Tension

N^*_{brace} = 1503 kN Compression.

$N^* \leq \phi N_t$.

$\leq \phi A_g f_y$ & $\phi 0.85 k_t A_n f_u$.

$\leq 0.9 \times A_g \times 300$ & $0.9 \times 0.85 \times 1 \times A_n \times 440$.

$1277\ kN \leq \frac{270 A_g}{1000}$ & $\frac{336.6}{1000} A_n$.

$A_g \geq 4729\ mm^2$ ✓

$A_n \geq 3794\ mm^2$ ✓ for 310UC96.8.

$N^* \leq \phi N_s$

$\leq \phi N_c$.

$L_e = k_e L$ $k_e = 1.0$

$= 7.68m$. $L = 7.677m$.

$N^* = 1503\ kN$ requires 310UC96.8

$\phi N_s = 3340\ kN$

$\phi N_c = 1606\ kN$. $L_e = 7.68m$ ✓

Brace
310UB 96.8

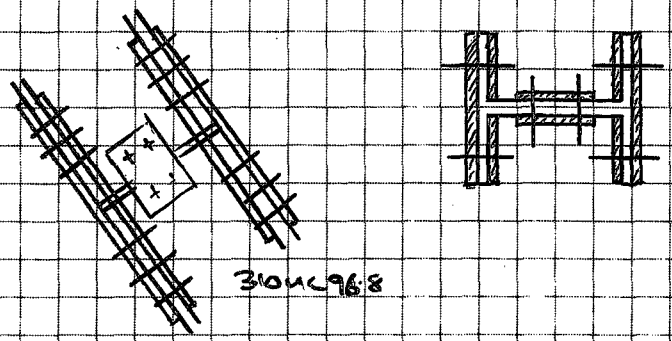
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Calculation Sheet

Job name: Queensgate Redevelopment
Subject: IC-brace frame Zone ZJ
By: Dean Saunders

Job No: 2711265/109
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Brace Splice.



Bolts. friction Type. $\mu = 0.35$

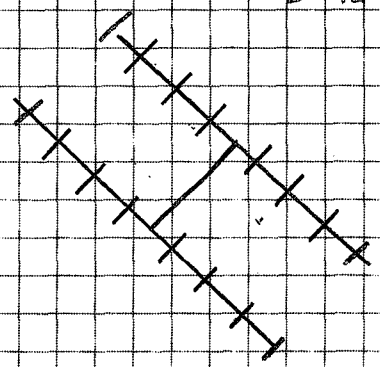
$V^* \leq \phi V_f$. $V^* = 1503$ kN.

M20 $\phi V_f = 35.5$ kN Single shear.
M24 $\phi V_f = 51.5$ kN " "

Bolts in Double shear.

$$\# M_{24} \text{ needed} = \frac{1503}{2 \times 51.5 \text{ kN}}$$

$= 14.6$ bolts. place 8 per flange.



Calculation Sheet

Job name: Queenscliff Redevelopment
Subject: K-brace Zone 25
By: DWS

Job No: 2711265/109
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Date: 6-5-04.

flange plates. use full width plates ext & 100 mm wide on inside.

Tension $1803\ kN \leq 0.9 \times 300 \times 2\left([245 + 2\times100]t - 4\times32\times t\right)$

$\leq 0.9\times300\times2\ (445 - 128)t$

$1503\times10^3 \leq 171{,}180t$

$t \geq 8.8\ mm$

Try 16mm thick plate. critical section is int plate 100×16.

$N^t \leq \phi N_s$
$\leq \phi N_c = \phi \alpha_c N_s$.

$N_s = k_f A_n f_y$
$= 1\times100\times16\times300$.
$= 480\ kN$

$L_c = 0.7\times150$
$= 105\ mm$

$r = \sqrt{\frac{16^2}{12}}$
$= 4.61$

$N^c \leq \phi N_c$
$\leq 0.9\times0.983\times480\ kN$.
$\leq 399\ kN$

$\lambda_n = 25$
$\alpha_b = 1$
$\alpha_c = 0.983$

$N^t = 752/4$
$= 188\ kN$ ✓ $188 \leq 399\ kN$ ✓

Plate bearing

$V_b = 3.2\, t_p d_b f_{up}$. uc Governs.
$= 3.2\times16\times24\times440$.
$= 520\ kN$

$94\ kN \leq 0.9\times520\ kN$ ✓

241059
7850 kg/m3

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace Zone 2J
By: DWS.

Job No: 2711265/109
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Plate tension.

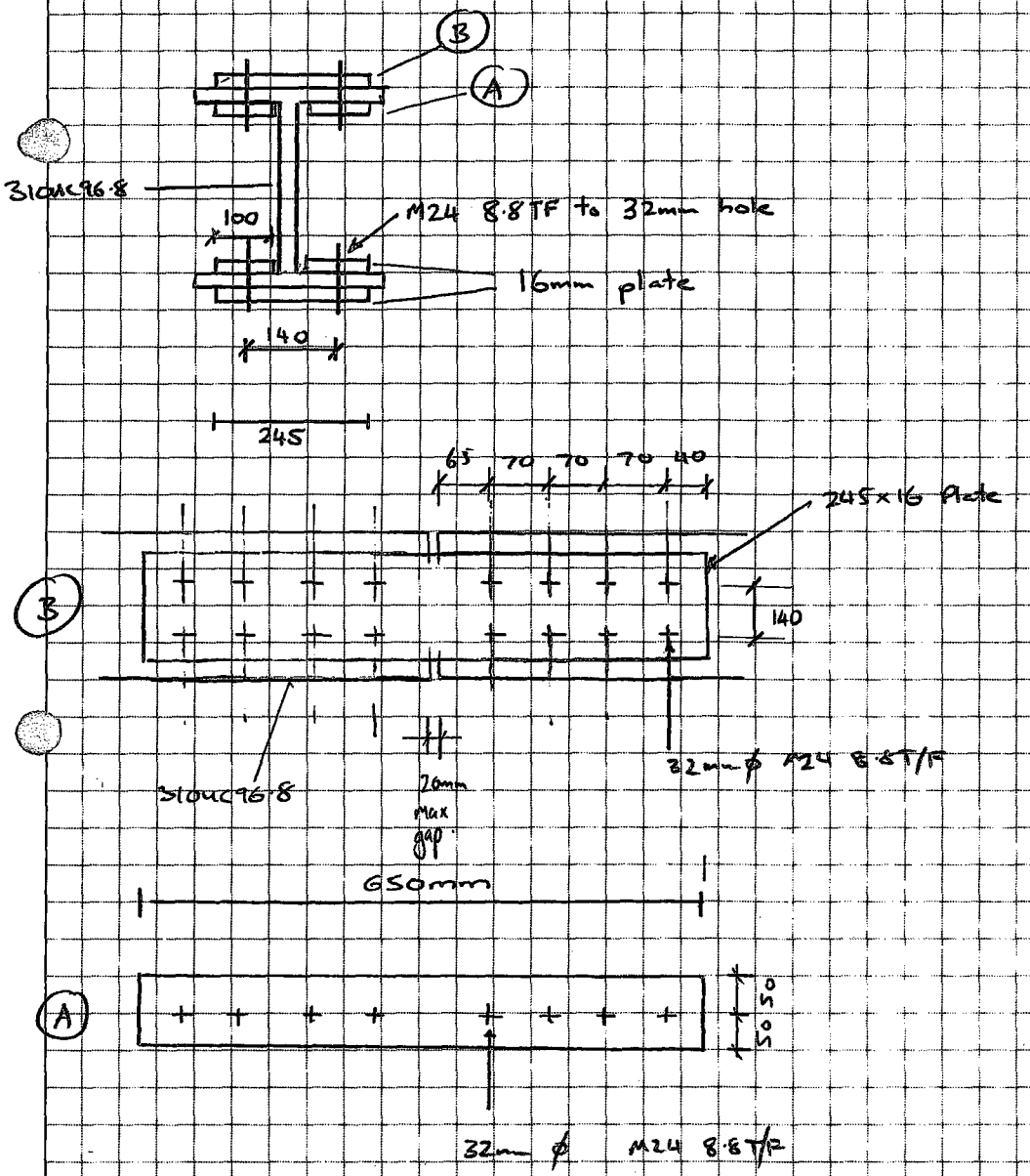
$V_p = 9 t_p f_{yp}$.

$= 40 \times 15.4 \times 440$.

$= 271$ kN

94 kN $\leq 0.9 \times 271$ kN ✓

Plate ok.

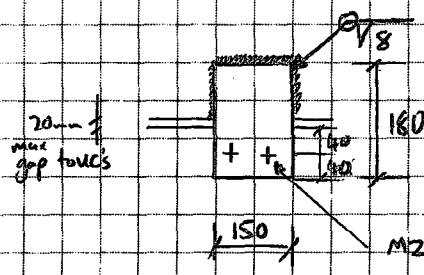


Calculation Sheet

Job name: Queensgate Redevelopment
Subject: k-brace Zone 7.5
By: DWS.

Job No: 2711265/109.
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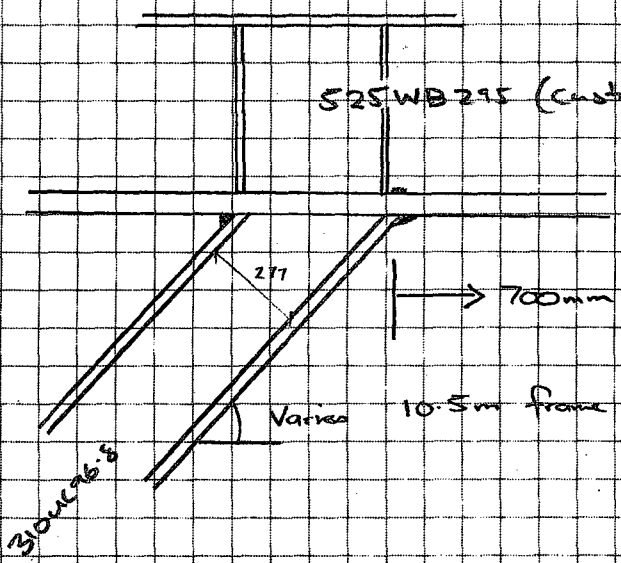
Web plate (erection plate)



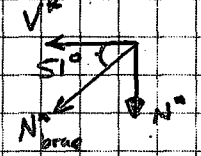
180x150x10 Plate.

M20 with hole tolerance for erection.

Brace to Beam Connection:



$N^*_{brace} = 1503\ kN$



$V^* = 941\ kN$ cover by web fillet weld + butt weld.
$N^* = 1172\ kN$ cover by flange butt weld

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Calculation Sheet

Job name: Queensgate Redevelopment
Subject: A-brace Zone 25
By: DMS.

Job No: 2711265/109
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B+H welds.

$n^* \leq \phi n_w$

$\leq 0.8 \times 0.6 \times 15.4 \times 410.$

$\leq 3.03 \text{ kN/mm}$

available length = 2×305
= 610mm

$n^* \leq \phi n_w$.

$\leq 3.03 \times 610$.

$\leq 1848 \text{ kN}$

remaining force to provide

$1172 + 941 - 1848 \text{kN} = 265 \text{kN}$ min

Provide 8mm fillet weld SP

$\phi_{nw} = 0.8 \times 0.6 \times \frac{8}{\sqrt{2}} \times 410.$

$= 1.110 \text{kN/mm}.$

$L = 277\text{mm} \cdot \frac{1}{\sin\theta}$ $\theta \geq 51°$

$L \geq 356\text{mm}$ use 340mm each side

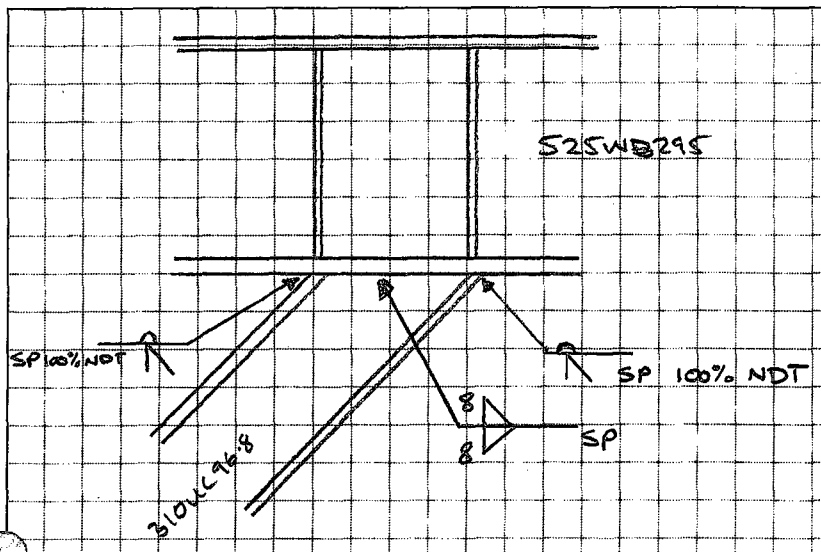
$\phi V_w = 1.11 \times 2 \times 340\text{mm}$
$= 754 \text{ kN}.$

$265 \text{ kN} \leq 754 \text{kN}$ ✓

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace Zone 3J
By: DWS

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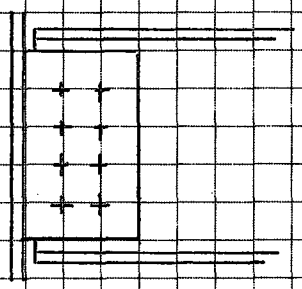
Calculation Sheet

Job name: Queensgate Redevelopment. Job No: 2711265/109

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By: Dean Saunders. Date: 10-5-04

Beam to Column Connection.

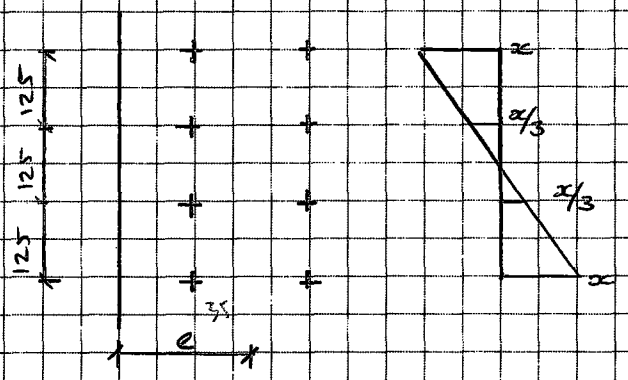


Loads:

$N^* = 699kN$

$V^* = 160kN.$

Try M24 bolts.



$e = 10 + 40 + \frac{70}{2}$

$= 85mm$

$V^*e = x \times 375 + \frac{x}{3} \times 125$

$160 \times 85 = 416\frac{2}{3}x$

$x = 32.6\ kN.$

$N^* = \frac{699kN}{8} + \frac{32.6kN}{2}$

$= 103.7\ kN$

$V^* = \frac{160}{8}$

$= 20\ kN$

$V^*_{bolt} = \sqrt{103.7^2 + 20^2}$

$= 106\ kN$

$V^*_{bolt} \leq \phi V_f$

$106kN \leq 133kN$ ✓

Calculation Sheet

Job name:

Subject:

By: Dean Saunders

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$N^*_t \leq \phi N_{t\,plate} \quad \& \quad V^* \leq \phi V_{t\,plate}$

$700kN \leq 0.9 \times t_t \times (375 + 80 - 4 \times 26) \times 300$

$t_t \geq \frac{700,000}{94,770}$

$\geq 7.4\,mm$

use 10mm plate.

10mm plate.

$V^* \leq \phi V_t$

$160kN \leq 0.9 \times 0.6 \times 300 \times 10 \times (375 + 80 - 4 \times 26)$

$160kN \leq 568kN$ ✓

$\left(\frac{N^*}{\phi N_t}\right)^2 + \left(\frac{V^*}{\phi V_t}\right)^2 \leq 1$

$\left(\frac{700}{927.7}\right)^2 + \left(\frac{160}{568}\right)^2 \leq 1$

$(0.739)^2 + (0.282)^2 \leq 1$

$0.624 \leq 1$ ✓ ok.

Check plate bearing & tearout.

$V^*_{bolt} \leq \phi V_b$

$\leq 0.9 \times 3.2 \, d_f t_p f_{up}$

$\leq 0.9 \times 3.2 \times 10 \times 24 \times 440$

$106kN \leq 304kN.$ ✓

$V^*_{bolt} \leq 9 k_t t_p f_{up}$

$\leq 40 \times 10 \times 440$

$106kN \leq 176kN$ ✓

241059

Calculation Sheet

Job name:

Subject:

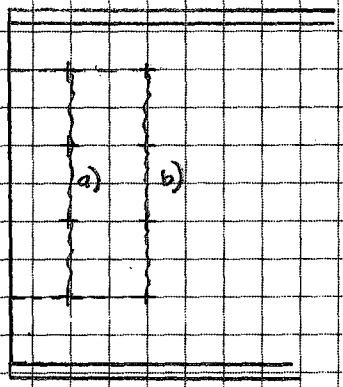
By: Dean Saunders.

Job No: 2711265/009

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Date: 11-5-04

Check ultimate fracture of WB web.



a) $F_{web} = 0.6 \times f_y A_w + A_t f_{up}$.

$= 0.6 \times 300 \times 2(40-13) \times 10 + 10 \times (375 - 4 \times 26) \times 440$

$= 1289.6\ kN.$

b) $F_{web} = 0.6 \times 300 \times 2 \times 10 (410 - 39) + 10 \times (375 - 4 \times 26) \times 440$

$= 1448\ kN.$

$N^* \leq \phi F_{web\ b)}$ $\quad 699kN \leq 1303\ kN$ ✓

$\frac{N^*}{2} \leq \phi F_{web\ a)}$. $\quad \frac{699}{2} kN \leq 1160\ kN$ ✓

Web o.k.

Weld to Column:

Weld $L = 3 \times 125 + 2 \times 40$.

$= 455mm.$

$V^* = 699kN + 160kN$.

$= 859kN.$

$V^* \leq \phi V_w$.

use butt weld.

$859kN \leq \phi V_w \times L$

$\leq 0.8 \times 455 \times 0.6 \times 10 \times 410.$

$859kN \leq 895\ kN$ ✓

10mm butt weld with backing strip.

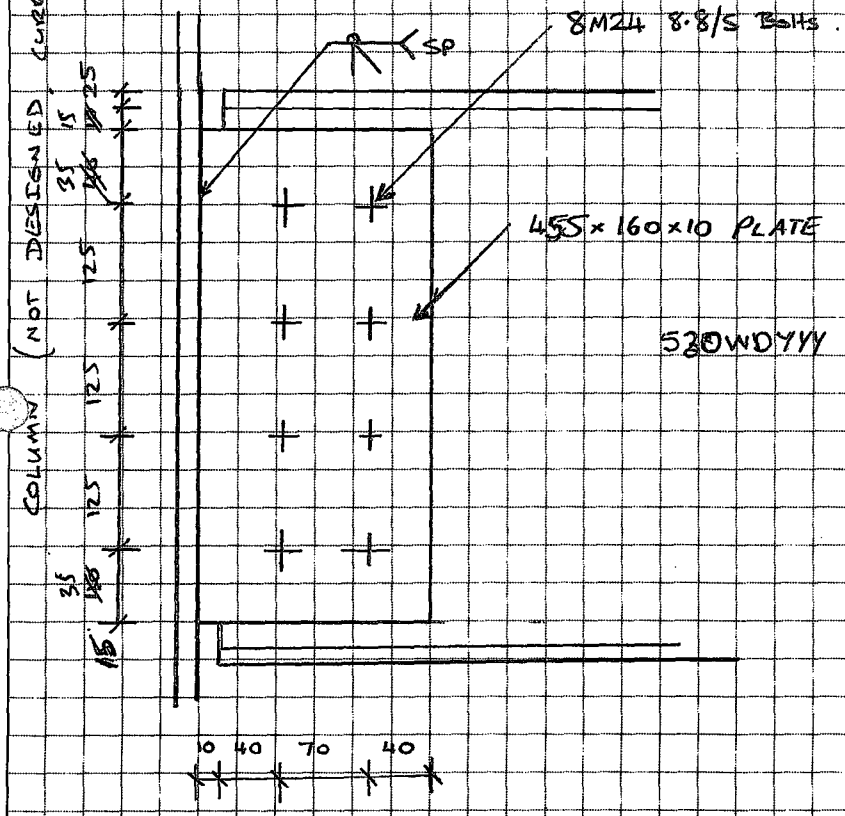
Calculation Sheet

Job name: Queensgate mall Redevelopment.
Subject: K-brace frame design.
By: Dean Sanders.

Job No: 3711265/109
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Date: 11-5-04

Beam to Column Connection

(NOT DESIGNED CURRENTLY)



241059

L = 10.5 m UDL

2711265/10-1
Pages 24 - 26

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.500	0.000	0.000
3	0.000	5.910	0.000
4	10.500	5.910	0.000
5	4.900	5.910	0.000
6	5.600	5.910	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.900
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.900
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.677
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.677

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Column 310 UB 4	S1	Not applicable	No	Manual
2	Beam 700 WB 130	S2	Not applicable	No	Manual
3	Brace 310 UC 13	S3	Not applicable	No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.2100E+03	1.5700E+05	7.6500E+06	8.6400E+07	INFINITE	INFINITE	0.00
2	1.6600E+04	1.5100E+06	5.2100E+07	1.4000E+09	INFINITE	INFINITE	0.00
3	1.7500E+04	2.5200E+06	1.0700E+08	3.2900E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	3	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000
	4	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000
	5	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	4.899	-0.278	0.000	0.000	0.000	-0.001
4	4.899	0.278	0.000	0.000	0.000	-0.001
5	3.981	-0.524	0.000	0.000	0.000	0.001
6	3.981	0.524	0.000	0.000	0.000	0.001

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.004	-0.146	0.000	0.000	0.000	0.000
4	-0.004	-0.146	0.000	0.000	0.000	0.000
5	0.004	-0.160	0.000	0.000	0.000	0.000
6	-0.004	-0.160	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	4.903	-0.424	0.000	0.000	0.000	-0.001
4	4.894	0.132	0.000	0.000	0.000	-0.001
5	3.985	-0.685	0.000	0.000	0.000	0.001
6	3.977	0.364	0.000	0.000	0.000	0.001

Dmax

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	49.007	2.557	0.000	0.000	0.000	-9.884
	3	49.007	2.557	0.000	0.000	0.000	5.227
2	2	-49.007	2.557	0.000	0.000	0.000	-9.884
	4	-49.007	2.557	0.000	0.000	0.000	5.227
3	3	621.863	49.007	0.000	0.000	0.000	5.227
	5	621.863	49.007	0.000	0.000	0.000	245.362
4	5	0.000	-701.036	0.000	0.000	0.000	245.362
	6	0.000	-701.036	0.000	0.000	0.000	-245.362
5	6	-621.863	49.007	0.000	0.000	0.000	-245.362
	4	-621.863	49.007	0.000	0.000	0.000	-5.227
6	1	-974.309	0.000	0.000	0.000	0.000	0.000
	5	-974.309	0.000	0.000	0.000	0.000	0.000
7	2	974.309	0.000	0.000	0.000	0.000	0.000
	6	974.309	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	25.792	-0.383	0.000	0.000	0.000	0.751
	3	25.792	-0.383	0.000	0.000	0.000	-1.515
2	2	25.792	0.383	0.000	0.000	0.000	-0.751
	4	25.792	0.383	0.000	0.000	0.000	1.515
3	3	0.383	25.792	0.000	0.000	0.000	-1.515
	5	0.383	-37.908	0.000	0.000	0.000	-31.198
4	5	35.585	4.550	0.000	0.000	0.000	-31.198
	6	35.585	-4.550	0.000	0.000	0.000	-31.198
5	6	0.383	37.908	0.000	0.000	0.000	-31.198
	4	0.383	-25.792	0.000	0.000	0.000	-1.515
6	1	55.153	0.000	0.000	0.000	0.000	0.000
	5	55.153	0.000	0.000	0.000	0.000	0.000
7	2	55.153	0.000	0.000	0.000	0.000	0.000
	6	55.153	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	74.799	2.173	0.000	0.000	0.000	-9.133
	3	74.799	2.173	0.000	0.000	0.000	3.712
2	2	-23.215	2.940	0.000	0.000	0.000	-10.635
	4	-23.215	2.940	0.000	0.000	0.000	6.741
3	3	622.247	74.799	0.000	0.000	0.000	3.712
	5	622.247	11.099	0.000	0.000	0.000	214.164
4	5	35.585	-696.486	0.000	0.000	0.000	214.164
	6	35.585	-705.586	0.000	0.000	0.000	-276.561
5	6	-621.480	86.915	0.000	0.000	0.000	-276.561
	4	-621.480	23.215	0.000	0.000	0.000	-6.741
6	1	-919.156	0.000	0.000	0.000	0.000	0.000
	5	-919.156	0.000	0.000	0.000	0.000	0.000
7	2	1029.462	0.000	0.000	0.000	0.000	0.000
	6	1029.462	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	-701.036	0.000	0.000	0.000	9.884
2	-624.420	701.036	0.000	0.000	0.000	9.884
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

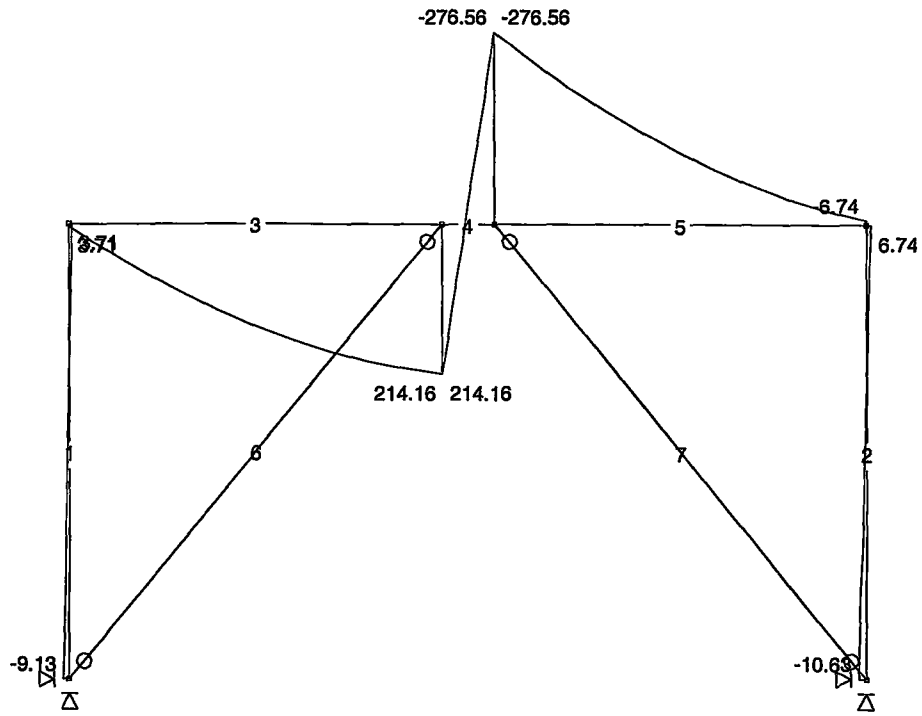
Load case 2 (Linear): G+Qu

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	35.585	68.250	0.000	0.000	0.000	-0.751
2	-35.585	68.250	0.000	0.000	0.000	0.751
Load	0.000	-136.500	0.000			
Reac	0.000	136.500	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-588.835	-632.786	0.000	0.000	0.000	9.133
2	-660.005	769.286	0.000	0.000	0.000	10.635
Load	1248.840	-136.500	0.000			
Reac	-1248.840	136.500	0.000			

All combination load cases:
3

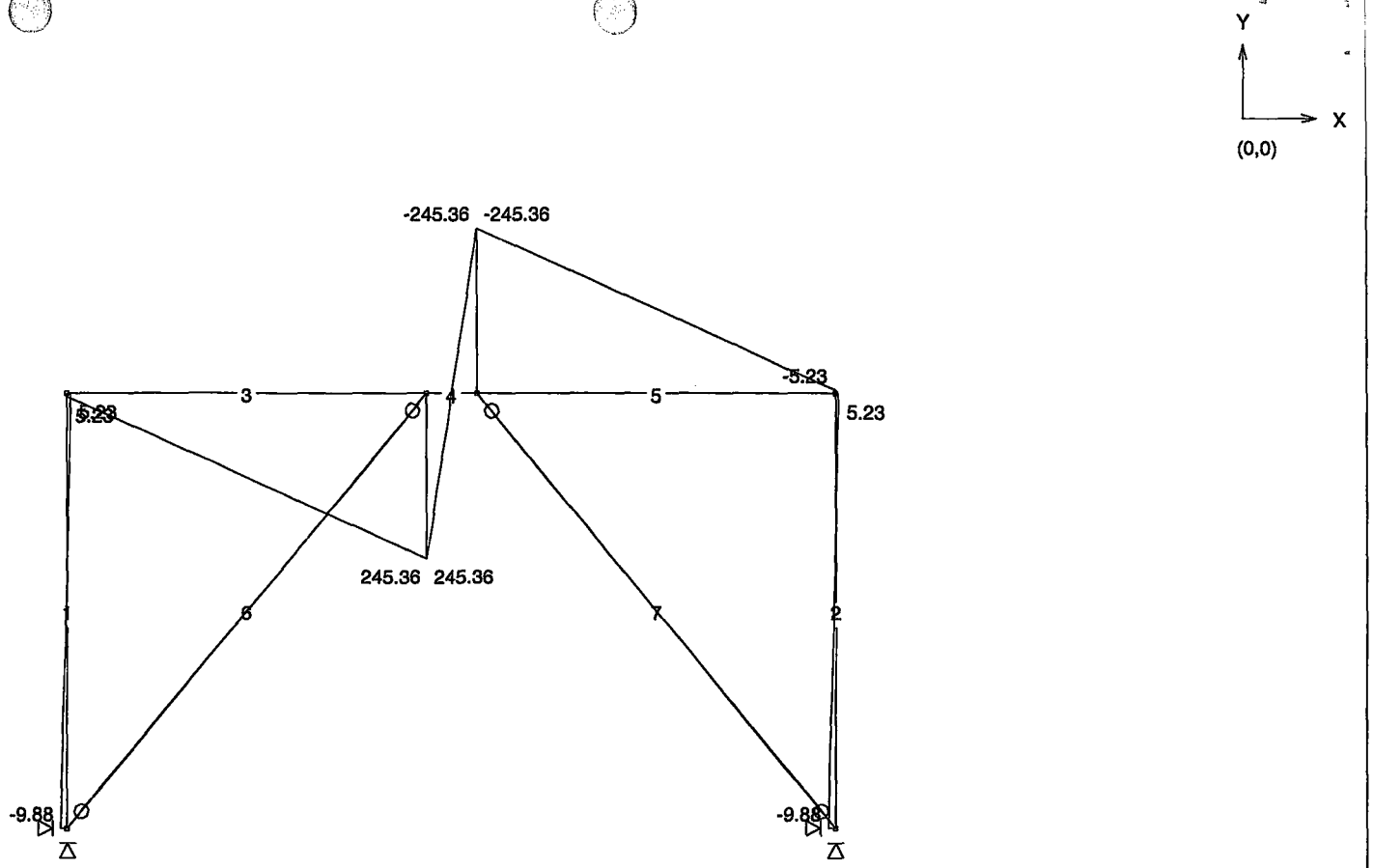


Moment
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 1
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

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Load cases:
■ 1 —— E loads



Moment
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 1
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

L=10.5m . Point loads 2711265/10

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NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.500	0.000	0.000
3	0.000	5.910	0.000
4	10.500	5.910	0.000
5	4.900	5.910	0.000
6	5.600	5.910	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.900
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.900
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.677
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.677

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Column 310 UB 4 S1		Not applicable	No	Manual
2	Beam 700 WB 130 S2		Not applicable	No	Manual
3	Brace 310 UC 13 S3		Not applicable	No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.2100E+03	1.5700E+05	7.6500E+06	8.6400E+07	INFINITE	INFINITE	0.00
2	1.6600E+04	1.5100E+06	5.2100E+07	1.4000E+09	INFINITE	INFINITE	0.00
3	1.7500E+04	2.5200E+06	1.0700E+08	3.2900E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
2	3	1	G	2.100	0.000	-58.200	0.000
					0.000	0.000	0.000
	3	2	G	4.200	0.000	-58.200	0.000
					0.000	0.000	0.000
	5	1	G	0.700	0.000	-58.200	0.000
					0.000	0.000	0.000
	5	2	G	2.800	0.000	-58.200	0.000
					0.000	0.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	4.899	-0.278	0.000	0.000	0.000	-0.001
4	4.899	0.278	0.000	0.000	0.000	-0.001
5	3.981	-0.524	0.000	0.000	0.000	0.001
6	3.981	0.524	0.000	0.000	0.000	0.001

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.009	-0.168	0.000	0.000	0.000	0.000
4	-0.009	-0.168	0.000	0.000	0.000	0.000
5	0.008	-0.328	0.000	0.000	0.000	0.000
6	-0.008	-0.328	0.000	0.000	0.000	0.000

Dmax

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	4.908	-0.446	0.000	0.000	0.000	-0.001
4	4.890	0.110	0.000	0.000	0.000	-0.001
5	3.989	-0.852	0.000	0.000	0.000	0.001
6	3.973	0.197	0.000	0.000	0.000	0.001

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	49.007	2.557	0.000	0.000	0.000	-9.884
	3	49.007	2.557	0.000	0.000	0.000	5.227
2	2	-49.007	2.557	0.000	0.000	0.000	-9.884
	4	-49.007	2.557	0.000	0.000	0.000	5.227
3	3	621.863	49.007	0.000	0.000	0.000	5.227
	5	621.863	49.007	0.000	0.000	0.000	245.362
4	5	0.000	-701.036	0.000	0.000	0.000	245.362
	6	0.000	-701.036	0.000	0.000	0.000	-245.362
5	6	-621.863	49.007	0.000	0.000	0.000	-245.362
	4	-621.863	49.007	0.000	0.000	0.000	-5.227
6	1	-974.309	0.000	0.000	0.000	0.000	0.000
	5	-974.309	0.000	0.000	0.000	0.000	0.000
7	2	974.309	0.000	0.000	0.000	0.000	0.000
	6	974.309	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	29.581	-0.794	0.000	0.000	0.000	1.555
	3	29.581	-0.794	0.000	0.000	0.000	-3.136
2	2	29.581	0.794	0.000	0.000	0.000	-1.555
	4	29.581	0.794	0.000	0.000	0.000	3.136
3	3	0.794	29.581	0.000	0.000	0.000	-3.136
	5	0.794	-86.819	0.000	0.000	0.000	-61.887
4	5	72.775	0.000	0.000	0.000	0.000	-61.887
	6	72.775	0.000	0.000	0.000	0.000	-61.887
5	6	0.794	86.819	0.000	0.000	0.000	-61.887
	4	0.794	-29.581	0.000	0.000	0.000	-3.136
6	1	112.778	0.000	0.000	0.000	0.000	0.000
	5	112.778	0.000	0.000	0.000	0.000	0.000
7	2	112.778	0.000	0.000	0.000	0.000	0.000
	6	112.778	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	78.589	1.763	0.000	0.000	0.000	-8.329
	3	78.589	1.763	0.000	0.000	0.000	2.091
2	2	-19.426	3.350	0.000	0.000	0.000	-11.439
	4	-19.426	3.350	0.000	0.000	0.000	8.362
3	3	622.657	78.589	0.000	0.000	0.000	2.091
	5	622.657	-37.811	0.000	0.000	0.000	183.475
4	5	72.775	-701.036	0.000	0.000	0.000	183.475
	6	72.775	-701.036	0.000	0.000	0.000	-307.250
5	6	-621.070	135.826	0.000	0.000	0.000	-307.250
	4	-621.070	19.426	0.000	0.000	0.000	-8.362
6	1	-861.531	0.000	0.000	0.000	0.000	0.000
	5	-861.531	0.000	0.000	0.000	0.000	0.000
7	2	1087.087	0.000	0.000	0.000	0.000	0.000
	6	1087.087	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	701.036	0.000	0.000	0.000	9.884
2	-624.420	701.036	0.000	0.000	0.000	9.884
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

Load case 2 (Linear): G+Qu

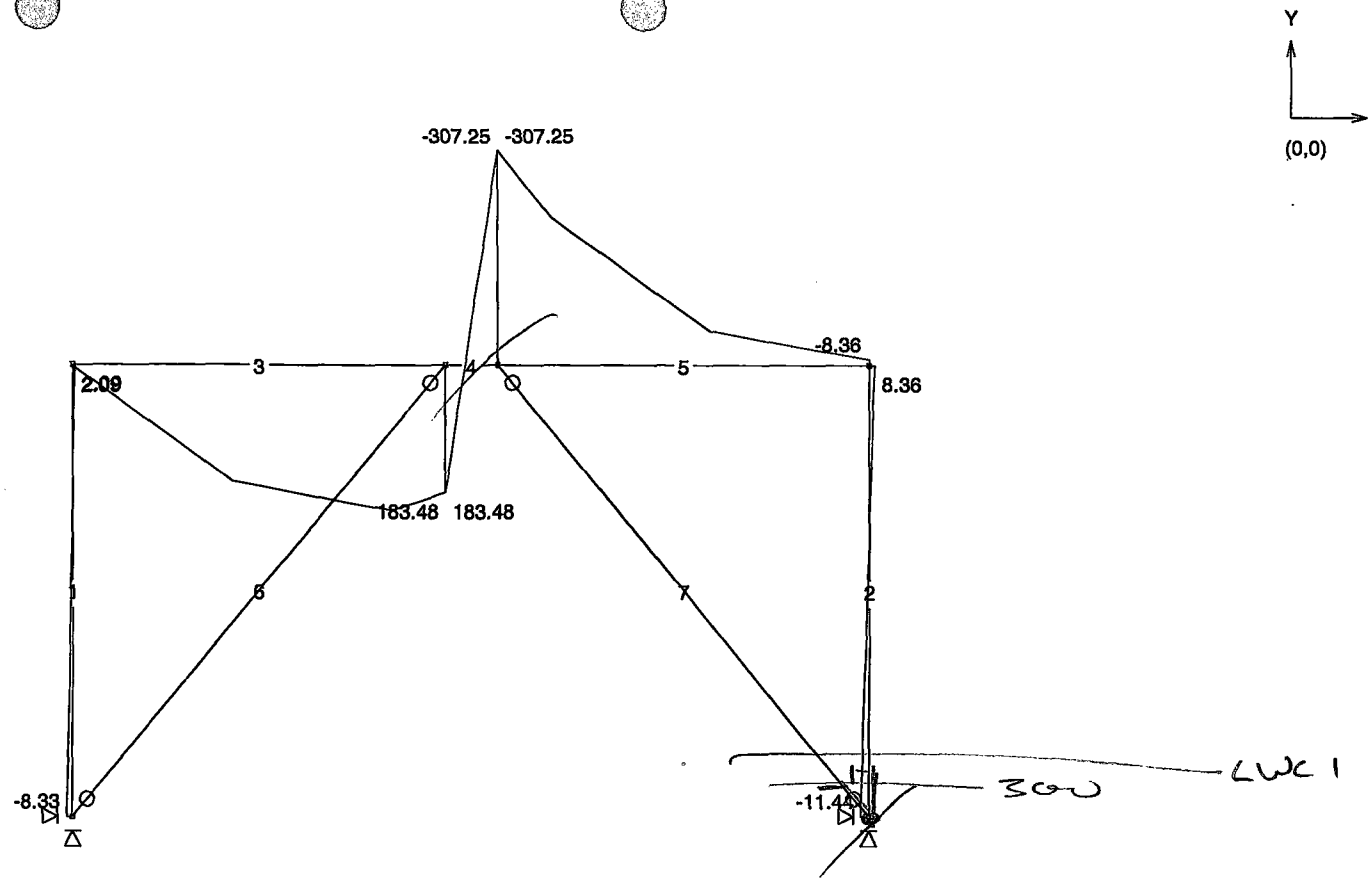
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	72.775	116.400	0.000	0.000	0.000	-1.555
2	-72.775	116.400	0.000	0.000	0.000	1.555
Load	0.000	-232.800	0.000			
Reac	0.000	232.800	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-551.645	-584.636	0.000	0.000	0.000	8.329
2	-697.195	817.436	0.000	0.000	0.000	11.439
Load	1248.840	-232.800	0.000			
Reac	-1248.840	232.800	0.000			

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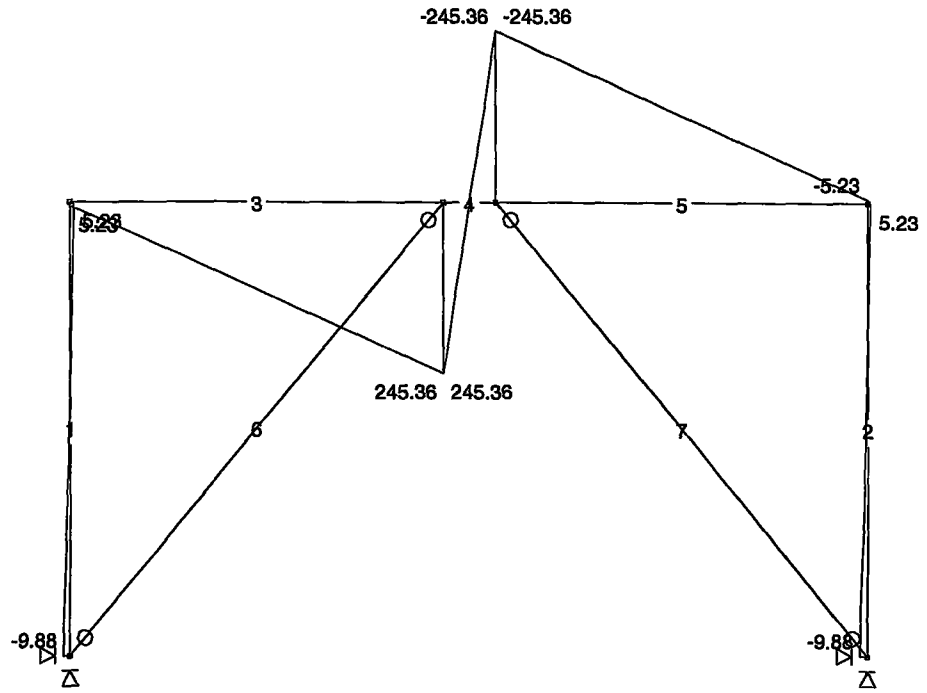
All combination load cases:
■ 3



No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 2
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

Load cases:
■ 1 E loads



No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 2
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

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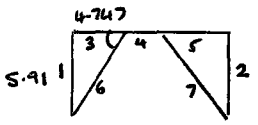
L = 10.194m

2711265/04

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NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.194	0.000	0.000
3	0.000	5.910	0.000
4	10.194	5.910	0.000
5	4.747	5.910	0.000
6	5.447	5.910	0.000



MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.747
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.747
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.580
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.580

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Column 310 UB 4	S1	Not applicable	No	Manual
2	Beam 700 WB 130	S2	Not applicable	No	Manual
3	Brace 310 UC 13	S3	Not applicable	No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.2100E+03	1.5700E+05	7.6500E+06	8.6400E+07	INFINITE	INFINITE	0.00
2	1.6600E+04	1.5100E+06	5.2100E+07	1.4000E+09	INFINITE	INFINITE	0.00
3	1.7500E+04	2.5200E+06	1.0700E+08	3.2900E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
2	3	1	G	1.800	0.000	-58.200	0.000
					0.000	0.000	0.000
	3	2	G	3.600	0.000	-58.200	0.000
					0.000	0.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	4.977	-0.295	0.000	0.000	0.000	-0.001
4	4.977	0.295	0.000	0.000	0.000	-0.001
5	4.088	-0.525	0.000	0.000	0.000	0.001
6	4.088	0.525	0.000	0.000	0.000	0.001

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.148	-0.261	0.000	0.000	0.000	0.000
4	0.140	0.055	0.000	0.000	0.000	0.000
5	0.147	-0.258	0.000	0.000	0.000	0.000
6	0.140	-0.033	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	5.125	-0.557	0.000	0.000	0.000	-0.001
4	5.117	0.351	0.000	0.000	0.000	-0.001
5	4.234	-0.783	0.000	0.000	0.000	0.002
6	4.228	0.492	0.000	0.000	0.000	0.002

Dmax

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	52.079	2.644	0.000	0.000	0.000	-10.133
	3	52.079	2.644	0.000	0.000	0.000	5.493
2	2	-52.079	2.644	0.000	0.000	0.000	-10.133
	4	-52.079	2.644	0.000	0.000	0.000	5.493
3	3	621.776	52.079	0.000	0.000	0.000	5.493
	5	621.776	52.079	0.000	0.000	0.000	252.711
4	5	0.000	-722.030	0.000	0.000	0.000	252.711
	6	0.000	-722.030	0.000	0.000	0.000	-252.711
5	6	-621.776	52.079	0.000	0.000	0.000	-252.711
	4	-621.776	52.079	0.000	0.000	0.000	-5.493
6	1	-992.900	0.000	0.000	0.000	0.000	0.000
	5	-992.900	0.000	0.000	0.000	0.000	0.000
7	2	992.900	0.000	0.000	0.000	0.000	0.000
	6	992.900	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	46.043	-0.993	0.000	0.000	0.000	1.810
	3	46.043	-0.993	0.000	0.000	0.000	-4.060
2	2	-9.753	-0.172	0.000	0.000	0.000	0.200
	4	-9.753	-0.172	0.000	0.000	0.000	-0.816
3	3	0.993	46.043	0.000	0.000	0.000	-4.060
	5	0.993	-70.357	0.000	0.000	0.000	-23.764
4	5	32.583	-31.027	0.000	0.000	0.000	-23.764
	6	32.583	-31.027	0.000	0.000	0.000	-45.482
5	6	-0.172	9.753	0.000	0.000	0.000	-45.482
	4	-0.172	9.753	0.000	0.000	0.000	0.816
6	1	50.446	0.000	0.000	0.000	0.000	0.000
	5	50.446	0.000	0.000	0.000	0.000	0.000
7	2	52.306	0.000	0.000	0.000	0.000	0.000
	6	52.306	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	98.122	1.651	0.000	0.000	0.000	-8.323
	3	98.122	1.651	0.000	0.000	0.000	1.433
2	2	-61.832	2.472	0.000	0.000	0.000	-9.933
	4	-61.832	2.472	0.000	0.000	0.000	4.677
3	3	622.769	98.122	0.000	0.000	0.000	1.433
	5	622.769	-18.278	0.000	0.000	0.000	228.947
4	5	32.583	-753.057	0.000	0.000	0.000	228.947
	6	32.583	-753.057	0.000	0.000	0.000	-298.193
5	6	-621.948	61.832	0.000	0.000	0.000	-298.193
	4	-621.948	61.832	0.000	0.000	0.000	-4.677
6	1	-942.454	0.000	0.000	0.000	0.000	0.000
	5	-942.454	0.000	0.000	0.000	0.000	0.000
7	2	1045.206	0.000	0.000	0.000	0.000	0.000
	6	1045.206	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	722.030	0.000	0.000	0.000	10.133
2	-624.420	722.030	0.000	0.000	0.000	10.133
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

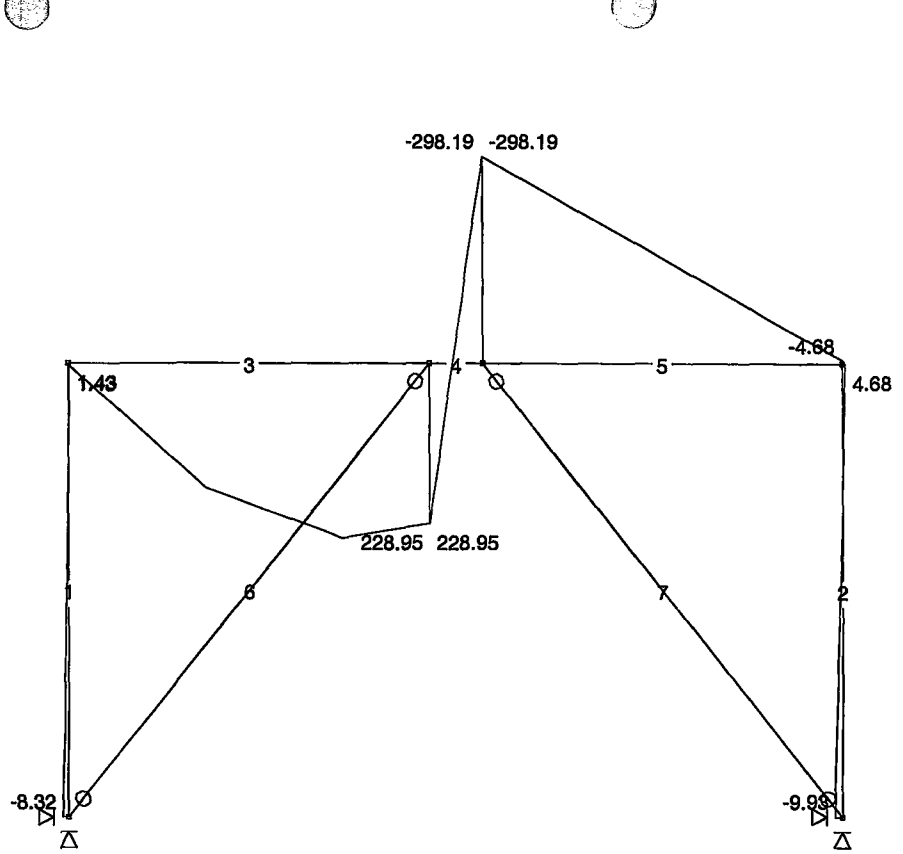
Load case 2 (Linear): G+Qu

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	32.583	85.373	0.000	0.000	0.000	-1.810
2	-32.583	31.027	0.000	0.000	0.000	-0.200
Load	0.000	-116.400	0.000			
Reac	0.000	116.400	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-591.837	-636.657	0.000	0.000	0.000	8.323
2	-657.003	753.057	0.000	0.000	0.000	9.933
Load	1248.840	-116.400	0.000			
Reac	-1248.840	116.400	0.000			

All combination load cases:
■ 3

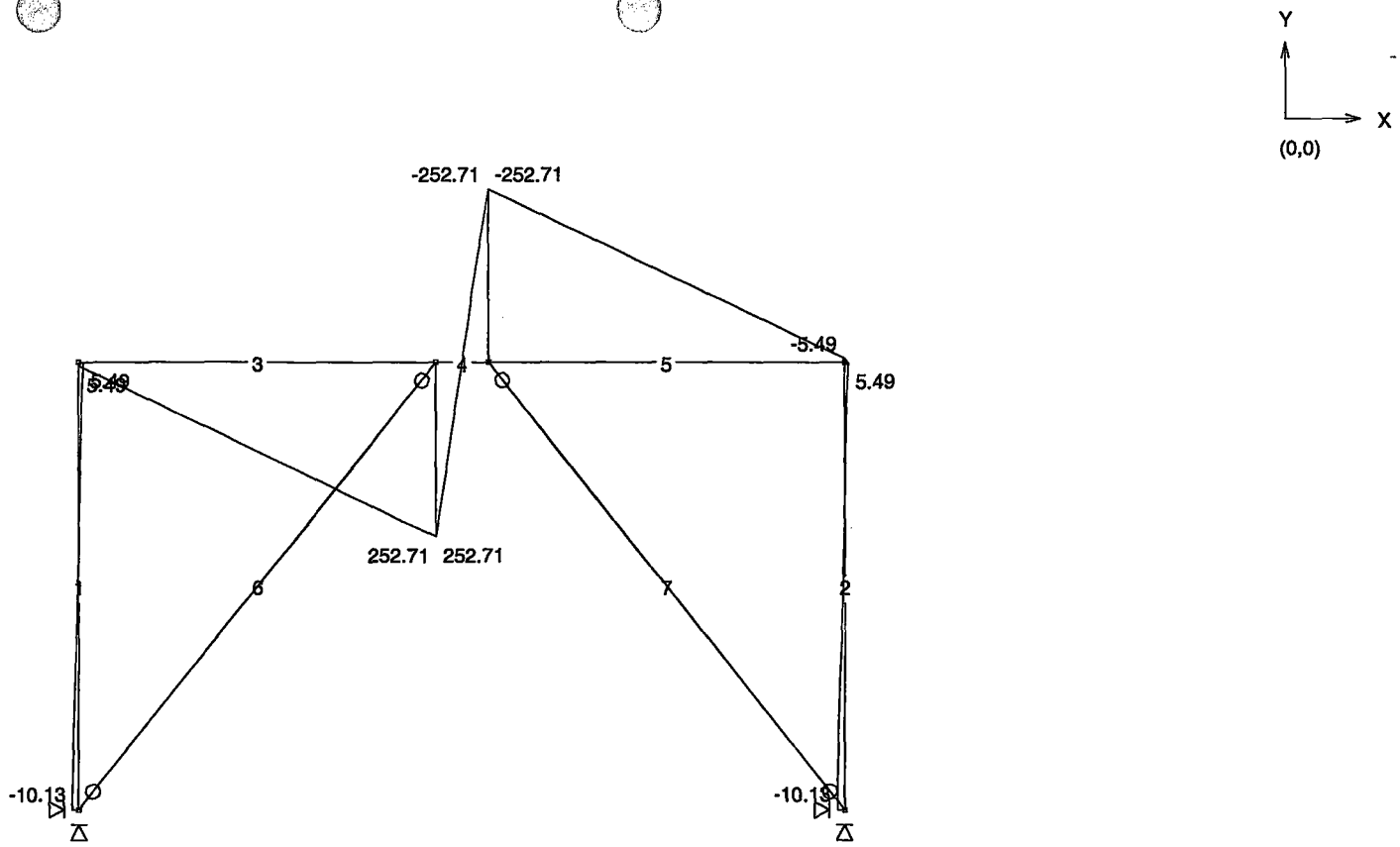


G+Qu+E
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 3
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

241059

Load cases:
■ 1 —— E loads



E only
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 3
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design Zone 2J
By: Dean Saunders

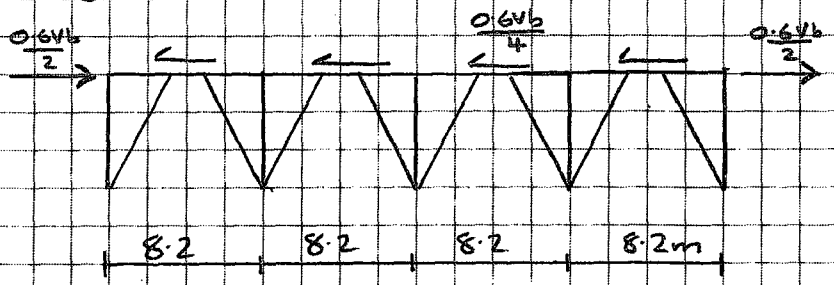
Job No: 2711265/109
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K-brace frames on (S4) & (S11)

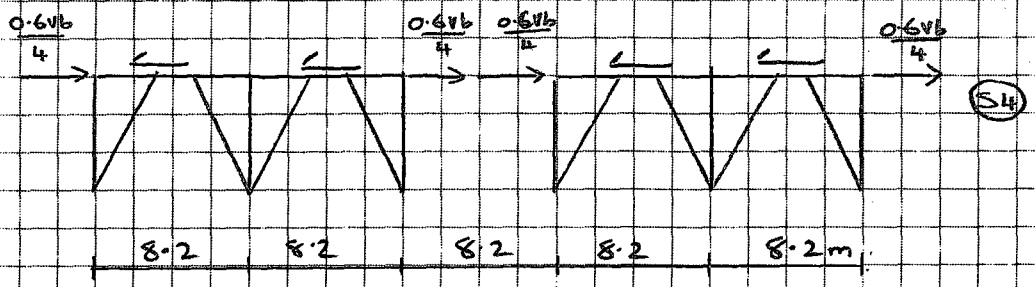
4 bays with a loading of 0.6Vb

$\mu = 6$, soft soil, $T = 0.1$ sec

$V_b = 7493$kN

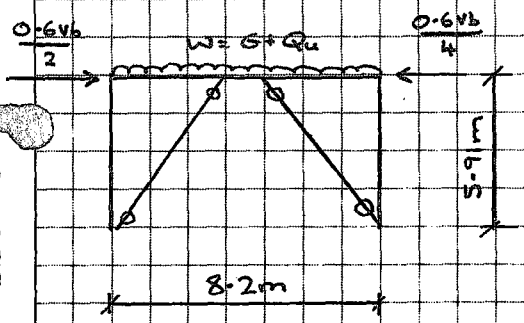


(S11)



(S4)

Grid (S11)



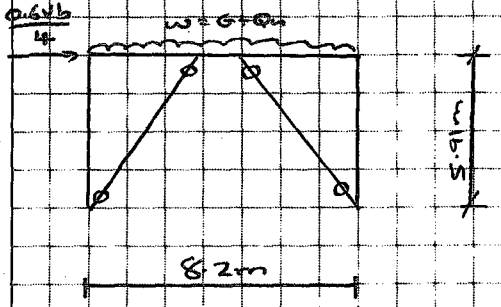
$w = G + Q_u$

$G = 1.0194 \times (0.135 \times 23.5 + 0.5)$
$= 3.74$ kN/m

$Q_u = 0.4 \times 6.5$kPa $\times 1.0194$
$= 2.65$kN/m

$w = 6.39$ kN/m

Grid (S4)



$w = G + Q_u$

$G = 2.06 \times (0.135 \times 23.5 + 0.5)$
$= 7.57$ kN/m

$Q = 0.4 \times 6.5 \times 2.06$
$= 5.36$ kN/m

$w = 12.93$ kN/m

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Calculation Sheet

Job name:
Subject:
By: DWS

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Space Gears Model

Beam	525WB Custom
Col	310UB110
Brace	310UC96.8
active link	700mm

---

max = 7.055mm = 0.13% ok ✓

max allowable = 20mm = 0.4%

$V^* = 811\,kN$

$M^* = 299\,kNm$

$N^* = \frac{0.6V_b}{2}$

$N^* = 224.8\,kN$

check Custom WB.

$\phi V_v = 769.5\,kN$

$V^* \not\leq \phi V_v$ must increase section. use 12mm thick web.

$\phi V_v = 0.9 \times 0.6 \times 300 \times 475 \times 12$
$= 923\,kN$

$V^* \leq \phi V_v$

$811\,kN \leq 923\,kN$ ✓

$\frac{475}{12} \leq 72.5$

$39.6 \leq 72.5$ ✓ must add stiffeners

Compactness.

web $\lambda_e = \frac{475}{12} \times \sqrt{\frac{300}{250}} \leq 82$

$43.4 \leq 82$ ✓ web compact

flange $5.7 \leq 8$ ✓ flange compact as previous

compact section.

Calculation Sheet

Job name:
Subject:
By: DJS

Job No: 2711265/109
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Section 12.11

$d_b \le e \le 1.6 \frac{M_s}{V_w}$.

$475 \le 700 \le \frac{1.6 \times 295 \times 25 \times 500 \times 30}{92300}$ ← flanges only.

$475 \le 700 \le 1917\text{mm}$ ✓

$V_{sol} = 1026\text{kN}$
$M_{sol} = 1106\text{kNm}$
$N_{sol} = 4425\text{kN}$.

$N^* \le \phi N_{sol}$

$2248\text{kN} \le 0.9 \times 4425\text{kN}$
$2248\text{kN} \le 3982.5\text{kN}$ ✓

combined section

$M^* \le \phi M_{sol}\left(1 - \frac{N^*}{\phi N_{sol}}\right)$

$299\text{kNm} \le 0.9 \times 1106\left(1 - \frac{2248}{3982.5}\right)$

$299\text{kNm} \le 433\text{kNm}$ ✓

use custom WB

$d = 525$
$b = 295$
$t_f = 25$
$t_w = 12$

web stiffening Requirements:

full width

$t_{stiff} \ge 0.75\, t_w$.

$\ge 0.75 \times 12$.

$\ge 9\text{mm}$ ∴ use 10mm plate.

$s \le 38\, t_{wb} - d_b/5$

$\le 38 \times 12 - 525/5$

$\le 351\text{mm}$.

241059

Calculation Sheet

Job name:

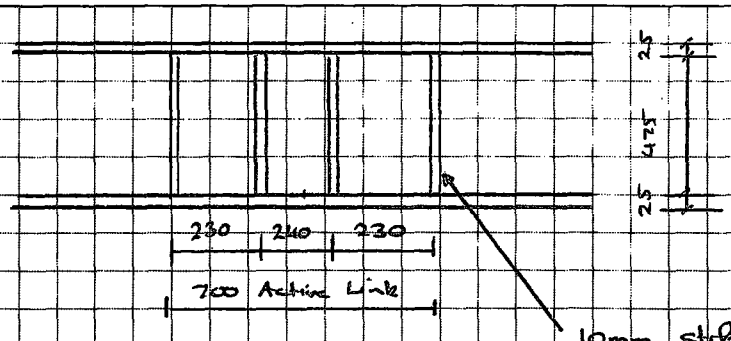
Subject:

By: DWS

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weld.

$\phi A_s f_{us} = 0.9 \times 10 \times 142.5 \times 300$

$= 385\,kN$

web $385\,kN \leq \phi v_w L$

$\leq 0.8 \times 0.6 \times 410 \times \frac{t_L}{\sqrt{2}} \times 475 \times 2$

$385\,kN \leq 132\,t_L$

$t_L \geq 2.9mm$ Provide 6mm F.W S.P full Length.

flange $\frac{385kN}{4} \leq 0.835\,kN/mm \times L \times 2$

$L \geq 58mm$ provide full length

6mm F.W S.P all around.

Lateral Restraint:

2.5% of $\phi f_y A_{fb} = 49.8\,kN$.

12.6.2.3.1

$\frac{L}{r_y} \leq 30\sqrt{\frac{250}{300}}$

$L \leq 27.4\,r_y$

$\leq 27.4 \times 74$

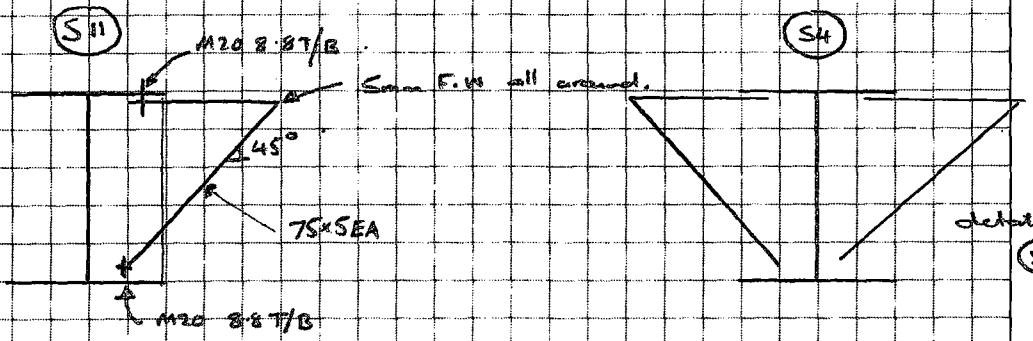
$\leq 2.0m$ no additional required.

Calculation Sheet

Job name:
Subject:
By: DWS

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Bolts & welds are per shear design force (SG).



$V^{o}_{link} = \phi_{mo} V_{ow}$

$= 1.40 \times 1026\,kN$

$= 1436.4\,kN$

$\phi^{o} = 1.40$

V^{o}_{link}
$= 1436.4\,kN$

Brace Capacity Design:

$N^{*}_{brace} = \phi^{o} N_{brace\,E} + N_{brace\,G,Qu}$

$= \pm 1.4 \times 1041.2 + 39\,kN$

$= 1497\,kN\,(c)$ or $1002\,kN\,(T)$

N^{*}_{brace}
$= 1497\,kN$ Comp.
$= 1002\,kN$ Tens.

Beam Capacity Design

$N^{*}_{beam} = 0.8 \times 1.40 \times \frac{0.6 V_b}{2}$

$= 2517\,kN$

$M^{c}_{beam} = 0.8 \times 1.4 \times 282\,kNm$
$= 316\,kNm$

$M^{*}_{beam} = 316\,kNm + 16.86\,kNm$
$= 333\,kNm$

$V^{c}_{beam} = 1.4 \times 72.8\,kN$
$= 102\,kN$

$V^{*}_{beam} = 102\,kN + 28.5\,kN$
$= 130.4\,kN$

Beam
$N^{*} = 2517\,kN$
$M^{*} = 333\,kNm$
$V^{*} = 130.4\,kN$

241059

Calculation Sheet

Job name:
Subject:
By: DWS

Job No: 2711265/109
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Date: 7-5-04

Collector beam design

$N^* = 2517\ kN$
$M^* = 333\ kNm$
$V^* = 130.4\ kN.$

$M^* \le \phi M_{sx}$
$\le \phi M_{bx}$

Compact Section

$\le \phi M_{sx}$
$\le \phi f_y Z_x$
$\le 1180\ kNm$

$\phi M_{bx} = 1799\ kNm$ $\quad \beta = 0 \quad \alpha_s = 1.75$

$333\ kNm \le 1180\ kNm$ ✓ M^* ok.

$V^* \le \phi V_w$

$130 \le 923\ kN$ ✓ V^* ok.

$N^* \le \phi N_c$
$\le \phi N_s$

$\phi N_s = 0.9 k_f A_n f_y$ web $\lambda_e = \frac{475}{12}\sqrt{\frac{300}{250}}$

$= 0.9 \times 0.946 \times 1935 \times 300$ $= 43.4$

$= 4944\ kN$ $b_e = 475 \times \frac{35}{43.4}$

$= 383\ mm.$

$k_f = \frac{A_e}{A_g} = \frac{295 \times 20 \times 2 + 383 \times 12}{20450}$

$= 0.946$

$\phi N_c = \phi \alpha_c N_s$ $\quad \lambda_n = 55.4$
$\alpha_b = 1$
$\alpha_c = 0.711$

$= 0.9 \times 0.711 \times 4944\ kN$
$= 3164\ kN$

$2517 \le 3164\ kN$ ✓ N^* ok.

Calculation Sheet

Job name:
Subject:
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Combined Action.

$$\delta_b = \frac{C_m}{1 - \left[\frac{N^*}{N_{om}}\right]} \geq 1$$

$\beta_L = -0.5$
$C_m = 0.8$

$$= \frac{0.8}{1 - \left[\frac{2517}{144,579}\right]}$$

$$N_{om} = \frac{\pi^2 EI}{(k_e L)^2}$$

$$= \frac{\pi^2 \times 200 GPa \times 1030 \times 10^6 mm^4}{(3750)^2}$$

$$= 144,579 kN$$

$$= 1$$

$M^* \leq \phi M_{cx}$.

$\leq \phi M_{sx} \left(1 - \frac{N^*}{\phi N_s}\right)$.

$\leq 0.9 \times 1180 \left(1 - \frac{2517}{4944}\right)$

$333 kNm \leq 521 kNm$ ✓

Beam o.k.

241059

Calculation Sheet

Job name:
Subject:
By: DWS

Job No: 2711265/109
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Date: 7-5-04

Brace Design

Try 310UC96.8 for consistency

N^*_{brace} = 1497kN Comp
= 1002kN Tens.

$N^* \le \phi N_t$

$\le \phi A_g f_y$ & $\phi 0.85 k_t A_n f_u$

$\le 0.9 \times 12400 \times 300$ & $0.9 \times 0.85 \times 1 \times 10,428 \times 440$

1002kN $\le$ 3348kN & 3510kN.

$N^* \le \phi N_s$
$\le \phi N_c$

$L_e = k_e L$ $k_e = 1.0$
$L = 7.0m$
$= 7.0m$

from design aid, need 310UC96.8 as min section.

$\phi N_s = 3340kN$
$\phi N_c = 1810kN$ $L_e = 7.0m$

$N^* \le \phi N_c$

1497 $\le$ 1810kN ✓

Brace
310UC96.8

Brace Splice

friction bolts $\mu = 0.35$

$V^* = 1497$ less than for (5G) Brace use same splice detail.

Brace Splice as per frame on (5G).

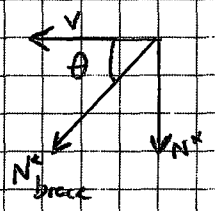
Calculation Sheet

Job name:
Subject:
By: DWS

Job No: 2711265/109
Page No: 41 of
Date: 7-5-04.

Brace to Beam Connection:

N^*_{brace} = 1497 kN.



$\theta \approx 57.6°$ V^* = 802 kN N^* = 1264 kN

fillet + butt weld
butt welded only.

Butt weld

ϕN_w = 1848 kN.

$N^* \leq \phi N_w$

1264 ≤ 1848 ✓

Remaining force = 802 + 1264 − 1848
= 218 kN.

Provide 8mm fillet weld SP

ϕv_w = 1.11 kN/mm.

$L = \frac{277}{\sin\theta}$.

= 328 mm. use 310mm.

ϕV_w = 1.11 × 2 × 310.
= 688 kN

218 kN ≤ 688 kN ✓

Beam to Column Connect.

as per frame (5A) & (5F)

Brace to beam as per frame of (5F)

241059

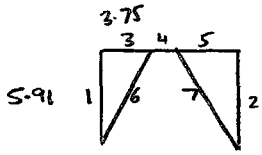
L = 8·2m

Pages 42-44

SPACE GASS 9.03 (Win) — BECA CARTER HOLLINGS & FERNER LTD
Job: P:\27\27112265\SGass\Queensgate Zone ZJ K-brace frame 4
Designer: Date: 7 May 2004, 9:23 am

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	8.200	0.000	0.000
3	0.000	5.910	0.000
4	8.200	5.910	0.000
5	3.750	5.910	0.000
6	4.450	5.910	0.000



MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	3.750
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	3.750
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	6.999
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	6.999

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle	Type	Flipped	Source
1	Column 310 UB 4	S1	Not applicable		No	Manual
2	Custom 525WB	S2	Not applicable		No	Manual
3	310 UC 96.8	S3	Not applicable		No	Aust300

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.2100E+03	1.5700E+05	7.6500E+06	8.6400E+07	INFINITE	INFINITE	0.00
2	1.9500E+04	3.1220E+06	1.0701E+08	1.0120E+09	INFINITE	INFINITE	0.00
3	1.2400E+04	9.2800E+05	7.2900E+07	2.2300E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	561.980	0.000	0.000	0.000	0.000	0.000
	4	561.980	0.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	3	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000
	4	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000
	5	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	7.053	-0.413	0.000	0.000	0.000	-0.001
4	7.053	0.413	0.000	0.000	0.000	-0.001
5	6.516	-0.655	0.000	0.000	0.000	0.002
6	6.516	0.655	0.000	0.000	0.000	0.002

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.002	-0.114	0.000	0.000	0.000	0.000
4	-0.002	-0.114	0.000	0.000	0.000	0.000
5	0.002	-0.132	0.000	0.000	0.000	0.000
6	-0.002	-0.132	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	7.055	-0.526	0.000	0.000	0.000	-0.001
4	7.051	0.299	0.000	0.000	0.000	-0.001
5	6.518	-0.786	0.000	0.000	0.000	0.002
6	6.515	0.523	0.000	0.000	0.000	0.002

Δmax.

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	72.778	4.136	0.000	0.000	0.000	-15.126
	3	72.778	4.136	0.000	0.000	0.000	9.316
2	2	-72.778	4.136	0.000	0.000	0.000	-15.126
	4	-72.778	4.136	0.000	0.000	0.000	9.316
3	3	557.844	72.778	0.000	0.000	0.000	9.316
	5	557.844	72.778	0.000	0.000	0.000	282.234
4	5	0.000	-806.384	0.000	0.000	0.000	282.234
	6	0.000	-806.384	0.000	0.000	0.000	-282.234
5	6	-557.844	72.778	0.000	0.000	0.000	-282.234
	4	-557.844	72.778	0.000	0.000	0.000	-9.316
6	1	-1041.210	0.000	0.000	0.000	0.000	0.000
	5	-1041.210	0.000	0.000	0.000	0.000	0.000
7	2	1041.210	0.000	0.000	0.000	0.000	0.000
	6	1041.210	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	20.017	-0.256	0.000	0.000	0.000	0.502
	3	20.017	-0.256	0.000	0.000	0.000	-1.010
2	2	20.017	0.256	0.000	0.000	0.000	-0.502
	4	20.017	0.256	0.000	0.000	0.000	1.010
3	3	0.256	20.017	0.000	0.000	0.000	-1.010
	5	0.256	-28.470	0.000	0.000	0.000	-16.860
4	5	21.192	4.525	0.000	0.000	0.000	-16.860
	6	21.192	-4.525	0.000	0.000	0.000	-16.860
5	6	0.256	28.470	0.000	0.000	0.000	-16.860
	4	0.256	-20.017	0.000	0.000	0.000	-1.010
6	1	39.078	0.000	0.000	0.000	0.000	0.000
	5	39.078	0.000	0.000	0.000	0.000	0.000
7	2	39.078	0.000	0.000	0.000	0.000	0.000
	6	39.078	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	92.795	3.880	0.000	0.000	0.000	-14.624
	3	92.795	3.880	0.000	0.000	0.000	8.305
2	2	-52.761	4.392	0.000	0.000	0.000	-15.628
	4	-52.761	4.392	0.000	0.000	0.000	10.326
3	3	558.100	92.795	0.000	0.000	0.000	8.305
	5	558.100	44.308	0.000	0.000	0.000	265.374
4	5	21.192	-801.859	0.000	0.000	0.000	265.374
	6	21.192	-810.910	0.000	0.000	0.000	-299.095
5	6	-557.588	101.249	0.000	0.000	0.000	-299.095
	4	-557.588	52.761	0.000	0.000	0.000	-10.326
6	1	-1002.132	0.000	0.000	0.000	0.000	0.000
	5	-1002.132	0.000	0.000	0.000	0.000	0.000
7	2	1080.287	0.000	0.000	0.000	0.000	0.000
	6	1080.287	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

(0·6Vb / x) × 1·6

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-561.980	806.384	0.000	0.000	0.000	15.126
2	-561.980	806.384	0.000	0.000	0.000	15.126
Load	1123.960	0.000	0.000			
Reac	-1123.960	0.000	0.000			

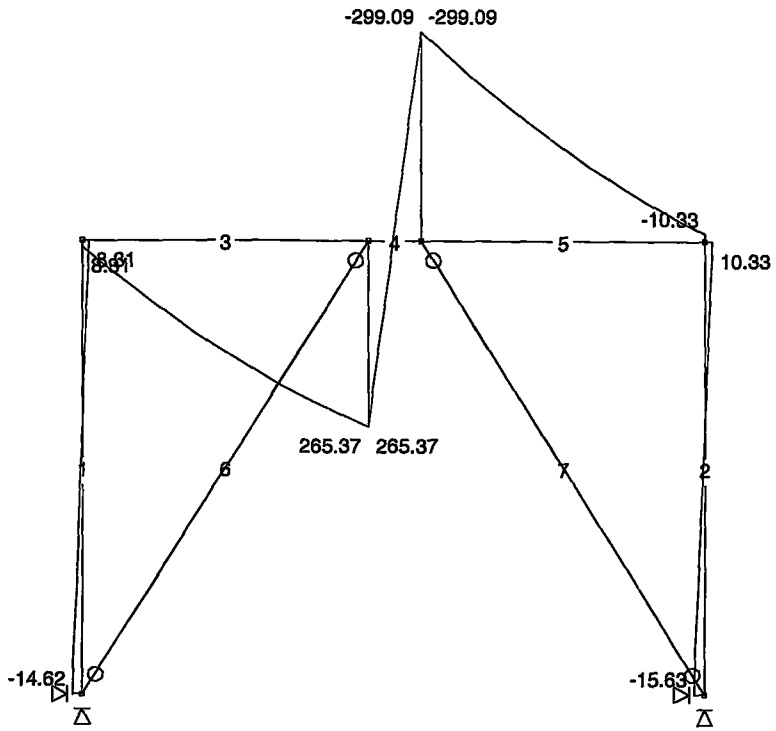
Load case 2 (Linear): G+Qu

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	21.192	53.013	0.000	0.000	0.000	-0.502
2	-21.192	53.013	0.000	0.000	0.000	0.502
Load	0.000	-106.026	0.000			
Reac	0.000	106.026	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-540.788	-753.371	0.000	0.000	0.000	14.624
2	-583.172	859.397	0.000	0.000	0.000	15.628
Load	1123.960	-106.026	0.000			
Reac	-1123.960	106.026	0.000			

Load cases:
3

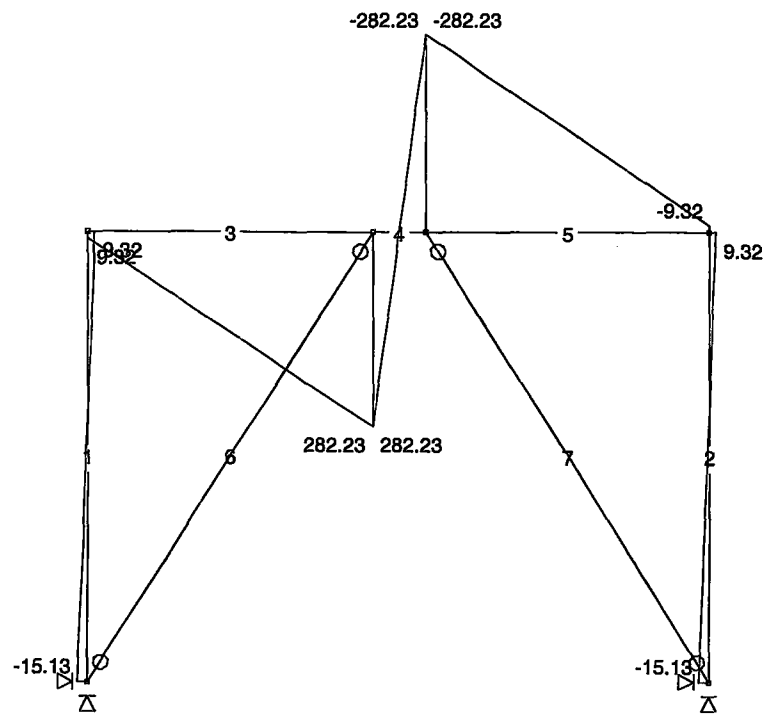


E+G+Qu
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 4
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

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Load cases:
■ 1 —— E loads



E
No general restraint

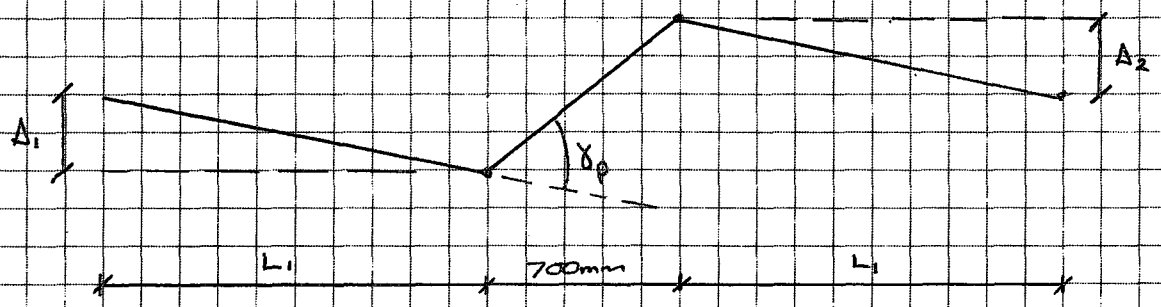
Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 4
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design.
By: Dean Saunders.

Job No: 2711265/109
Page No: 45 of
Date: 11-5-04

12.11.8.3 Checks



$$\gamma_p = \text{ATAN}\left(\frac{\Delta_1}{L_1}\right) + \text{ATAN}\left(\frac{\Delta_1 + \Delta_2}{700}\right)$$

Frame Case	Δ_1	Δ_2	L_1	γ_p (radians)
1	0.734	0.438	4900.	0.0018
2	0.91	0.218	4900	0.0018.
3	0.793	0.497	4747	0.0020
4	0.516	0.368	3750	0.0014.

γ_p ok final. ✓

$\gamma_p \leq \pm 0.09$ radians for $\epsilon \leq 16 M_{sp}/N_w$.

Deflections:

Elastic Space Gaus Analysis of individual frames.

In line of (E4) & (E11) $\Delta_{max} = 7.0$mm.

In line of (E4) & (E6) $\Delta_{max} = 6.7$mm.

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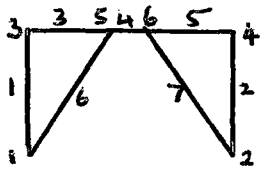
L=10.5m UDL Final. 6-46

2711265/109

Pages 46-47

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.500	0.000	0.000
3	0.000	5.910	0.000
4	10.500	5.910	0.000
5	4.900	5.910	0.000
6	5.600	5.910	0.000



MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.900
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.900
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.677
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.677

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle	Type	Flipped	Source
1	Column 250 UC 8 S1		Not applicable		No	Manual
2	Beam 525WB Cust S2		Not applicable		No	Manual
3	Brace 310 UC 96 S3		Not applicable		No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	1.1400E+04	1.0400E+06	4.8400E+07	1.4300E+08	INFINITE	INFINITE	0.00
2	1.9500E+04	3.1220E+06	1.0701E+08	1.0120E+09	INFINITE	INFINITE	0.00
3	1.2400E+04	9.2800E+05	7.2900E+07	2.2300E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	3	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000
	4	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000
	5	1	GI	0.000%	100.000%	0.000	-13.000	0.000
						0.000	-13.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.344	-0.125	0.000	0.000	0.000	-0.001
4	6.344	0.125	0.000	0.000	0.000	-0.001
5	5.566	-0.711	0.000	0.000	0.000	0.002
6	5.566	0.711	0.000	0.000	0.000	0.002

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.004	-0.070	0.000	0.000	0.000	0.000
4	-0.004	-0.070	0.000	0.000	0.000	0.000
5	0.003	-0.218	0.000	0.000	0.000	0.000
6	-0.003	-0.218	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.349	-0.195	0.000	0.000	0.000	-0.001
4	6.340	0.055	0.000	0.000	0.000	-0.001
5	5.569	-0.929	0.000	0.000	0.000	0.002
6	5.563	0.493	0.000	0.000	0.000	0.002

Δmax.

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	48.219	4.776	0.000	0.000	0.000	-19.798
	3	48.219	4.776	0.000	0.000	0.000	8.427
2	2	-48.219	4.776	0.000	0.000	0.000	-19.798
	4	-48.219	4.776	0.000	0.000	0.000	8.427
3	3	619.644	48.219	0.000	0.000	0.000	8.427
	5	619.644	48.219	0.000	0.000	0.000	244.702
4	5	0.000	-699.147	0.000	0.000	0.000	244.702
	6	0.000	-699.147	0.000	0.000	0.000	-244.702
5	6	-619.644	48.219	0.000	0.000	0.000	-244.702
	4	-619.644	48.219	0.000	0.000	0.000	-8.427
6	1	-970.833	0.000	0.000	0.000	0.000	0.000
	5	-970.833	0.000	0.000	0.000	0.000	0.000
7	2	970.833	0.000	0.000	0.000	0.000	0.000
	6	970.833	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	27.025	-0.987	0.000	0.000	0.000	1.938
	3	27.025	-0.987	0.000	0.000	0.000	-3.898
2	2	27.025	0.987	0.000	0.000	0.000	-1.938
	4	27.025	0.987	0.000	0.000	0.000	3.898
3	3	0.987	27.025	0.000	0.000	0.000	-3.898
	5	0.987	-36.675	0.000	0.000	0.000	-27.539
4	5	35.167	4.550	0.000	0.000	0.000	-27.539
	6	35.167	-4.550	0.000	0.000	0.000	-27.539
5	6	0.987	36.675	0.000	0.000	0.000	-27.539
	4	0.987	-27.025	0.000	0.000	0.000	-3.898
6	1	53.551	0.000	0.000	0.000	0.000	0.000
	5	53.551	0.000	0.000	0.000	0.000	0.000
7	2	53.551	0.000	0.000	0.000	0.000	0.000
	6	53.551	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	75.244	3.788	0.000	0.000	0.000	-17.860
	3	75.244	3.788	0.000	0.000	0.000	4.529
2	2	-21.194	5.763	0.000	0.000	0.000	-21.736
	4	-21.194	5.763	0.000	0.000	0.000	12.324
3	3	620.632	75.244	0.000	0.000	0.000	4.529
	5	620.632	11.544	0.000	0.000	0.000	217.162
4	5	35.167	-694.597	0.000	0.000	0.000	217.162
	6	35.167	-703.697	0.000	0.000	0.000	-272.241
5	6	-618.657	84.894	0.000	0.000	0.000	-272.241
	4	-618.657	21.194	0.000	0.000	0.000	-12.324
6	1	-917.281	0.000	0.000	0.000	0.000	0.000
	5	-917.281	0.000	0.000	0.000	0.000	0.000
7	2	1024.384	0.000	0.000	0.000	0.000	0.000
	6	1024.384	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	-699.147	0.000	0.000	0.000	19.798
2	-624.420	699.147	0.000	0.000	0.000	19.798
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

Load case 2 (Linear): G+Qu

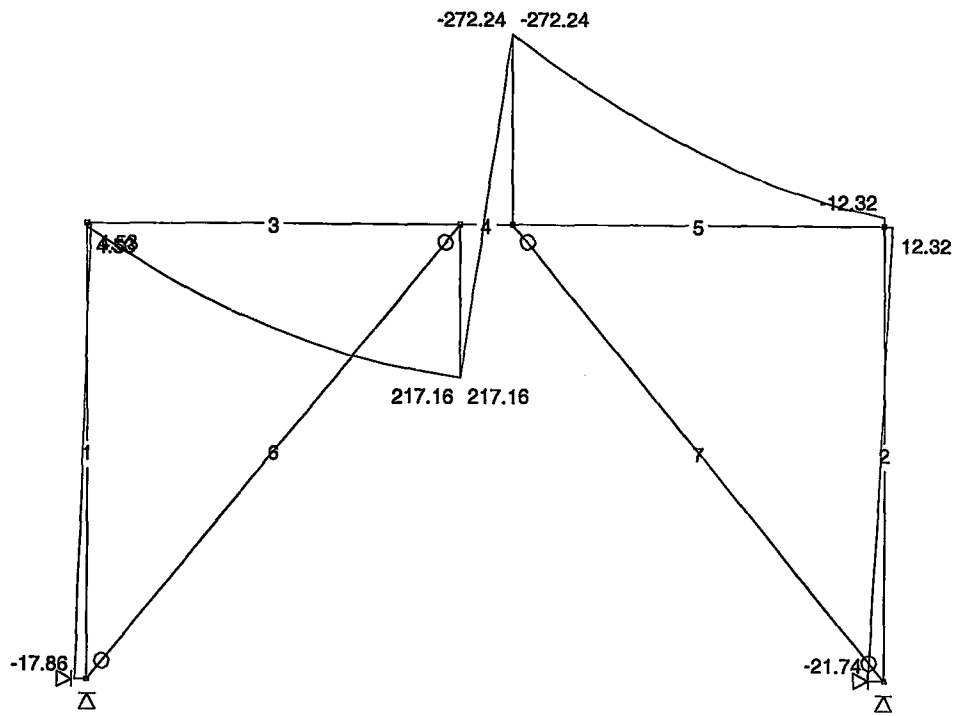
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	35.167	68.250	0.000	0.000	0.000	-1.938
2	-35.167	68.250	0.000	0.000	0.000	1.938
Load	0.000	-136.500	0.000			
Reac	0.000	136.500	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-589.253	-630.897	0.000	0.000	0.000	17.860
2	-659.587	767.397	0.000	0.000	0.000	21.736
Load	1248.840	-136.500	0.000			
Reac	-1248.840	136.500	0.000			

Loads ok.
to Preliminary.

All combination load cases:
3



Frame on SA G+Q+E
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 1
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

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L=10.5m Point loads Filed. 6-4

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Drawn.

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.500	0.000	0.000
3	0.000	5.910	0.000
4	10.500	5.910	0.000
5	4.900	5.910	0.000
6	5.600	5.910	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.900
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	4.900
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.900
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.677
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.677

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle	Type	Flipped	Source
1	Column 250 UC 8 S1		Not applicable		No	Manual
2	Beam 525WB Cust S2		Not applicable		No	Manual
3	Brace 310 UC 96 S3		Not applicable		No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	1.1400E+04	1.0400E+06	4.8400E+07	1.4300E+08	INFINITE	INFINITE	0.00
2	1.9500E+04	3.1220E+06	1.0701E+08	1.0120E+09	INFINITE	INFINITE	0.00
3	1.2400E+04	9.2800E+05	7.2900E+07	2.2300E+08	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
2	3	1	G	2.100	0.000	-58.200	0.000
					0.000	0.000	0.000
	3	2	G	4.200	0.000	-58.200	0.000
					0.000	0.000	0.000
	5	1	G	0.700	0.000	-58.200	0.000
					0.000	0.000	0.000
	5	2	G	2.800	0.000	-58.200	0.000
					0.000	0.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.344	-0.125	0.000	0.000	0.000	-0.001
4	6.344	0.125	0.000	0.000	0.000	-0.001
5	5.566	-0.711	0.000	0.000	0.000	0.002
6	5.566	0.711	0.000	0.000	0.000	0.002

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.009	-0.082	0.000	0.000	0.000	0.000
4	-0.009	-0.082	0.000	0.000	0.000	0.000
5	0.006	-0.449	0.000	0.000	0.000	0.000
6	-0.006	-0.449	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.353	-0.207	0.000	0.000	0.000	-0.002
4	6.335	0.043	0.000	0.000	0.000	-0.001
5	5.572	-1.160	0.000	0.000	0.000	0.002
6	5.559	0.261	0.000	0.000	0.000	0.002

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	48.219	4.776	0.000	0.000	0.000	-19.798
	3	48.219	4.776	0.000	0.000	0.000	8.427
2	2	-48.219	4.776	0.000	0.000	0.000	-19.798
	4	-48.219	4.776	0.000	0.000	0.000	8.427
3	3	619.644	48.219	0.000	0.000	0.000	8.427
	5	619.644	48.219	0.000	0.000	0.000	244.702
4	5	0.000	-699.147	0.000	0.000	0.000	244.702
	6	0.000	-699.147	0.000	0.000	0.000	-244.702
5	6	-619.644	48.219	0.000	0.000	0.000	-244.702
	4	-619.644	48.219	0.000	0.000	0.000	-8.427
6	1	-970.833	0.000	0.000	0.000	0.000	0.000
	5	-970.833	0.000	0.000	0.000	0.000	0.000
7	2	970.833	0.000	0.000	0.000	0.000	0.000
	6	970.833	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	31.453	-1.893	0.000	0.000	0.000	3.715
	3	31.453	-1.893	0.000	0.000	0.000	-7.474
2	2	31.453	1.893	0.000	0.000	0.000	-3.715
	4	31.453	1.893	0.000	0.000	0.000	7.474
3	3	1.893	31.453	0.000	0.000	0.000	-7.474
	5	1.893	-84.947	0.000	0.000	0.000	-57.053
4	5	72.323	0.000	0.000	0.000	0.000	-57.053
	6	72.323	0.000	0.000	0.000	0.000	-57.053
5	6	1.893	84.947	0.000	0.000	0.000	-57.053
	4	1.893	-31.453	0.000	0.000	0.000	-7.474
6	1	110.346	0.000	0.000	0.000	0.000	0.000
	5	110.346	0.000	0.000	0.000	0.000	0.000
7	2	110.346	0.000	0.000	0.000	0.000	0.000
	6	110.346	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	79.673	2.883	0.000	0.000	0.000	-16.083
	3	79.673	2.883	0.000	0.000	0.000	0.953
2	2	-16.766	6.669	0.000	0.000	0.000	-23.513
	4	-16.766	6.669	0.000	0.000	0.000	15.901
3	3	621.537	79.673	0.000	0.000	0.000	0.953
	5	621.537	-36.727	0.000	0.000	0.000	187.648
4	5	72.323	-699.147	0.000	0.000	0.000	187.648
	6	72.323	-699.147	0.000	0.000	0.000	-301.755
5	6	-617.751	133.166	0.000	0.000	0.000	-301.755
	4	-617.751	16.766	0.000	0.000	0.000	-15.901
6	1	-860.486	0.000	0.000	0.000	0.000	0.000
	5	-860.486	0.000	0.000	0.000	0.000	0.000
7	2	1081.179	0.000	0.000	0.000	0.000	0.000
	6	1081.179	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	-699.147	0.000	0.000	0.000	19.798
2	-624.420	699.147	0.000	0.000	0.000	19.798
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

Load case 2 (Linear): G+Qu

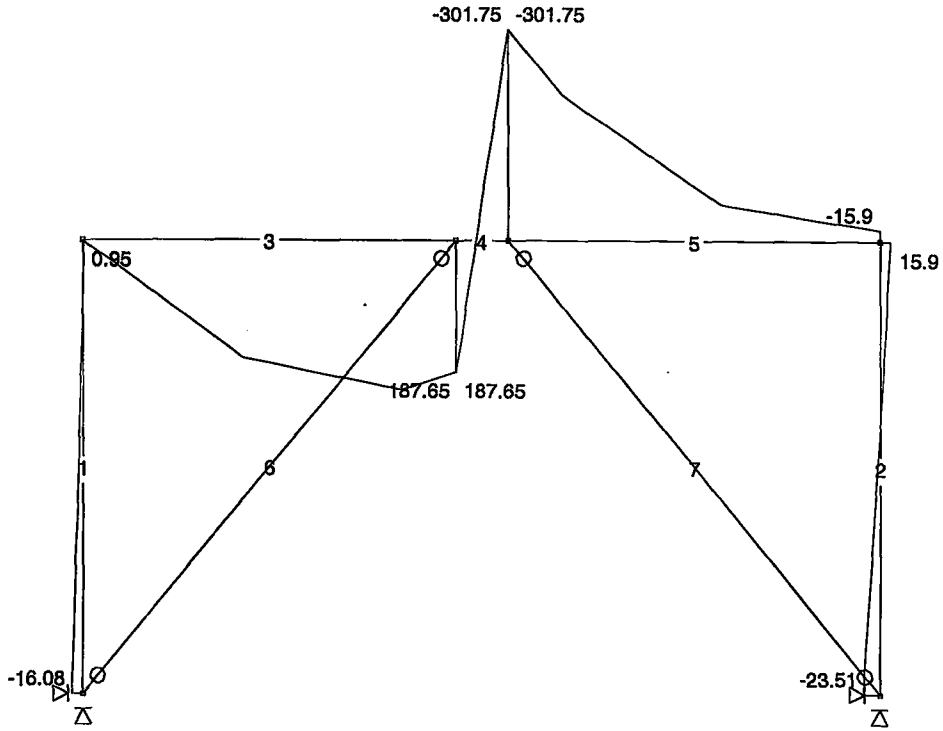
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	72.323	116.400	0.000	0.000	0.000	-3.715
2	-72.323	116.400	0.000	0.000	0.000	3.715
Load	0.000	-232.800	0.000			
Reac	0.000	232.800	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-552.097	-582.747	0.000	0.000	0.000	16.083
2	-696.743	815.547	0.000	0.000	0.000	23.513
Load	1248.840	-232.800	0.000			
Reac	-1248.840	232.800	0.000			

Loads ok to Preliminary

All combination load cases:
■ 3



Frame on SG G+Q+E
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 2
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

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6-12

L = 10.194m final. 2711265 Pages 50-5

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.194	0.000	0.000
3	0.000	5.910	0.000
4	10.194	5.910	0.000
5	4.747	5.910	0.000
6	5.447	5.910	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	4.747
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	4.747
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	7.580
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	7.580

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Column 250 UC 8	S1	Not applicable	No	Manual
2	Beam 525WB Cust	S2	Not applicable	No	Standard shape
3	310 UC 96.8	S3	Not applicable	No	Aust300

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	1.1400E+04	1.0400E+06	4.8400E+07	1.4300E+08	INFINITE	INFINITE	0.00
2	1.9500E+04	3.1220E+06	1.0701E+08	1.0120E+09	INFINITE	INFINITE	0.00
3	1.2400E+04	9.2800E+05	7.2900E+07	2.2300E+08	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
2	I-beam	525.000	295.000	295.000	10.000	25.000

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	624.420	0.000	0.000	0.000	0.000	0.000
	4	624.420	0.000	0.000	0.000	0.000	0.000

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
2	3	1	G	1.800	0.000	-58.200	0.000
					0.000	0.000	0.000
	3	2	G	3.600	0.000	-58.200	0.000
					0.000	0.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.466	-0.133	0.000	0.000	0.000	-0.001
4	6.466	0.133	0.000	0.000	0.000	-0.001
5	5.712	-0.710	0.000	0.000	0.000	0.002
6	5.712	0.710	0.000	0.000	0.000	0.002

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.193	-0.124	0.000	0.000	0.000	-0.001
4	0.185	0.024	0.000	0.000	0.000	0.000
5	0.190	-0.340	0.000	0.000	0.000	0.000
6	0.184	-0.056	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.659	-0.257	0.000	0.000	0.000	-0.002
4	6.651	0.157	0.000	0.000	0.000	-0.001
5	5.902	-1.050	0.000	0.000	0.000	0.002
6	5.896	0.654	0.000	0.000	0.000	0.002

Dunca

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	51.195	4.968	0.000	0.000	0.000	-20.377
	3	51.195	4.968	0.000	0.000	0.000	8.986
2	2	-51.195	4.968	0.000	0.000	0.000	-20.377
	4	-51.195	4.968	0.000	0.000	0.000	8.986
3	3	619.452	51.195	0.000	0.000	0.000	8.986
	5	619.452	51.195	0.000	0.000	0.000	252.007
4	5	0.000	-720.021	0.000	0.000	0.000	252.007
	6	0.000	-720.021	0.000	0.000	0.000	-252.007
5	6	-619.452	51.195	0.000	0.000	0.000	-252.007
	4	-619.452	51.195	0.000	0.000	0.000	-8.986
6	1	-989.188	0.000	0.000	0.000	0.000	0.000
	5	-989.188	0.000	0.000	0.000	0.000	0.000
7	2	989.188	0.000	0.000	0.000	0.000	0.000
	6	989.188	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	47.855	-2.366	0.000	0.000	0.000	4.345
	3	47.855	-2.366	0.000	0.000	0.000	-9.637
2	2	-9.349	-0.362	0.000	0.000	0.000	0.411
	4	-9.349	-0.362	0.000	0.000	0.000	-1.729
3	3	2.366	47.855	0.000	0.000	0.000	-9.637
	5	2.366	-68.545	0.000	0.000	0.000	-20.742
4	5	32.285	-31.296	0.000	0.000	0.000	-20.742
	6	32.285	-31.296	0.000	0.000	0.000	-42.650
5	6	-0.362	9.349	0.000	0.000	0.000	-42.650
	4	-0.362	9.349	0.000	0.000	0.000	1.729
6	1	47.777	0.000	0.000	0.000	0.000	0.000
	5	47.777	0.000	0.000	0.000	0.000	0.000
7	2	52.133	0.000	0.000	0.000	0.000	0.000
	6	52.133	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	99.049	2.603	0.000	0.000	0.000	-16.032
	3	99.049	2.603	0.000	0.000	0.000	-0.651
2	2	-60.544	4.606	0.000	0.000	0.000	-19.966
	4	-60.544	4.606	0.000	0.000	0.000	7.257
3	3	621.817	99.049	0.000	0.000	0.000	-0.651
	5	621.817	-17.351	0.000	0.000	0.000	231.265
4	5	32.285	-751.317	0.000	0.000	0.000	231.265
	6	32.285	-751.317	0.000	0.000	0.000	-294.657
5	6	-619.814	60.544	0.000	0.000	0.000	-294.657
	4	-619.814	60.544	0.000	0.000	0.000	-7.257
6	1	-941.411	0.000	0.000	0.000	0.000	0.000
	5	-941.411	0.000	0.000	0.000	0.000	0.000
7	2	1041.321	0.000	0.000	0.000	0.000	0.000
	6	1041.321	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-624.420	720.021	0.000	0.000	0.000	20.377
2	-624.420	720.021	0.000	0.000	0.000	20.377
Load	1248.840	0.000	0.000			
Reac	-1248.840	0.000	0.000			

Load case 2 (Linear): G+Qu

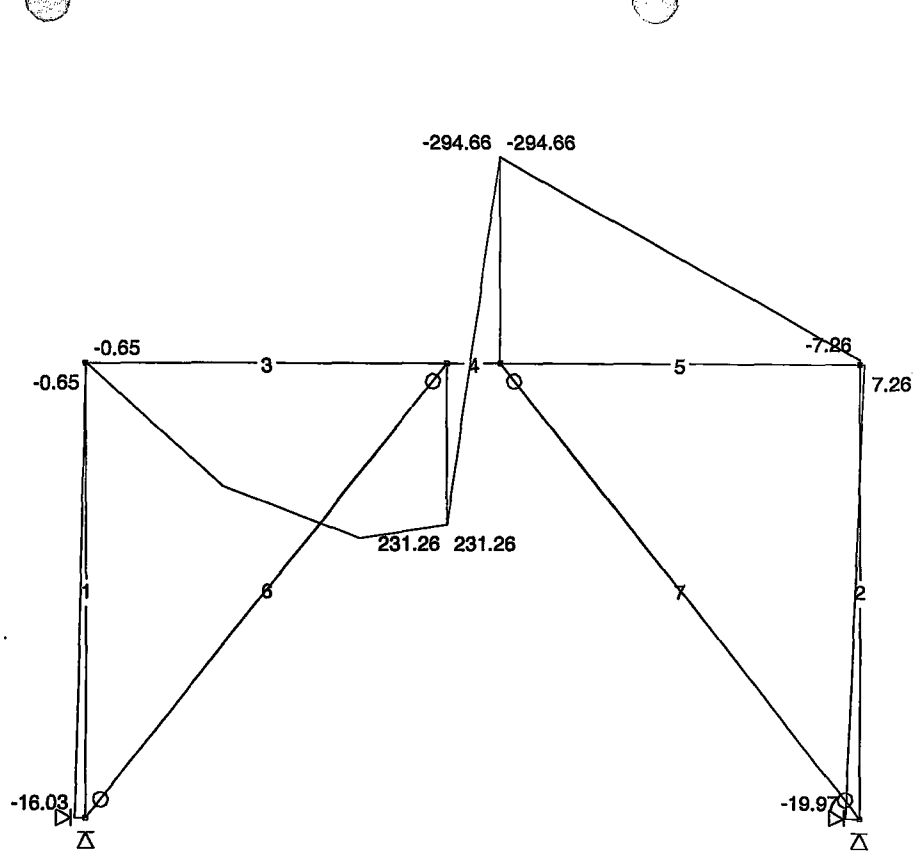
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	32.285	85.104	0.000	0.000	0.000	-4.345
2	-32.285	31.296	0.000	0.000	0.000	-0.411
Load	0.000	-116.400	0.000			
Reac	0.000	116.400	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-592.135	-634.917	0.000	0.000	0.000	16.032
2	-656.705	751.317	0.000	0.000	0.000	19.966
Load	1248.840	-116.400	0.000			
Reac	-1248.840	116.400	0.000			

Loads ok to Preliminary.

All combination load cases:
■ 3



Frame on SG G+Q+E
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 3
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

241059

L = 8.2m final.

2711265/1
Pages 52-

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	8.200	0.000	0.000
3	0.000	5.910	0.000
4	8.200	5.910	0.000
5	3.750	5.910	0.000
6	4.450	5.910	0.000

Δmax.

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	5.910
2	0.00			Norm	2	4	1	1	FFFFFF	FFFFFF	5.910
3	0.00			Norm	3	5	2	1	FFFFFF	FFFFFF	3.750
4	0.00			Norm	5	6	2	1	FFFFFF	FFFFFF	0.700
5	0.00			Norm	6	4	2	1	FFFFFF	FFFFFF	3.750
6	0.00			Norm	1	5	3	1	FFFFFR	FFFFFR	6.999
7	0.00			Norm	2	6	3	1	FFFFFR	FFFFFR	6.999

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDF						
2	FFDDDF						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Column 250 UC 8	S1	Not applicable	No	Manual
2	Custom 525WB	S2	Not applicable	No	Standard shape
3	310 UC 96.8	S3	Not applicable	No	Aust300

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	1.1400E+04	1.0400E+06	4.8400E+07	1.4300E+08	INFINITE	INFINITE	0.00
2	2.0450E+04	3.2528E+06	1.0704E+08	1.0298E+09	INFINITE	INFINITE	0.00
3	1.2400E+04	9.2800E+05	7.2900E+07	2.2300E+08	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
2	I-beam	525.000	295.000	295.000	12.000	25.000

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	561.980	0.000	0.000	0.000	0.000	0.000
	4	561.980	0.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	3	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000
	4	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000
	5	1	GI	0.000%	100.000%	0.000	-12.930	0.000
						0.000	-12.930	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1: E loads
1.000 * Load case 2: G+Qu

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Linear): E loads

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.960	-0.185	0.000	0.000	0.000	-0.001
4	6.960	0.185	0.000	0.000	0.000	-0.001
5	6.451	-0.627	0.000	0.000	0.000	0.002
6	6.451	0.627	0.000	0.000	0.000	0.002

Load case 2 (Linear): G+Qu

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	0.002	-0.054	0.000	0.000	0.000	0.000
4	-0.002	-0.054	0.000	0.000	0.000	0.000
5	0.002	-0.128	0.000	0.000	0.000	0.000
6	-0.002	-0.128	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000
3	6.963	-0.239	0.000	0.000	0.000	-0.001
4	6.958	0.131	0.000	0.000	0.000	-0.001
5	6.453	-0.755	0.000	0.000	0.000	0.002
6	6.449	0.499	0.000	0.000	0.000	0.002

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): E loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	71.391	6.386	0.000	0.000	0.000	-23.980
	3	71.391	6.386	0.000	0.000	0.000	13.764
2	2	-71.391	6.386	0.000	0.000	0.000	-23.980
	4	-71.391	6.386	0.000	0.000	0.000	13.764
3	3	555.594	71.391 ✓	0.000	0.000	0.000	13.764
	5	555.594	71.391 ✓	0.000	0.000	0.000	281.479 ✓
4	5	0.000	-804.225	0.000	0.000	0.000	281.479
	6	0.000	-804.225 ✓	0.000	0.000	0.000	-281.479
5	6	-555.594	71.391 ✓	0.000	0.000	0.000	-281.479
	4	-555.594	71.391	0.000	0.000	0.000	-13.764
6	1	-1037.009 ✓	0.000	0.000	0.000	0.000	0.000
	5	-1037.009	0.000	0.000	0.000	0.000	0.000
7	2	1037.009	0.000	0.000	0.000	0.000	0.000
	6	1037.009	0.000	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	20.883	-0.496	0.000	0.000	0.000	0.973
	3	20.883	-0.496	0.000	0.000	0.000	-1.957
2	2	20.883	0.496	0.000	0.000	0.000	-0.973
	4	20.883	0.496	0.000	0.000	0.000	1.957
3	3	0.496	20.883	0.000	0.000	0.000	-1.957
	5	0.496	-27.605	0.000	0.000	0.000	-14.561
4	5	20.883	4.525	0.000	0.000	0.000	-14.561 ✓
	6	20.883	-4.525 ✓	0.000	0.000	0.000	-14.561 ✓
5	6	0.496	27.605 ✓	0.000	0.000	0.000	-14.561
	4	0.496	20.883	0.000	0.000	0.000	-1.957
6	1	38.053 ✓	0.000	0.000	0.000	0.000	0.000
	5	38.053	0.000	0.000	0.000	0.000	0.000
7	2	38.053	0.000	0.000	0.000	0.000	0.000
	6	38.053	0.000	0.000	0.000	0.000	0.000

Load case 3 (Linear):

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	92.273	5.891	0.000	0.000	0.000	-23.007
	3	92.273	5.891	0.000	0.000	0.000	11.807
2	2	-50.508	6.882	0.000	0.000	0.000	-24.953
	4	-50.508	6.882	0.000	0.000	0.000	15.720 ✓
3	3	556.089	92.273	0.000	0.000	0.000	11.807
	5	556.089	43.786	0.000	0.000	0.000	266.918
4	5	20.883	-799.699 ✓	0.000	0.000	0.000	266.918
	6	20.883	-808.750	0.000	0.000	0.000	-296.039
5	6	-555.098	98.995	0.000	0.000	0.000	-296.039 ✓
	4	-555.098	50.508	0.000	0.000	0.000	-15.720
6	1	-998.956	0.000	0.000	0.000	0.000	0.000
	5	-998.956	0.000	0.000	0.000	0.000	0.000
7	2	1075.061	0.000	0.000	0.000	0.000	0.000
	6	1075.061	0.000	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): E loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-561.980	-804.225 ✓	0.000	0.000	0.000	23.980
2	-561.980 ✓	804.225 ✓	0.000	0.000	0.000	23.980
Load	1123.960	0.000	0.000			
Reac	-1123.960	0.000	0.000			

Load case 2 (Linear): G+Qu

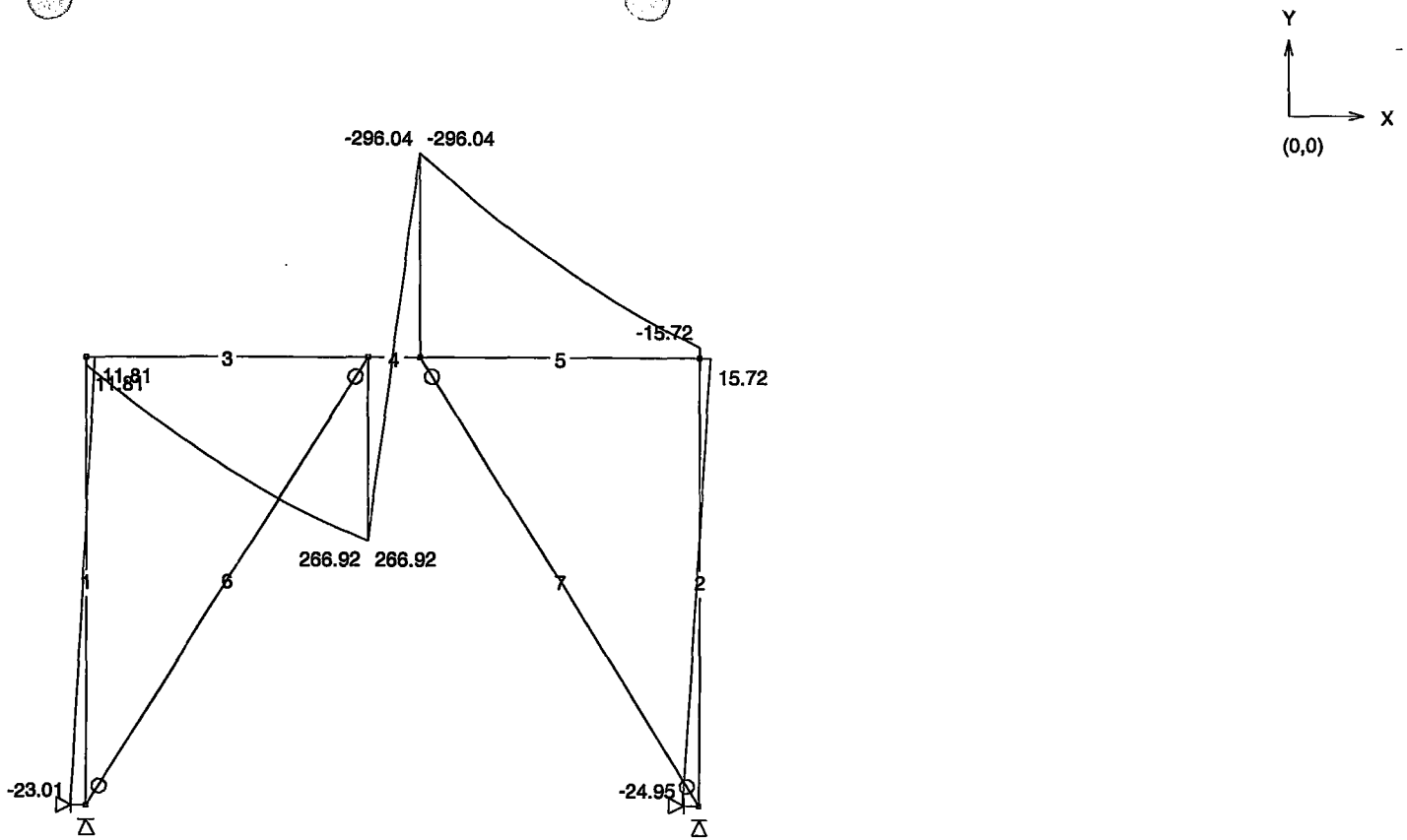
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	20.883	53.013	0.000	0.000	0.000	-0.973
2	-20.883	53.013	0.000	0.000	0.000	0.973
Load	0.000	-106.026	0.000			
Reac	0.000	106.026	0.000			

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-541.097	-751.212	0.000	0.000	0.000	23.007
2	-582.863	857.238	0.000	0.000	0.000	24.953
Load	1123.960	-106.026	0.000			
Reac	-1123.960	106.026	0.000			

Loads ok.
to Preliminary.

All combination load cases:
3



Frame on S4 & S11 G+Q+E
No general restraint

Job: P:\27\2711265\SGass\Queensgate Zone ZJ K-brace frame 4
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 11, Shear: None, Axial: None, Torsion: None

241059

6-45

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: K-brace frame design
By: Dean Saunders.

Job No: 2711265/109
Page No: 54 of
Date: 11-5-04.

Column Design:

Load Cases:

a) $1.2G + 1.6Q$

b) $G + Q_u + E_u$

(S4) $V_c^* (eq) = 1.4 \times 72.8 kN = 102 kN.$ + 1.4 x 260kN for roof bracing

(SA) $V_c^* (eq) = 1.4 \times 52 kN = 72.9 kN.$ + 1.4 x 260kN for roof bracing

Loads from Jamil's load takeoffs 11-5-04

a) $1.2 \times 555kN + 1.6 \times 546 = 1540 kN$

b) (S4) $452 + 0.4 \times 556 + 102 = 776 kN$ + 1.4 x 260 = 1140 kN

(SA) $555 + 0.4 \times 546 + 72.9 = 846.3 kN$ + 1.4 x 260 = 1210 kN. O.K ✓

Design for $1.2G + 1.6Q = 1540 kN.$

Design.

$N^* \leq \phi N_s$.

$N^* < \phi N_c$

$Le = ke\, L$ $\quad ke = 0.85$, $L = 5.91m.$

$Le = 5.02m.$

From design Tables 250UC89.5

Columns 250UC89.5

6-55

NOTES

1. ALL LEVELS TO BE CONFIRMED ON SITE PRIOR TO CONSTRUCTION.
2. REFER S141 FOR PILE CAP TYPES AND PILE CAP DETAILS.
   i.e. PC1A ETC.
3. GB1 - GROUND BEAMS 800x400
   5-DH32 TOP & BOTTOM
   4 LEG R12-200 STIRRUPS
   GB2 - 500x400, 170kg/m³
4. REFER S452 FOR EBF 'K' BRACE AND BASEPLATE DETAILS.
5. SEE TABLE BELOW FOR COLUMN SIZES:

COLUMN	SIZE
C1A	250x6 SHS CONCRETE FILLED
C2A	250x9 SHS CONCRETE FILLED
C3A	310UC137
C4A	325x9.5 CHS
C5A	410UB54 PORTAL LEG

- ALL COLUMNS FROM FOUNDATIONS TO UNDERSIDE LEVEL 2 TO BE COATED WITH INTUMECENT PAINT TO ACHIEVE 60 MINUTE FRR
   a) COLUMNS TO BE FIRE RATED TO 60 MINUTES FRR WITH INTUMECENT PAINT.
   b) COAT UNDERGROUND PORTIONS OF ALL COLUMNS WITH 3 COATS MULSEAL INCLUDING BASEPLATES PROTECT AGAINST DAMAGE DURING BACKFILLING
6. TOP OF ALL PILE CAPS TO BE RL 4.700 i.e 300 BELOW TOP OF LEVEL 1 SLAB UNO
7. ALLOW TO ALTER 5 NO. PILE CAPS DUE TO POSSIBLE CLASH WITH EXISTING FOUNDATIONS
8. ALL PILES TO BE DESIGNED WITH A MINIMUM CONFINEMENT STEEL TO PROVIDE DUCTILITY = 6
   EACH CONCRETE PILE TO BE DESIGNED TO RESIST A LATERAL SEISMIC SHEAR LOAD OF 60kN
   PILE LONGITUDINAL REINFORCEMENT TO BE DEVELOPED IN THE PILE CAP.

D=370 L=452. D=555 L=546 D=452 L=556. D=426 L=495 D=342 L=442 D=382 L=484 D=380 L=442 D=376 L=334 D=350 L=397 D=312 L=319 D=231 L=97.6 D=217 L=110. T D=248 L=284

LVL1 Column loads. (unfactored)

250UC89-5

WORKING COPY

NOT FOR CONSTRUCTION

PRELIMINARY

WATERLOO ROAD / QUEENS DRIVE / BLOOMFIELD TERRACE / KNIGHTS ROAD / KNIGHTS STREET / ZJ

24105

NOTE
ALL DIMENSIONS TO BE CHECKED ON SITE.
WRITTEN DIMENSIONS ONLY TO BE USED.
REFER TO ALL DETAIL DWGS, STRUCTURAL MECHANICAL & SERVICE DWGS BEFORE COMMENCING WORK.
REFER ANY DISCREPANCIES TO ARCHITECTS.

24105?

Rev.	Date	By	Description
1	29.3.04	HJB	BILL OF QUANTITIES

Westfield (New Zealand) Limited
L3 cnr REMUERA RD & NUFFIELD ST, New Market, AUCKLAND, New Zealand
Telephone : (09) 978 5050
Fax Num. : (09) 978 5070
Project Architect Mark OakleyChecked

Beca Carter Hollings & Ferner Ltd
Tel 04-473 7551 Fax 04-472 7911
Drawn by P DAVIES

Other Consultants
Architects
Tolrad Design Group Limited
Tel 04-385 8187 Fax 04-385 1834
Services Engineer
Montgomery Watson NZ Ltd
Tel 04-381 6700 Fax 04-381 6739

Project QUEENSGATE REDEVELOPMENT
Title ZONE J FOUNDATIONS PLAN

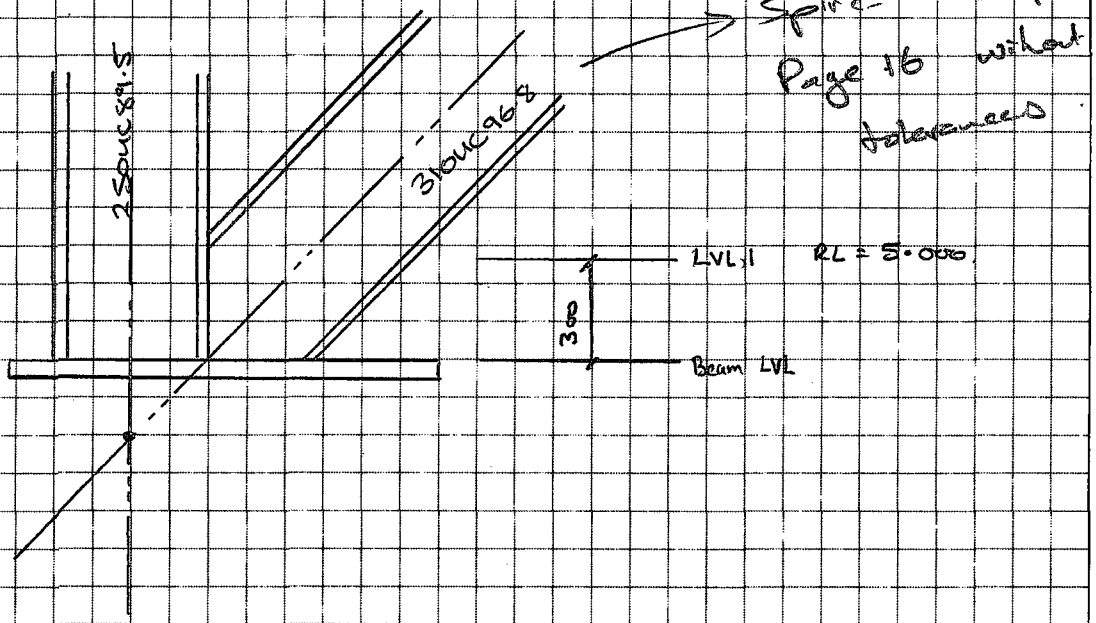
Scale	Project/Job Number	Computer Ref.	Drawing Number
1:200	2711265	S100-ZJ	S100-ZJ
Library Ref. M:/QG/CD	Sheet 35R	Date Plotted 29/03/04	Revision 1

Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: K-brace frame design
By: Dean Saunders

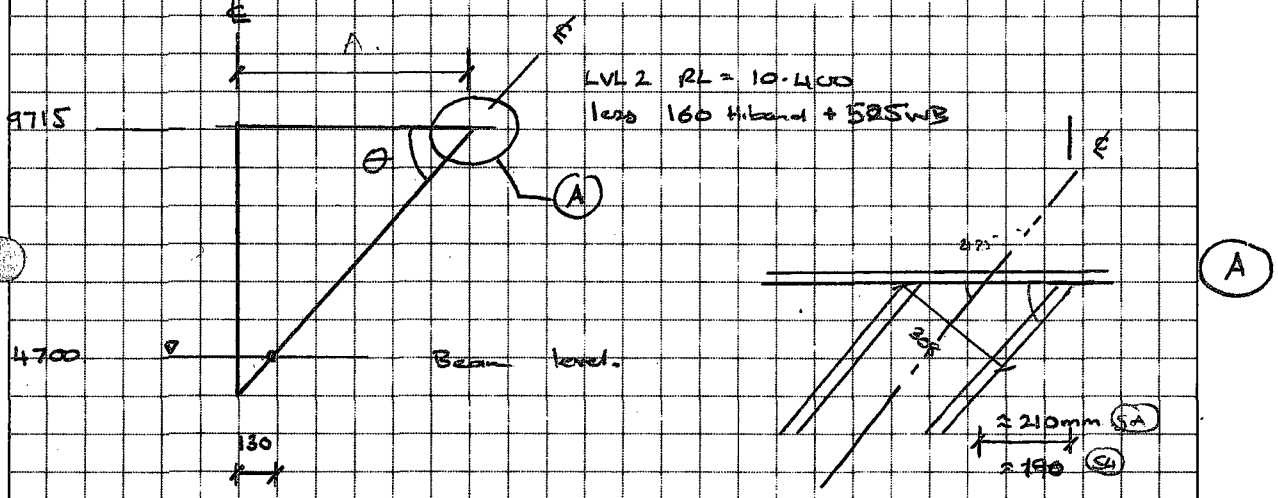
Job No: 2711265/109
Page No: 56 of
Date: 11-5-04

Column & Brace Base Connection :



Spirac - as per Page 16 without exceptions

Dimensions :



(SA) & (SG) $\theta = \text{INVTAN}\left(\frac{9715-4700}{4900-130-210}\right)$

L = 10.5 = 47.7° L = 10.194 θ = 48.7°

(SG) $\theta = \text{INVTAN}\left(\frac{9715-4700}{3790-130-190}\right)$

L = 8.4 = 55.6°

241059

Calculation Sheet

Job name:

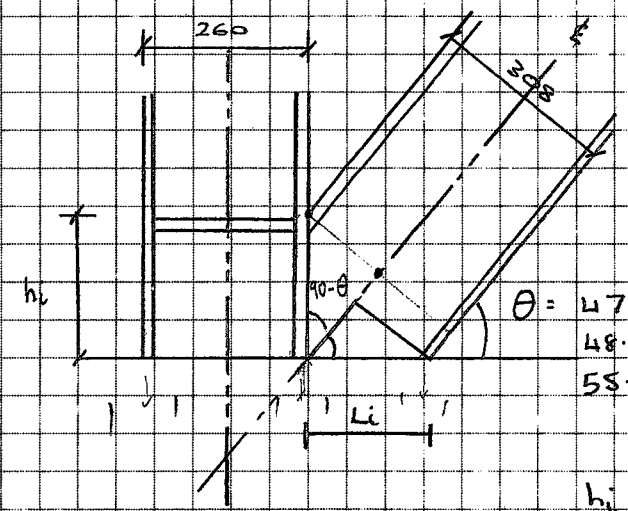
Subject:

By: Dean Saunders.

Job No: 2711265/109

Page No: 57 of

Date: 11-5-04



θ = 47.7° L = 10.5

48.7° L = 10.194

55.6° L = 8.4

$L_i = 154/\sin\theta$

$h_i = 154/\sin(90-\theta)$

= 208mm L = 10.5m = 229mm L = 10.5m

= 205mm L = 10.194m = 233mm L = 10.194m

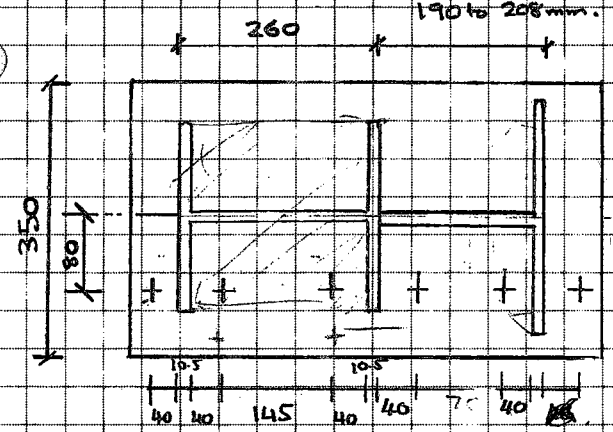
= 187mm L = 8.4m. = 273mm L = 8.4m.

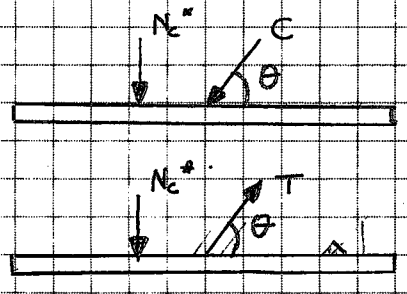
NOTE IN ALL CASES WEB STIFFENER TO COLUMN (250UC89.5) will be below floor LVL.

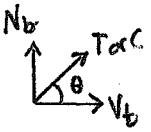
Provide 12mm plate to match off 310UC96.8 flange. angle off.

12mm plate 8mm FW Both sides.

Design Plate.







$N_c^* = G + Q_u + E_u$

= 846.3kN

Span	θ	C	N_{bc}^*	V_{bc}^*	T	N_{tc}^*	M_{tc}^*
1	47.7°	1503	1112	1012	1277	945	859
2	48.7°	1503	1129	992	1277	959	843
3	55.6°	1497	1235	846	1002	827	566

Conservatively design for N_{bc}^* = 1235kN & V_{bc}^* = 1012kN

N_{tc}^* = 959kN & V_{tc}^* = 859kN

Calculation Sheet

Job name:

Subject:

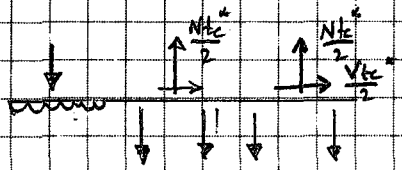
By: Dean Saunders.

Job No: 2711265/109

Page No: 58 of

Date: 11-5-04

Idealise loading as three point loads resisted by bearing bolts



4 bolts act to each point load.

$\frac{N_{tc}^*}{2} = 480kN$

$\frac{V_{tc}^*}{2} = 430kN$

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1$$

Try ϕ24mm 8.8 rod.

$$\left(\frac{430}{4\times133}\right)^2 + \left(\frac{480}{4\times234}\right)^2 \leq 1$$

$0.652 + 0.263 \leq 1$

$0.914 \leq 1$ ✓

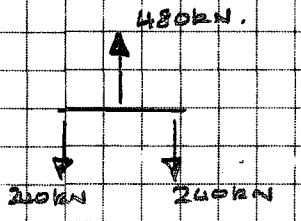
Check shear under compression load.

$\frac{V_{bc}^*}{2} = 506kN.$

$V_f^* \leq \phi V_f.$

$506kN \leq 4\times133kN$

$\leq 532kN$ ✓



$V_p^* = 240kN$

$M_p^* = 240\times0.05$

$= 12kNm.$



Work cont.

241059

Calculation Sheet

Job name:

Subject:

By: Dean Saunders.

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Date: 11-5-04

$V_f^* \leq \phi V_w$

$240kN \leq 0.9 \times 0.6 \times 300 \times t_p \times 256$

$t_p \geq \frac{240,000}{41,472}$.

≥ 5.8 kN no allowance made for axial load.

Prying action recommendation $1.25 \times d_f$.
$= 30$mm plate.

$M_f^* \leq \phi 2 f_y$.

$M_f^* \leq 0.9 \times \frac{bd^2}{6} \times f_y$.

≥ 20 mm ✓

$12 kNm \leq 0.9 \times \frac{(350 - 2 \times 26) \times 30^2}{6} \times 280$.

$12 kNm \leq 11.26$ kNm ✗

try $t_p = 40$mm

$12 kNm \leq 0.9 \times \frac{(350 - 52) \times 40^2}{6} \times 280$.

$12 kNm \leq 20 kNm$ ✓

use 40 mm thick plate.

Check bearing of bolts.

$V^* \leq \phi V_b$.

$\leq 0.9 \times 3.2 \times 40 \times 24 \times 440$.
$\leq 1216 kN$/bolt

$V^* \leq \phi V_p$.

$\leq 0.9 \times 40 \times 40 \times 440$

$\frac{430kN}{4} \leq 633 kN$ ✓

Calculation Sheet

Job name:

Subject:

By: Dean Saunders.

Job No: 2711265/109

Page No: 60 of 61

Date: 11-5-04

check bearing stress on Concrete beam

load $N_{bc}^{*} = \frac{1235}{2}$ kN.

$= 618$ kN

Area $= (256+80) \times (173+80)$

$= 32,693\ mm^2$.

Stress $= \frac{618,000\ N}{32,693\ mm^2}$

$= 18.9$ MPa. ✓ $\leq f'_c \times 0.8$ —

241059

Calculation Sheet

Job name:

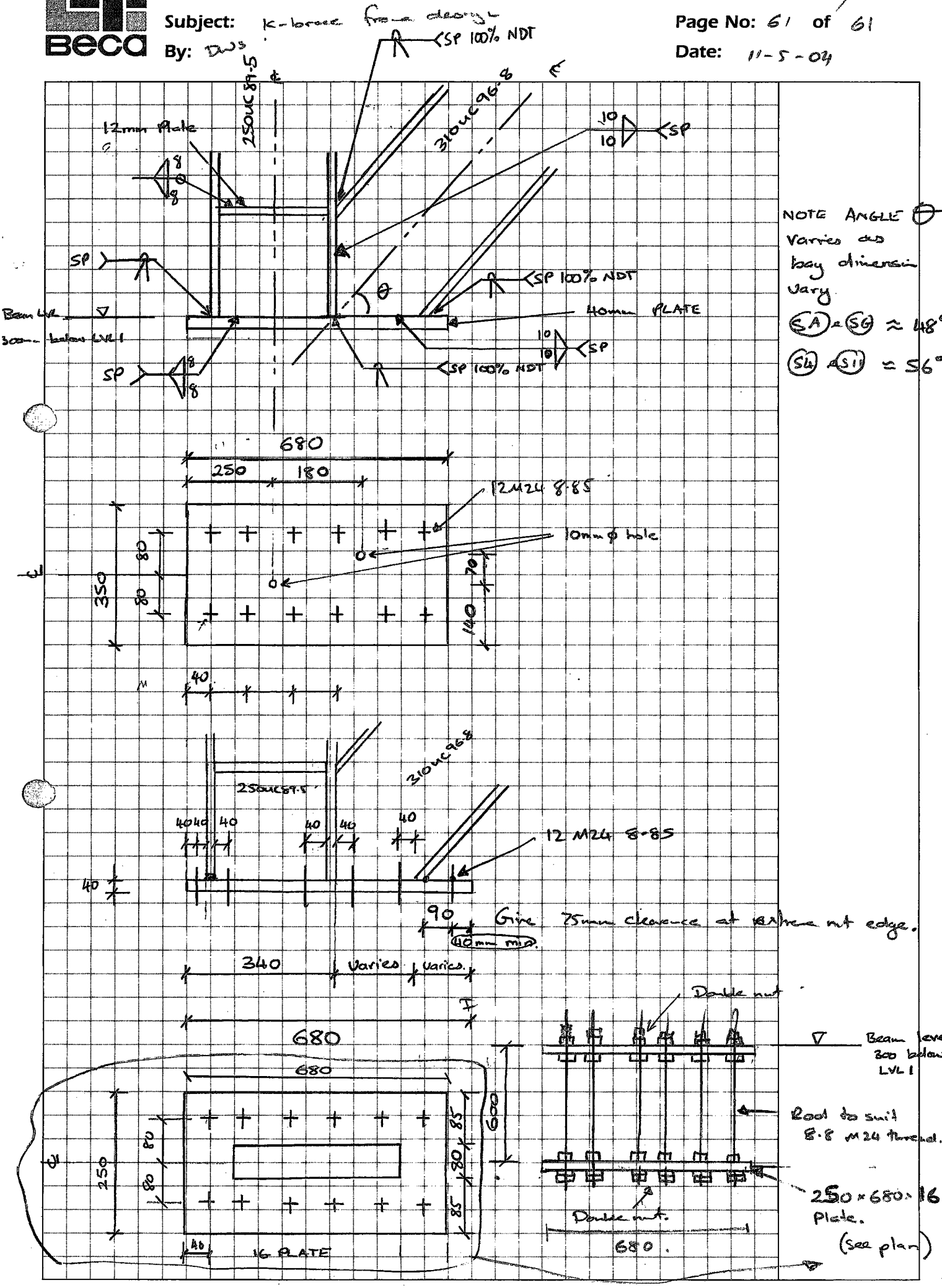
Subject: K-bracee from design

By: DWS

Job No: 2711265/109

Page No: 61 of 61

Date: 11-5-04



Column Load Taking

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1	12.00	6.00	1.00	12.00	6.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	12.00	6.00	12.00	6.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	130.00	154.00	1.00	130.00	154.00		
No of Similar floor including this			1	Total Load	137.80	154.00	149.80	160.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------	6.00	46.08			
	Beam GB2 500 X 400	4.80		5.50	26.40			
	Pile Cap 700 X 2100	35.28		1.00	35.28			
	Cladding	0.15		62.10	9.32			
No of Similar floor including this			1	Total Load	117.08	0.00	266.87	160.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	266.87	160.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	266.87	160.00

Total Short Term Live Load 112.00 Total Long Term Live Load 61.60

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1	24.00	16.00	1.00	24.00	16.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.00	16.00	24.00	16.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	486.00	520.00	1.00	486.00	520.00		
No of Similar floor including this			1	Total Load	493.80	520.00	517.80	536.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		12.20	58.56			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		59.40	8.91			
No of Similar floor including this			1	Total Load	82.83	0.00	600.63	536.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	600.63	536.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	600.63	536.00

Total Short Term Live Load 375.20 Total Long Term Live Load 208.00

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SC **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1	20.00	13.00	1.00	20.00	13.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	20.00	13.00	20.00	13.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 6	0.46	------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	------	4.60	4.91			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	335.00	327.00	1.00	335.00	327.00		
No of Similar floor including this			1	Total Load	342.03	327.00	362.03	340.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		10.20	48.96			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		43.20	6.48			
No of Similar floor including this			1	Total Load	70.80	0.00	432.83	340.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	432.83	340.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	432.83	340.00

Total Short Term Live Load 238.00 Total Long Term Live Load 130.80

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SD **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1	16.00	9.00	1.00	16.00	9.00		
	Portal Frame b/w S1 & S2							
	Cladding	0.15						
	M&E Plants							
No of Similar floor including this			1	Total Load	16.00	9.00	16.00	9.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 6	0.46	------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	------	4.60	4.91			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	319.00	302.00	1.00	319.00	302.00		
No of Similar floor including this			1	Total Load	326.03	302.00	342.03	311.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		10.20	48.96			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		43.20	6.48			
No of Similar floor including this			1	Total Load	70.80	0.00	412.83	311.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	412.83	311.00
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	412.83	311.00

Total Short Term Live Load 217.70 Total Long Term Live Load 120.80

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1	17.00	11.00	1.00	17.00	11.00		
	Portal Frame b/w S1 & S2							
	Cladding	0.15						
	M&E Plants							
No of Similar floor including this			1	Total Load	17.00	11.00	17.00	11.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	326.00	309.00	1.00	326.00	309.00		
No of Similar floor including this			1	Total Load	333.03	309.00	350.03	320.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		10.00	48.00			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		43.20	6.48			
No of Similar floor including this			1	Total Load	69.84	0.00	419.87	320.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	419.87	320.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	419.87	320.00

Total Short Term Live Load 224.00 Total Long Term Live Load 123.60

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SF.5 **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1	11.00	5.00	1.00	11.00	5.00		
	Portal Frame b/w S1 & S2							
	Cladding	0.15						
	M&E Plants							
No of Similar floor including this			1	Total Load	11.00	5.00	11.00	5.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	34.00	28.00	1.00	34.00	28.00		
No of Similar floor including this			1	Total Load	41.03	28.00	52.03	33.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		10.00	48.00			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		21.60	3.24			
No of Similar floor including this			1	Total Load	66.60	0.00	118.63	33.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	118.63	33.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	118.63	33.00

Total Short Term Live Load 23.10 Total Long Term Live Load 11.20

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S1 / SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1	18.00	11.00	1.00	18.00	11.00		
	Portal Frame b/w S1 & S2							
	Cladding	0.15						
	M&E Plants							
No of Similar floor including this			1	Total Load	18.00	11.00	18.00	11.00
Level 2	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 900 WB 175	1.75						
	Beam 410 UB 54	0.54						
	Beam 460 UB 67	0.67						
	From Frame Analy:	312.00	297.00	1.00	312.00	297.00		
No of Similar floor including this			1	Total Load	319.03	297.00	337.03	308.00
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		14.00	67.20			
	Pile Cap 800 X 800	15.36		1.00	15.36			
	Cladding	0.15		43.20	6.48			
No of Similar floor including this			1	Total Load	89.04	0.00	426.07	308.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	426.07	308.00
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	426.07	308.00

Total Short Term Live Load 215.60 Total Long Term Live Load 118.80

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2	24.00	25.00	0.75	18.00	18.75		
	Portal Frame b/w S2 & S3	24.00	25.00	0.39	9.25	9.64		
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	27.25	28.39	27.25	28.39
Level 2	Slab (1) 140	3.36	6.50	74.40	249.98	483.60		
	Slab (2)							
	SDL	1.00	------	74.40	74.40			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 6	0.46	------	4.50	2.07			
	Filled Conc Dia in r 238	1.07	------	4.50	4.80			
	Beam 900 WB 175	1.75		12.40	21.70			
	Beam 800 UB 132	1.32		6.20	8.18			
	Beam 460 UB 67	0.67		30.00	20.10			
	Cladding	0.15		78.00	11.70			
No of Similar floor including this			1	Total Load	392.94	483.60	420.20	511.99
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------	12.00	92.16			
	Beam GB2 500 X 400	4.80		6.20	29.76			
	Pile Cap 850 X 850	17.34		1.00	17.34			
	Cladding	0.15		54.00	8.10			
No of Similar floor including this			1	Total Load	147.36	0.00	567.56	511.99
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	567.56	511.99
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	567.56	511.99

Total Short Term Live Load 358.39 Total Long Term Live Load 193.44

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2	24.00	25.00	0.75	18.00	18.75		
	Portal Frame b/w S2 & S3	24.00	25.00	0.39	9.25	9.64		
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	27.25	28.39	27.25	28.39
Level 2	Slab (1) 140	3.36	6.50	74.40	249.98	483.60		
	Slab (2)							
	SDL	1.00	------	74.40	74.40			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 6	0.46	------	4.50	2.07			
	Filled Conc Dia in r 238	1.07	------	4.50	4.80			
	Beam 900 WB 175	1.75		12.40	21.70			
	Beam 800 UB 132	1.32		6.20	8.18			
	Beam 460 UB 67	0.67		30.00	20.10			
	Cladding	0.15		78.00	11.70			
No of Similar floor including this			1	Total Load	392.94	483.60	420.20	511.99
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------	12.00	92.16			
	Beam GB2 500 X 400	4.80		6.20	29.76			
	Pile Cap 850 X 850	17.34		1.00	17.34			
	Cladding	0.15		54.00	8.10			
No of Similar floor including this			1	Total Load	147.36	0.00	567.56	511.99
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	567.56	511.99
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	567.56	511.99

Total Short Term Live Load 358.39 Total Long Term Live Load 193.44

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2	0.00	0.00	0.75	0.00			
	Portal Frame b/w S2 & S3	0.00	0.00	0.39	0.00			
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	123.60	415.30	803.40		
	Slab (2)							
	SDL	1.00	------	123.60	123.60			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	------	4.50	4.57			
	Beam 900 WB 175	1.75		18.20	31.85			
	Beam 410 UB 54	0.54		24.60	13.28			
	Beam 460 UB 67	0.67		30.00	20.10			
	Cladding							
No of Similar floor including this			1	Total Load	611.76	803.40	611.76	803.40
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		12.20	58.56			
	Pile Cap 700 X 2100	35.28		1.00	35.28			
No of Similar floor including this			1	Total Load	93.84	0.00	705.60	803.40
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	705.60	803.40
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	705.60	803.40

Total Short Term Live Load 562.38 Total Long Term Live Load 321.36

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SC **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S2	47.00	53.00	0.75	35.25	39.75		
	Portal Frame b/w S2 & S3	47.00	53.00	0.39	18.12	20.44		
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	60.87	60.19	60.87	60.19
Level 2	Slab (1) 140	3.36	6.50	98.40	330.62	639.60		
	Slab (2)							
	SDL	1.00	-------	98.40	98.40			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.40	3.00			
	Filled Conc Dia in mm 232	1.01	-------	4.40	4.46			
	Beam 1000 WB 215	2.15		12.00	25.80			
	Beam 410 UB 54	0.54		49.20	26.57			
	Cladding							
No of Similar floor including this			1	Total Load	488.85	639.60	549.73	699.79
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 600 X 400	7.68	-------	0.00				
	Beam GB2 500 X 400	4.80		10.10	48.48			
	Pile Cap 850 X 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	65.82	0.00	615.55	699.79
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	615.55	699.79
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	615.55	699.79

Total Short Term Live Load 489.85 Total Long Term Live Load 255.84

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SD **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S2	0.00	0.00	0.75	0.00			
	Portal Frame b/w S2 & S3	0.00	0.00	0.39	0.00			
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	98.40	330.62	639.60		
	Slab (2)							
	SDL	1.00	-------	98.40	98.40			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.50	3.06			
	Filled Conc Dia in mm 232	1.01	-------	4.50	4.57			
	Beam 900 WB 175	1.75		12.00	21.00			
	Beam 410 UB 54	0.54		49.20	26.57			
	Cladding							
No of Similar floor including this			1	Total Load	484.22	639.60	484.22	639.60
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		10.10	48.48			
	Pile Cap 850 X 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	65.82	0.00	550.04	639.60
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	550.04	639.60
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	550.04	639.60

Total Short Term Live Load 447.72 Total Long Term Live Load 255.84

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S	35.00	42.00	0.75	26.25	31.50		
	Portal Frame b/w S2 & S	35.00	42.00	0.39	13.50	16.19		
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	47.25	47.69	47.25	47.69
Level 2	Slab (1) 140	3.36	6.50	98.40	330.62	639.60		
	Slab (2)							
	SDL	1.00	-------	98.40	98.40			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.40	3.00			
	Filled Conc Dia in r 232	1.01	-------	4.40	4.46			
	Beam 1000 WB 215	2.15		12.00	25.80			
	Beam 410 UB 54	0.54		49.20	26.57			
	Cladding							
No of Similar floor including this			1	Total Load	488.85	639.60	536.10	687.29
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		10.10	48.48			
	Pile Cap 850 x 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	65.82	0.00	601.92	687.29
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	601.92	687.29
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	601.92	687.29

Total Short Term Live Load 481.11 Total Long Term Live Load 255.84

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S	0.00	0.00	0.75	0.00			
	Portal Frame b/w S2 & S	0.00	0.00	0.39	0.00			
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	98.40	330.62	639.60		
	Slab (2)							
	SDL	1.00	-------	98.40	98.40			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	-------	4.50	4.57			
	Beam 900 WB 175	1.75		12.00	21.00			
	Beam 410 UB 54	0.54		49.20	26.57			
	Cladding							
No of Similar floor including this			1	Total Load	484.22	639.60	484.22	639.60
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		14.20	68.16			
	Pile Cap 850 X 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	85.50	0.00	569.72	639.60
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	569.72	639.60
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	569.72	639.60

Total Short Term Live Load 447.72 Total Long Term Live Load 255.84

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S2 / SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S	21.00	21.00	0.75	15.75	15.75		
	Portal Frame b/w S2 & S	21.00	21.00	0.39	8.10	8.10		
	Cladding	0.15		51.60	7.74			
	M&E Plants							
No of Similar floor including this			1	Total Load	31.59	23.85	31.59	23.85
Level 2	Slab (1) 140	3.36	6.50	49.20	165.31	319.80		
	Slab (2)							
	SDL	1.00	-------	49.20	49.20			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	-------	4.50	4.57			
	Beam 900 WB 175	1.75		12.00	21.00			
	Beam 410 UB 54	0.54		24.60	13.28			
	Cladding	0.15		78.00	11.70			
No of Similar floor including this			1	Total Load	268.13	319.80	299.71	343.65
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------	6.00	46.08			
	Beam GB2 500 x 400	4.80		10.10	48.48			
	Pile Cap 850 x 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	111.90	0.00	411.61	343.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	411.61	343.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	411.61	343.65

Total Short Term Live Load 240.55 Total Long Term Live Load 127.92

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S3	24.00	25.00	0.61	14.75	15.36		
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	14.75	15.36	14.75	15.36
Level 2	Slab (1) 140	3.36	6.50	55.80	187.49	362.70		
	Slab (2)							
	SDL	1.00	-------	55.80	55.80			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.50	2.07			
	Filled Conc Dia in r 238	1.07	-------	4.50	4.80			
	Beam 900 WB 175	1.75		6.00	10.50			
	Beam 700 UB 132	1.32		3.00	3.96			
	Beam 460 UB 67	0.67		22.50	15.08			
	Cladding	0.15		58.50	8.78			
No of Similar floor including this			1	Total Load	288.47	362.70	303.22	378.06
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	9.00	69.12			
	Beam GB2 500 X 400	4.80		6.20	29.76			
	Pile Cap 580 X 850	11.83		1.00	11.83			
	Cladding	0.15		48.60	7.29			
No of Similar floor including this			1	Total Load	118.00	0.00	421.22	378.06
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	421.22	378.06
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	421.22	378.06

Total Short Term Live Load 264.64 Total Long Term Live Load 145.08

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	92.70	311.47	602.55		
	Slab (2)							
	SDL	1.00	------	92.70	92.70			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	------	4.50	4.57			
	Beam 900 WB 175	1.75		15.20	26.60			
	Beam 410 UB 54	0.54		18.45	9.96			
	Beam 460 UB 67	0.67		22.50	15.08			
	Cladding							
No of Similar floor including this			1	Total Load	463.44	602.55	463.44	602.55
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		9.20	44.16			
	Pile Cap 800 X 800	15.36		1.00	15.36			
No of Similar floor including this			1	Total Load	59.52	0.00	522.96	602.55
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	522.96	602.55
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	522.96	602.55

Total Short Term Live Load 421.79 Total Long Term Live Load 241.02

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SC **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S	47.00	53.00	0.61	28.88	32.56		
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	36.38	32.56	36.38	32.56
Level 2	Slab (1) 140	3.36	6.50	73.80	247.97	479.70		
	Slab (2)							
	SDL	1.00	------	73.80	73.80			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.40	3.00			
	Filled Conc Dia in r 232	1.01	------	4.40	4.46			
	Beam 1000 WB 215	2.15		9.00	19.35			
	Beam 410 UB 54	0.54		36.90	19.93			
	Cladding							
No of Similar floor including this			1	Total Load	368.50	479.70	404.88	512.26
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------					
	Beam GB2 500 x 400	4.80		7.10	34.08			
	Pile Cap 800 x 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	46.37	0.00	451.25	512.26
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	451.25	512.26
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	451.25	512.26

Total Short Term Live Load 358.58 Total Long Term Live Load 191.88

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject *Column S3 / SD* **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2	0.00	0.00	0.75	0.00			
	Portal Frame b/w S2 & S3	0.00	0.00	0.39	0.00			
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	73.80	247.97	479.70		
	Slab (2)							
	SDL	1.00	------	73.80	73.80			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	4.50	6.17			
	Filled Conc Dia in r		------	4.50	0.00			
	Beam 900 WB 175	1.75		9.00	15.75			
	Beam 410 UB 54	0.54		36.90	19.93			
	Cladding							
No of Similar floor including this			1	Total Load	363.61	479.70	363.61	479.70
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------					
	Beam GB2 500 x 400	4.80		7.10	34.08			
	Pile Cap 850 x 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	51.42	0.00	415.03	479.70
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	415.03	479.70
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	415.03	479.70

Total Short Term Live Load 335.79 Total Long Term Live Load 191.88

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S3	35.00	42.00	0.61	21.50	25.81		
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	29.00	25.81	29.00	25.81
Level 2	Slab (1) 140	3.36	6.50	73.80	247.97	479.70		
	Slab (2)							
	SDL	1.00	------	73.80	73.80			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	4.40	6.03			
	Filled Conc Dia in r		------	4.40	0.00			
	Beam 1000 WB 215	2.15		9.00	19.35			
	Beam 410 UB 54	0.54		36.90	19.93			
	Cladding							
No of Similar floor including this			1	Total Load	367.07	479.70	396.08	505.51
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------					
	Beam GB2 500 x 400	4.80		7.10	34.08			
	Pile Cap 900 x 2400	51.84		1.00	51.84			
No of Similar floor including this			1	Total Load	85.92	0.00	482.00	505.51
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	482.00	505.51
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	482.00	505.51

Total Short Term Live Load 353.85 Total Long Term Live Load 191.88

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	73.80	247.97	479.70		
	Slab (2)							
	SDL	1.00	-------	73.80	73.80			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	-------	4.50	4.57			
	Beam 900 WB 175	1.75		9.00	15.75			
	Beam 410 UB 54	0.54		36.90	19.93			
	Cladding							
No of Similar floor including this			1	Total Load	365.07	479.70	365.07	479.70
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		11.20	53.76			
	Pile Cap 800 X 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	66.05	0.00	431.12	479.70
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	431.12	479.70
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	431.12	479.70

Total Short Term Live Load 335.79 Total Long Term Live Load 191.88

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S3 / SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame b/w S1 & S2							
	Portal Frame b/w S2 & S	21.00	21.00	0.61	12.90	12.90		
	Cladding	0.15		38.70	5.81			
	M&E Plants							
No of Similar floor including this			1	Total Load	18.71	12.90	18.71	12.90
Level 2	Slab (1) 140	3.36	6.50	36.90	123.98	239.85		
	Slab (2)							
	SDL	1.00	------	36.90	36.90			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	------	4.50	4.57			
	Beam 900 WB 175	1.75		9.00	15.75			
	Beam 410 UB 54	0.54		18.45	9.96			
	Cladding	0.15		58.50	8.78			
No of Similar floor including this			1	Total Load	203.00	239.85	221.71	252.75
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	9.00	69.12			
	Beam GB2 500 x 400	4.80		4.10	19.68			
	Pile Cap 900 x 2400	51.84		0.80	41.47			
No of Similar floor including this			1	Total Load	130.27	0.00	351.98	252.75
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	351.98	252.75
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	351.98	252.75

Total Short Term Live Load 176.93 Total Long Term Live Load 95.94

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4.8 / SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S4	37.00	45.00	0.33	12.21	14.85		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	12.21	14.85	12.21	14.85
Level 2	Slab (1) 140	3.36	6.50	60.00	201.60	390.00		
	Slab (2) Staircase							
	SDL	1.00	------	60.00	60.00			
	115mm Brick 2.5	3.00	------	6.25	46.88			
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in mm 232	1.01	------	4.60	4.67			
	Beam 800 WB 122	1.22		12.00	14.64			
	Beam 410 UB 54	0.54		32.80	17.71			
	Cladding							
No of Similar floor including this			1	Total Load	348.63	390.00	360.84	404.85
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	7.50	57.60			
	Beam GB2 500 x 400	4.80		4.10	19.68			
	Pile Cap 850 x 850	17.34		0.80	13.87			
No of Similar floor including this			1	Total Load	91.15	0.00	451.99	404.85
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	451.99	404.85
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	451.99	404.85

Total Short Term Live Load 283.40 Total Long Term Live Load 156.00

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4.8 / SH **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	78.75	264.60	511.88		
	Slab (2) Staircase							
	SDL	1.00	-------	78.75	78.75			
	115mm Brick 2.5	3.00	-------	8.50	63.75			
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 800 WB 122	1.22		11.50	14.03			
	Beam 410 UB 54	0.54		32.80	17.71			
	Cladding							
No of Similar floor including this			1	Total Load	445.87	511.88	445.87	511.88
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		11.60	55.68			
	Pile Cap 850 X 850	17.34		0.80	13.87			
No of Similar floor including this			1	Total Load	69.55	0.00	515.42	511.88
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	515.42	511.88
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	515.42	511.88

Total Short Term Live Load 358.31 Total Long Term Live Load 204.75

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4.8 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	17.00	16.00	0.81	13.73	12.92		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		45.15	6.77			
No of Similar floor including this			1	Total Load	20.50	12.92	20.50	12.92
Level 2	Slab (1) 140	3.36	6.50	39.38	132.30	255.94		
	Slab (2) Staircase							
	SDL	1.00	-------	39.38	39.38			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 800 WB 122	1.22		34.00	41.48			
	Beam 410 UB 54	0.54		7.50	4.05			
	Pre-cast Cladding 175	4.20		42.00	176.40			
	Cladding	0.15		26.25	3.94			
No of Similar floor including this			1	Total Load	404.57	255.94	425.07	268.86
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		15.70	75.36			
	Pile Cap 850 X 850	17.34		0.80	13.87			
	Pre-cast Cladding 175	4.20		56.70	238.14			
No of Similar floor including this			1	Total Load	327.37	0.00	752.44	268.86
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	752.44	268.86
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	752.44	268.86

Total Short Term Live Load 188.20 Total Long Term Live Load 102.38

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject *Column S4 / SA* **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	24.00	25.00	1.00	24.00	25.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.00	25.00	24.00	25.00
Level 2	Slab (1) 140	3.36	6.50	51.46	172.91	334.49		
	Slab (2)							
	SDL	1.00	-------	51.46	51.46			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	7.44	-------	4.70	34.98			
	Filled Conc Dia in mm		-------	4.70	0.00			
	Beam 700 WB 132	1.32		8.30	10.96			
	Beam 410 UB 54	0.54		0.00				
	Beam 460 UB 67	0.67		20.75	13.90			
	Cladding	0.15		53.95	8.09			
No of Similar floor including this			1	Total Load	292.29	334.49	316.29	359.49
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	8.30	63.74			
	Beam GB2 500 X 400	4.80		6.20	29.76			
	Pile Cap 850 X 850	17.34		1.00	17.34			
	Cladding	0.15		44.82	6.72			
No of Similar floor including this			1	Total Load	117.57	0.00	433.86	359.49
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	433.86	359.49
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	433.86	359.49

Total Short Term Live Load 251.64 Total Long Term Live Load 133.80

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	85.49	287.25	555.69		
	Slab (2)							
	SDL	1.00	-------	85.49	85.49			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	7.44	-------	4.50	33.49			
	Filled Conc Dia in r		-------	4.50	0.00			
	Beam 900 WB 172	1.72		14.50	24.94			
	Beam 410 UB 54	0.54		12.30	6.64			
	Beam 460 UB 67	0.67		20.75	13.90			
	Cladding							
No of Similar floor including this			1	Total Load	451.71	555.69	451.71	555.69
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	10.30	79.10			
	Beam GB2 500 X 400	4.80		3.00	14.40			
	Pile Cap 900 X 2400	51.84		1.00	51.84			
	K Brace							
No of Similar floor including this			1	Total Load	145.34	0.00	597.05	555.69
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	597.05	555.69
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	597.05	555.69

Total Short Term Live Load 388.98 Total Long Term Live Load 222.27

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SC **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	47.00	53.00	1.00	47.00	53.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	54.50	53.00	54.50	53.00
Level 2	Slab (1) 140	3.36	6.50	68.06	228.68	442.39		
	Slab (2)							
	SDL	1.00	-------	68.06	68.06			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	7.44	-------	4.40	32.75			
	Filled Conc Dia in mm		-------	4.40	0.00			
	Beam 1000 WB 215	2.15		8.30	17.85			
	Beam 410 UB 54	0.54		24.60	13.28			
	Beam 700 WB 132	1.32		8.20	10.82			
	Cladding							
No of Similar floor including this			1	Total Load	371.44	442.39	425.94	495.39
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	8.20	62.98			
	Beam GB2 500 X 400	4.80		12.40	59.52			
	Pile Cap 850 X 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	139.84	0.00	565.78	495.39
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	565.78	495.39
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	565.78	495.39

Total Short Term Live Load 346.77 Total Long Term Live Load 176.96

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SD **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S4							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	68.06	228.68	442.39		
	Slab (2)							
	SDL	1.00	------	68.06	68.06			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	4.50	6.17			
	Filled Conc Dia in r		------	4.50	0.00			
	Beam 900 WB 175	1.75		8.30	14.53			
	Beam 410 UB 54	0.54		24.60	13.28			
	Beam 700 UB 132	1.32		8.20	10.82			
No of Similar floor including this			1	Total Load	341.54	442.39	341.54	442.39
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	8.20	62.98			
	Beam GB2 500 x 400	4.80		8.30	39.84			
	Pile Cap 900 x 2400	51.84		1.00	51.84			
No of Similar floor including this			1	Total Load	154.66	0.00	496.20	442.39
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	496.20	442.39
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	496.20	442.39

Total Short Term Live Load 309.67 Total Long Term Live Load 176.96

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	35.00	42.00	1.00	35.00	42.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	42.50	42.00	42.50	42.00
Level 2	Slab (1) 140	3.36	6.50	68.06	228.68	442.39		
	Slab (2)							
	SDL	1.00	-------	68.06	68.06			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 135	1.35		9.00	12.15			
	Beam 410 UB 54	0.54		24.60	13.28			
	Beam 700 UB 132	1.32		8.20	10.82			
No of Similar floor including this			1	Total Load	339.30	442.39	381.80	484.39
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------	8.20	62.98			
	Beam GB2 500 x 400	4.80		8.30	39.84			
	Pile Cap 850 x 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	120.16	0.00	501.96	484.39
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	501.96	484.39
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	501.96	484.39

Total Short Term Live Load 339.07 Total Long Term Live Load 176.96

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S4	0.00	0.00					
	Portal Frame b/w S2 & S	0.00	0.00					
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	68.06	228.68	442.39		
	Slab (2)							
	SDL	1.00	------	68.06	68.06			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 800 WB 135	1.35		8.30	11.21			
	Beam 410 UB 54	0.54		24.60	13.28			
	Beam 700 UB 132	1.32		8.20	10.82			
No of Similar floor including this			1	Total Load	339.85	442.39	339.85	442.39
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	8.20	62.98			
	Beam GB2 500 x 400	4.80		8.30	39.84			
	Pile Cap 850 x 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	120.16	0.00	460.01	442.39
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	460.01	442.39
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	460.01	442.39

Total Short Term Live Load 309.67 Total Long Term Live Load 176.96

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	37.00	45.00	1.00	37.00	45.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		30.10	4.52			
	M&E Plants							
No of Similar floor including this			1	Total Load	41.52	45.00	41.52	45.00
Level 2	Slab (1) 140	3.36	6.50	34.03	114.34	221.20		
	Slab (2) Staircase 225	5.40	4.00	17.00	91.80	68.00		
	SDL	1.00	-------	34.03	34.03			
	115mm Brick 2.5	3.00	-------	8.25	61.88			
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 122	1.22		8.30	10.13			
	Beam 410 UB 54	0.54		16.40	8.86			
	Cladding	0.15		45.50	6.83			
No of Similar floor including this			1	Total Load	334.15	289.20	375.67	334.20
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick 6.5	3.00	-------	8.25	160.88			
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	9.00	69.12			
	Beam GB2 500 X 400	4.80		4.10	19.68			
	Pile Cap 850 X 850	17.34		1.00	17.34			
No of Similar floor including this			1	Total Load	267.02	0.00	642.68	334.20
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	642.68	334.20
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	642.68	334.20

Total Short Term Live Load 233.94 Total Long Term Live Load 115.68

7-35

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** **2711265**

Subject *Column S4 / SH* **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S4	37.00	45.00	0.50	18.50	22.50		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		32.25	4.84			
	M&E Plants							
No of Similar floor including this			1	Total Load	23.34	22.50	23.34	22.50
Level 2	Slab (1) 140	3.36	6.50	39.75	133.56	258.38		
	Slab (2) Staircase 225	5.40	4.00	17.00	91.80	68.00		
	SDL	1.00	------	39.75	39.75			
	115mm Brick 2.5	3.00	------	7.75	58.13			
	230mm Brick	5.60	------					
	Column 250 SHS 6	0.46	------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	------	4.60	4.91			
	Beam 800 WB 122	1.22		4.00	4.88			
	Beam 410 UB 54	0.54		10.25	5.54			
	Cladding	0.15		48.75	7.31			
No of Similar floor including this			1	Total Load	347.99	326.38	371.33	348.88
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick 6.5	3.00	------	7.75	151.13			
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	7.50	57.60			
	Beam GB2 500 x 400	4.80		4.00	19.20			
	Pile Cap 800 x 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	240.21	0.00	611.54	348.88
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	611.54	348.88
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	611.54	348.88

Total Short Term Live Load 244.21 Total Long Term Live Load 130.55

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S4 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	17.00	16.00	1.00	17.00	16.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		38.92	5.84			
	M&E Plants							
No of Similar floor including this			1	Total Load	22.84	16.00	22.84	16.00
Level 2	Slab (1) 140	3.36	6.50	19.88	66.78	129.19		
	Slab (2) Staircase							
	SDL	1.00	-------	19.88	19.88			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 800 WB 122	1.22		4.00	4.88			
	Beam 410 UB 54	0.54		10.25	5.54			
	Cladding	0.15		18.10	2.72			
	Pre-cast Cladding 175	4.20		40.73	171.05			
No of Similar floor including this			1	Total Load	277.86	129.19	300.69	145.19
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick 6.5	3.00	-------	0.00				
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------	7.50	57.60			
	Beam GB2 500 x 400	4.80		4.00	19.20			
	Pile Cap 800 x 800	15.36		0.80	12.29			
	Pre-cast Cladding 175	4.20		48.87	205.25			
No of Similar floor including this			1	Total Load	294.34	0.00	595.04	145.19
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	595.04	145.19
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	595.04	145.19

Total Short Term Live Load 101.63 Total Long Term Live Load 51.68

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5.4 / SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	37.00	45.00	0.67	24.79	30.15		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.79	30.15	24.79	30.15
Level 2	Slab (1) 140	3.36	6.50	72.50	243.60	471.25		
	Slab (2) Staircase							
	SDL	1.00	-------	72.50	72.50			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		28.75	35.08			
	Beam 410 UB 54	0.54		12.30	6.64			
	Cladding							
No of Similar floor including this			1	Total Load	365.62	471.25	390.41	501.40
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------	6.25	48.00			
	Beam GB2 500 x 400	4.80		5.00	24.00			
	Pile Cap 850 x 850	17.34		0.80	13.87			
No of Similar floor including this			1	Total Load	85.87	0.00	476.28	501.40
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	476.28	501.40
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	476.28	501.40

Total Short Term Live Load 350.98 Total Long Term Live Load 188.50

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5/SA, S6/SA and S7/SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S4	24.00	25.00	1.00	24.00	25.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.00	25.00	24.00	25.00
Level 2	Slab (1) 140	3.36	6.50	65.72	220.82	427.18		
	Slab (2)							
	SDL	1.00	-------	65.72	65.72			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.70	6.44			
	Filled Conc Dia in r		-------	4.70	0.00			
	Beam 700 WB 130	1.30		10.60	13.78			
	Beam 800 WB 175	1.75		6.20	10.85			
	Beam 460 UB 67	0.67		26.50	17.76			
	Cladding	0.15		68.90	10.34			
No of Similar floor including this			1	Total Load	345.70	427.18	369.70	452.18
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	10.60	81.41			
	Beam GB2 500 X 400	4.80		6.20	29.76			
	Pile Cap 900 X 2400	51.84		1.00	51.84			
	Cladding	0.15		57.24	8.59			
No of Similar floor including this			1	Total Load	171.59	0.00	541.29	452.18
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	541.29	452.18
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	541.29	452.18

Total Short Term Live Load 316.53 Total Long Term Live Load 170.87

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5/SB, S6/SB and S7/SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S5							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	109.18	366.84	709.67		
	Slab (2)							
	SDL	1.00	------	109.18	109.18			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.50	3.06			
	Filled Conc Dia in r 232	1.01	------	4.50	4.57			
	Beam 900 WB 175	1.75		16.80	29.40			
	Beam 410 UB 54	0.54		20.50	11.07			
	Beam 460 UB 67	0.67		26.50	17.76			
	Cladding							
No of Similar floor including this			1	Total Load	541.88	709.67	541.88	709.67
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		11.50	55.20			
	Pile Cap 800 X 2050	39.36		1.00	39.36			
	K Brace							
No of Similar floor including this			1	Total Load	94.56	0.00	636.44	709.67
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	636.44	709.67
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	636.44	709.67

Total Short Term Live Load 496.77 Total Long Term Live Load 283.87

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5/SC, S6/SC, S7/SC, S8/SC, S9/SC & S10/SC **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	47.00	53.00	1.00	47.00	53.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	54.50	53.00	54.50	53.00
Level 2	Slab (1) 140	3.36	6.50	86.92	292.05	564.98		
	Slab (2)							
	SDL	1.00	-------	86.92	86.92			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		10.60	12.93			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	421.84	564.98	476.34	617.98
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		9.40	45.12			
	Pile Cap 800 X 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	57.41	0.00	533.75	617.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	533.75	617.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	533.75	617.98

Total Short Term Live Load 432.59 Total Long Term Live Load 225.99

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265
Subject Column S5/SD, S6/SD, S7/SD **Page no** Sheet 1 of
Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	86.92	292.05	564.98		
	Slab (2)							
	SDL	1.00	-------	86.92	86.92			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		10.60	12.93			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	421.84	564.98	421.84	564.98
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		9.40	45.12			
	Pile Cap 800 x 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	57.41	0.00	479.25	564.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	479.25	564.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	479.25	564.98

Total Short Term Live Load 395.49 Total Long Term Live Load 225.99

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5/SE, S6/SE, S7/SE, S10/SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	32.00	39.00	1.00	32.00	39.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	39.50	39.00	39.50	39.00
Level 2	Slab (1) 140	3.36	6.50	86.10	289.30	559.65		
	Slab (2)							
	SDL	1.00	-------	86.10	86.10			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 135	1.35		10.50	14.18			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	419.51	559.65	459.01	598.65
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		9.35	44.88			
	Pile Cap 800 X 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	57.17	0.00	516.18	598.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	516.18	598.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	516.18	598.65

Total Short Term Live Load 419.06 Total Long Term Live Load 223.86

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S5/SF, S6/SF, S7/SF, S10/SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	0.00	0.00					
	Portal Frame b/w S2 & S	0.00	0.00					
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	86.10	289.30	559.65		
	Slab (2)							
	SDL	1.00	-------	86.10	86.10			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 800 WB 135	1.35		10.50	14.18			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	418.74	559.65	418.74	559.65
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		13.45	64.56			
	Pile Cap 800 x 800	15.36		0.80	12.29			
No of Similar floor including this			1	Total Load	76.85	0.00	495.59	559.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	495.59	559.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	495.59	559.65

Total Short Term Live Load 391.76 Total Long Term Live Load 223.86

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S6.5/SG & S7.5/SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S5							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	60.90	204.62	395.85		
	Slab (2) Staircase							
	SDL	1.00	------	60.90	60.90			
	115mm Brick 2.5	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 800 WB 122	1.22		27.75	33.86			
	Beam 410 UB 54	0.54		12.30	6.64			
	Cladding							
No of Similar floor including this			1	Total Load	313.82	395.85	313.82	395.85
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------	5.25	40.32			
	Beam GB2 500 X 400	4.80		4.10	19.68			
	Pile Cap 850 X 850	17.34		0.80	13.87			
No of Similar floor including this			1	Total Load	73.87	0.00	387.69	395.85
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	387.69	395.85
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	387.69	395.85

Total Short Term Live Load 277.10 Total Long Term Live Load 158.34

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S6/SG & S7/SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S6	37.00	45.00	1.00	37.00	45.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	37.00	45.00	37.00	45.00
Level 2	Slab (1) 140	3.36	6.50	60.90	204.62	395.85		
	Slab (2) Staircase							
	SDL	1.00	-------	60.90	60.90			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		27.75	33.86			
	Beam 410 UB 54	0.54		12.30	6.64			
	Cladding							
No of Similar floor including this			1	Total Load	313.82	395.85	350.82	440.85
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------	5.25	40.32			
	Beam GB2 500 X 400	4.80		7.50	36.00			
	Pile Cap 850 X 850	17.34		0.80	13.87			
No of Similar floor including this			1	Total Load	90.19	0.00	441.01	440.85
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	441.01	440.85
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	441.01	440.85

Total Short Term Live Load 308.60 Total Long Term Live Load 158.34

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S6 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	17.00	16.00	0.19	3.23	3.04		
	Portal Frame @ S6	17.00	16.00	1.00	17.00	16.00		
	Cladding	0.15		53.75	8.06			
No of Similar floor including this			1	Total Load	28.29	19.04	28.29	19.04
Level 2	Slab (1) 140	3.36	6.50	86.25	289.80	560.63		
	Slab (2) Staircase							
	SDL	1.00	-------	86.25	86.25			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		49.40	60.27			
	Beam 410 UB 54	0.54						
	Pre-cast Cladding 175	4.20		34.50	144.90			
	Cladding	0.15		40.25	6.04			
No of Similar floor including this			1	Total Load	595.06	560.63	623.35	579.67
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		19.00	91.20			
	Pile Cap 700 x 2100	35.28		1.00	35.28			
	Pre-cast Cladding 175	4.20		62.10	260.82			
No of Similar floor including this			1	Total Load	387.30	0.00	1010.65	579.67
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	1010.65	579.67
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	1010.65	579.67

Total Short Term Live Load 405.77 Total Long Term Live Load 224.25

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S7 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S7	17.00	16.00	1.00	17.00	16.00		
	Portal Frame @ S6							
	Cladding	0.15		45.15	6.77			
No of Similar floor including this			1	Total Load	23.77	16.00	23.77	16.00
Level 2	Slab (1) 140	3.36	6.50	78.75	264.60	511.88		
	Slab (2) Staircase							
	SDL	1.00	-------	78.75	78.75			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		49.40	60.27			
	Beam 410 UB 54	0.54						
	Pre-cast Cladding 175	4.20						
	Cladding	0.15		68.25	10.24			
No of Similar floor including this			1	Total Load	421.66	511.88	445.43	527.88
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick 6.5	3.00	-------	0.00				
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		18.00	86.40			
	Pile Cap 850 X 850	17.34		0.80	13.87			
	Pre-cast Cladding 150	3.60		56.70	204.12			
No of Similar floor including this			1	Total Load	304.39	0.00	749.82	527.88
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	749.82	527.88
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	749.82	527.88

Total Short Term Live Load 369.51 Total Long Term Live Load 204.75

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S8/SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S8	24.00	25.00	1.00	24.00	25.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.00	25.00	24.00	25.00
Mezzar	Slab (1) 160	3.84	3.00	32.55	124.99	97.65		
Floor	Slab (2)							
	SDL	0.50	-------	32.55	16.28			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 132	1.32		10.60	13.99			
	Beam 500 UB 82	0.82		18.60	15.25			
	Internal Partition	1.00		15.50	15.50			
	Cladding	0.15		26.25	3.94			
No of Similar floor including this			1	Total Load	196.25	97.65	220.25	122.65
Level 2	Slab (1) 140	3.36	6.50	65.10	218.74	423.15		
	Slab (2)							
	SDL	1.00	-------	65.10	65.10			
	115mm Brick 3.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.70	6.44			
	Filled Conc Dia in r		-------	4.70	0.00			
	Beam 700 WB 130	1.30		16.70	21.71			
	Beam 800 WB 122	1.22		0.00				
	Beam 460 UB 67	0.67		26.25	17.59			
	Cladding	0.15		36.75	5.51			
No of Similar floor including this			1	Total Load	335.09	423.15	555.34	545.80
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	6.34	-------	10.50	66.53			
	Beam GB2 500 x 400	3.46		6.20	21.43			
	Pile Cap 850 x 850	17.34		1.00	17.34			
	Cladding	0.15		57.24	8.59			
No of Similar floor including this			1	Total Load	113.88	0.00	669.22	545.80
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	669.22	545.80

Total Short Term Live Load 382.06 Total Long Term Live Load 208.32

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265
Subject Column S8/SB **Page no** Sheet 1 of
Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S8							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Mezzar	Slab (1) 160	3.84	3.00	32.55	124.99	97.65		
Floor	Slab (2)							
	SDL	0.50	-------	32.55	16.28			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 325 CHS 9.5	0.74	-------	4.60	3.40			
	Filled Conc Dia in r 306	1.76	-------	4.60	8.12			
	Beam 800 WB 132	1.32		5.25	6.93			
	Beam 500 UB 82	0.82		18.60	15.25			
	Beam 460 UB 67	0.67						
	Internal Wall	1.00		28.63	28.63			
No of Similar floor including this			1	Total Load	203.59	97.65	203.59	97.65
Level 2	Slab (1) 140	3.36	6.50	108.15	363.38	702.98		
	Slab (2)							
	SDL	1.00	-------	108.15	108.15			
	115mm Brick 3.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 122	1.22		16.70	20.37			
	Beam 410 UB 54	0.54		20.50	11.07			
	Beam 460 UB 67	0.67		26.25	17.59			
	Cladding	0.15						
No of Similar floor including this			1	Total Load	526.87	702.98	730.46	800.63
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	6.34	-------					
	Beam GB2 500 x 400	3.46		11.45	39.57			
	Pile Cap 700 x 2100	28.22		1.00	28.22			
	Cladding	0.15						
No of Similar floor including this			1	Total Load	67.80	0.00	798.26	800.63
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	798.26	800.63

Total Short Term Live Load 560.44 Total Long Term Live Load 320.25

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265
Subject Column S8/SD, S9/SD, S10/SD **Page no** Sheet 1 of
Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	86.92	292.05	564.98		
	Slab (2)							
	SDL	1.00	-------	86.92	86.92			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		10.60	12.93			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	421.84	564.98	421.84	564.98
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		9.40	45.12			
	Pile Cap 700 x 2100	35.28		1.00	35.28			
No of Similar floor including this			1	Total Load	80.40	0.00	502.24	564.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	502.24	564.98
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	502.24	564.98

Total Short Term Live Load 395.49 Total Long Term Live Load 225.99

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S8/SE, S9/SE **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	32.00	39.00	1.00	32.00	39.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants	30.00		0.25	7.50			
No of Similar floor including this			1	Total Load	39.50	39.00	39.50	39.00
Level 2	Slab (1) 140	3.36	6.50	86.10	289.30	559.65		
	Slab (2)							
	SDL	1.00	-------	86.10	86.10			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 135	1.35		10.50	14.18			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	419.51	559.65	459.01	598.65
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		9.35	44.88			
	Pile Cap 700 X 2100	35.28		1.00	35.28			
No of Similar floor including this			1	Total Load	80.16	0.00	539.17	598.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	539.17	598.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	539.17	598.65

Total Short Term Live Load 419.06 Total Long Term Live Load 223.86

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S8/SF, S9/SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S5	0.00	0.00					
	Portal Frame b/w S2 & S	0.00	0.00					
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1) 140	3.36	6.50	86.10	289.30	559.65		
	Slab (2)							
	SDL	1.00	-------	86.10	86.10			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 6	0.46	-------	4.60	2.11			
	Filled Conc Dia in r 238	1.07	-------	4.60	4.91			
	Beam 800 WB 135	1.35		10.50	14.18			
	Beam 410 UB 54	0.54		41.00	22.14			
	Cladding							
No of Similar floor including this			1	Total Load	418.74	559.65	418.74	559.65
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		13.45	64.56			
	Pile Cap 700 X 2100	35.28		0.80	28.22			
No of Similar floor including this			1	Total Load	92.78	0.00	511.52	559.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	511.52	559.65
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	511.52	559.65

Total Short Term Live Load 391.76 Total Long Term Live Load 223.86

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S8/SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S6	37.00	45.00	1.00	37.00	45.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	37.00	45.00	37.00	45.00
Level 2	Slab (1) 140	3.36	6.50	53.29	179.05	346.37		
	Slab (2) Canopy 10.5	0.25	0.25	21.00	5.29	5.25		
	SDL	1.00	------	74.29	74.29			
	115mm Brick 2.5	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 800 WB 122	1.22		30.38	37.06			
	Beam 410 UB 54	0.54		16.40	8.86			
	Cladding							
No of Similar floor including this			1	Total Load	312.34	351.62	349.34	396.62
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	7.68	------	7.88	60.48			
	Beam GB2 500 x 400	4.80		7.50	36.00			
	Pile Cap 850 x 850	17.34		1.00	17.34			
	pre-cast Cladding 150	3.60		28.35	102.06			
No of Similar floor including this			1	Total Load	215.88	0.00	565.22	396.62
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	565.22	396.62
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	565.22	396.62

Total Short Term Live Load 277.63 Total Long Term Live Load 140.65

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S8 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S8	17.00	16.00	1.00	17.00	16.00		
	Portal Frame @ S6							
	Cladding	0.15		54.83	8.22			
No of Similar floor including this			1	Total Load	25.22	16.00	25.22	16.00
Level 2	Slab (1) 140	3.36	6.50	39.38	132.30	255.94		
	Slab (2) Canopy 10.5	0.25	0.25	39.38	9.92	9.84		
	SDL	1.00	-------	78.75	78.75			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		27.75	33.86			
	Beam 410 UB 54	0.54						
	Pre-cast Cladding 175	4.20						
	Cladding	0.15		82.88	12.43			
No of Similar floor including this			1	Total Load	275.06	265.78	300.28	281.78
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick 6.5	3.00	-------	0.00				
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	7.68	-------					
	Beam GB2 500 X 400	4.80		18.00	86.40			
	Pile Cap 850 X 850	17.34		0.80	13.87			
	Pre-cast Cladding 150	3.60		56.70	204.12			
No of Similar floor including this			1	Total Load	304.39	0.00	604.67	281.78
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	604.67	281.78
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	604.67	281.78

Total Short Term Live Load 197.25 Total Long Term Live Load 106.31

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S9/SA, S10/SA **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S9	24.00	25.00	1.00	24.00	25.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	24.00	25.00	24.00	25.00
Mezzar	Slab (1) 140	3.36	3.00	65.10	218.74	195.30		
Floor	Slab (2)							
	SDL	0.50	-------	65.10	32.55			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 132	1.32		10.60	13.99			
	Beam 500 UB 82	0.82		31.00	25.42			
	Beam 460 UB 67	0.67						
	Cladding	0.15		26.25	3.94			
No of Similar floor including this			1	Total Load	302.44	195.30	326.44	220.30
Level 2	Slab (1) 160	3.84	6.50	65.10	249.98	423.15		
	Slab (2)							
	SDL	1.00	-------	65.10	65.10			
	115mm Brick 3.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.70	6.44			
	Filled Conc Dia in r		-------	4.70	0.00			
	Beam 700 WB 130	1.30		16.70	21.71			
	Beam 500 UB 82	0.82		0.00				
	Beam 460 UB 67	0.67		26.25	17.59			
	Cladding	0.15		42.00	6.30			
No of Similar floor including this			1	Total Load	367.12	423.15	693.56	643.45
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	6.14	-------	10.50	64.51			
	Beam GB2 500 X 400	3.26		6.20	20.24			
	Pile Cap 850 X 850	17.34		1.00	17.34			
	Cladding	0.15		56.70	8.51			
No of Similar floor including this			1	Total Load	110.59	0.00	804.15	643.45
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	804.15	643.45
Total Short Term Live Load	450.42			Total Long Term Live Load	247.38			

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S9/SB, S10/SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S9							
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Mezzar	Slab (1) 140	3.36	3.00	65.10	218.74	195.30		
Floor	Slab (2)							
	SDL	0.50	-------	65.10	32.55			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 325 CHS 9.5	0.74	-------	4.60	3.40			
	Filled Conc Dia in mm 306	1.76	-------	4.60	8.12			
	Beam 800 WB 132	1.32		10.60	13.99			
	Beam 500 UB 82	0.82		31.00	25.42			
	Beam 460 UB 67	0.67						
	Internal Wall	1.00		26.25	26.25			
No of Similar floor including this			1	Total Load	328.47	195.30	328.47	195.30
Level 2	Slab (1) 160	3.84	6.50	108.15	415.30	702.98		
	Slab (2)							
	SDL	1.00	-------	108.15	108.15			
	115mm Brick 3.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 122	1.22		16.70	20.37			
	Beam 410 UB 54	0.54		20.50	11.07			
	Beam 460 UB 67	0.67		26.25	17.59			
	Cladding	0.15						
No of Similar floor including this			1	Total Load	578.78	702.98	907.25	898.28
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 X 400	6.14	-------					
	Beam GB2 500 X 400	3.26		11.45	37.37			
	Pile Cap 700 X 2100	27.22		1.00	27.22			
	Cladding	0.15						
No of Similar floor including this			1	Total Load	64.59	0.00	971.84	898.28
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	971.84	898.28

Total Short Term Live Load 628.79 Total Long Term Live Load 359.31

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S9/SG & S10/SG **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S9	22.00	21.00	1.00	22.00	21.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		45.15	6.77			
	M&E Plants							
No of Similar floor including this			1	Total Load	28.77	21.00	28.77	21.00
Level 2	Slab (1) 140	3.36	6.50	43.05	144.65	279.83		
	Slab (2) Canopy 10.5	0.25	0.25	73.50	18.52	18.38		
	SDL	1.00	-------	79.80	79.80			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 310 UC 137	1.37	-------	4.60	6.30			
	Filled Conc Dia in r		-------	4.60	0.00			
	Beam 800 WB 122	1.22		10.50	12.81			
	Beam 410 UB 54	0.54		20.50	11.07			
	Pre-cast Cladding							
	Cladding	0.15		68.25	10.24			
No of Similar floor including this			1	Total Load	283.39	298.20	312.16	319.20
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------	10.50	80.64			
	Beam GB2 500 x 400	4.80		7.50	36.00			
	Pile Cap 850 x 850	17.34		1.00	17.34			
	Pre-cast Cladding 150	3.60		56.70	204.12			
No of Similar floor including this			1	Total Load	338.10	0.00	650.26	319.20
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	650.26	319.20
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	650.26	319.20

Total Short Term Live Load 223.44 Total Long Term Live Load 119.28

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S9 / SK **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S8							
	Portal Frame @ S6							
	Cladding							
No of Similar floor including this			1	Total Load	0.00	0.00	0.00	0.00
Level 2	Slab (1)							
	Slab (2) Canopy 10.5	0.25	0.25	39.38	9.92	9.84		
	SDL	1.00	-------	39.38	39.38			
	115mm Brick 2.5	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	5.40	3.68			
	Filled Conc Dia in r 232	1.01	-------	5.40	5.48			
	Beam WB 122	1.22						
	Beam 410 UB 54	0.54						
	Pre-cast Cladding 175	4.20						
	Cladding	0.15		19.13	2.87			
No of Similar floor including this			1	Total Load	61.32	9.84	61.32	9.84
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick 6.5	3.00	-------	0.00				
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		12.75	61.20			
	Pile Cap 850 x 850	17.34		0.80	13.87			
	Pre-cast Cladding 150	3.60		56.70	204.12			
No of Similar floor including this			1	Total Load	279.19	0.00	340.51	9.84
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	340.51	9.84
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	340.51	9.84

Total Short Term Live Load 6.89 Total Long Term Live Load 3.94

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S11/SA, **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S11	12.00	6.00	1.00	12.00	6.00		
	Portal Frame b/w S2 & S3							
	Cladding							
	M&E Plants							
No of Similar floor including this			1	Total Load	12.00	6.00	12.00	6.00
Mezzar	Slab (1) 140	3.36	3.00	31.62	106.24	94.86		
Floor	Slab (2)							
	SDL	0.50	------	31.62	15.81			
	115mm Brick 2.5	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	2.60	1.77			
	Filled Conc Dia in r 232	1.01	------	2.60	2.64			
	Beam 800 WB 132	1.32		5.10	6.73			
	Beam 500 UB 82	0.82		18.60	15.25			
	Beam 460 UB 67	0.67						
	Cladding	0.15		28.25	4.24			
No of Similar floor including this			1	Total Load	152.68	94.86	164.68	100.86
Level 2	Slab (1) 160	3.84	6.50	31.62	121.42	205.53		
	Slab (2)							
	SDL	1.00	------	31.62	31.62			
	115mm Brick 3.5	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	4.70	6.44			
	Filled Conc Dia in r		------	4.70	0.00			
	Beam 700 WB 130	1.30		11.45	14.89			
	Beam 500 UB 82	0.82		0.00				
	Beam 460 UB 67	0.67		15.50	10.39			
	Cladding	0.15		61.02	9.15			
No of Similar floor including this			1	Total Load	193.90	205.53	358.59	306.39
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 x 400	6.14	------	11.30	69.43			
	Beam GB2 500 x 400	3.26						
	Pile Cap 800 x 800	15.36		1.00	15.36			
	Cladding	0.15		14.70	2.21			
	Pre-cast cladding 150	3.60		49.60	178.56			
No of Similar floor including this			1	Total Load	265.55	0.00	624.14	306.39
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	624.14	306.39

Total Short Term Live Load 214.47 Total Long Term Live Load 120.16

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S11/SB **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S11	23.00	15.00	1.00	23.00	15.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		44.29	6.64			
	M&E Plants							
No of Similar floor including this			1	Total Load	29.64	15.00	29.64	15.00
Mezzar	Slab (1) 160	3.84	3.00	31.62	121.42	94.86		
Floor	Slab (2)							
	SDL	0.50	------	31.62	15.81			
	115mm Brick 2.5	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	2.60	3.56			
	Filled Conc Dia in r		------	2.60	0.00			
	Beam 800 WB 132	1.32		11.30	14.92			
	Beam 500 UB 82	0.82		18.60	15.25			
	Internal Wall	1.00		12.75	12.75			
	Cladding	0.15		25.75	3.86			
No of Similar floor including this			1	Total Load	187.57	94.86	217.22	109.86
Level 2	Slab (1) 140	3.36	6.50	52.53	176.50	341.45		
	Slab (2)							
	SDL	1.00	------	52.53	52.53			
	115mm Brick 3.5	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	1.37	------	4.70	6.44			
	Filled Conc Dia in r		------	4.70	0.00			
	Beam 700 WB 130	1.30		10.30	13.39			
	Beam 800 WB 122	1.22		5.10	6.22			
	Beam 460 UB 67	0.67		20.95	14.04			
	Cladding	0.15		46.35	6.95			
No of Similar floor including this			1	Total Load	276.07	341.45	493.29	451.31
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	6.34	------	6.20	39.28			
	Beam GB2 500 X 400	3.46		5.10	17.63			
	Pile Cap 850 X 850	17.34		1.00	17.34			
	Cladding	0.15		14.42	2.16			
	Pre-cast cladding 150	3.60		30.24	108.86			
No of Similar floor including this			1	Total Load	185.28	0.00	678.56	451.31
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	678.56	451.31

Total Short Term Live Load 315.91 Total Long Term Live Load 174.52

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265
Subject Column S11/SC, S11/SD, S11/SE, S11/SF **Page no** Sheet 1 of
Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity Dead	Load Intensity Live	Area / Lengt	Total Load from Eleme Dead	Total Load from Eleme Live	Cumulative Load Dead	Cumulative Load Live
Roof	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Portal Frame @ S11	19.00	12.00	1.00	19.00	12.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		32.80	4.92			
	M&E Plants							
No of Similar floor including this			1	Total Load	23.92	12.00	23.92	12.00
Level 2	Slab (1) 140	3.36	6.50	41.82	140.52	271.83		
	Slab (2)							
	SDL	1.00	-------	41.82	41.82			
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column 250 SHS 9	0.68	-------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	-------	4.60	4.67			
	Beam 800 WB 122	1.22		5.30	6.47			
	Beam 410 UB 54	0.54		16.40	8.86			
	Beam 700 WB 132	1.32		8.20	10.82			
	Cladding	0.15		53.30	8.00			
No of Similar floor including this			1	Total Load	224.28	271.83	248.20	283.83
Level 1	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column		-------					
	Beam GB1 800 x 400	7.68	-------					
	Beam GB2 500 x 400	4.80		8.20	39.36			
	Pile Cap 850 x 850	17.34		1.00	17.34			
	Pre-cast cladding 150	3.60		44.28	159.41			
	Cladding							
No of Similar floor including this			1	Total Load	216.11	0.00	464.30	283.83
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	464.30	283.83
	Slab (1)							
	Slab (2)							
	SDL		-------					
	115mm Brick	3.00	-------					
	230mm Brick	5.60	-------					
	Column / Wall		-------					
	Beam x		-------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	464.30	283.83

Total Short Term Live Load 198.68 Total Long Term Live Load 108.73

241059

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265

Subject Column S11/SF **Page no** Sheet 1 of

Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S11	19.00	12.00	1.00	19.00	12.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		32.80	4.92			
	M&E Plants							
No of Similar floor including this			1	Total Load	23.92	12.00	23.92	12.00
Level 2	Slab (1) 140	3.36	6.50	41.82	140.52	271.83		
	Slab (2)							
	SDL	1.00	------	41.82	41.82			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 250 SHS 9	0.68	------	4.60	3.13			
	Filled Conc Dia in r 232	1.01	------	4.60	4.67			
	Beam 800 WB 122	1.22		5.30	6.47			
	Beam 410 UB 54	0.54		16.40	8.86			
	Beam 700 WB 132	1.32		8.20	10.82			
	Cladding	0.15		53.30	8.00			
No of Similar floor including this			1	Total Load	224.28	271.83	248.20	283.83
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		8.20	39.36			
	Pile Cap 900 X 2400	51.84		1.00	51.84			
	Pre-cast cladding 150	3.60		44.28	159.41			
	Cladding							
No of Similar floor including this			1	Total Load	250.61	0.00	498.80	283.83
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	498.80	283.83
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	498.80	283.83

Total Short Term Live Load 198.68 Total Long Term Live Load 108.73

LOAD TAKING SHEET

Job name: *Queensgate Redevelopment, Zone J* **Job No.** 2711265
Subject Column S11/SG **Page no** Sheet 1 of
Calculated by Jamil Khan **Reviewed By** Hamish Brookie **Date** 25/05/2004

Level	Element	Load Intensity		Area / Lengt	Total Load from Eleme		Cumulative Load	
		Dead	Live		Dead	Live	Dead	Live
Roof	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Portal Frame @ S11	11.00	5.00	1.00	11.00	5.00		
	Portal Frame b/w S2 & S3							
	Cladding	0.15		39.99	6.00			
	M&E Plants							
No of Similar floor including this			1	Total Load	17.00	5.00	17.00	5.00
Level 2	Slab (1) Stairs 250	6.00	4.00	20.91	125.46	83.64		
	Slab (2) canopy 10.5	0.25	0.25	35.70	9.00	8.93		
	SDL	1.00	------	20.91	20.91			
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column 310 UC 137	7.44	------	4.60	34.23			
	Filled Conc Dia in r 35	0.02	------	4.60	0.11			
	Beam 800 WB 122	1.22		5.10	6.22			
	Beam 410 UB 54	0.54		8.20	4.43			
	Beam 700 WB 132	1.32		4.10	5.41			
	Cladding	0.15		59.80	8.97			
No of Similar floor including this			1	Total Load	214.74	92.57	231.74	97.57
Level 1	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column		------					
	Beam GB1 800 X 400	7.68	------					
	Beam GB2 500 X 400	4.80		9.20	44.16			
	Pile Cap 900 X 2400	51.84		1.00	51.84			
	Pre-cast cladding 150	3.60		49.68	178.85			
	Cladding							
No of Similar floor including this			1	Total Load	274.85	0.00	506.59	97.57
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	x							
No of Similar floor including this			1	Total Load	0.00	0.00	506.59	97.57
	Slab (1)							
	Slab (2)							
	SDL		------					
	115mm Brick	3.00	------					
	230mm Brick	5.60	------					
	Column / Wall		------					
	Beam x		------					
	Beam x							
No of Similar floor including this			1	Total Load	0.00	0.00	506.59	97.57

Total Short Term Live Load 68.30 Total Long Term Live Load 37.03

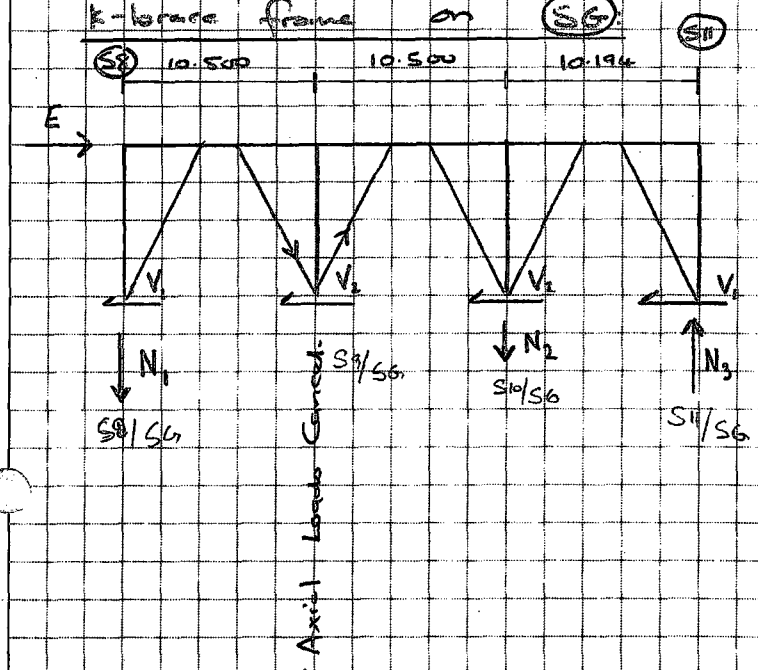
241059

Calculation Sheet

Job name: Queensgate Mall Redevelopment
Subject: E Loads at foundation ONLY.
By: Dean Saunders.

Job No: 2711265/109
Page No: 1 of
Date: 10-5-04

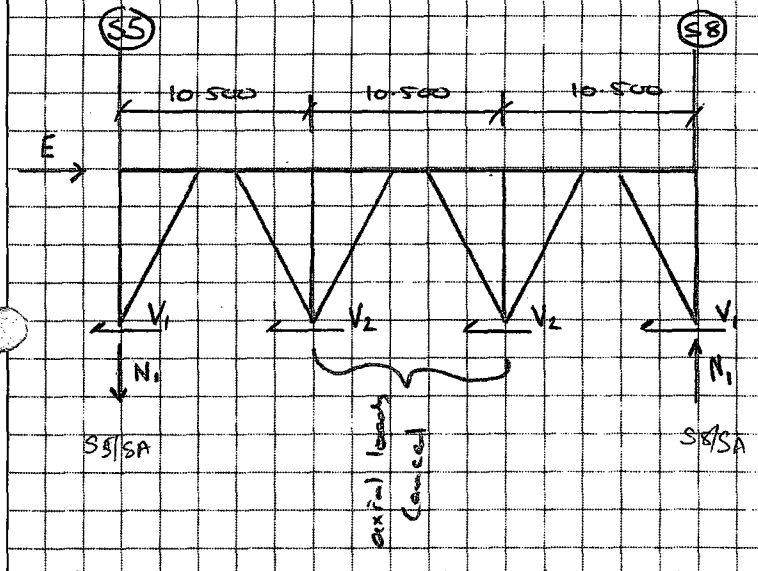
k-brace frame on (SG)



$V_1 = 625\,kN$
$V_2 = 1250\,kN$

NOTE TENSION OR COMPRESSION.
$N_1 = 701\,kN$
$N_2 = 21\,kN$
$N_3 = 722\,kN$

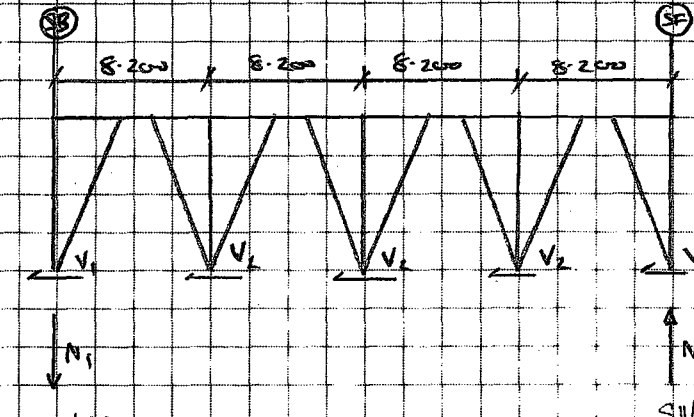
k-brace frame on (SA)



$V_1 = 625\,kN$
$V_2 = 1250\,kN$

NOTE TENSION OR COMPRESSION
$N_1 = 701\,kN$

k-brace frame on (S11)



$V_1 = 562\,kN$
$V_2 = 1124\,kN$

NOTE TENSION OR COMPRESSION
$N_1 = 806\,kN$

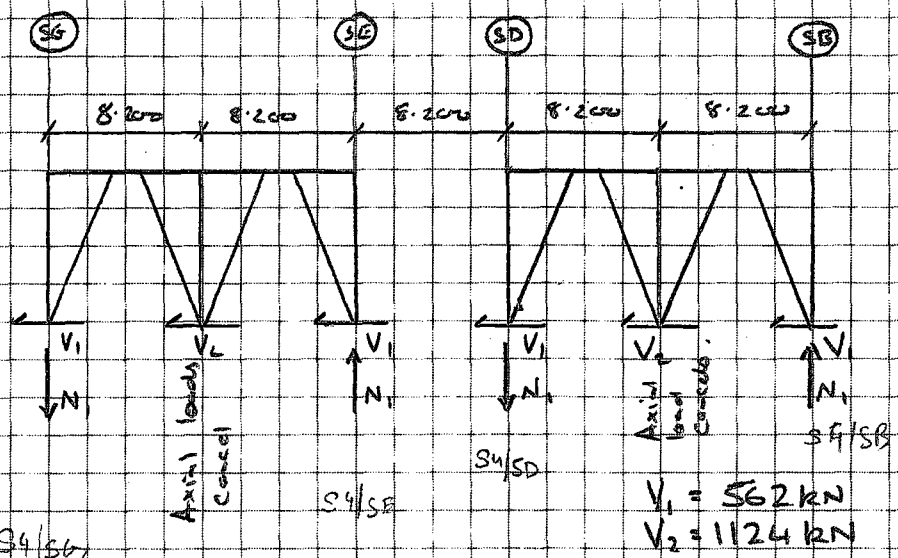
241059

Calculation Sheet

Job name: Queensgate Mall Redevelopment
Subject: E loads at foundation
By: Dave Saunders.

Job No: 2711265/109
Page No: 2 of
Date: 10-5-04

1c - brace frame on (S4)



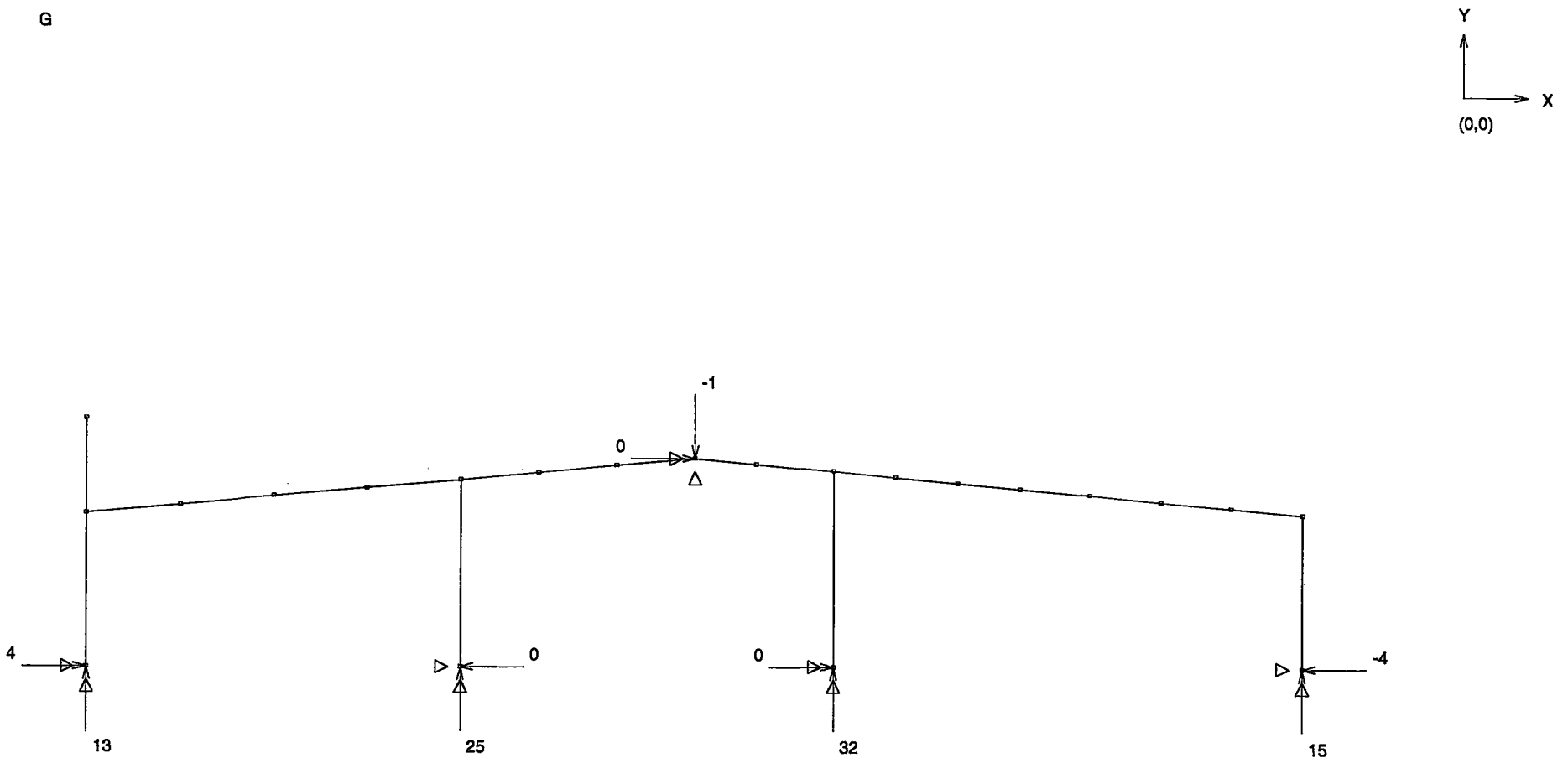
$V_1 = 562\,kN$

$V_2 = 1124\,kN$

NOTE TENSION AND COMPRESSION

$N_1 = 806\,kN$

Load cases:
■ 3 —— G

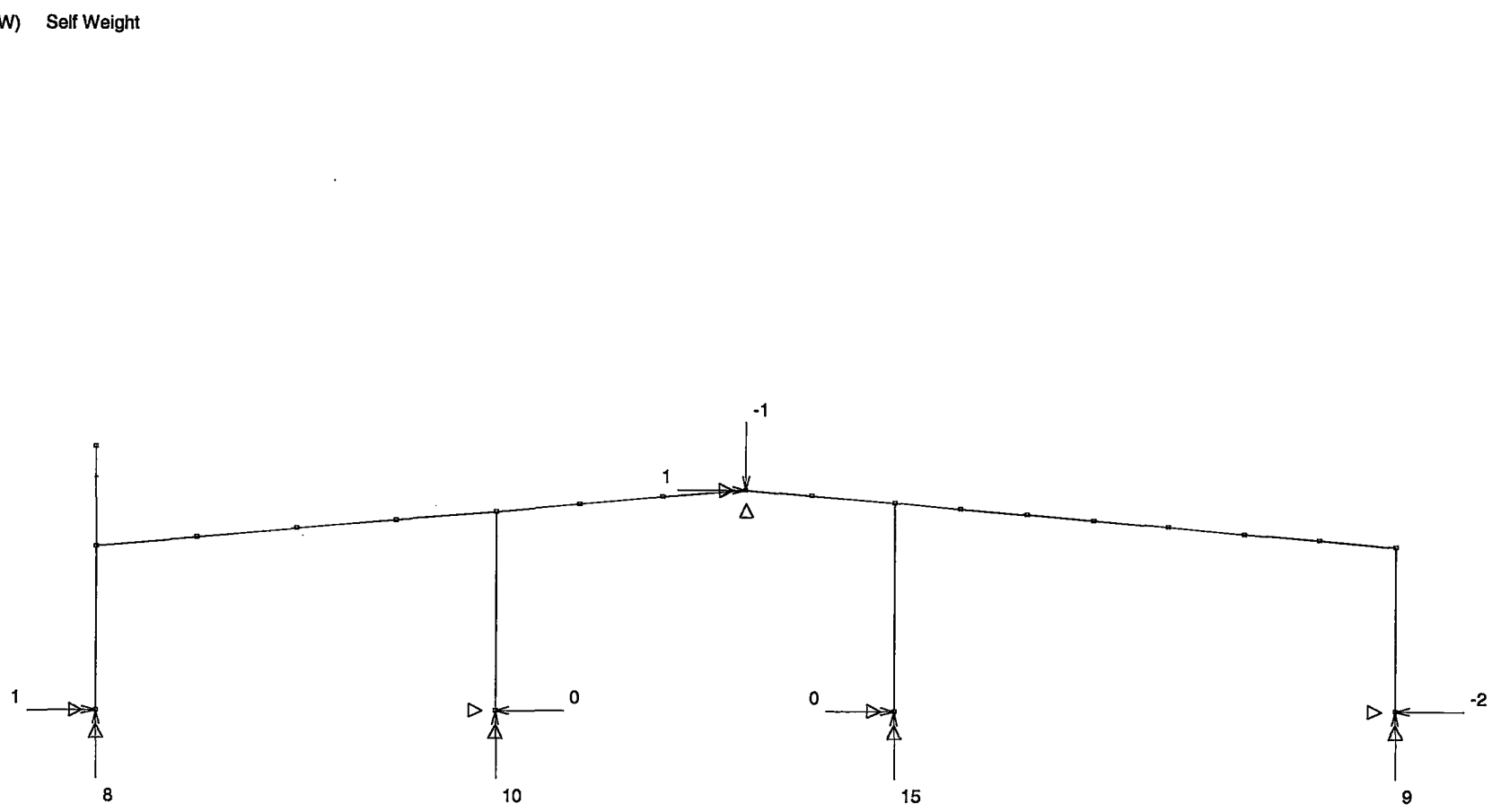


No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units: Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales: Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

241059
241059

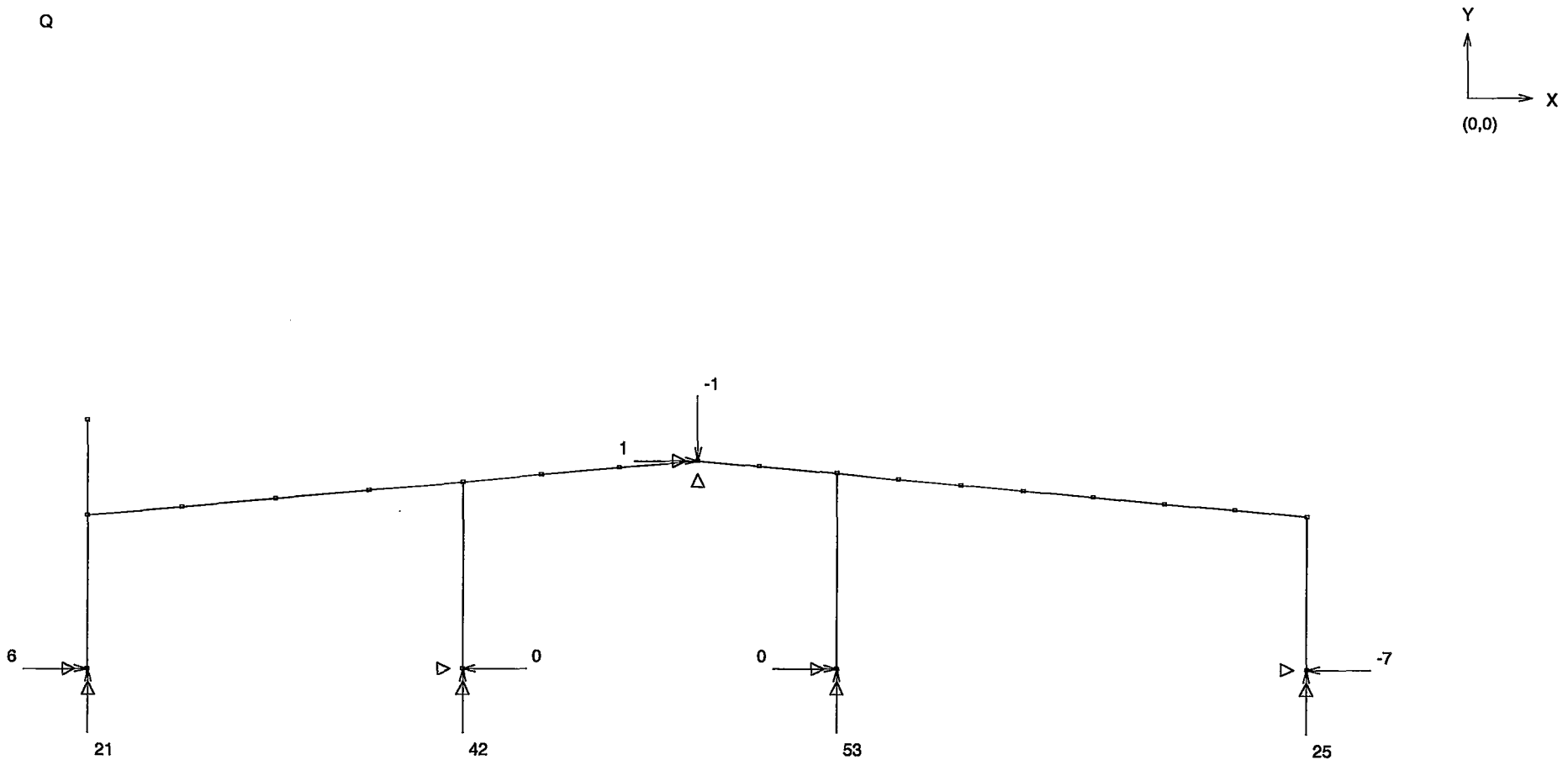
Load cases:
■ 4 ——— (SW) Self Weight



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 5 Q

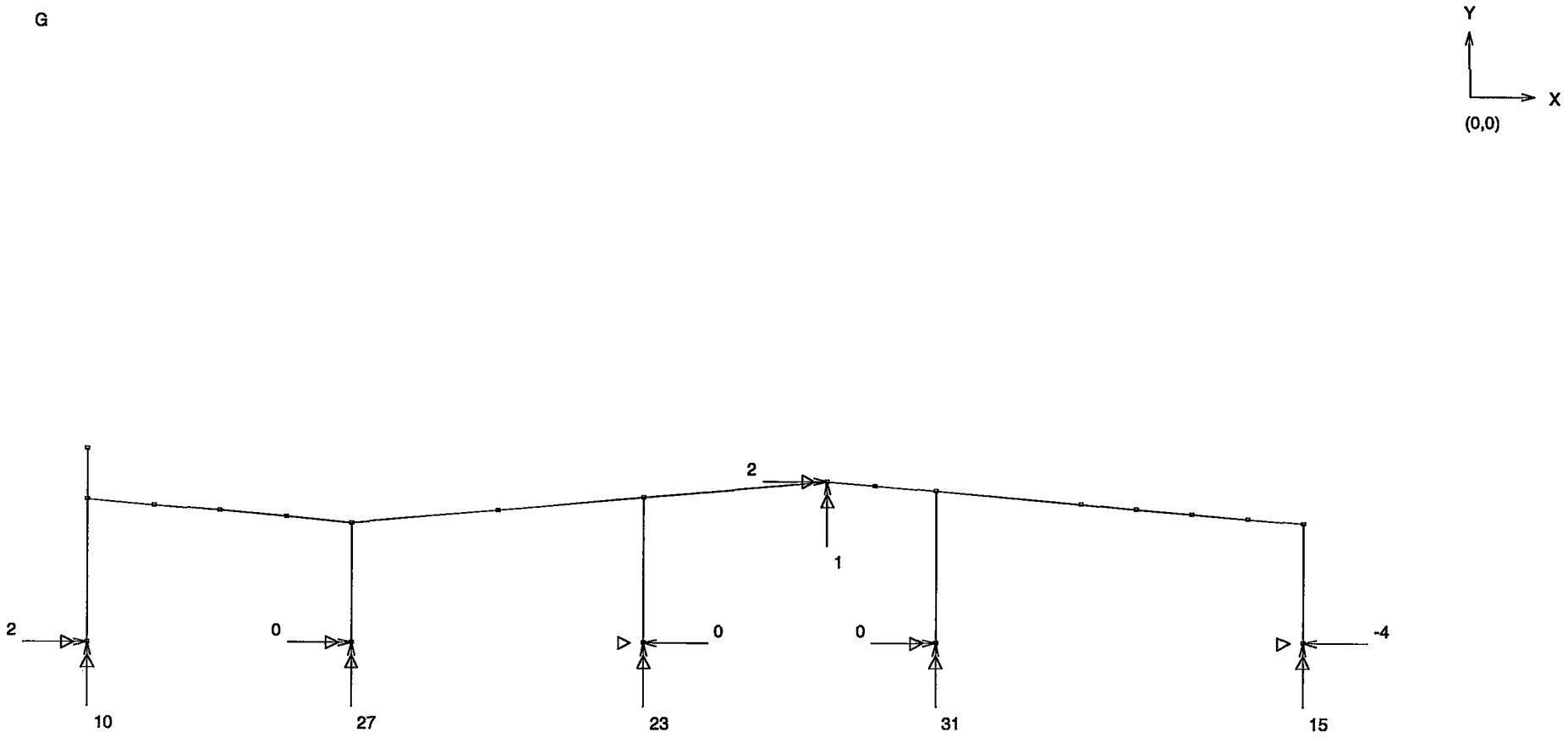


2A1059

No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

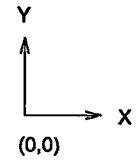
Load cases:
■ 3 ——— G

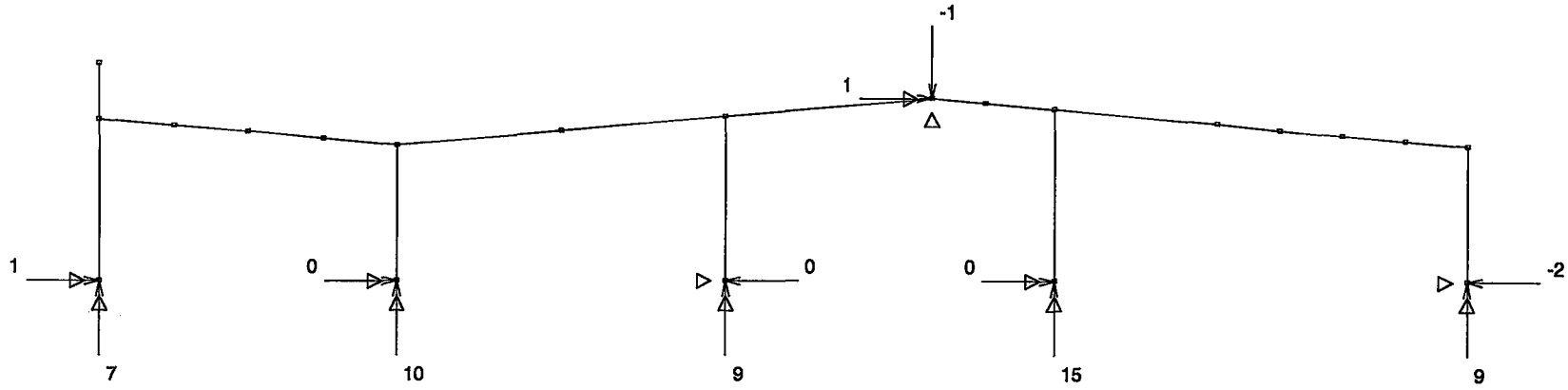


No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S5 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:360, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 4 ——— (SW) Self Weight



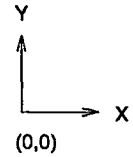


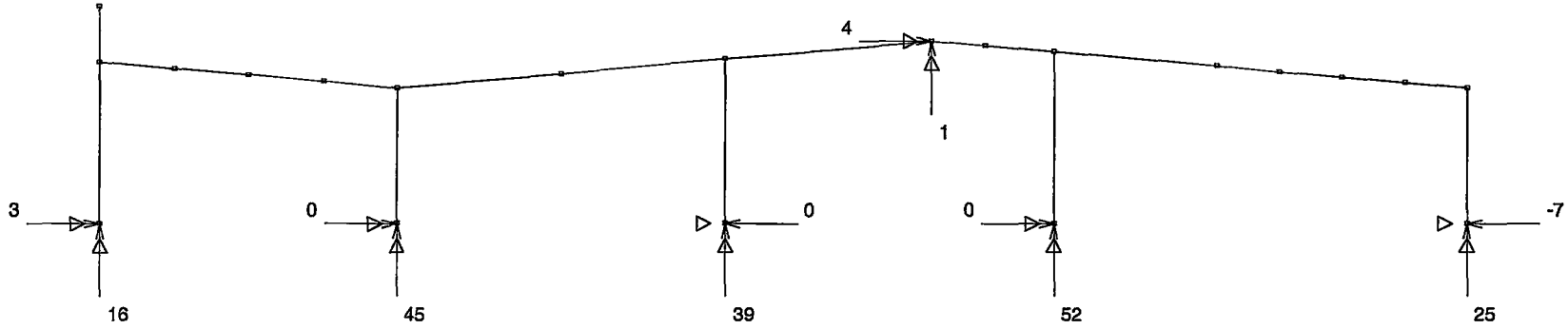
241059

No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S5 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:360, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 5 Q

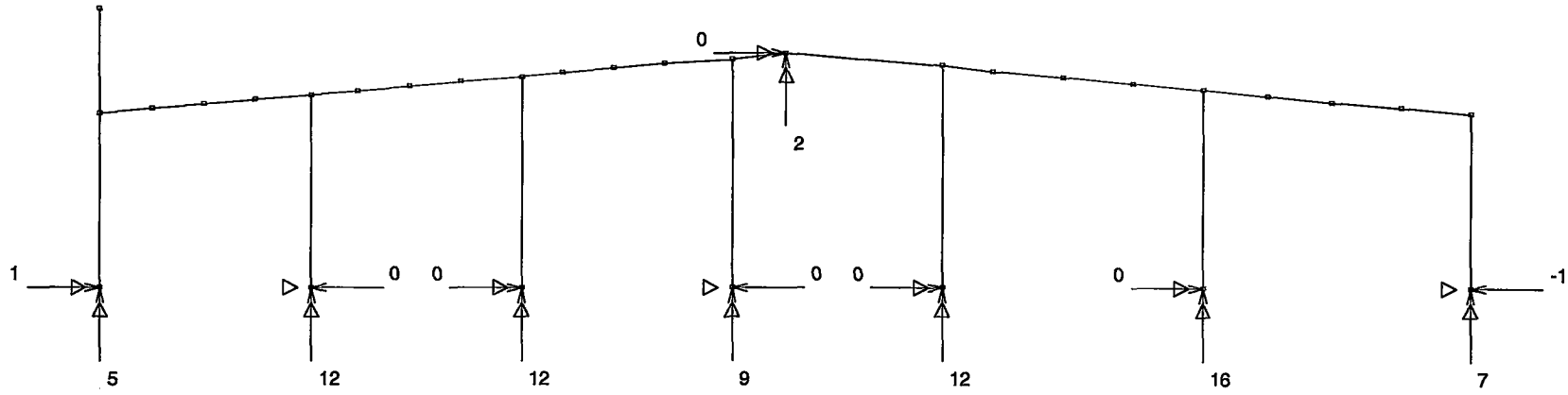




No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S5 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:360, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 3 ——— G

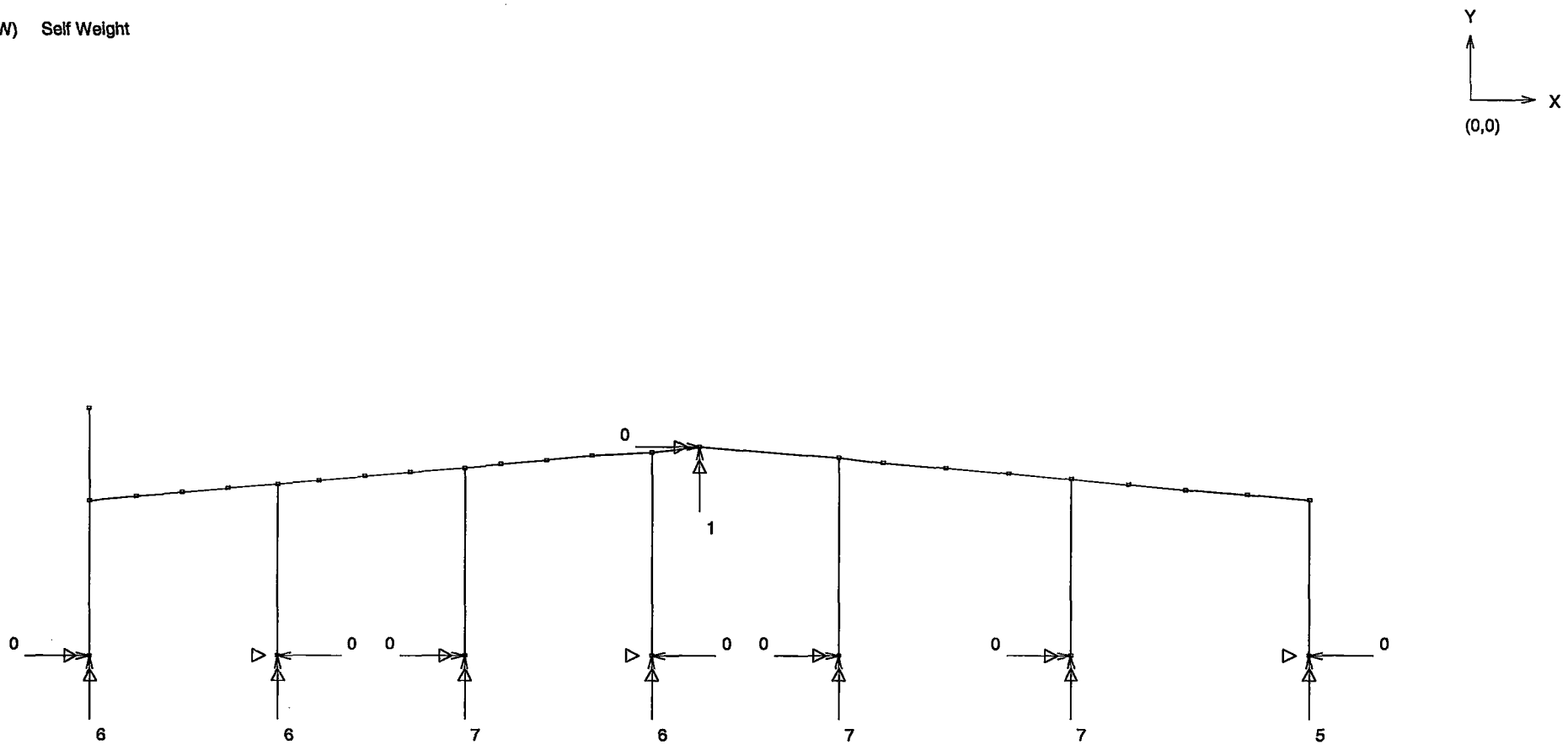


241059

Portal frame @ GL S11
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

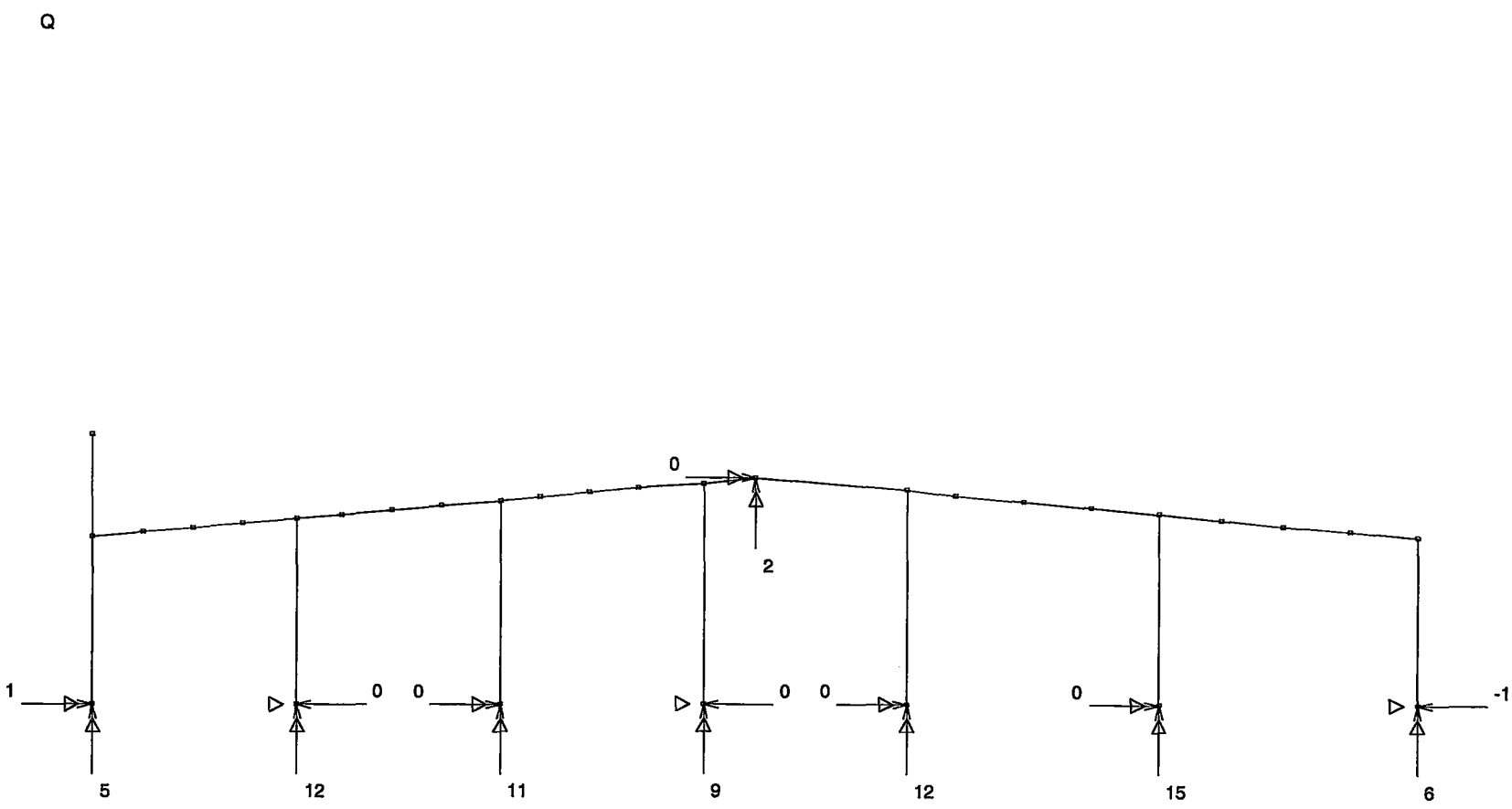
Load cases:
■ 4 ——— (SW) Self Weight



Portal frame @ GL S11
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

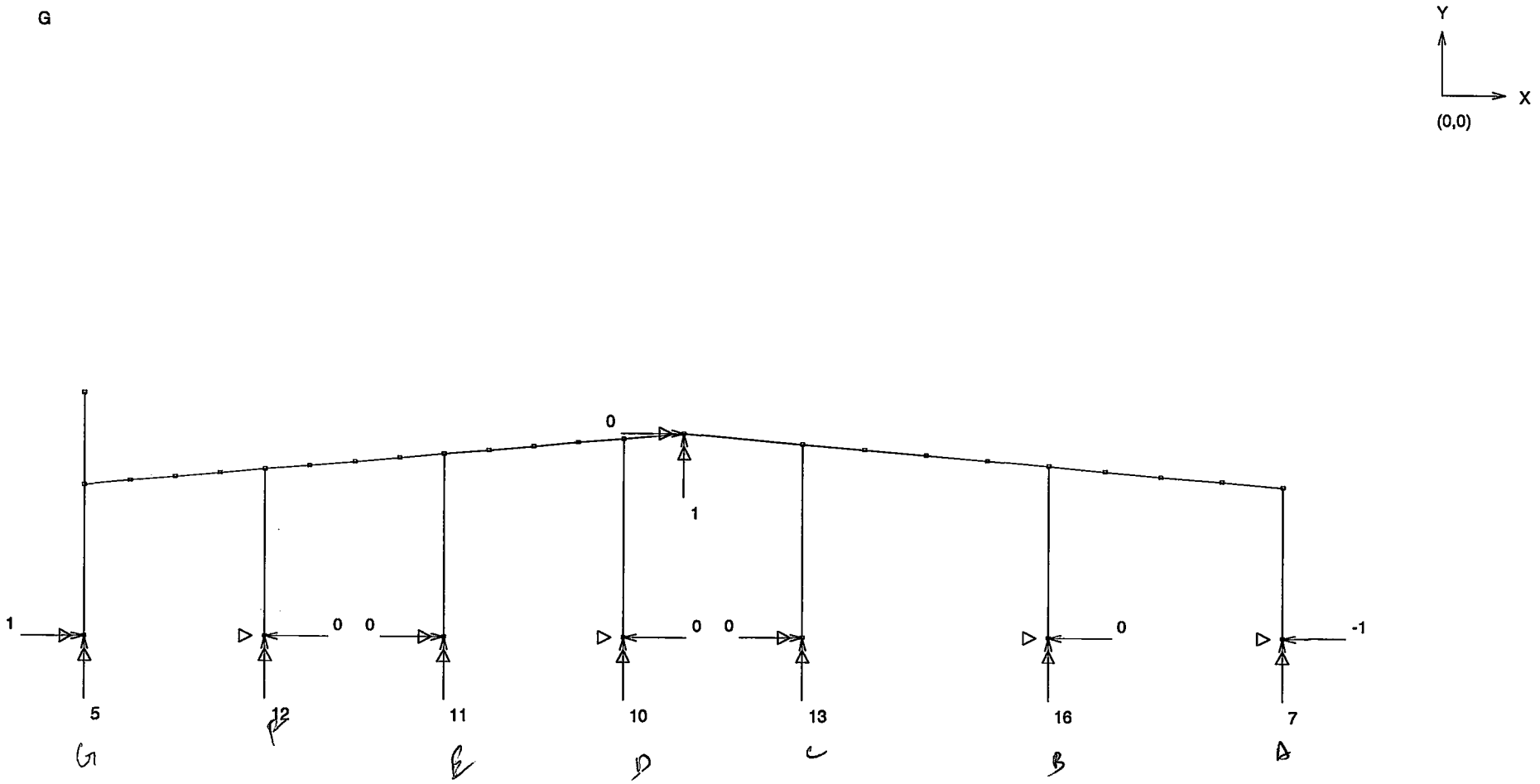
Load cases:
■ 5 —— Q



241059

Portal frame @ GL S11
No general restraint

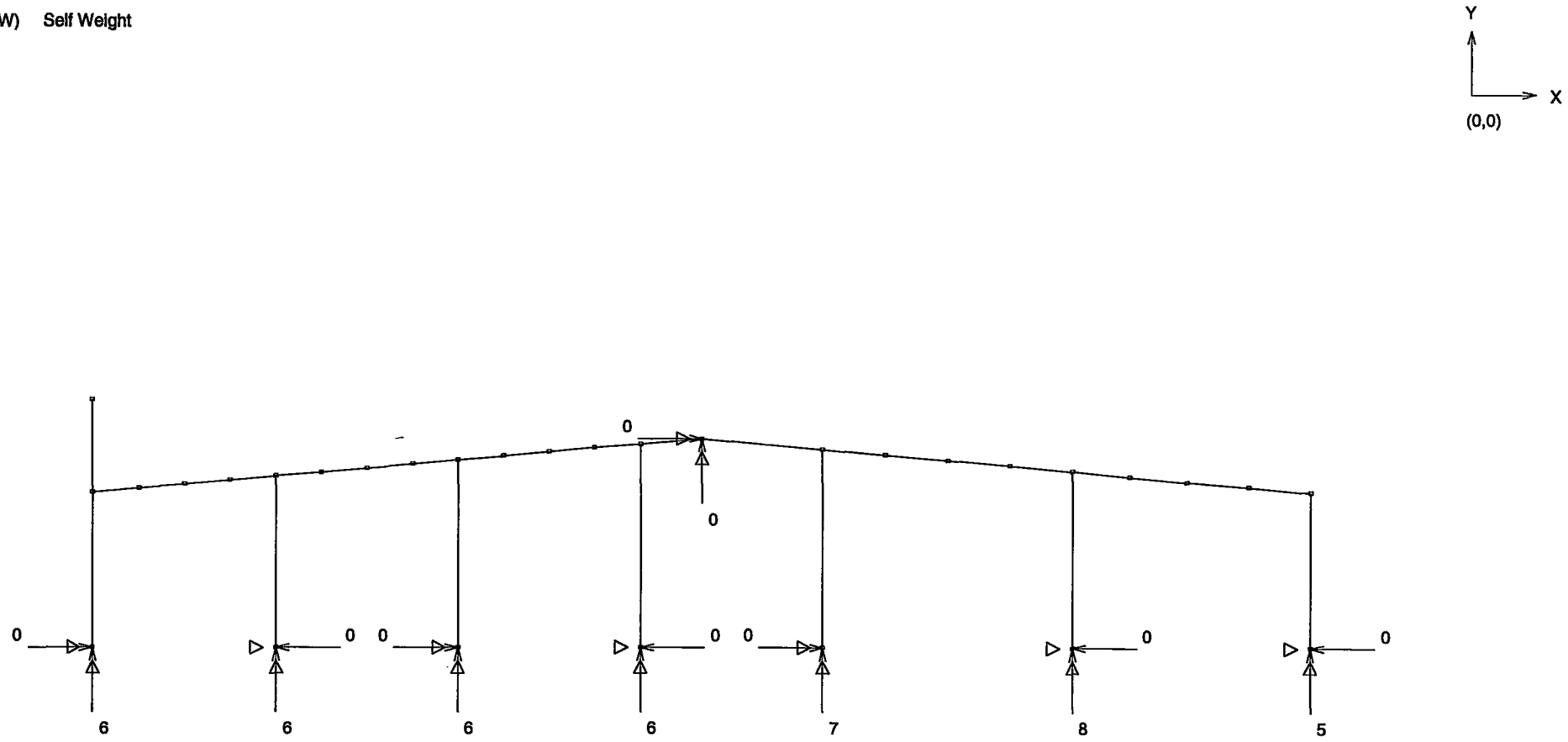
Load cases:
■ 3 ——— G



Portal Frame @ GL S1
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 4 ——— (SW) Self Weight

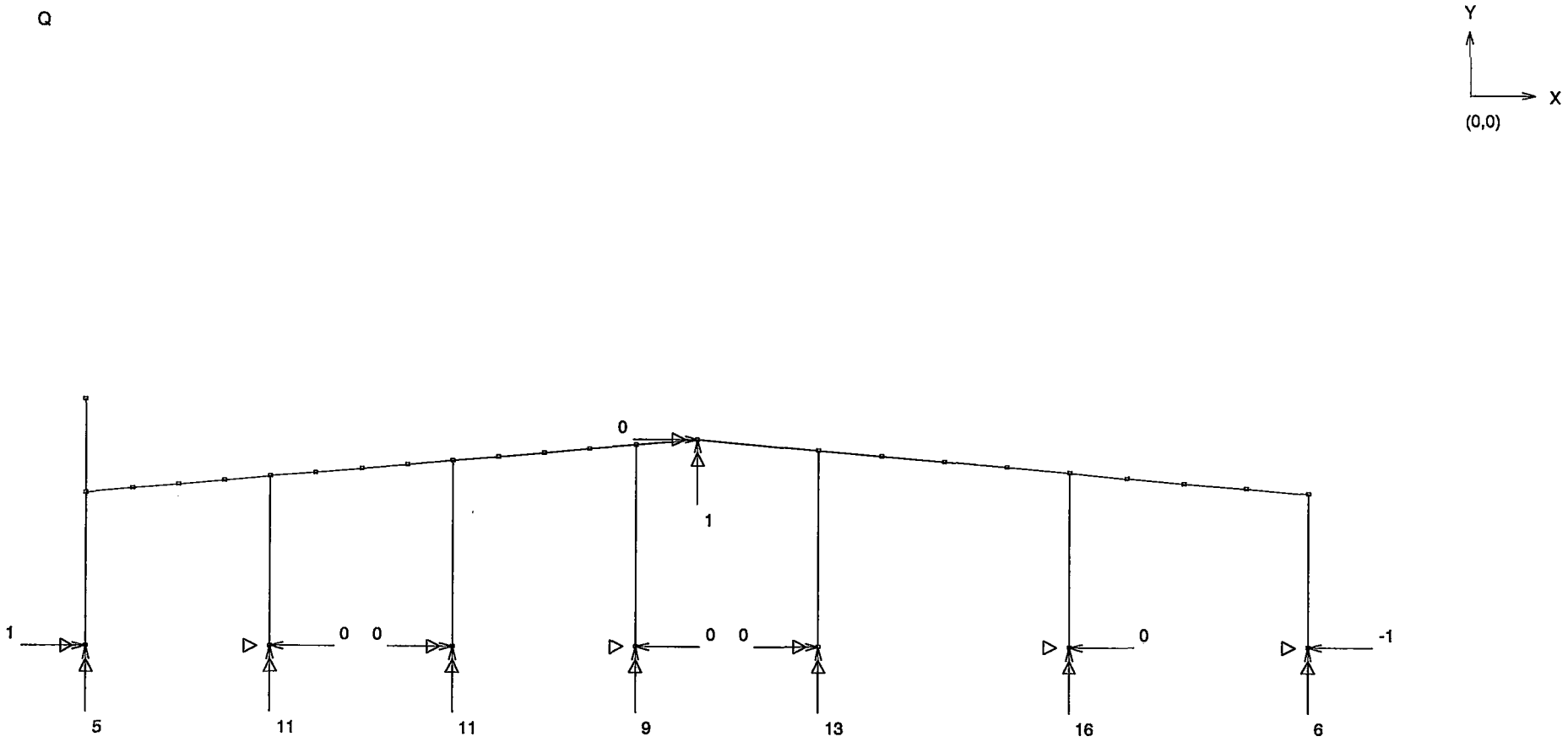


241059

Portal Frame @ GL S1
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

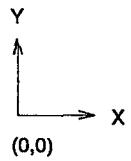
Load cases:
■ 5 ——— Q

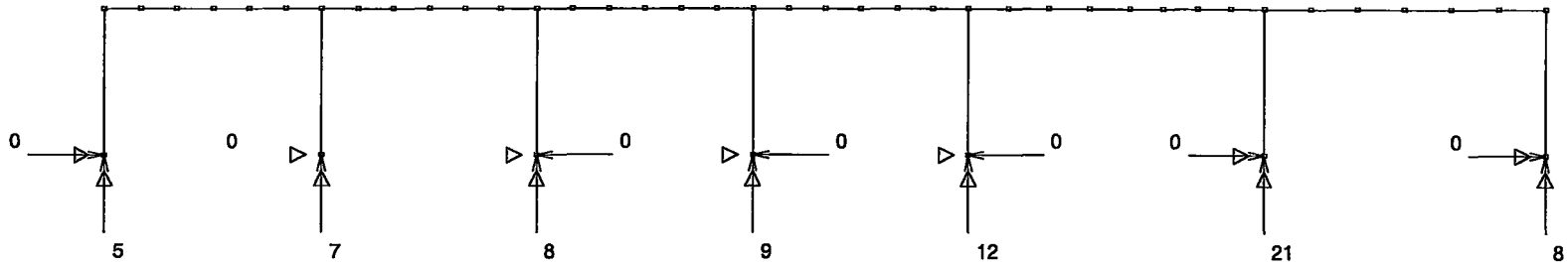


Portal Frame @ GL S1
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 4 ——— (SW) Self Weight





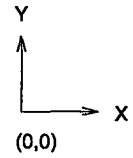
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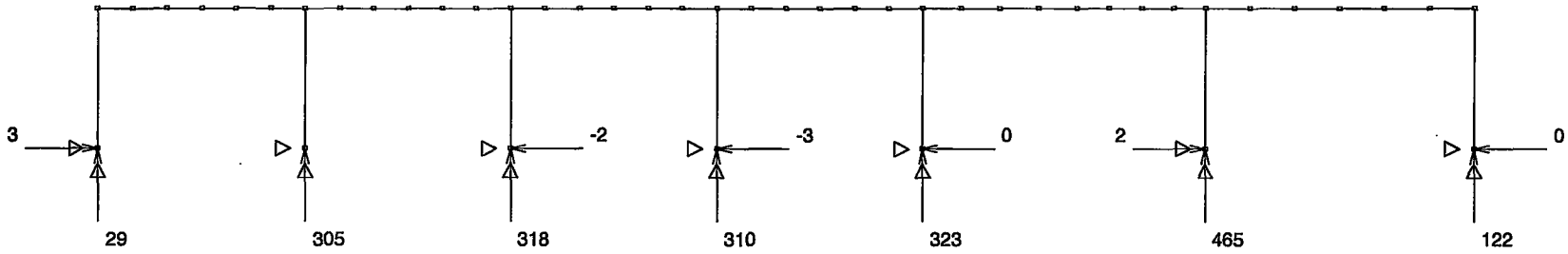
L2 frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 01b
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

J-25

Load cases:
■ 3 ——— G





L2 frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 01b
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Summary of Column Loads
For
Pile Design

241059

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J*

Subject **Summary of Column Loads with Combinations of SLS & ULS For Piles**

Calculat Jamil Khan **Reviewed By** Hamish Brookie

Job No. 2711265

Page no Sheet 1 of 3

Date 22/06/2004

S No.	Column Marks	Dead load	Live Load			E.Q. Load	Pile	SLS Combinations		ULS Gravity Load		Pile capacity	ϕP_U /	ULS Gravuty & Seismic Loads		Pile capacity	ϕP_U /	$\phi(-P_U)$ /
		G	Q	$\Psi_S Q$	$\Psi_L Q$	Eu	Arrangement	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	$1.2G+1.6Q$	ϕP_U (ϕ = 0.65)	(1.2G + 1.6Q)	$G+\Psi_U Q+Eo$	$G+\Psi_U Q-Eo$	ϕP_U (ϕ = 0.90)	(G + Q + E0)	(G + Q - E0)
1	S1/SA	267	160	112	62	---	2 - 300 Sq	379	328	328	576	1755	3.05	328		2430	7.40	
2	S1/S5	601	536	375	208	88	1 - 450 Sq	976	809	897	1578	1723	1.09	949		2385	2.51	
3	S1/SC	433	340	238	131	88	1 - 400 Sq	671	564	652	1063	1365	1.28	704		1890	2.68	
4	S1/SD	413	311	218	121	88	1 - 350 Sq	631	534	622	993	1040	1.05	674		1440	2.14	
5	S1/SE	420	320	224	124	88	1 - 350 Sq	644	543	631	1016	1040	1.02	684		1440	2.10	
6	S1/SF	426	308	216	119	---	1 - 350 Sq	642	545	545	1004	1040	1.04	545		1440	2.64	
7	S1/SF.5	119	33	23	11	---	1 - 300 Sq	142	130	130	195	878	4.50	130		1215	9.36	
8	S2/SA	568	512	358	193	---	1 - 450 Sq	926	761	761	1500	1723	1.15	761		2385	3.13	
9	S2/SB	706	803	562	321	---	2 - 350 Sq	1268	1027	1027	2132	2080	0.98	1027		2880	2.80	
10	S2/SC	616	700	490	256	---	2 - 350 Sq	1105	871	871	1858	2080	1.12	871		2880	3.31	
11	S2/SD	550	640	448	256	---	2 - 300 Sq	998	806	806	1683	1755	1.04	806		2430	3.02	
12	S2/SE	602	687	481	256	---	2 - 350 Sq	1083	858	858	1822	2080	1.14	858		2880	3.36	
13	S2/SF	570	640	448	256	---	1 - 450 Sq	1017	826	826	1707	1723	1.01	826		2385	2.89	
14	S2/SG	412	344	241	128	64	1 - 350 Sq	652	540	604	1044	1040	1.00	642		1440	2.24	
15	S3/SA	421	378	265	145	---	1 - 400 Sq	686	566	566	1110	1365	1.23	566		1890	3.34	
16	S3/SB	523	603	422	241	---	1 - 450 Sq	945	764	764	1592	1723	1.08	764		2385	3.12	
17	S3/SC	451	512	359	192	---	1 - 400 Sq	810	643	643	1361	1365	1.00	643		1890	2.94	
18	S3/SD	415	480	336	192	---	1 - 400 Sq	751	607	607	1266	1365	1.08	607		1890	3.11	
19	S3/SE	482	506	354	192	---	1 - 450 Sq	836	674	674	1387	1723	1.24	674		2385	3.54	
20	S3/SF	431	480	336	192	---	1 - 400 Sq	767	623	623	1285	1365	1.06	623		1890	3.03	
21	S3/SG	352	253	177	96	119	1 - 300 Sq	529	448	567	827	878	1.06	638		1215	1.90	
22	S4/SA	434	359	252	134	137	1 - 400 Sq	686	568	705	1096	1365	1.25	787		1890	2.40	
23	S4/SB	597	556	389	222	806	4 - Screw in	986	819	1625	1606	2080	1.30	2109	470	2880	1.37	4.82
24	S4/SC	566	495	347	177	---	1 - 450 Sq	913	743	743	1472	1723	1.17	743		2385	3.21	
25	S4/SD	496	442	310	177	806	3 - Screw in	806	673	1479	1303	1560	1.20	1963	616	2160	1.10	2.76
26	S4/SE	502	484	339	177	806	3 - Screw in	841	679	1485	1377	1560	1.13	1969	611	2160	1.10	2.79
27	S4/SF	460	442	310	177	---	1 - 400 Sq	770	637	637	1260	1365	1.08	637		1890	2.97	
28	S4/SG	643	334	234	116	1066	4 - Screw in	877	758	1824	1306	2080	1.59	2464	947	2880	1.17	2.39
29	S4/SH	612	349	244	131	260	1 - 400 Sq	856	742	1002	1292	1365	1.06	1158		1890	1.63	
30	S4/SK	595	145	102	52	260	1 - 350 Sq	697	647	907	946	1040	1.10	1063		1440	1.36	
31	S5/SA	541	452	317	171	838	3 - Screw in	858	712	1550	1373	1560	1.14	2053	629	2160	1.05	2.71
32	S5/SB	636	710	497	284	---	2 - 350 Sq	1133	920	920	1899	2080	1.10	920		2880	3.13	
33	S5/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
34	S5/SD	479	565	395	226	---	1 - 450 Sq	875	705	705	1479	1723	1.16	705		2385	3.38	
35	S5/SE	516	599	419	224	---	1 - 450 Sq	935	740	740	1577	1723	1.09	740		2385	3.22	

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J*

Subject **Summary of Column Loads with Combinations of SLS & ULS For Piles**

Calculat Jamil Khan Reviewed By Hamish Brookie

Job No. 2711265

Page no Sheet 2 of 3

Date 22/06/2004

S No.	Column Marks	Dead load	Live Load			E.Q. Load	Pile	SLS Combinations		ULS Gravity Load		Pile capacity	ϕP_U /	ULS Gravuty & Seismic Loads		Pile capacity	ϕP_U /	$\phi(-P_U)$ /
		G	Q	$\Psi_s Q$	$\Psi_L Q$	Eu	Arrangement	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	$1.2G+1.6Q$	$\phi P_U (\phi=0.65)$	$(1.2G+1.6Q)$	$G+\Psi_U Q+Eo$	$G+\Psi_U Q-Eo$	$\phi P_U (\phi=0.90)$	$(G+Q+E0)$	$(G+Q-E0)$
36	S5/SF	496	560	392	224	---	1 - 450 Sq	887	719	719	1490	1723	1.16	719		2385	3.32	
37	S4.8/SG	452	405	283	156	---	1 - 400 Sq	735	608	608	1190	1365	1.15	608		1890	3.11	
38	S4.8/SH	515	512	358	205	---	1 - 450 Sq	874	720	720	1438	1723	1.20	720		2385	3.31	
39	S4.8/SK	752	269	188	102	---	1 - 400 Sq	941	855	855	1333	1365	1.02	855		1890	2.21	
40	S5.4/SG	476	501	351	189	---	1 - 450 Sq	827	665	665	1374	1723	1.25	665		2385	3.59	
41	S6/SA	541	452	317	171	---	1 - 450 Sq	858	712	712	1373	1723	1.25	712		2385	3.35	
42	S6/SB	636	710	497	284	---	2 - 350 Sq	1133	920	920	1899	2080	1.10	920		2880	3.13	
43	S6/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
44	S6/SD	479	565	395	226	---	1 - 450 Sq	875	705	705	1479	1723	1.16	705		2385	3.38	
45	S6/SE	516	599	419	224	---	1 - 450 Sq	935	740	740	1577	1723	1.09	740		2385	3.22	
46	S6/SF	496	560	392	224	---	1 - 450 Sq	887	719	719	1490	1723	1.16	719		2385	3.32	
47	S6/SG	441	441	309	158	---	1 - 400 Sq	750	599	599	1235	1365	1.11	599		1890	3.15	
48	S6/SK	1011	580	406	224	---	2 - 400 Sq	1416	1235	1235	2140	2730	1.28	1235		3780	3.06	
49	S6.5/SG	388	396	277	158	---	1 - 400 Sq	665	546	546	1099	1365	1.24	546		1890	3.46	
50	S7/SA	541	452	317	171	137	1 - 450 Sq	858	712	849	1373	1723	1.25	931		2385	2.56	
51	S7/SB	636	710	497	284	---	2 - 350 Sq	1133	920	920	1899	2080	1.10	920		2880	3.13	
52	S7/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
53	S7/SD	479	565	395	226	---	1 - 450 Sq	875	705	705	1479	1723	1.16	705		2385	3.38	
54	S7/SE	516	599	419	224	---	1 - 450 Sq	935	740	740	1577	1723	1.09	740		2385	3.22	
55	S7/SF	496	560	392	224	---	1 - 450 Sq	887	719	719	1490	1723	1.16	719		2385	3.32	
56	S7/SG	441	441	309	158	---	1 - 400 Sq	750	599	599	1235	1365	1.11	599		1890	3.15	
57	S7/SK	750	528	370	205	42	1 - 450 Sq	1119	955	997	1744	1723	0.99	1022		2385	2.33	
58	S7.5/SG	388	396	277	158	---	2 - 300 Sq	665	546	546	1099	1755	1.60	546		2430	4.45	
59	S8/SA	669	546	382	208	838	4 - Screw ir	1051	878	1716	1676	2080	1.24	2218	463	2880	1.30	4.90
60	S8/SB	798	801	560	320	---	2 - 400 Sq	1359	1119	1119	2239	2730	1.22	1119		3780	3.38	
61	S8/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
62	S8/SD	502	565	395	226	---	2 - 300 Sq	898	728	728	1507	1755	1.16	728		2430	3.34	
63	S8/SE	539	599	419	224	---	2 - 300 Sq	958	763	763	1605	1755	1.09	763		2430	3.18	
64	S8/SF	512	560	392	224	---	2 - 300 Sq	903	735	735	1509	1755	1.16	735		2430	3.30	
65	S8/SG	565	397	278	141	961	3 - Screw ir	843	706	1667	1313	1560	1.19	2243	832	2160	0.96	2.05
66	S8/SK	605	282	197	106	260	1 - 400 Sq	802	711	971	1176	1365	1.16	1127		1890	1.68	
67	S9/SA	804	643	450	247	658	2 - 350 Sq	1255	1052	1710	1994	2080	1.04	2104	1	2880	1.37	0.00
68	S9/SB	972	898	629	359	---	2 - 400 Sq	1601	1331	1331	2603	2730	1.05	1331		3780	2.84	
69	S9/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
70	S9/SD	502	565	395	226	---	2 - 300 Sq	898	728	728	1507	1755	1.16	728		2430	3.34	

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J*

Subject **Summary of Column Loads with Combinations of SLS & ULS For Piles**

Calculat Jamil Khan **Reviewed By** Hamish Brookie

Job No. 2711265

Page no Sheet 3 of 3

Date 22/06/2004

S No.	Column Marks	Dead load	Live Load			E.Q. Load	Pile	SLS Combinations		ULS Gravity Load		Pile capacity	ϕP_U /	ULS Gravuty & Seismic Loads		Pile capacity	ϕP_U /	$\phi(-P_U)$ /
		G	Q	$\Psi_s Q$	$\Psi_L Q$	Eu	Arrangement	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	1.2G + 1.6Q	ϕP_U (ϕ = 0.65)	(1.2G + 1.6Q)	$G+\Psi_U Q+Eo$	$G+\Psi_U Q-Eo$	ϕP_U (ϕ = 0.90)	(G + Q + E0)	(G + Q - E0)
71	S9/SE	539	599	419	224	---	2 - 300 Sq	958	763	763	1605	1755	1.09	763		2430	3.18	
72	S9/SF	512	560	392	224	---	2 - 300 Sq	903	735	735	1509	1755	1.16	735		2430	3.30	
73	S9/SG	650	319	223	119	183	1 - 400 Sq	874	770	953	1291	1365	1.06	1062		1890	1.78	
74	S9/SK	341	10	7	4	---	1 - 300 Sq	347	344	344	424	878	2.07	344		1215	3.53	
75	S10/SA	804	643	450	247	658	2 - 350 Sq	1255	1052	1710	1994	2080	1.04	2104	1	2880	1.37	0.00
76	S10/SB	972	898	629	359	---	2 - 400 Sq	1601	1331	1331	2603	2730	1.05	1331		3780	2.84	
77	S10/SC	534	618	433	226	---	1 - 450 Sq	966	760	760	1629	1723	1.06	760		2385	3.14	
78	S10/SD	502	565	395	226	---	2 - 300 Sq	898	728	728	1507	1755	1.16	728		2430	3.34	
79	S10/SE	516	599	419	224	---	1 - 450 Sq	935	740	740	1577	1723	1.09	740		2385	3.22	
80	S10/SF	496	560	392	224	---	1 - 450 Sq	887	719	719	1490	1723	1.16	719		2385	3.32	
81	S10/SG	650	319	223	119	204	1 - 400 Sq	874	770	974	1291	1365	1.06	1096		1890	1.72	
82	S11/SA	624	306	214	120	137	1 - 400 Sq	839	744	881	1239	1365	1.10	963		1890	1.96	
83	S11/SB	679	451	316	175	806	3 - Screw in	994	853	1659	1536	1560	1.02	2143	437	2160	1.01	3.90
84	S11/SC	464	284	199	109	118	1 - 350 Sq	663	573	691	1011	1040	1.03	762		1440	1.89	
85	S11/SD	464	284	199	109	118	1 - 350 Sq	663	573	691	1011	1040	1.03	762		1440	1.89	
86	S11/SE	464	284	199	109	---	1 - 350 Sq	663	573	573	1011	1040	1.03	573		1440	2.51	
87	S11/SF	499	284	199	109	924	3 - Screw in	697	608	1532	1053	1560	1.48	2086	871	2160	1.04	1.95
88	S11/SG	507	98	68	37	905	3 - Screw in	575	544	1449	764	1560	2.04	1992	904	2160	1.08	1.88

Pile Size	Ultimate Pile Compression Load	Ultimate Pile Tension Load	Total No of Piles of each Size
300 Sq	1350 KN	0 KN	23
350 Sq	1600 KN	0 KN	24
400 Sq	2100 KN	0 KN	25
450 Sq	2650 KN	0 KN	28
Screw in Piles	800 KN	630 KN	33

Oversterngth Factor for Seismic ULS Loads	1.60
ϕ Factor for (1.2G+1.6Q) Load Combinatic	0.65
ϕ Factor for ($G+Q+E_U$) Load Combination	0.90
Total No of Piles	133

Calculation Sheet

Job name:	**Queensgate Shopping Mall Redevelopment, Wellington**			Job No:	**2711265**
Subject:	**Zone J, Pile Ultimate End Bearing Resistance**			Page No:	**1 of 1**
Design By	Jamil ur Rahman Khan	Reviewed By	**Hamish Brookie**	Date:	26/05/2004

Pile Type ***Pre-cast Reinforced Driven Piles*** **Type of Soil** ***Sand & Gravel***

The ultimate end bearing resistance of short driven piles may be estimated by using the following relation

$q_b = (40 * D_b / B) * N < 400\ N$ (Sand & Gravel)

$q_b = (40 * D_b / B) * N < 300\ N$ (non-plastic silt)

Pile Size	Q_u (KN)	q_u (MPa)	B' (mm)	B (m)	Db (m)	N	q_b (KPa)	q_b (MPa)
300 Sq	1350	15.00	300	0.27	4.00	38	15200	15.20
350 Sq	1600	13.06	350	0.31	4.00	33	13200	13.20
400 Sq	2100	13.13	400	0.35	4.00	33	13200	13.20
450 Sq	2650	13.09	450	0.40	4.00	33	13200	13.20

Where,

B'	Width of pile
B	Equivalent Diameter of Pile
D_b	Depth of embedment of pile into bearing stratum
Q_U	Ultimate Load carrying capacity of a single pile
q_U	Ultimate stresses in Pile in (MPa)
q_b	Ultimate end bearing resistance

8-5

PILE FOUNDATION
DESIGN NOTES

Office of the Chief Designing Engineer
Civil Engineering Division
Ministry of Works and Development
Wellington
NEW ZEALAND

Civil Engineering Division
CDP 812/B : 1981

241059

(a) Static Cone Penetration Test

It has been widely reported that for small diameter driven piles the cone point bearing resistance, q_c, will provide a reliable estimate of the ultimate end bearing capacity of driven piles. This approach is often preferable to laboratory and theoretical determination of bearing capacity, due to the sensitivity of the analysis to the soil friction angle. Allowing for embedment into the bearing stratum the bearing capacity is given by:

$$q_p = q_c \frac{Db}{10\,B} \leq q_c \qquad (4\text{-}11)$$

Kerisel (1961) has shown however that a correction should be made where the pile diameter exceeds 500 mm.

Readings greater than 10 MPa should be taken as 10 MPa while smaller readings should be averaged over the soil from four pile diameters above the pile tip to one diameter below. Care should be taken where the bearing stratum is thin and where necessary the approach outlined in 4.2.2 should be used.

Skin friction as measured by the sleeve of a Dutch Cone Penetrometer will provide a conservative estimate for driven piles. Alternatively, skin friction may be determined by estimating $\emptyset'$ and K_o and applying the approach outlined in section 4.2.3.

End bearing capacity for bored piles is estimated using the factors given in section 4.2.2. Skin friction for bored piles is estimated by working back from the skin friction of a driven pile using the factors given in Table 3.

(b) Standard Penetration Test

The ultimate end bearing resistance of driven piles may be estimated conservatively as:

$$q_p = \frac{40\,Db}{B} N \leq 400\,N \text{ (sand and gravel)} \qquad (4\text{-}12)$$
$$\text{or} \leq 300\,N \text{ (non-plastic silt)}$$

Where q_p is given in kPa.

Reasonable results are obtained for short piles using this formula. However for longer piles it will over-estimate the bearing resistance and a depth correction factor should be applied. An improved estimate is obtained by multiplying the N value by the correction factor C_N as proposed by Peck (1974).

$$C_N = 0.77 \log_{10} \frac{2000}{P_o} \qquad (4\text{-}13)$$

Where P_o is the effective overburden pressure in kPa. Corrections should also be made where the effects of pore pressure may influence the N-value as outlined in Table 2.

For bored piles, the correction factors based on relative density as defined in section 4.2.2 should be applied to estimate end bearing capacity.

PILE CHECK FOR LATERAL EARTHQUAKE FORCES

Developed by Jamil ur Rahman Khan

Project Ref. No. 2711265

Project Title: Queensgate Mall, Zone J

Design By: Jamil

I) INPUT DATA

a) Floor Data

First Floor Level	103.00 m
Natural Ground Level	103.00 m
Ground Water Table	103.00 m
Lowest Basement Slab Soffit Level	103.00 m
Typical Grid line Spacing	10.50 m

b) Soil Data

Soil density	18 KN/m^3
ϕ	30
Cu	15 KN/m^2
Es	2.40E+05 KN/m^2
Ep	2.60E+07 KN/m^2

c) Pile Data

Pile Dia (d in m)		No. of Piles	Pile Length (L in m)	Pile Capacity (P in KN)	Pile Re-bars
0.30	Square	23	8	1350	8DH20
0.35	Square	24	8	1600	8DH20
0.40	Square	25	8	2100	8DH25
0.45	Square	28	8	2650	8DH28
0.20	Circle	33	8	800	---

d) Soil Parameters

Ko	0.50
Ka	0.33
Kp	3.00
Live Load Surcharge	0.00 KN/m^2
Density of Water	9.81 KN/m^3

II) LATERAL UNBALANCE FORCE

a) From Surcharge Live Load	0 KN
b) From Soil	0 KN
c) From Ground water	0 KN
d) Seismic Base Shear	13000 KN
Total Force	13000 KN

Shear Stiffness of Pile Group {Σ (n * d^2)}	15.72 m^2
Required Factor of Safety for Induced Moment	1.50

The foundation piles will be subjected to the lateral force due to earthqukae forces. 'This lateral forces has to be resisted by the structure itself without transmitting any force to the any neighbouring structure. The lateral force will be resisted by

1) Frictional force between lowest slab & soil
2) Passive resistance against pile cap & ground beams.
3) Pile lateral load resistance capacity.

1) Frictinal resistance Between Lowest Slab & Soil

Total Floor are for Lowest Slab	5800 m^2	Tickness of Slab	150 mm	μ = 0.25
Total Frictional Resistance	5220 KN			

2) Passive Reistance Against Pilecaps and Ground Beams

Total No of Pile caps	107	Wdith	0.80 m	Depth	0.80 m	Kp	3.0
Total Ground Beams	4	Length	103.00 m	Depth	0.50 m	Kp	3.0
Total Passive Resistance	823 KN						

3) Lateral load on Piles

iii) LATERAL FORCE (H) & DISPLACEMENT (ΔH) OF SINGLE PILE (Fixed - Head Conditions)

Pile Size (m)	n * A	H_U (KN)	L/d	K_R	I_{PF}	ΔH (Hori. Disp.) (mm)	H = Total Force * (d^2 / Σ (n * d^2))
0.30	2.07	40	27	1.79E-05	12.00	0.25	K_P = (E_P * I_P) / (E_s * L^4)
0.35	2.94	54	23	3.31E-05	10.00	0.28	I_{PF} Obtained from Graphs
0.40	4.00	71	20	5.64E-05	9.00	0.33	$\Delta H = I_{PF} * H / (Es * L)$
0.45	5.67	90	18	9.04E-05	20.00	0.93	f = (Hu / (1.5γK_pd))^.5
0.20	1.04	14	40	2.08E-06	20.00	0.14	Mu = Hu *(1.5d + 0.67f) / 2
							M_{UR} is Ultimate Structural Capacity at P_U

iv) ULTIMATE MOMENT (M_u) INDUCED BY LATERAL FORCE ON SINGLE PILE (Fixed - Head Conditions)

Pile Dia (m)	Pile Re-bars	H_U (KN)	f	M_U (KN-m)	P_U (KN)	M_{UR} (KN-m)	FOS = M_{ur} / M_u	Remarks
0.30	8DH20	40	1.64	31	1350	120	3.89	> 1.50 Okay
0.35	8DH20	54	1.91	49	1600	175	3.57	> 1.50 Okay
0.40	8DH25	71	2.18	73	2100	250	3.42	> 1.50 Okay
0.45	8DH28	90	2.46	104	2650	350	3.36	> 1.50 Okay
0.20	---	14	0.86	6	800	10	1.64	> 1.50 Okay

241059

Design
Of
Pile Caps

241059

Calculation Sheet

Job name:

Subject:

By:

Job No:

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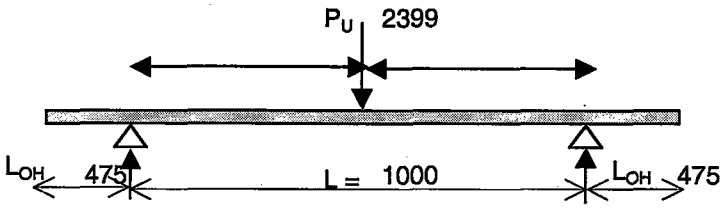
24059

Pilecap Typ	Pilecap Dimensions L	B	H	Pile typ	P_u KN	$M_u = P_u(0.05)$ KN-m	l_a mm	$A_s = \frac{M_u}{\phi l_a f_y}$ mm²	Provided Reinforcem
PC1A	800	800	800	350Sq	1440	72	630	269	4DH16
PC1B	800	800	800	400Sq	1890	94.5	630	355	4DH16
PC1C	850	850	1000	450Sq	2385	119.25	830	444	4DH16
PC1D	800	800	800	300Sq	1215	60.75	630	227.8	4DH16
PC1E	1000	800	800	350Sq	1440	72	630	269	4DH16
PC1F	1000	800	800	400Sq	1890	94.5	630	355	4DH16
PC1F	1000	800	1000	450Sq	2385	119.25	830	444	4DH16

Calculation Sheet

Job name: **Queensgate Shopping Mall Redevelopment, Wellington** Job No: **2711265**
Subject: **Zone J, Pilecap Design** Page No: **1 of 1**
Design By Jamil ur Rahman Khan Reviewed By **Hamish Brookie** Date: 18/05/2004

Pilecape Data			**Pile Data**	
Type		**PC2A**	Type	RC Driven Square
Length	L	1850 mm	No of piles in a Group	2
Width	b	700 mm	Size	**300** mm
Depth	h	**1000** mm	Geotechnical Capacity	1350 KN
Central Span	$L_{Central}$	900 mm	Ultimate Capacity	878 KN
Overhang Span	L_{OH}	475 mm	Over strength Capacity	1215 KN
Material data				
Design Concrete Strength	f'_c	**25** MPa	Vertical Shear Link Size	**16** mm
Concrete Density	γ_c	24 KN/m^3	Vertical Shear Link Sp s_v	**275** mm
Reinforcement Yield Strength	f_y	**500** MPa	Horizontal Shear Link Size	**16** mm
Flexural Reinforcement Size		**25** mm	Horizontal Shear Link Sp s_{vh}	**275** mm



Analysis

Maximum Allowable Eccentricity	e	**50** mm	Maximum Column Load	P_u	2399 KN
Effective Central Span	L_e	1000 mm	-ve Moment	$-M_u$	2 KN-mm
Clear Concrete cover	Cc	75	+ve Moment	$+M_u$	602 KN-mm
Effective Depth	d	896.5 mm	Shear	V_u	1207 KN

Flexure Design

$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	1.26
$\rho = 1/m \{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0026
Required As = ρ bd	1629 mm^2	Required No of Re-bars	3.32 Nos
Provided As	1963 mm2 Okay	Provided No of Re-bars	**4** Nos Okay
ρ_w = As / (b d)	0.003		

Shear Design

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			1.92 MPa Okay
$v_n - v_c$			0.21 MPa
$v_c = 1/7(3.5\text{-}2.5M^*/(V^* d))((f'_c)^{0.5}+120\rho_w(V^* d/M^*)) < 0.5(f'_c)^{0.5}$			1.71 MPa
$(A_v/s_v)_{mim} = b / fy$	1.40 mm^2/mm	$(A_{vh}/s_{vh})_{mim} = b / fy$	1.40 mm^2/mm
$(A_v/s_v)_{Provided}$	1.46 mm^2/mm	$(A_{vh}/s_{vh})_{Provided}$	1.46 mm^2/mm
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			1.04 MPa Okay

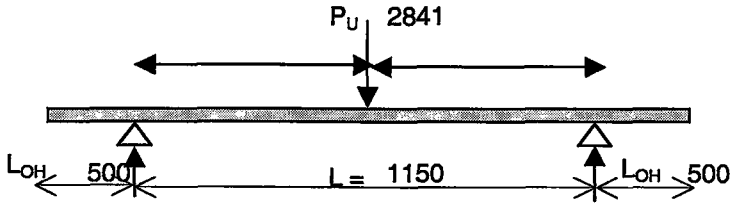
Design Summary

Bottom Flexural Reinforcement	4 DH25	Top Reinforcement	4 DH16
Vertical Shear Links	DH16 - 275	Horizontal Shear Links	DH16 - 275

Calculation Sheet

Job name: **Queensgate Shopping Mall Redevelopment, Wellington** Job No: **2711265**
Subject: **Zone J, Pilecap Design** Page No: **1 of 1**
Design By Jamil ur Rahman Khan Reviewed By Hamish Brookie Date: 18/05/2004

Pilecape Data			**Pile Data**	
Type		**PC2B**	Type	RC Driven Square
Length	L	2050 mm	No of piles in a Group	2
Width	b	800 mm	Size	**350** mm
Depth	h	**1000** mm	Geotechnical Capacity	1600 KN
Central Span	$L_{Central}$	1050 mm	Ultimate Capacity	1040 KN
Overhang Span	L_{OH}	500 mm	Over strength Capacity	1440 KN
Material data				
Design Concrete Strength	f'_c	**25** MPa	Vertical Shear Link Size	**16** mm
Concrete Density	γ_c	24 KN/m^3	Vertical Shear Link Sp s_v	**250** mm
Reinforcement Yield Strength	f_y	**500** MPa	Horizontal Shear Link Size	**16** mm
Flexural Reinforcement Size		**28** mm	Horizontal Shear Link Sp s_{vh}	**250** mm



Analysis					
Maximum Allowable Eccentricity	e	**50** mm	Maximum Column Load	P_u	2841 KN
Effective Central Span	L_e	1150 mm	-ve Moment	$-M_u$	2 KN-mm
Clear Concrete cover	Cc	75	+ve Moment	$+M_u$	820 KN-mm
Effective Depth	d	895 mm	Shear	V_u	1430 KN

Flexure Design

$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	1.50
$\rho = 1/m\,\{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0031
Required As = ρ bd	2237 mm^2	Required No of Re-bars	3.63 Nos
Provided As	2463 mm2 Okay	Provided No of Re-bars	**4** Nos Okay
ρ_w = As / (b d)	0.003		

Shear Design

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			2.00 MPa Okay
$v_n - v_c$			0.47 MPa
$v_c = 1/7(3.5\text{-}2.5M^*/(V^*d))((f'_c)^{0.5}+120\rho_w(V^*d/M^*)) < 0.5(f'_c)^{0.5}$			1.53 MPa
$(A_v/s_v)_{mim} = b / fy$	1.60 mm^2/mm	$(A_{vh}/s_{vh})_{mim} = b / fy$	1.60 mm^2/mm
$(A_v/s_v)_{Provided}$	1.61 mm^2/mm	$(A_{vh}/s_{vh})_{Provided}$	1.61 mm^2/mm
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			1.01 MPa Okay

Design Summary

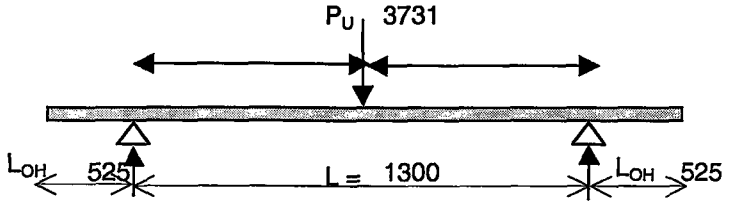
Bottom Flexural Reinforcement	4 DH28	Top Reinforcement	4 DH16
Vertical Shear Links	DH16 - 250	Horizontal Shear Links	DH16 - 250

241059

Calculation Sheet

Job name: **Queensgate Shopping Mall Redevelopment, Wellington** Job No: **2711265**
Subject: **Zone J, Pilecap Design** Page No: **1 of 1**
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Pilecape Data			**Pile Data**	
Type		**PC2C**	Type	RC Driven Square
Length	L	2250 mm	No of piles in a Group	2
Width	b	900 mm	Size	**400** mm
Depth	h	**1000** mm	Geotechnical Capacity	2100 KN
Central Span	$L_{Central}$	1200 mm	Ultimate Capacity	1365 KN
Overhang Span	L_{OH}	525 mm	Over strength Capacity	1890 KN
Material data				
Design Concrete Strength	f'_c	**25** MPa	Vertical Shear Link Size	**16** mm
Concrete Density	γ_c	24 KN/m^3	Vertical Shear Link Sp s_v	**200** mm
Reinforcement Yield Strength	f_y	**500** MPa	Horizontal Shear Link Size	**16** mm
Flexural Reinforcement Size		**28** mm	Horizontal Shear Link Sp s_{vh}	**200** mm



Analysis					
Maximum Allowable Eccentricity	e	**50** mm	Maximum Column Load	P_u	3731 KN
Effective Central Span	L_e	1300 mm	-ve Moment	$-M_u$	3 KN-mm
Clear Concrete cover	Cc	75	+ve Moment	$+M_u$	1217 KN-mm
Effective Depth	d	895 mm	Shear	V_u	1879 KN

Flexure Design

$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	1.99
$\rho = 1/m\,\{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0042
Required As = ρ bd	3365 mm^2	Required No of Re-bars	5.46 Nos
Provided As	3695 mm2 Okay	Provided No of Re-bars	**6** Nos Okay
ρ_w = As / (b d)	0.005		

Shear Design

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			2.33 MPa Okay
$v_n - v_c$			0.94 MPa
$v_c = 1/7(3.5 - 2.5M^*/(V^* d))((f'_c)^{0.5} + 120\rho_w(V^* d/M^*)) < 0.5(f'_c)^{0.5}$			1.39 MPa
$(A_v/s_v)_{mim} = b / fy$	1.80 mm^2/mm	$(A_{vh}/s_{vh})_{mim} = b / fy$	1.80 mm^2/mm
$(A_v/s_v)_{Provided}$	2.01 mm^2/mm	$(A_{vh}/s_{vh})_{Provided}$	2.01 mm^2/mm
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			1.12 MPa Okay

Design Summary

Bottom Flexural Reinforcement	6 DH28	Top Reinforcement	6 DH16
Vertical Shear Links	DH16 - 200	Horizontal Shear Links	DH16 - 200

Calculation Sheet

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Subject: **Zone J, Pilecap Design** Page No: **1 of 2**
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Pilecape Data

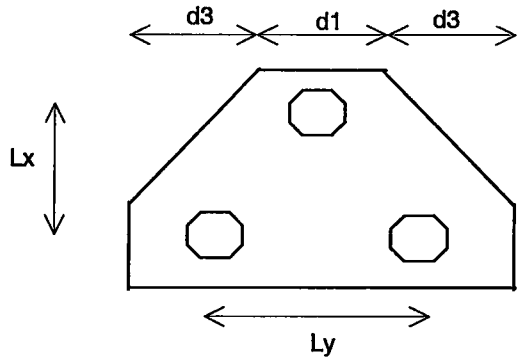
Type		PC3A
Dimension 1	d1	1000 mm
Dimension 2	d2	1300 mm
Dimension 3	d3	750
Depth	h	**1000** mm
Span in X Direction	Lx	1300 mm
Span in Y Direction	L_Y	1500 mm

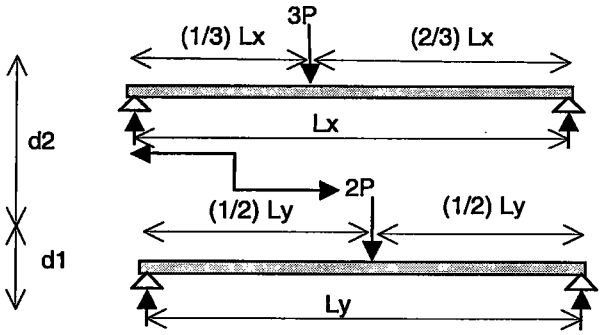
Pile Data

Type	Screw in Piles
No of piles in a Group	3
Size	**219** mm
Geotechnical Capacity	800 KN
Ultimate Capacity	520 KN
Over strength Capacity	720 KN

Material data

Design Concrete Strength	f'_c	**25** MPa	Vertical Shear Link Size	**13** mm
Concrete Density	γ_c	24 KN/m^3	Vertical Shear Link Sp s_v	**250** mm
Reinforcement Yield Strength	f_y	**500** MPa	Horizontal Shear Link Size	**13** mm
Flexural Reinforcement Size in X-Dir		**25** mm	Horizontal Shear Link Sp s_{vh}	**250** mm
Flexural Reinforcement Size in Y-Dir		**25** mm		





Analysis

Maximum Column Load (3P)	P_u	2045 KN	Moment in X-Dir.	M_{ux}	636 KN-mm
Maximum Allowable Eccentricity	e	**50** mm	Shear in X-Dir.	V_{ux}	1364 KN
Effective Span in X-Direction	L_{ex}	1400 mm	Moment in Y-Dir.	M_{uy}	545 KN-mm
Effective Span in X-Direction	L_{ey}	1600 mm	Shear in Y-Dir.	V_{uy}	682 KN
Clear Concrete cover	Cc	75			

Flexure Design in X-Direction

Effective Depth in X-dir d	887.5 mm	Effective Width in X-dir b	1500 mm
$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	0.63
$\rho = 1/m \{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0025
Required As = ρ bd	3328 mm^2	Required No of Re-bars	6.78 Nos
Provided As	3436 mm2 Okay	Provided No of Re-bars	**7** Nos Okay
ρ_w = As / (b d)	0.0026		

Shear Design in X-Direction

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			1.02 MPa Okay
$v_c = 1/7(3.5\text{-}2.5M^*/(V^* d))((f'_c)^{0.5}+120\rho_w(V^* d/M^*)) < 0.5(f'_c)^{0.5}$			1.74 MPa
$v_n - v_c$			No Shear R/f Req.
$(A_v/s_v)_{mim} = b / fy$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{mim} = b / fy$	No Shear R/f Req.
$(A_v/s_v)_{Provided}$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{Provided}$	No Shear R/f Req.
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			No Shear R/f Req.

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Calculation Sheet

Job name:	**Queensgate Shopping Mall Redevelopment, Wellington**	Job No:	**2711265**
Subject:	**Zone J, Pilecap Design**	Page No:	**2 of 2**
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		Date:	18/05/2004

Flexure Design in Y-Direction

Effective Depth in X-dir d	912.5 mm	Effective Width in X-dir b	1000 mm
$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	0.77
$\rho = 1/m \{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0025
Required As = ρ bd	2281 mm^2	Required No of Re-bars	4.65 Nos
Provided As	2945 mm2 Okay	Provided No of Re-bars	**6** Nos Okay
ρ_w = As / (b d)	0.0032		

Shear Design in Y-Direction

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			0.75 MPa Okay
$v_c = 1/7(3.5\text{-}2.5M^*/(V^*d))((f'_c)^{0.5}+120\rho_w(V^*d/M^*)) < 0.5(f'_c)^{0.5}$			1.00 MPa
$v_n - v_c$			No Shear R/f Req.
$(A_v/s_v)_{mim} = b / fy$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{mim} = b / fy$	No Shear R/f Req.
$(A_v/s_v)_{Provided}$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{Provided}$	No Shear R/f Req.
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			No Shear R/f Req.

Design Summary

Bottom X-Dir Flexural Reinforcement	7 DH25	Top X-Dir Reinforcement	7 DH13
Bottom Y-Dir Flexural Reinforcement	6 DH25	Top Y-Dir Reinforcement	6 DH13
Vertical Shear Links		Horizontal Shear Links	

Calculation Sheet

Job name: **Queensgate Shopping Mall Redevelopment, Wellington** Job No: **2711265**
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Pilecape Data

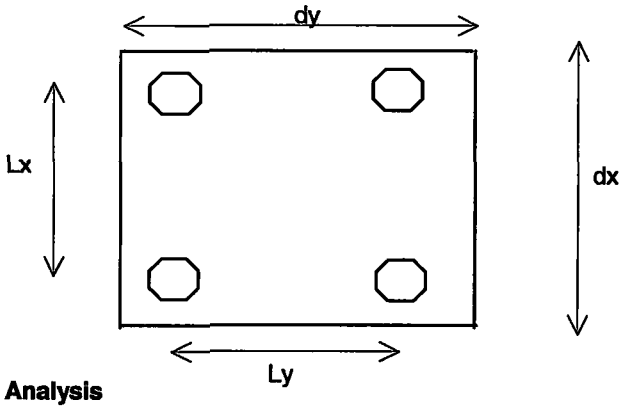
Type		PC4A
Dimension 1	dx	2500 mm
Dimension 2	dy	2500 mm
Depth	h	1000 mm
Span in X Direction	Lx	1500 mm
Span in Y Direction	Ly	1500 mm

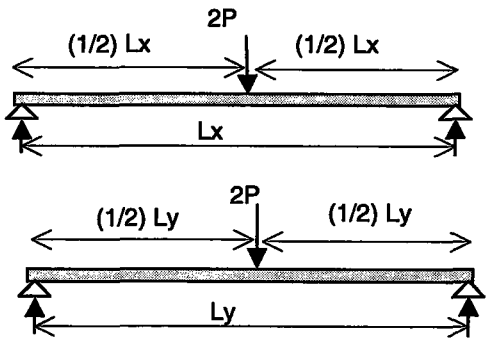
Pile Data

Type	Screw in Piles
No of piles in a Group	4
Size	219 mm
Geotechnical Capacity	800 KN
Ultimate Capacity	520 KN
Over strength Capacity	720 KN

Material data

Design Concrete Strength	f'_c	25 MPa	Vertical Shear Link Size	16 mm
Concrete Density	γ_c	24 KN/m^3	Vertical Shear Link Sp s_v	250 mm
Reinforcement Yield Strength	f_y	500 MPa	Horizontal Shear Link Size	16 mm
Flexural Reinforcement Size in X-Dir		25 mm	Horizontal Shear Link Sp s_{vh}	250 mm
Flexural Reinforcement Size in Y-Dir		25 mm		





Analysis

Maximum Column Load in X-Dir (2P)		1365 KN	Moment in X-Dir.	M_{ux}	558 KN-mm
Maximum Column Load in Y-Dir (2P)		1365 KN	Shear in X-Dir.	V_{ux}	683 KN
Maximum Allowable Eccentricity	e	50 mm	Moment in Y-Dir.	M_{uy}	558 KN-mm
Clear Concrete cover	Cc	75 mm	Shear in Y-Dir.	V_{uy}	683 KN

Flexure Design in X-Direction

Effective Depth in X-dir d	912.5 mm	Effective Width in X-dir b	2500 mm
$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	0.32
$\rho = 1/m\,\{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0025
Required As = ρ bd	5703 mm^2	Required No of Re-bars	11.62 Nos
Provided As	5890 mm2 Okay	Provided No of Re-bars	12 Nos Okay
ρ_w = As / (b d)	0.0026		

Shear Design in X-Direction

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			0.30 MPa Okay
$v_c = 1/7(3.5\text{-}2.5M^*/(V^* d))((f'_c)^{0.5}+120\rho_w(V^* d/M^*)) < 0.5(f'_c)^{0.5}$			0.96 MPa
$v_n - v_c$			No Shear R/f Req.
$(A_v/s_v)_{mim} = b / fy$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{mim} = b / fy$	No Shear R/f Req.
$(A_v/s_v)_{Provided}$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{Provided}$	No Shear R/f Req.
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			No Shear R/f Req.

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Calculation Sheet

Job name:	**Queensgate Shopping Mall Redevelopment, Wellington**	Job No:	**2711265**
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		Date:	18/05/2004

Flexure Design in Y-Direction

Effective Depth in X-dir d	887.5 mm	Effective Width in X-dir b	2500 mm
$m = f_y / (0.85\, f'_c)$	23.53	$Ru = M_u / (\phi\, b\, d^2)$	0.33
$\rho = 1/m \{1 - (1 - (2\, m\, R_u) / f_y)^{0.5}\} > \{(f'_c)^{1/2} / (4\, fy)\}$			0.0025
Required As = ρ bd	5547 mm^2	Required No of Re-bars	11.30 Nos
Provided As	5890 mm2 Okay	Provided No of Re-bars	**12** Nos Okay
ρ_w = As / (b d)	0.0027		

Shear Design in Y-Direction

$v_{n\text{-}Max} = \text{Min}(1.1(f'_c)^{1/2}, 0.2\, f'_c, 9.0)$			5.00 MPa
$v_u = V_u / (b\, d)$			0.31 MPa Okay
$v_c = 1/7(3.5\text{-}2.5M^*/(V^*d))((f'_c)^{0.5}+120\rho_w(V^*d/M^*)) < 0.5(f'_c)^{0.5}$			0.90 MPa
$v_n - v_c$			No Shear R/f Req.
$(A_v/s_v)_{min} = b / fy$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{min} = b / fy$	No Shear R/f Req.
$(A_v/s_v)_{Provided}$	No Shear R/f Req.	$(A_{vh}/s_{vh})_{Provided}$	No Shear R/f Req.
$1/(12b)((1+L_n/d)fy(A_v/s_v)+(11-L_n/d)f_y(A_{vh}/s_{vh}))$			No Shear R/f Req.

Design Summary

Bottom X-Dir Flexural Reinforcement	12 DH25	Top X-Dir Reinforcement	12 DH16
Bottom Y-Dir Flexural Reinforcement	12 DH25	Top Y-Dir Reinforcement	12 DH16
Vertical Shear Links		Horizontal Shear Links	

Design
Of
Ground Beam

241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

The over strength Seismic base Shear will be resisted by

i) Frictional resistence between lowest Slab & Soil.
ii) Pile lateral resistance
iii) Passive Resistence against pilecaps and Ground beams.

The Ground beam will transfer the K-brace Shear to foundation.
For design of ground beam, we ignore the passive resistence against pilecaps & Ground beam.

Therefore, K-brace Shear will induce an axial force in ground beam GB1, which will transfer to foundation piles and Ground Slab,

Foundation Piles are designed to take an Ultimate lateral load of 120 kN.

For our design of Ground beams we take this resistence as 60 kN. per pile. (Same for all type of piles)

Frictional resistence between Ground Slab and Soil is taken as μ (weight of Slab).

$= 0.25\ (0.15 \times 24) = 0.9\ kPa$

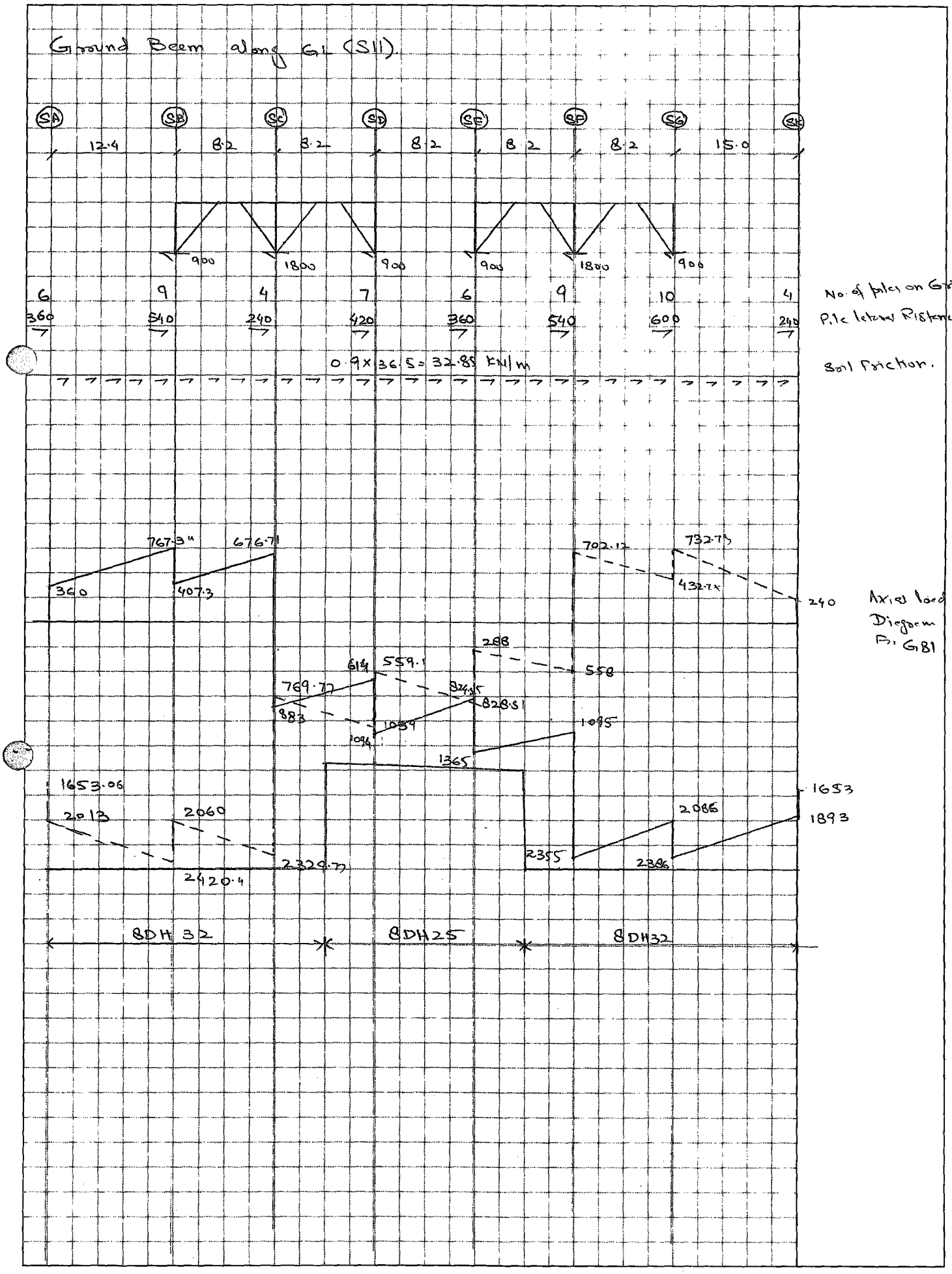
241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



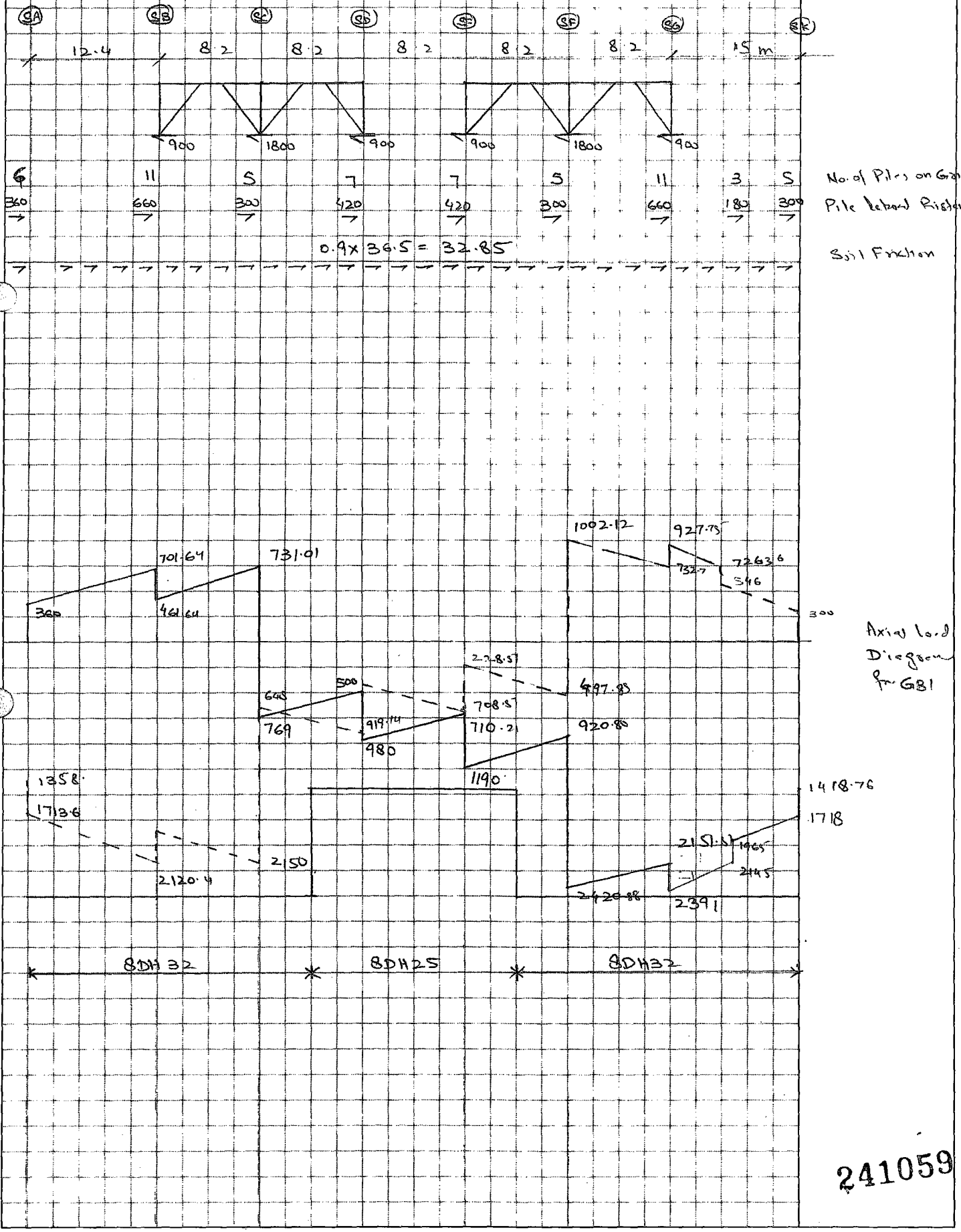
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Ground Beam along GL (S4)



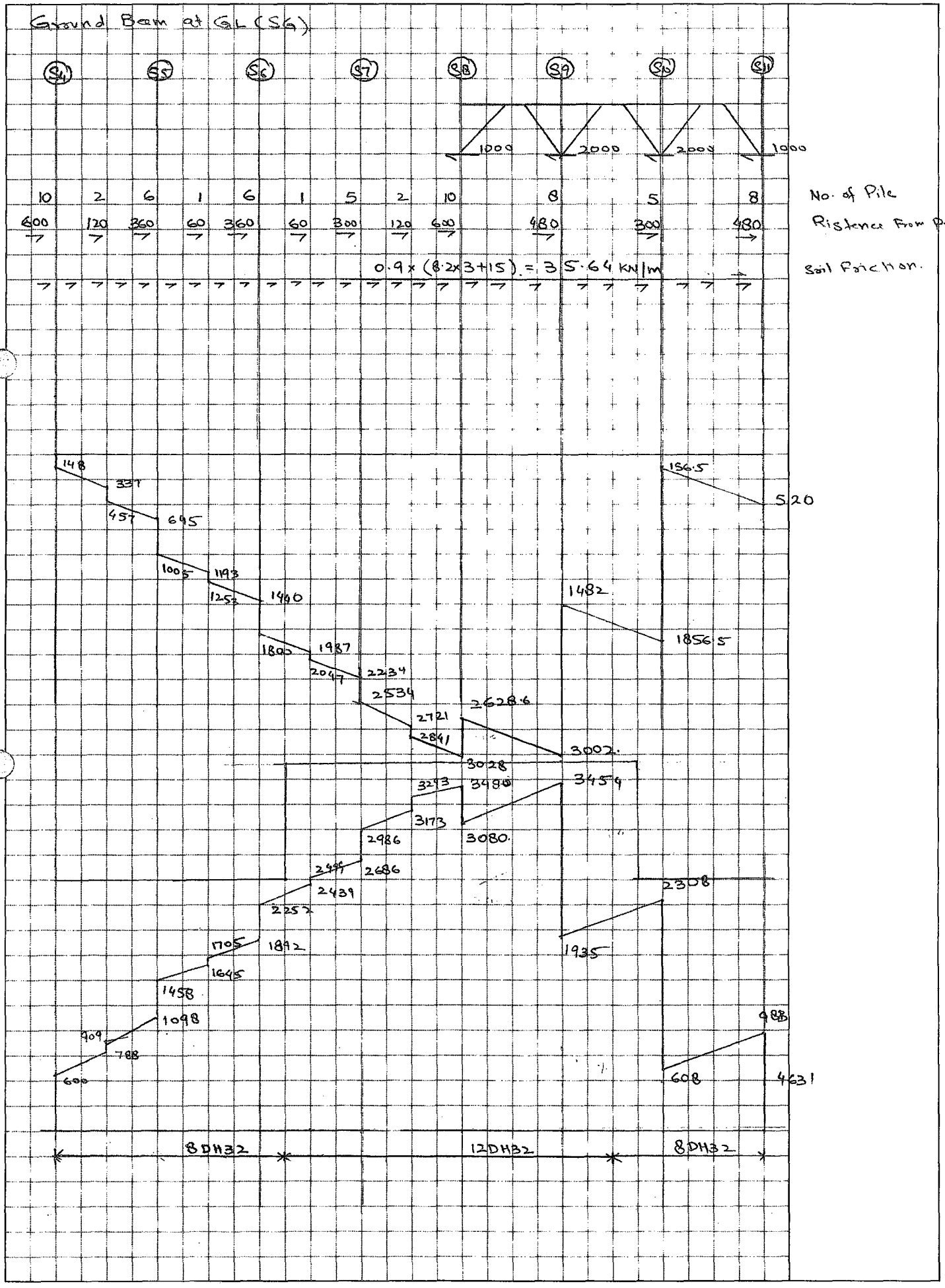
241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Beca Carter Hollings & Ferner

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall, Zone J — JOB NO: 2711265
SUBJECT: Ground Beam GB1 with 12HD32 Re-bars — PAGE NO:
BY: Jamil Khan — DATE: 16/6/04

Moment - Axial Force Curve for a Rectangular Beam-Column

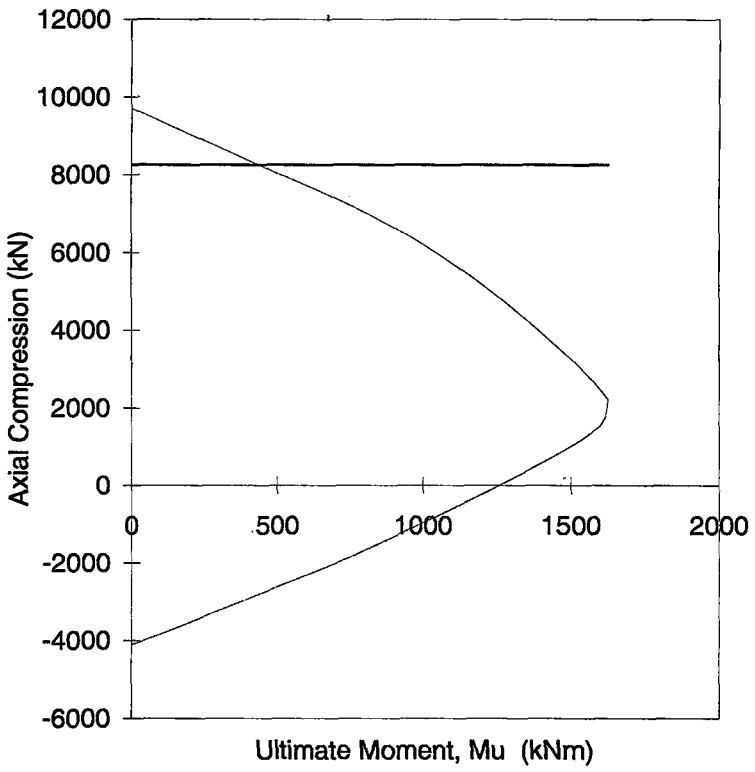
h =	800 mm	c =	20.0 mm	Layer		Bar Area (mm^2)	Depth (mm)	Strain	Stress (MPa)	Force (kN)
b =	400 mm	Pi =	-4681.0 kN							
d =	715 mm	Mi =	56.6 kNm	1	4 DH 32	3217	75	0.0083	500.0	1608.5
d'=	85 mm	ø =	0.8500	2	2 DH 32	1608	175	0.0233	500.0	804.2
Fy =	500 MPa	Pu =	-3978.8 kN	3	2 DH 32	1608	625	0.0908	500.0	804.2
fc'=	25 MPa	Mu =	48.1 kNm	4	4 DH 32	3217	725	0.1058	500.0	1608.5
ø max =	0.85	α_1 =	0.850							
ø min =	0.85	β_1 =	0.850							

Ag = 0.3200 m^2
Ast= 9650.97 mm^2
AstFy= 4825.49 kN

øFyc= 9707.3 kN — Mpc= 0.00 kNm
øFyt = 4101.7 kN — Mpt = 0.00 kNm
$0.85øP_o$= 8251.2 kN

cb = 390.0 mm — step1= 19.5 mm
cmax= 941.2 mm — step2= 27.6 mm

c	Pu	Mu	c	Pu	Mu
	-3979	48		-3979	48
0	-4102	0	941	9707	0
20	-3982	47	914	8065	491
39	-3862	92	886	7833	569
59	-2838	429	859	7597	643
78	-2192	641	831	7414	697
98	-1784	770	803	7169	766
117	-1208	931	776	6918	832
137	-763	1054	748	6661	897
156	-399	1153	721	6426	952
176	-89	1236	693	6155	1012
195	182	1306	666	5876	1070
215	397	1360	638	5587	1127
234	621	1413	610	5287	1182
254	828	1460	583	4975	1236
273	1023	1502	555	4648	1291
293	1208	1539	528	4306	1345
312	1385	1571	500	3944	1400
332	1555	1600	473	3559	1457
351	1762	1616	445	3145	1516
371	1999	1620	418	2684	1574
390	2224	1624	390	2224	1624



Limit Lines:

8251.2	0
8251.2	1624

241059

Beca Carter Hollings & Ferner

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall, Zone J — JOB NO: 2711265
SUBJECT: Ground Beam GB1 with 10HD32 Re-bars — PAGE NO:
BY: Jamil Khan — DATE: 16/6/04

Moment - Axial Force Curve for a Rectangular Beam-Column

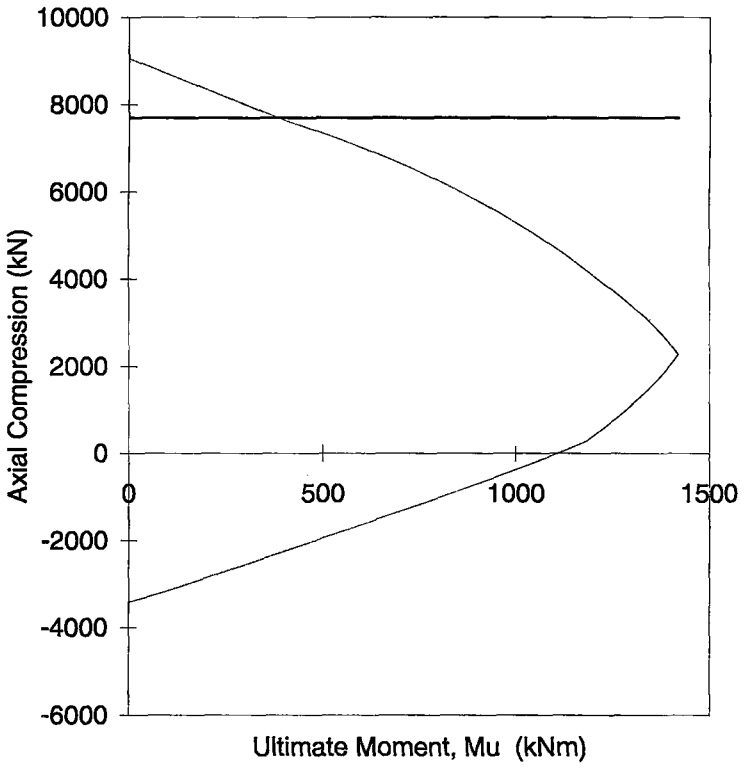
h =	800 mm	c =	20.0 mm	Layer		Bar Area	Depth	Strain	Stress	Force
b =	400 mm	Pi =	-3876.7 kN			(mm²)	(mm)		(MPa)	(kN)
d =	715 mm	Mi =	56.6 kNm	1	4 DH 32	3217	75	0.0083	500.0	1608.5
d'=	85 mm	ø =	0.8500	2	2 DH 32	1608	400	0.0570	500.0	804.2
Fy =	500 MPa	Pu =	-3295.2 kN	3	4 DH 32	3217	725	0.1058	500.0	1608.5
fc'=	25 MPa	Mu =	48.1 kNm							
ø max =	0.85	α_1 =	0.850							
ø min =	0.85	β_1 =	0.850							

Ag = 0.3200 m²
Ast= 8042.48 mm²
AstFy= 4021.24 kN

øFyc= 9052.8 kN — Mpc= 0.00 kNm
øFyt = 3418.1 kN — Mpt = 0.00 kNm
0.85øP_o= 7694.9 kN

cb = 390.0 mm — step1= 19.5 mm
cmax= 941.2 mm — step2= 27.6 mm

c	Pu	Mu	c	Pu	Mu
	-3295	48		-3295	48
0	-3418	0	941	9053	0
20	-3298	47	914	7633	400
39	-3179	92	886	7412	475
59	-2154	429	859	7187	547
78	-1509	641	831	7018	597
98	-1132	763	803	6786	662
117	-801	869	776	6549	725
137	-531	952	748	6308	786
156	-299	1022	721	6061	843
176	-92	1082	693	5808	899
195	98	1133	666	5548	952
215	275	1179	638	5281	1003
234	545	1220	610	5004	1052
254	813	1256	583	4717	1101
273	1060	1288	555	4419	1148
293	1289	1317	528	4107	1194
312	1506	1343	500	3779	1241
332	1710	1366	473	3433	1288
351	1906	1385	445	3091	1336
371	2093	1403	418	2679	1382
390	2274	1417	390	2274	1417



Limit Lines:

7694.9	0
7694.9	1417

Beca Carter Hollings & Ferner

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall, Zone J
SUBJECT: Ground Beam GB1 with 8HD32 Re-bars
BY: Jamil Khan

JOB NO: 2711265
PAGE NO:
DATE: 16/6/04

Moment - Axial Force Curve for a Rectangular Beam-Column

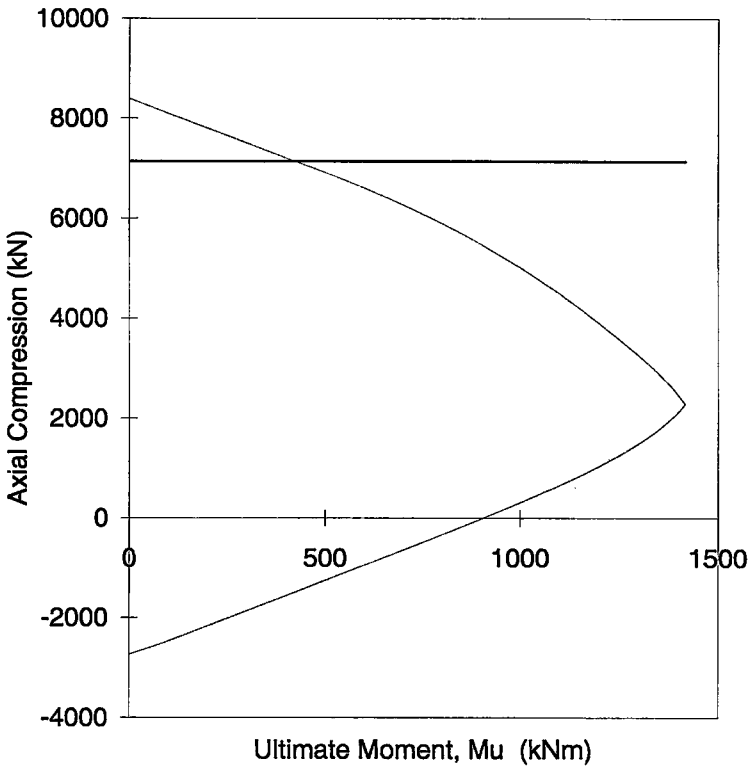
h =	800 mm	c =	20.0 mm	Layer		Bar Area (mm²)	Depth (mm)	Strain	Stress (MPa)	Force (kN)
b =	400 mm	Pi =	-3072.5 kN							
d =	715 mm	Mi =	56.6 kNm	1	4 DH 32	3217	75	0.0083	500.0	1608.5
d'=	85 mm	ø =	0.8500	2	0 DH 32	0	175			
Fy =	500 MPa	Pu =	-2611.6 kN	3	0 DH 32	0	625			
fc'=	25 MPa	Mu =	48.1 kNm	4	4 DH 32	3217	725	0.1058	500.0	1608.5
ø max =	0.85	α_1 =	0.850							
ø min =	0.85	β_1 =	0.850							

Ag = 0.3200 m²
Ast= 6433.98 mm²
AstFy= 3216.99 kN

øFyc= 8398.2 kN Mpc= 0.00 kNm
øFyt = 2734.4 kN Mpt = 0.00 kNm
$0.85øP_o$= 7138.5 kN

cb = 390.0 mm step1= 19.5 mm
cmax= 941.2 mm step2= 27.6 mm

c	Pu	Mu	c	Pu	Mu
	-2612	48		-2612	48
0	-2734	0	941	8398	0
20	-2615	47	914	7200	400
39	-2495	92	886	6991	475
59	-1471	429	859	6778	547
78	-825	641	831	6621	597
98	-448	763	803	6403	662
117	-118	869	776	6181	725
137	152	952	748	5955	786
156	385	1022	721	5725	843
176	592	1082	693	5491	899
195	782	1133	666	5250	952
215	959	1179	638	5004	1003
234	1127	1220	610	4750	1052
254	1287	1256	583	4489	1101
273	1441	1288	555	4218	1148
293	1591	1317	528	3937	1194
312	1737	1343	500	3644	1241
332	1880	1366	473	3336	1288
351	2020	1385	445	3008	1336
371	2159	1403	418	2644	1382
390	2295	1417	390	2295	1417



Limit Lines:

7138.5	0
7138.5	1417

241059

Beca Carter Hollings & Ferner

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall, Zone J
SUBJECT: Ground Beam GB1 with 8HD25 Re-bars
BY: Jamil Khan

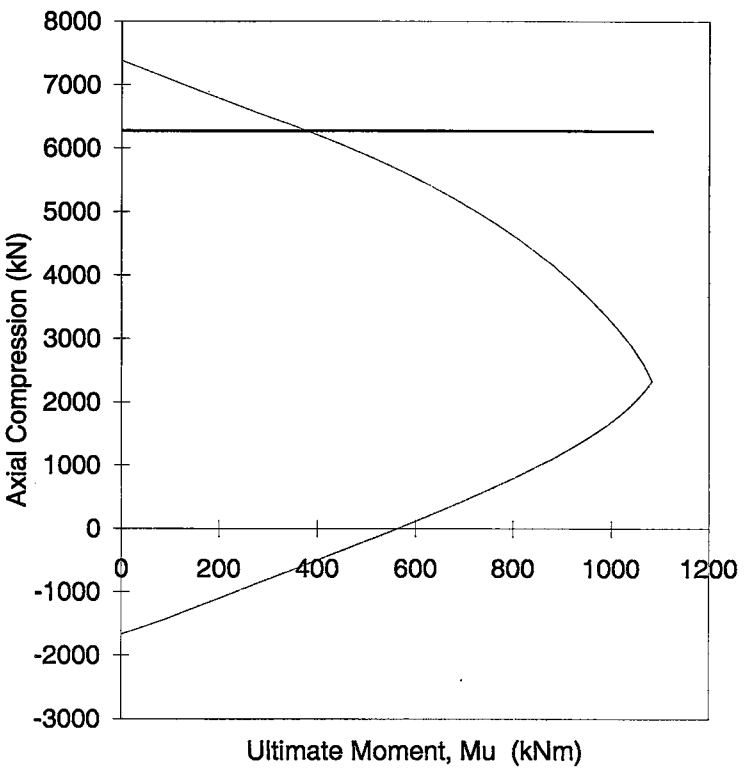
JOB NO: 2711265
PAGE NO:
DATE: 16/6/04

Moment - Axial Force Curve for a Rectangular Beam-Column

h =	800 mm	c =	20.0 mm	Layer		Bar Area (mm²)	Depth (mm)	Strain	Stress (MPa)	Force (kN)
b =	400 mm	Pi =	-1819.0 kN							
d =	715 mm	Mi =	56.6 kNm	1	4 DH 25	1963	75	0.0083	500.0	981.7
d'=	85 mm	ø =	0.8500	2	0 DH 25	0	175			
Fy =	500 MPa	Pu =	-1546.1 kN	3	0 DH 25	0	625			
fc'=	25 MPa	Mu =	48.1 kNm	4	4 DH 25	1963	725	0.1058	500.0	981.7
ø max =	0.85	α_1 =	0.850							
ø min =	0.85	β_1 =	0.850							

Ag =	0.3200	m²	
Ast=	3926.99	mm²	
AstFy=	1963.5	kN	
øFyc=	7378.0	kN	Mpc= 0.00 kNm
øFyt =	1669.0	kN	Mpt = 0.00 kNm
0.85øP_o=	6271.3	kN	
cb =	390.0	mm	step1= 19.5 mm
cmax=	941.2	mm	step2= 27.6 mm

c	Pu	Mu	c	Pu	Mu
	-1546	48		-1546	48
0	-1669	0	941	7378	0
20	-1549	47	914	6581	270
39	-1429	92	886	6387	340
59	-758	314	859	6192	406
78	-317	459	831	6030	457
98	-40	549	803	5830	517
117	208	628	776	5629	573
137	420	693	748	5425	626
156	608	748	721	5219	676
176	781	797	693	5010	723
195	944	840	666	4797	767
215	1099	878	638	4581	809
234	1248	913	610	4360	848
254	1392	944	583	4135	884
273	1533	972	555	3904	918
293	1671	997	528	3666	951
312	1807	1019	500	3421	982
332	1941	1039	473	3167	1011
351	2073	1056	445	2901	1040
371	2204	1071	418	2613	1066
390	2334	1084	390	2334	1084



Limit Lines:

6271.3	0
6271.3	1084

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Connection between Ground Beams & Slab at K-brace Section

The interface R/f for connection can be designed using Shear friction concept.

$V_u = 1000 + 2000 + 2000 + 1000 = 6000\ kN$

$h = 10500 + 10500 + 10200 = 31200\ mm$

$b = 150$

$$\text{Shear Stress} = \frac{V_u}{bh} = \frac{6000 \times 1000}{150 \times 31200} = 1.28\ MPa$$

$$\therefore v_u = 1.28\ MPa$$

$f'_c = 25\ MPa$

$$v_{u\,max} = \text{Min}\left(0.2 f'_c,\ 1.1\sqrt{f'_c},\ 9\right)$$

$0.2 f'_c = 5\ MPa$

$1.1\sqrt{f'_c} = 5.5\ MPa$

$\therefore v_{u\,max} = 5\ MPa.\quad v_u < v_{u\,max}$

$$A_{vf} = \left(\frac{V^*}{\phi \mu_f} - N^*\right)\frac{1}{f_y} = \frac{V^*}{\phi \mu_f f_y}$$

$\phi = 0.75$

$\mu_f = 1.4\lambda$ $\quad \lambda = 1.0$ for Normal conc.

$f_y = 500$

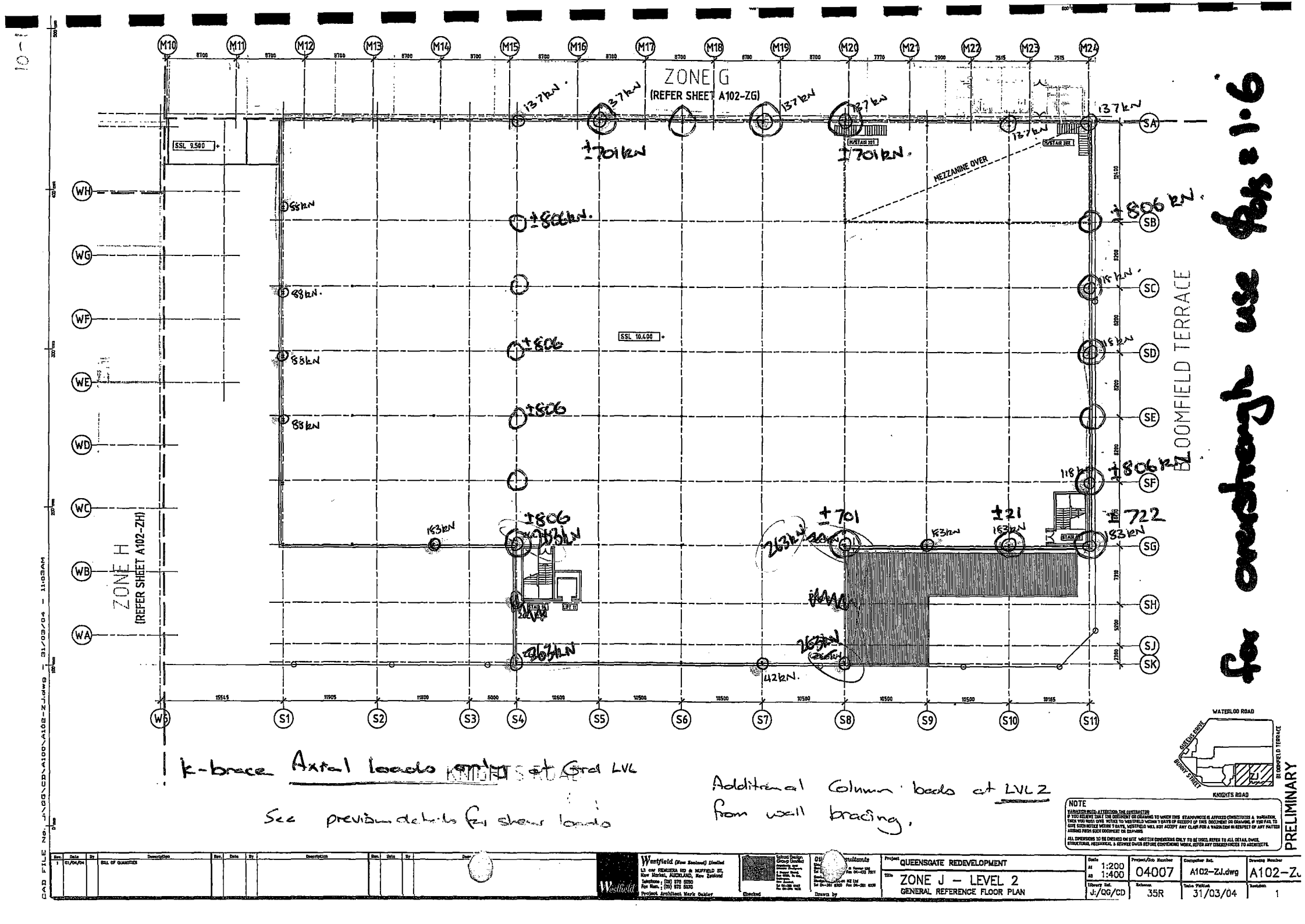
$$A_{vf} = \frac{6000 \times 1000}{0.75 \times 1.4 \times 1 \times 500} = 11428.57\ mm^2$$

$$A_{vf} = \frac{11428.57}{31.2} = 366.3\ mm^2/m.$$

Use D12-300 $\quad A_{vf} = 377\ mm^2/m$

418

241059



k-brace Axial loads ~~KNOWN~~ at Grd LVL

See previous details for shear loads

Additional Column loads at LVL 2 from wall bracing.

for comment use only 1:16

PRELIMINARY

WATERLOO ROAD
QUEENS DRIVE
BUNNY STREET
BLOOMFIELD TERRACE
KNIGHTS ROAD

NOTE
VARIATION NOTE: ATTENTION THE CONTRACTOR
IF YOU BELIEVE THAT THE DOCUMENT OR DRAWING TO WHICH THIS STATEMENT IS ATTACHED CONSTITUTES A VARIATION, THEN YOU MUST GIVE NOTICE TO WESTFIELD WITHIN 7 DAYS OF RECEIPT OF THIS DOCUMENT OR DRAWING, IF YOU FAIL TO GIVE SUCH NOTICE WITHIN 7 DAYS, WESTFIELD WILL NOT ACCEPT ANY CLAIM FOR A VARIATION IN RESPECT OF ANY MATTER ARISING FROM SUCH DOCUMENT OR DRAWING.
ALL DIMENSIONS TO BE CHECKED ON SITE. WRITTEN DIMENSIONS ONLY TO BE USED. REFER TO ALL DETAIL DRAWINGS, STRUCTURAL, MECHANICAL & SERVICES DRGS BEFORE COMMENCING WORK. REFER ANY DISCREPANCIES TO ARCHITECTS.

Rev.	Date	By	Description
1	01/04/04		BILL OF QUANTITIES

Westfield (New Zealand) Limited
L3 cnr RENUERA RD & NUFFIELD ST, New Market, AUCKLAND, New Zealand
Telephone: (09) 978 9090
Fax Num.: (09) 978 9070
Project Architect Mark Oakley

Checked

Drawn by

Project: QUEENSGATE REDEVELOPMENT
Title: ZONE J – LEVEL 2 GENERAL REFERENCE FLOOR PLAN

Scale	Project/Job Number	Computer Ref.	Drawing Number
A1 1:200 A3 1:400	04007	A102-ZJ.dwg	A102-Z
Library Ref. J:/QG/CD	Release 35R	Date Plotted 31/03/04	Revision 1

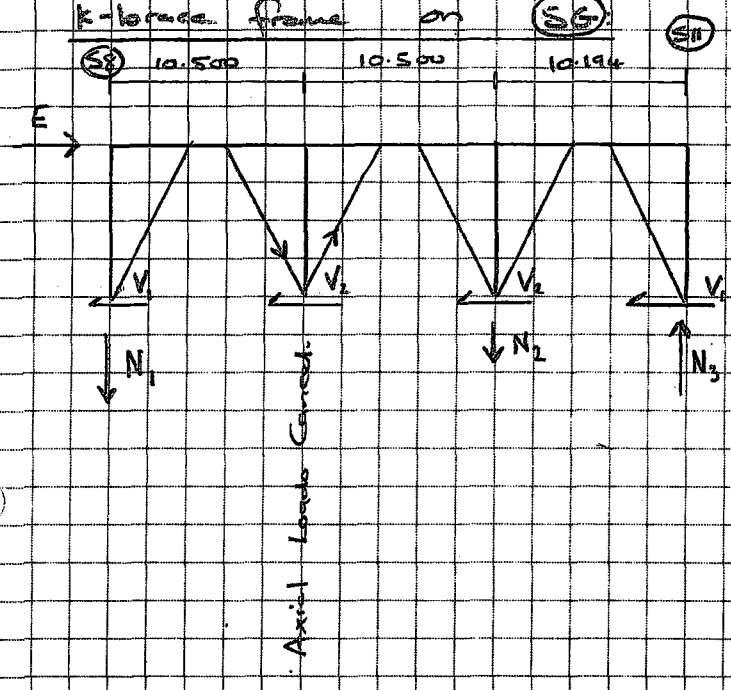
CAD FILE No. J:\QG\CD\A100\A102-ZJ.dwg 31/03/04 - 11:03AM

Calculation Sheet

Job name: Queensgate Mall Redevelopment
Subject: E Loads at foundation ONLY.
By: Dean Saunders. No O/S.

Job No: 2711265/109
Page No: 1 of 2
Date: 10-5-04

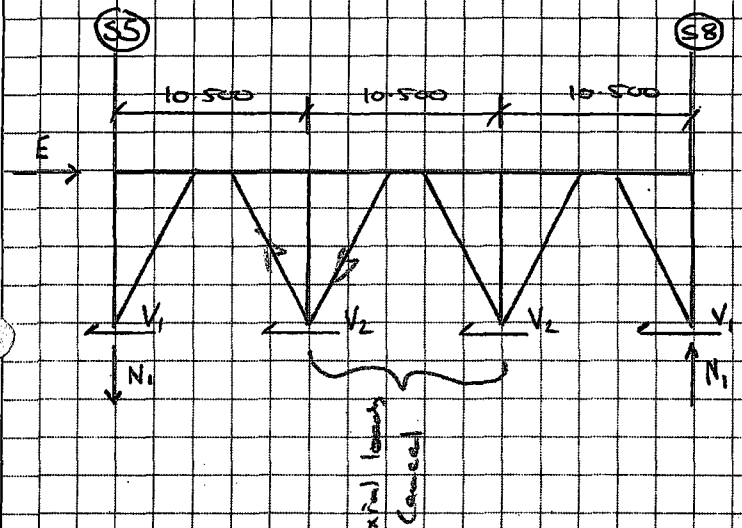
K-brace frame on (5G)



$V_1 = 625\,kN$
$V_2 = 1250\,kN$

NOTE TENSION OR COMPRESSION.
$N_1 = 701\,kN$
$N_2 = 21\,kN$
$N_3 = 722\,kN$

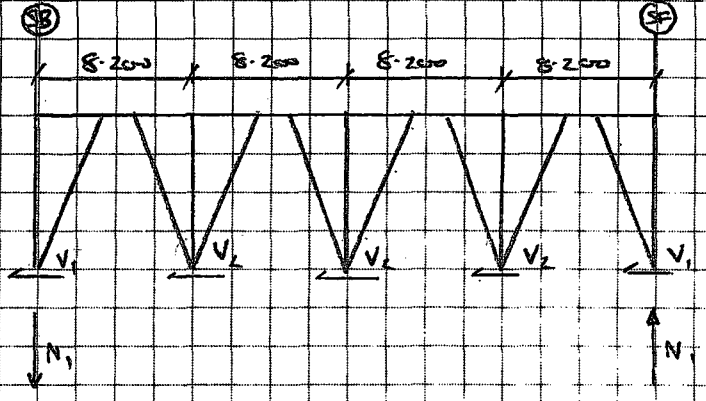
K-brace frame on (5A)



$V_1 = 625\,kN$
$V_2 = 1250\,kN$

NOTE TENSION OR COMPRESSION
$N_1 = 701\,kN$

K-brace frame on (511)



$V_1 = 562\,kN$
$V_2 = 1124\,kN$

NOTE TENSION OR COMPRESSION
$N_1 = 806\,kN$

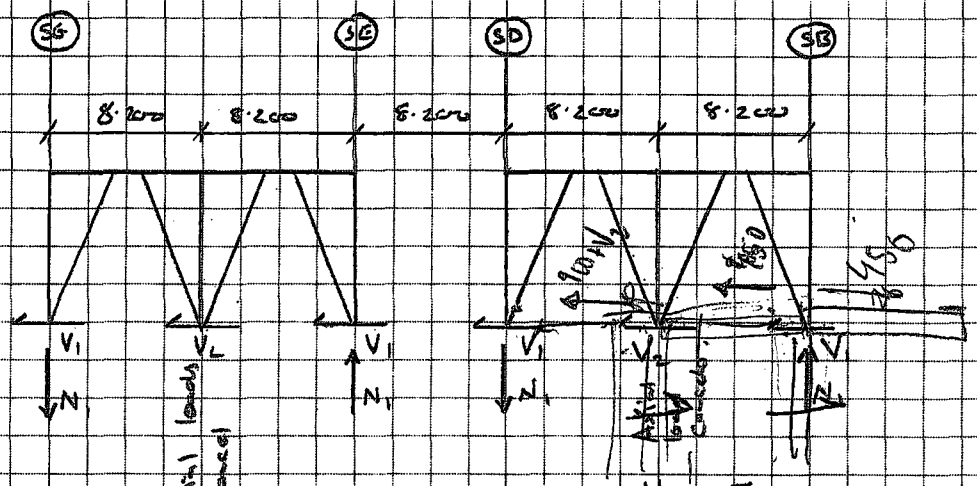
241059

Calculation Sheet

Job name: Queensgate Mall Redevelopment
Subject: E loads at foundation
By: Dean Saunders.

Job No: 2711265/109
Page No: 2 of 2
Date: 10-5-04

Ic - brace frame on (S4)



$V_1 = 562$ kN
$V_2 = 1124$ kN

NOTE TENSION AND COMPRESSION

$N_1 = 806$ kN

O/S = 1.6. factored Vb already included.

1.67

(SA) & (SG)

$N^* = 1155$ kN

$V^* = 2000$ kN INTERNAL
$= 1000$ kN EXTERNAL.

(S4) (S11).

$N^* = 1290$ kN.

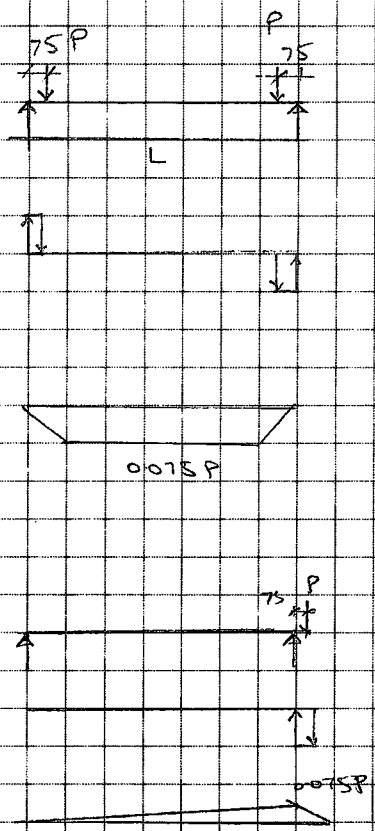
$V^* = 1798$ kN INTERNAL →
900 kN EXTERNAL.

Calculation Sheet

Job name: Queensgate Mall
Subject: Zone J, Ground Beam
By:

Job No:
Page No: of
Date:

G32



$\phi = 0.9$

$P_{max} = 2660$ kN

$M_{u\,max} = 0.075 \times 2660 = 199.5$ kN-m

$b = 400 \quad h = 500 \quad d = 500 - 75 - 20/2 = 415$

$f'_c = 25$ MPa $\quad f_y = 500$ MPa

$m = \frac{f_y}{0.85 f'_c} = 23.53$ $\quad R_u = \frac{M_u}{\phi b d^2} = \frac{199.5 \times 10^6}{.85 \times 400 \times 415^2}$

$R_u = 3.407$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 m R_u}{f_y}}\right) = \frac{1}{23.53}\left(1 - \sqrt{1 - \frac{2 \times 23.53 \times 3.56}{500}}\right)$

$\rho = 0.00747$ $\quad \rho_{min} = \frac{\sqrt{f'_c}}{4 f_y} = 0.0025$

$A_s = \rho b d = 1240.12$ mm^2

use 4DH20

241059

Beca Carter Hollings & Ferner Ltd

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall
SUBJECT: Zone J, Ground Beam GB2, Without Axial Force
BY: Jamil Khan

JOB NO: 2711265
PAGE NO:
DATE: 13-May-04

Ultimate Moment Strength of Regular Reinforced Concrete Columns

h =	500 mm	Fy =	500 MPa	Layer	Bars	Area (mm^2)	Depth (mm)	Strain	Stress (net)
bw =	400 mm	Es =	200000 MPa						
bt =	400 mm	fc'=	30 MPa	1	4D20	1256	85	0.000227	45.4
bb =	400 mm	ec =	0.003	2	4D20	1256	415	0.012756	500.0
Tt =	0 mm	Yo =	250 mm						
Tb =	0 mm	Ratio =	1						

ø =	0.85	β_1 =	0.850
		α_1 =	0.850

Pu =	0.0 kN	Pi =	0.0 kN
Mu =	206.1 kNm	Mi =	242.5 kNm

Ag =	200000 mm^2	AstFy=	1256.0 kN
Ast=	2512 mm^2	Fyc=	6291.9 kN

Iteration variable:		Convergence Check:	
c =	79.02 mm	P =	0.0 kN
a =	67.16 mm	Err =	-2.1E-07

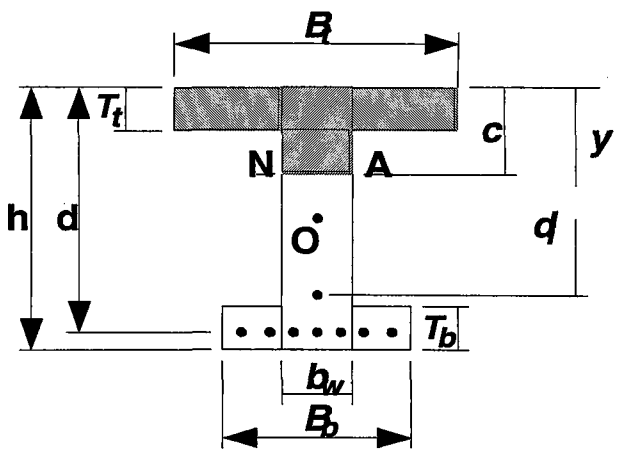
Concrete Compression Block Actions

	Force	Moment
web	685.1 kN	148 kNm
top	0.0 kN	0 kNm
bot	0.0 kN	0 kNm
totals:		
Cc =	685.1 kN	Mc = 148 kNm

Steel Totals

Ts =	685.1 kN	Ms =	94 kNm

This spreadsheet template has not had a Q.C. check yet.
Check all results carefully before using them.



Beca Carter Hollings & Ferner Ltd

EXCEL WORKSHEET

JOB NAME: Queensgate Shopping Mall
SUBJECT: Zone J, Ground Beam GB2, With Axial Force
BY: Jamil Khan

JOB NO: 2711265
PAGE NO:
DATE: 13-May-04

Ultimate Moment Strength of Regular Reinforced Concrete Columns

h =	500 mm	Fy =	500 MPa	Layer	Bars	Area (mm^2)	Depth (mm)	Strain	Stress (net)
bw =	400 mm	Es =	200000 MPa						
bt =	400 mm	fc'=	30 MPa	1	4D20	1256	85	-0.002038	-382.2
bb =	400 mm	ec =	0.003	2	4D20	1256	415	0.001695	339.1
Tt =	0 mm	Yo =	250 mm						
Tb =	0 mm	Ratio =	1						

ø =	0.85	β_1 =	0.850
		α_1 =	0.850

Pu =	2000.0 kN	Pi =	2352.9 kN
Mu =	395.4 kNm	Mi =	465.1 kNm

Ag =	200000 mm^2	AstFy=	1256.0 kN
Ast=	2512 mm^2	Fyc=	6291.9 kN

Iteration variable:		Convergence Check:	
c =	265.15 mm	P =	2352.9 kN
a =	225.38 mm	Err =	-0.000139

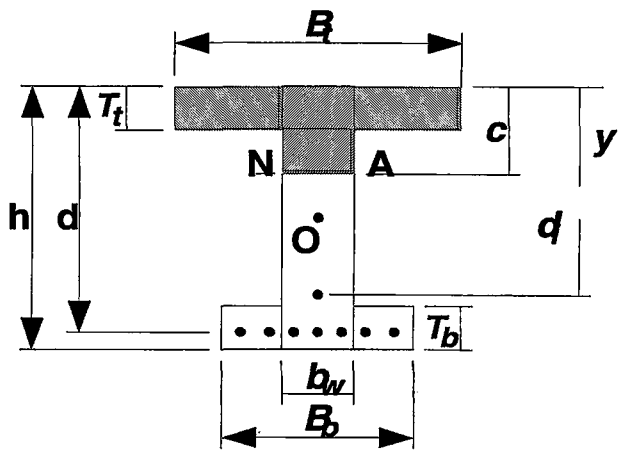
Concrete Compression Block Actions

	Force		Moment
web	2298.9 kN		316 kNm
top	0.0 kN		0 kNm
bot	0.0 kN		0 kNm
totals:			
Cc =	2298.9 kN	Mc =	316 kNm

Steel Totals

Ts =	-54.1 kN	Ms =	149 kNm

This spreadsheet template has not had a Q.C. check yet.
Check all results carefully before using them.



Design
Of
Ground Columns

241059

Original Tender Provisions with externally applied Fire Protection

Type	Structural Section	Weight Kg/m	Total No Columns	Steel Wt Kg	Conc. Vol m^3
C1A	250x6 SHS 25 MPa Concrete Filled	45	23	5589	7.04
C2A	250x9 SHS 25 MPa Concrete Filled	65.9	40	14234	11.63
C3A	310UC137	137	23	17015	
C4A	325x9.5 CHS	74	3	1199	
C5A	410UB54	54	0	0	
			89	38038	18.66

Total Structural Steel weight 38038Kg
Total Normal 25MPa in filled Concrete Volume 18.66 m^3
Total Externally applied fire protection (for 1Hr Fire Resistance) QS to Advise

New Proposal without externally applied Fire Protection

Type	Structural Section	Weight Kg/m	Total No Columns	Steel Wt Kg	Conc Vol m^3
CA1	250x6 SHS 60 MPa Fibre R/f Concrete Filled	45	89	21627	27.22

Total Structural Steel weight 21627Kg
Total Fibre Reinforced 69 MPa in filled Concrete Volume 27.22m^3
Total Externally applied fire protection (for 1Hr Fire Resistance) 00.00

Fibre Reinforced Concrete

The steel fibre reinforcement spread uniformly throughout the concrete and forming approximately 5% by dry mass of the constituents, before addition of water. The Fibre shape and dimension should be such as to provide adequate pullout strength. Typically, fibres should be 0.5mm diameter, not longer than 38mm, and have crimped flats or hooked ends to ensure adequate pullout resistance.

241059

Calculation Sheet

Job name: Queensgate Shopping Mall Job No: 2711265
Subject: Zone J, Ground Floor Col. Page No: 1 of 4
By: Jamil Khan Date: 20 May, 2004

Fire Limit State Axial load Capacity of SHS with Filled con.

Axial load at Fire Limit State

Using 250×250×6 SHS.

For 60min Fire Resistence

$\eta = 0.509$ for Plain & Reinforced conc. fill.

$\eta = 0.678$ for Fibre Reinforced conc. fill.

$$\frac{F_f + 6\left(\frac{M_{fx}}{d_x} + \frac{M_{fy}}{d_y}\right)}{k_f \cdot N_{uf}} \leq \eta$$

For our case $M_{fx} = M_{fy} = 0$ (For internal cols)

$\therefore F_f \leq \eta k_f N_{uf}$.

Floor to Floor height = 5.4m.
Generally Depth of Floor beams = 400m.

$\therefore L_e = 5.0$ m.

From Table 3.6 of Design Manual for Concrete Filled Columns Part 2,

For 250×6 SHS for Plain and Fibre Reinforced col.

$f_{cu} = 40$ MPa $N_{uf} = 1447$ kN $l_{Ef} = 5.03$ m

$f_{cu} = 60$ MPa $N_{uf} = 2170$ kN $l_{Ef} = 5.03$ m

$$\lambda = \frac{l_e}{l_{Ef}} = \frac{5.0}{5.03} = 1.0$$

Calculation Sheet

Job name: Queensgate Shopping Mall **Job No:** 2711265
Subject: Zone J, Ground Floor col. **Page No:** 2 of 4
By: Jamil Khan **Date:** 20 May 2004

From Table 3.8.1

For $\rho = 2\%$ 4 Bar Cage

250×250×6

$f_{cu} = 40$ MPa $N_{uf} = 1969$ kN $L_{Eq} = 5.21$ m $\rightarrow \lambda = 0.96$

$f_{cu} = 60$ MPa $N_{uf} = 2656$ kN $L_{Eq} = 5.15$ m $\rightarrow \lambda = 0.97$

From Table 3.8.2

For $\rho = 3\%$ 4 Bar Cage

$f_{cu} = 40$ MPa $N_{uf} = 2231$ kN $L_{Eq} = 5.23 \rightarrow \lambda = 0.956$

$f_{cu} = 60$ MPa $N_{uf} = 2929$ kN $L_{Eq} = 5.11 \rightarrow \lambda = 0.97$

From Table 3.8.3

For $\rho = 4.0\%$ 4 Bar Cage

$f_{cu} = 40$ $N_{uf} = 2732$ kN $L_{Eq} = 5.20$ m $\rightarrow \lambda = 0.9$

$f_{cu} = 60$ $N_{uf} = 3414$ kN $L_{Eq} = 5.15$ m $\rightarrow \lambda = 0.9$

For $\lambda = 0.9$ $k_f = 0.74$

$\lambda = 1.0$ $k_f = 0.67$.

We will take $k_f = 0.67$

$$F_f \leq \eta k_f N_{uf}$$

$$F_f \leq 0.67 \eta N_{uf}$$

241059

Calculation Sheet

Job name: Queensgate Shopping Mall. **Job No:** 2711265
Subject: Zone J, Ground Floor Col. **Page No:** 3 of 4
By: Jamil Khan **Date:** 20 May 2004

Type of Cone	η	f'_{cu} MPa	N_{uf} kN	F_f kN
Plain Conc.	0.509	40	1447	493
		60	2170	740
Fibre Reinforced Conc	0.678	40	1447	657.
		60	2170	986 ✓
Bar Reinforced Conc $\rho=2\%$	0.509	40	1949	645
		60	2656	906
Bar Reinforced Conc. $\rho=3\%$	0.509	40	2231	761
		60	2929	999
Bar Reinforced Conc $\rho=4\%$	0.509	40	2732	932
		60	3414	1164

Calculation Sheet

Job name: Queensgate Shopping Mall. Job No: 2711285

Subject: Zone J, Ground Floor Col. Page No: 4 of 4

By: Date: 20, May 2004

Ultimate Axial load capacity of SHS col with fibre R/f con fill

For 250x250x6 SHS.

From Table 4.2 of SHS Design Manual for Conc. Filled Cols Part 4

f_y	f_{cu}	α_c	N_u	l_E	M_u
275	40	0.411	2480	8.213	150.2
	60	0.511	2990	7.993	154.6

For f_{cu} = 40 MPa $\lambda_x = \frac{l_e}{l_E} = \frac{5.0}{8.213} = 0.608$

for $\lambda_x = 0.608$ $k_{lx} = 0.89$

$N = k_{lx} N_u = 2207.2$ kN

for f_{cu} = 60 MPa $\lambda_x = \frac{l_e}{l_E} = \frac{5.0}{7.99} = 0.625$

for $\lambda_x = 0.625$ $k_{lx} = 0.88$

$N = k_{lx} N_u = 0.88 \times 2990 = 2631.2$ kN.

241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Fire Resistance for C4A col. (using Kodur & Lie Approach)

Column C4A 325×9.5 CHS.

Applied Fire Limit State load = 1267 kN

External Dia of Column = 325 mm

Length of column = L = 5000 mm

f'_c = 60 MPa

K = Effective Length of column

From Table 4.2-22 for Col. 325×9.5 CHS

Le for applied

Le = 6 m P_u = 2840 kN

Le = 8 m P_u = 2490 kN

∴ For P_u = 2526 Le = 7.8 m

As L < Le ∴ K for this case is 1.0

f_1 = For Fibre Reinforced conc. fill = 0.08

Fire Resistance

$$R = \frac{f_1 (f'_c + 20)}{KL - 1000} D^2 \sqrt{\frac{D}{c}}$$

$$R = \frac{0.08 (60+20)}{1.0 \times 5000 - 1000} \times 325^2 \sqrt{\frac{325}{1267}}$$

$$R = \frac{0.08 \times 80 \times 325^2}{5000 - 1000} \times \sqrt{0.255} = 85.32 \text{ min}$$

$R_{req'd} > 60$ min.

Fire Resistance for CFA col. (Using BS code approach)

In infill hollow section, the fire limit state load resistance is based on the axial capacity of infill conc. Currently we have tables only for SHS. For CHS we workout the equivalent square infill conc. section and then use the code tables.

Circular col. = 325 x 9.5

Area of concrete in-fill in col. = $(325-9.5)^2 \frac{\pi}{4}$

= 78541.54 mm²

Equivalent Sq. conc section = 271.18 x 271.12.

∴ For 300x300x16 SHS the infill conc section is 268x268. So we use the L_{Ef} and N_{uf} values

From Table 3.6

For 300x300x16 SHS with f_{cu} = 60 MPa

L_{Ef} = 5.66 N_{uf} = 2751 kN

L_e = 5.0 For col.

$$\lambda = \frac{L_e}{L_{Ef}} = \frac{5.0}{5.66} = 0.883.$$

For λ = 0.9 K_f = 0.74

$$F_f = \eta K_f N_{uf}$$

F_f = 0.678 x 0.74 x 2751 = 1380.2 kN.

F_f for 1hr Fire Resistance = 1380 kN > applied load of 1267 kN

Ref: 241059

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* — Job No. 2711265

Subject **Summary of Column Loads For Column Design of Zone J** — Page no Sheet 1 of 3

Calculat Jamil Khan — Reviewed By — Date ########

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	Ψ_s Q	Ψ_L Q	Eu	G+Ψ_s Q	G + Ψ_L Q	G + Ψ_U Q + E_U	1.2G + 1.6Q	G + Ψ_U Q	G + Ψ_U Q + E_O	G + Ψ_U Q - E_O	Column Type	Ultimate	Fire Resistance	
1	S1/SA	150	160	112	62	---	262	211	211	436	211	211		C1A	2630	985	Okay
2	S1/S5	518	536	375	208	88	893	726	814	1479	726	867		C1A	2630	985	Okay
3	S1/SC	362	340	238	131	88	600	493	581	978	493	634		C1A	2630	985	Okay
4	S1/SD	342	311	218	121	88	560	463	551	908	463	604		C1A	2630	985	Okay
5	S1/SE	350	320	224	124	88	574	474	562	932	474	614		C1A	2630	985	Okay
6	S1/SF	337	308	216	119	---	553	456	456	897	456	456		C1A	2630	985	Okay
7	S1/SF.5	52	33	23	11	---	75	63	63	115	63	63		C1A	2630	985	Okay
8	S2/SA	420	512	358	193	---	779	614	614	1323	614	614		C1A	2630	985	Okay
9	S2/SB	612	803	562	321	---	1174	933	933	2020	933	933		C1A	2630	985	Okay
10	S2/SC	550	700	490	256	---	1040	806	806	1779	806	806		C1A	2630	985	Okay
11	S2/SD	484	640	448	256	---	932	740	740	1604	740	740		C1A	2630	985	Okay
12	S2/SE	536	687	481	256	---	1017	792	792	1743	792	792		C1A	2630	985	Okay
13	S2/SF	484	640	448	256	---	932	740	740	1604	740	740		C1A	2630	985	Okay
14	S2/SG	300	344	241	128	64	540	428	492	909	428	530		C1A	2630	985	Okay
15	S3/SA	303	378	265	145	---	568	448	448	969	448	448		C1A	2630	985	Okay
16	S3/SB	463	603	422	241	---	885	704	704	1520	704	704		C1A	2630	985	Okay
17	S3/SC	405	512	359	192	---	763	597	597	1305	597	597		C1A	2630	985	Okay
18	S3/SD	364	480	336	192	---	699	555	555	1204	555	555		C1A	2630	985	Okay
19	S3/SE	396	506	354	192	---	750	588	588	1284	588	588		C1A	2630	985	Okay
20	S3/SF	365	480	336	192	---	701	557	557	1206	557	557		C1A	2630	985	Okay
21	S3/SG	222	253	177	96	119	399	318	437	670	318	508		C1A	2630	985	Okay
22	S4/SA	316	359	252	134	137	568	450	587	955	450	669		C1A	2630	985	Okay
23	S4/SB	452	556	389	222	806	841	674	1480	1431	674	1964	616	C1A	2630	985	Okay
24	S4/SC	426	495	347	177	---	773	603	603	1304	603	603		C1A	2630	985	Okay
25	S4/SD	342	442	310	177	806	651	518	1324	1118	518	1808	771	C1A	2630	985	Okay
26	S4/SE	382	484	339	177	806	721	559	1365	1233	559	1848	731	C1A	2630	985	Okay
27	S4/SF	340	442	310	177	---	650	517	517	1116	517	517		C1A	2630	985	Okay
28	S4/SG	376	334	234	116	1066	610	491	1557	986	491	2197	1214	C1A	2630	985	Okay
29	S4/SH	371	349	244	131	260	616	502	762	1004	502	918		C1A	2630	985	Okay
30	S4/SK	301	145	102	52	260	402	352	612	593	352	768	64	C1A	2630	985	Okay
31	S5/SA	370	452	317	171	838	686	541	1379	1167	541	1881	800	C1A	2630	985	Okay
32	S5/SB	542	710	497	284	---	1039	826	826	1786	826	826		C1A	2630	985	Okay
33	S5/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
34	S5/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay
35	S5/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* Job No. 2711265

Subject **Summary of Column Loads and Combinations for SLS & ULS** Page no Sheet 2 of 3

Calculat Jamil Khan Reviewed By Date ########

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	$\Psi_s Q$	$\Psi_L Q$	Eu	$G+\Psi_s Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	$1.2G+1.6Q$	$G+\Psi_U Q$	$G+\Psi_U Q+E_O$	$G+\Psi_U Q-E_O$	Column Type	Ultimate	Fire Resistance	
36	S5/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
37	S4.8/SG	361	405	283	156	---	644	517	517	1081	517	517		C1A	2630	985	Okay
38	S4.8/SH	446	512	358	205	---	804	651	651	1354	651	651		C1A	2630	985	Okay
39	S4.8/SK	425	269	188	102	---	613	527	527	940	527	527		C1A	2630	985	Okay
40	S5.4/SG	390	501	351	189	---	741	579	579	1271	579	579		C1A	2630	985	Okay
41	S6/SA	370	452	317	171	---	686	541	541	1167	541	541		C1A	2630	985	Okay
42	S6/SB	542	710	497	284	---	1039	826	826	1786	826	826		C1A	2630	985	Okay
43	S6/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
44	S6/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay
45	S6/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay
46	S6/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
47	S6/SG	351	441	309	158	---	659	509	509	1126	509	509		C1A	2630	985	Okay
48	S6/SK	623	580	406	224	---	1029	848	848	1675	848	848		C1A	2630	985	Okay
49	S6.5/SG	314	396	277	158	---	591	472	472	1010	472	472		C1A	2630	985	Okay
50	S7/SA	370	452	317	171	137	686	541	678	1167	541	760		C1A	2630	985	Okay
51	S7/SB	542	710	497	284	---	1039	826	826	1786	826	826		C1A	2630	985	Okay
52	S7/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
53	S7/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay
54	S7/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay
55	S7/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
56	S7/SG	351	441	309	158	---	659	509	509	1126	509	509		C1A	2630	985	Okay
57	S7/SK	445	528	370	205	42	815	650	692	1379	650	717		C1A	2630	985	Okay
58	S7.5/SG	314	396	277	158	---	591	472	472	1010	472	472		C1A	2630	985	Okay
59	S8/SA	555	546	382	208	838	937	764	1602	1540	764	2104	577	C1A	2630	985	Okay
60	S8/SB	730	801	560	320	---	1291	1051	1051	2158	1051	1051		C4A	2630	1380	Okay
61	S8/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
62	S8/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay
63	S8/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay
64	S8/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
65	S8/SG	349	397	278	141	961	627	490	1451	1054	490	2028	1048	C1A	2630	985	Okay
66	S8/SK	300	282	197	106	260	498	407	667	811	407	823	9	C1A	2630	985	Okay
67	S9/SA	694	643	450	247	260	1144	941	1201	1862	941	1357		C1A	2630	985	Okay
68	S9/SB	907	898	629	359	---	1536	1267	1267	2526	1267	1267		C4A	2630	1380	Okay
69	S9/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
70	S9/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* Job No. 2711265

Subject **Summary of Column Loads and Combinations for SLS & ULS** Page no Sheet 3 of 3

Calculat Jamil Khan Reviewed By Date ########

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	$\Psi_s Q$	$\Psi_L Q$	Eu	$G+\Psi_s Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	1.2G + 1.6Q	$G+\Psi_U Q$	$G+\Psi_U Q+E_O$	$G+\Psi_U Q-E_O$	Column Type	Ultimate	Fire Resistance	
71	S9/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay
72	S9/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
73	S9/SG	312	319	223	119	183	536	431	614	885	431	724		C1A	2630	985	Okay
74	S9/SK	61	10	7	4	---	68	65	65	89	65	65		C1A	2630	985	Okay
75	S10/SA	694	643	450	247	137	1144	941	1078	1862	941	1160		C1A	2630	985	Okay
76	S10/SB	907	898	629	359	---	1536	1267	1267	2526	1267	1267		C4A	2630	1380	Okay
77	S10/SC	476	618	433	226	---	909	702	702	1560	702	702		C1A	2630	985	Okay
78	S10/SD	422	565	395	226	---	817	648	648	1410	648	648		C1A	2630	985	Okay
79	S10/SE	459	599	419	224	---	878	683	683	1509	683	683		C1A	2630	985	Okay
80	S10/SF	419	560	392	224	---	810	643	643	1398	643	643		C1A	2630	985	Okay
81	S10/SG	312	319	223	119	204	536	431	635	885	431	758		C1A	2630	985	Okay
82	S11/SA	359	306	214	120	137	573	479	616	921	479	698		C1A	2630	985	Okay
83	S11/SB	493	451	316	175	806	809	668	1474	1314	668	1957	622	C1A	2630	985	Okay
84	S11/SC	248	284	199	109	118	447	357	475	752	357	546		C1A	2630	985	Okay
85	S11/SD	248	284	199	109	118	447	357	475	752	357	546		C1A	2630	985	Okay
86	S11/SE	248	284	199	109	---	447	357	357	752	357	357		C1A	2630	985	Okay
87	S11/SF	248	284	199	109	924	447	357	1281	752	357	1835	1121	C1A	2630	985	Okay
88	S11/SG	232	98	68	37	905	300	269	1174	434	269	1717	1179	C1A	2630	985	Okay

CA1 250x6 SHS with 60MPa Fibre R/f Conc

Over strength Factor for Seismic ULS Loads 1.60

ϕ Factor for (1.2G+1.6Q) Load Combinatic 0.65

ϕ Factor for ($G+Q+E_U$) Load Combination 0.90

Design
Of
First Floor Columns

241059

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* Job No. 2711265

Subject **Summary of Column Loads For Column Design of Zone J @ First Floor Level** Page no Sheet 1 of 3

Calculat Jamil Khan Reviewed By Date

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	$\Psi_S Q$	$\Psi_L Q$	Eu	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	1.2G + 1.6Q	$G+\Psi_U Q$	$G+\Psi_U Q+E_O$	$G+\Psi_U Q-E_O$	Column Type	Ultimate	Fire Resistance	
1	S1/SA	12	6	4	0	---	16	12	12	24	0	12		C6A	1185		Okay
2	S1/S5	24	16	11	0	88	35	24	112	54	0	165	117	C6A	1185		Okay
3	S1/SC	20	13	9	0	88	29	20	108	45	0	161	121	C6A	1185		Okay
4	S1/SD	16	9	6	0	88	22	16	104	34	0	157	125	C6A	1185		Okay
5	S1/SE	17	11	8	0	88	25	17	105	38	0	158	124	C6A	1185		Okay
6	S1/SF	18	11	8	0	---	26	18	18	39	0	18		C6A	1185		Okay
7	S1/SF.5	11	5	4	0	---	15	11	11	21	0	11		C6A	1185		Okay
8	S2/SA	27	28	20	0	---	47	27	27	78	0	27		C6A	1185		Okay
9	S2/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
10	S2/SC	61	60	42	0	---	103	61	61	169	0	61		C5A	245		Okay
11	S2/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
12	S2/SE	47	48	33	0	---	81	47	47	133	0	47		C5A	245		Okay
13	S2/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
14	S2/SG	32	24	17	0	64	48	32	96	76	0	134	71	C6A	1185		Okay
15	S3/SA	15	15	11	0	---	25	15	15	42	0	15		C6A	1185		Okay
16	S3/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
17	S3/SC	36	33	23	0	---	59	36	36	96	0	36		C5A	245		Okay
18	S3/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
19	S3/SE	29	26	18	0	---	47	29	29	76	0	29		C5A	245		Okay
20	S3/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
21	S3/SG	19	13	9	0	119	28	19	138	43	0	209	172	C6A	1185		Okay
22	S4/SA	24	25	18	0	137	42	24	161	69	0	243	195	C6A	1185		Okay
23	S4/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
24	S4/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
25	S4/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
26	S4/SE	43	42	29	0	---	72	43	43	118	0	43		C5A	245		Okay
27	S4/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
28	S4/SG	42	45	32	0	260	73	42	302	122	0	458	374	C6A	1185		Okay
29	S4/SH	23	23	16	0	260	39	23	283	64	0	439	393	C6A	1185		Okay
30	S4/SK	23	16	11	0	260	34	23	283	53	0	439	393	C6A	1185		Okay
31	S5/SA	24	25	18	0	137	42	24	161	69	0	243	195	C6A	1185		Okay
32	S5/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
33	S5/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
34	S5/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
35	S5/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* Job No. 2711265

Subject **Summary of Column Loads For Column Design of Zone J @ First Floor Level** Page no Sheet 2 of 3

Calculat Jamil Khan Reviewed By Date

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	$\Psi_S Q$	$\Psi_L Q$	Eu	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	1.2G + 1.6Q	$G+\Psi_U Q$	$G+\Psi_U Q+E_O$	$G+\Psi_U Q-E_O$	Column Type	Ultimate	Fire Resistance	
36	S5/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
37	S4.8/SG	12	15	10	0	---	23	12	12	38	0	12		C5A	245		Okay
38	S4.8/SH	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
39	S4.8/SK	21	13	9	0	---	30	21	21	45	0	21		C6A	1185		Okay
40	S5.4/SG	25	30	21	0	---	46	25	25	78	0	25		C5A	245		Okay
41	S6/SA	24	25	18	0	---	42	24	24	69	0	24		C6A	1185		Okay
42	S6/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
43	S6/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
44	S6/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
45	S6/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay
46	S6/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
47	S6/SG	37	45	32	0	---	69	37	37	116	0	37		C5A	245		Okay
48	S6/SK	28	19	13	0	---	42	28	28	64	0	28		C6A	1185		Okay
49	S6.5/SG	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
50	S7/SA	24	25	18	0	137	42	24	161	69	0	243	195	C6A	1185		Okay
51	S7/SB	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
52	S7/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
53	S7/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
54	S7/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay
55	S7/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
56	S7/SG	37	45	32	0	---	69	37	37	116	0	37		C5A	245		Okay
57	S7/SK	24	16	11	0	42	35	24	66	54	0	91	43	C6A	1185		Okay
58	S7.5/SG	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
59	S8/SA	220	123	86	0	2?1	306	220	511	(461)	0	686	245	C7A			NG
60	S8/SB	204	98	68	0	154	272	204	358	401	0	450	43	---			NG
61	S8/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
62	S8/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
63	S8/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay
64	S8/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
65	S8/SG	37	45	32	0	260	69	57	297	116	0	453	379	C6A	1185		Okay
66	S8/SK	25	16	11	0	260	36	25	285	56	0	441	391	C6A	1185		Okay
67	S9/SA	326	220	154	0	154	481	326	480	(744)	0	573		C7A			NG
68	S9/SB	328	195	137	0	154	465	328	482	(707)	0	575		---			NG
69	S9/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
70	S9/SD	0	0	0	0	---	0	0		0	0	0	0	---			Okay

LOAD TAKING SUMMARY SHEET

Job nam *Queensgate Redevelopment, Zone J* Job No. 2711265

Subject **Summary of Column Loads For Column Design of Zone J @ First Floor Level** Page no Sheet 3 of 3

Calculat Jamil Khan Reviewed By Date

S No.	Column Marks	Dead load	Live Load			E.Q. Load	SLS Combinations		ULS Gravity Load		Fire Load	Over strength Combination		Column Capacity			
		G	Q	$\Psi_S Q$	$\Psi_L Q$	E_U	$G+\Psi_S Q$	$G+\Psi_L Q$	$G+\Psi_U Q+E_U$	$1.2G+1.6Q$	$G+\Psi_U Q$	$G+\Psi_U Q+E_O$	$G+\Psi_U Q-E_O$	Column Type	Ultimate	Fire Resistance	
71	S9/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay
72	S9/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
73	S9/SG	29	21	15	0	183	43	29	212	68	0	322	264	C6A	1185		Okay
74	S9/SK	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
75	S10/SA	326	220	154	0	291	481	326	617	744	0	792	139	C7A			NG
76	S10/SB	328	195	137	0	154	465	328	482	707	0	575		---			NG
77	S10/SC	55	53	37	0	---	92	55	55	150	0	55		C5A	245		Okay
78	S10/SD	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
79	S10/SE	40	39	27	0	---	67	40	40	110	0	40		C5A	245		Okay
80	S10/SF	0	0	0	0	---	0	0	0	0	0	0	0	---			Okay
81	S10/SG	29	21	15	0	204	43	29	233	68	0	355	298	C6A	1185		Okay
82	S11/SA	165	101	71	0	137	235	165	302	359	0	384	55	C7A			NG
83	S11/SB	217	110	77	0	---	294	217	217	436	0	217		C7A			NG
84	S11/SC	24	12	8	0	118	32	24	142	48	0	213	165	C6A	1185		Okay
85	S11/SD	24	12	8	0	118	32	24	142	48	0	213	165	C6A	1185		Okay
86	S11/SE	24	12	8	0	---	32	24	24	48	0	24		C6A	1185		Okay
87	S11/SF	24	12	8	0	118	32	24	142	48	0	213	165	C6A	1185		Okay
88	S11/SG	17	5	4	0	183	20	17	200	28	0	310	276	C6A	1185		Okay

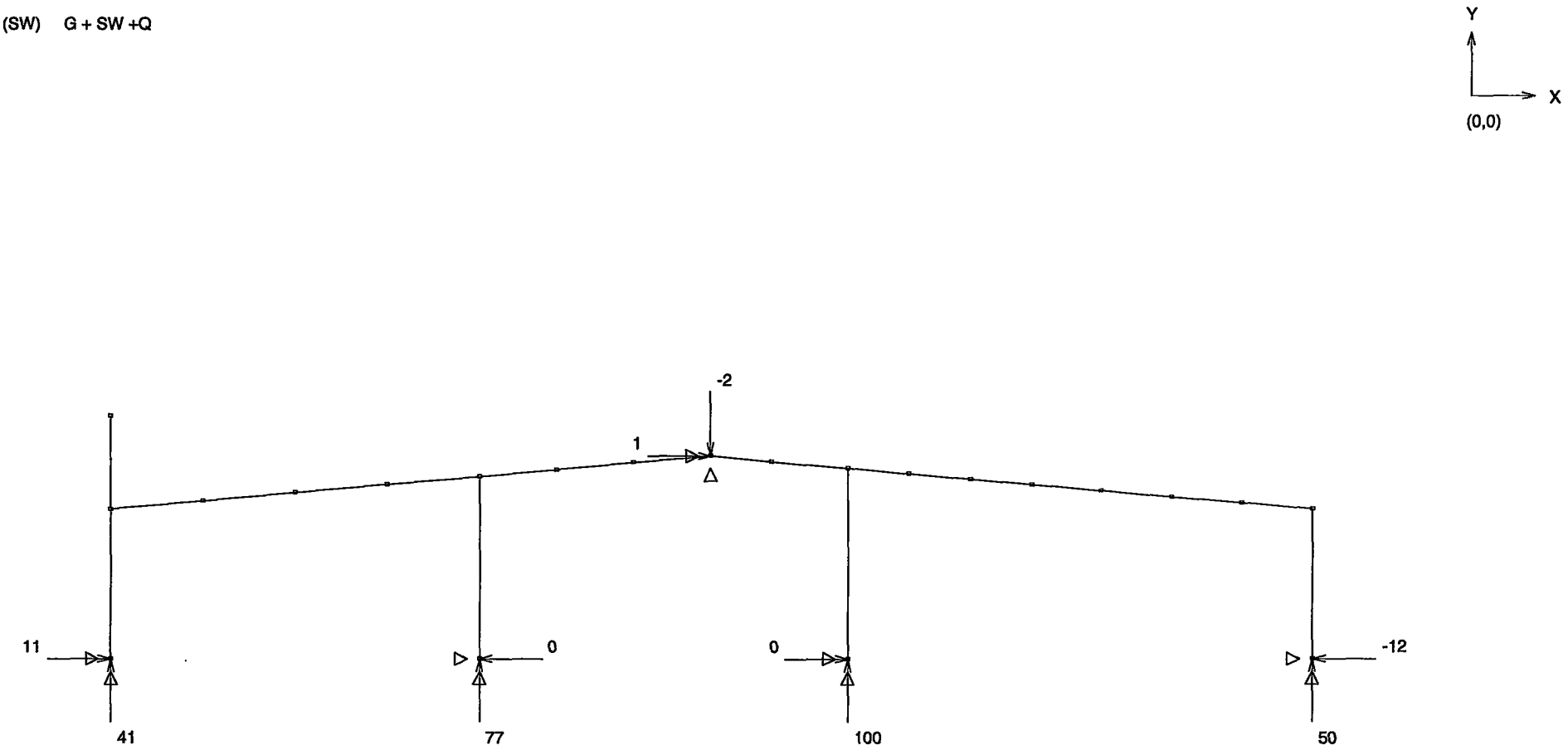
CA1 250x6 SHS with 60MPa Fibre R/f Conc

Over strength Factor for Seismic ULS Loads 1.60

241059

12-3

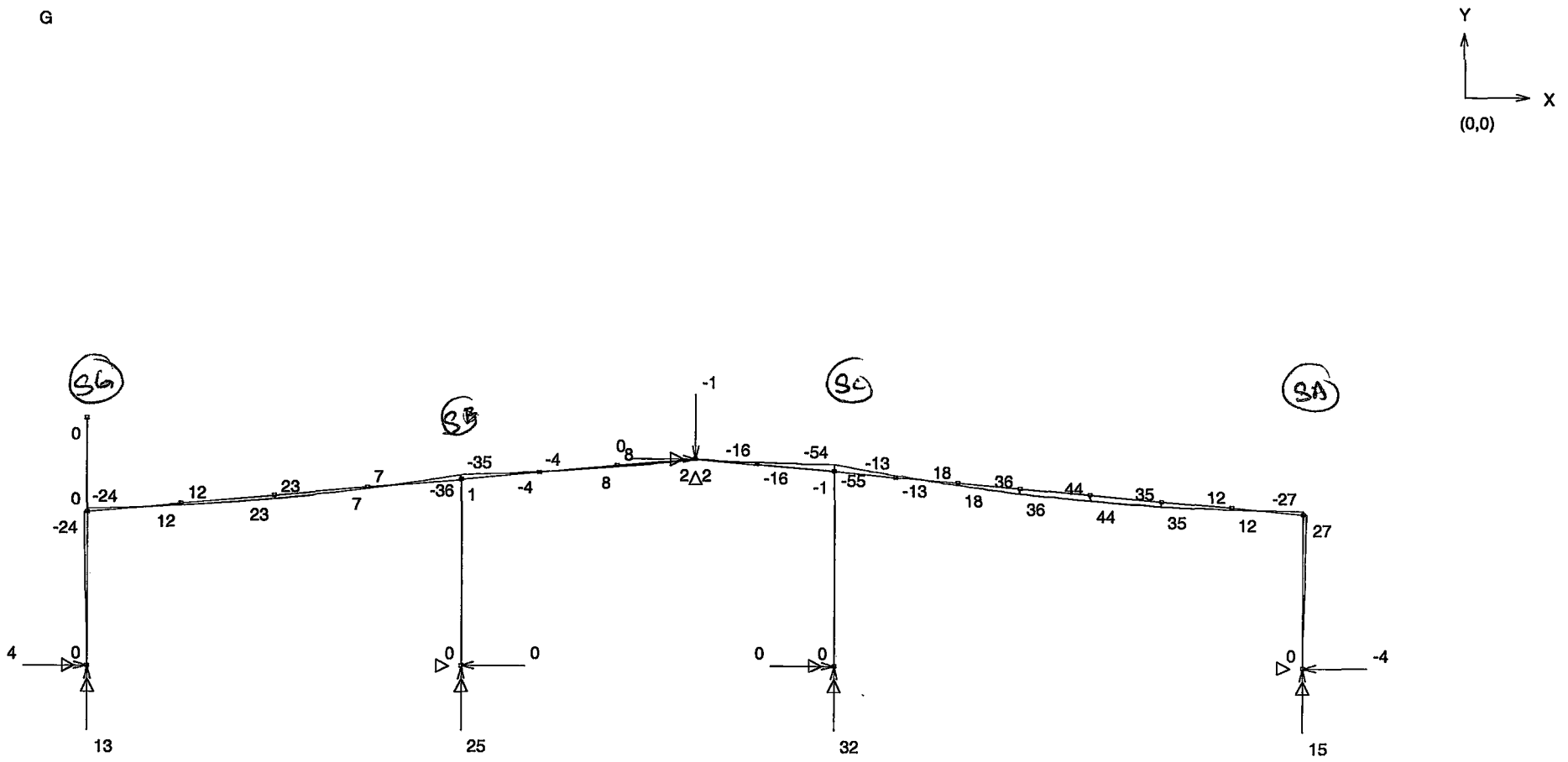
Load cases:
■ 14 ——— (SW) G + SW +Q



Portal frame @ GL S9, S10
No general restraint

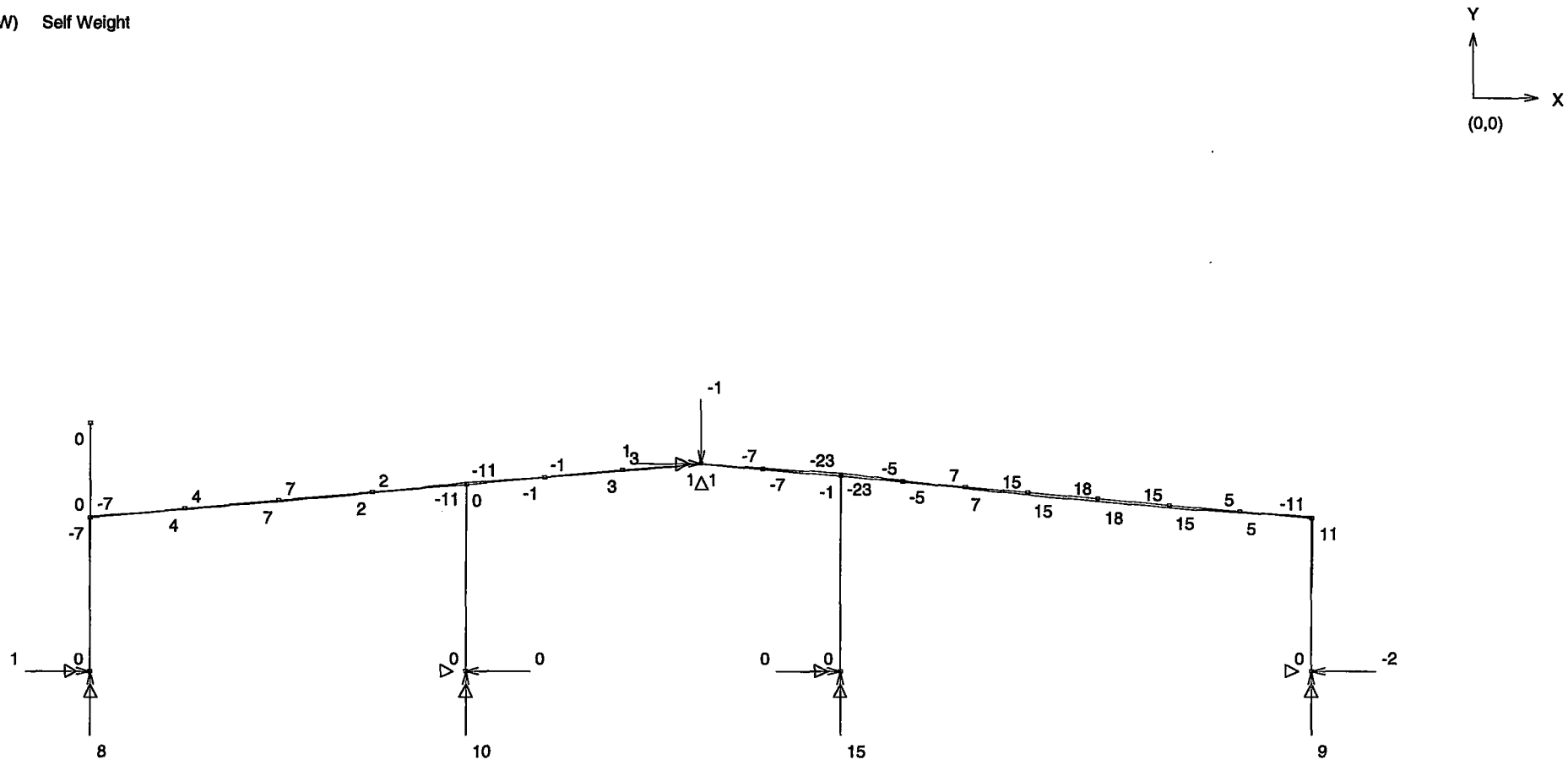
Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 3 ——— G



Dead Load Forces
No general restraint

Load cases:
■ 4 ——— (SW) Self Weight



Selfweigth Forces
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

i) Load combination $1.2G + 1.6Q$

$M_u = 107$ kN-m $\quad P_u = 50$ kN

ii) Load combination $0.9G + W_{WIND\,ULT}$

$M_u = 61$ kN-m $\quad P_u = -6$ kN

iii) Load combination $G + Q + E_u$ $\quad Q_{s.c.}$ Roof $= 0$

$G + E_u$

$M_u = 38$ kN-m $\quad P_u = 302$ kN

Section 310 UB 40.4

Effective Length

Flexural buckling (x-axis) = 6.5 m

Flexural buckling (y-axis) = ~~6.5~~ 3.25 m

Lateral buckling 3.25 m

} $1.2G+1.6Q$

Change to 4m ∴ need to Revised Section from 310 UB 40 to 310 UB 46 Pl. refer to revised calc

Design action effect

$N^* = 50$ kN

$M_x^* = 107$ kN-m

$M_y^* = 5$ kN-m

i) Section capacity check (Sec 8.3.3.1)

From table 8.1-5

$\phi N_s = 1430$ kN

$\phi M_{sx} = 182$ kN-m

$\phi M_{sy} = 40$ kN-m

$$n = \frac{N^*}{\phi N_s} = 0.035$$

Using Table 8.1-5

$\phi M_{rx} = 208(1-n) = 200.72 > \phi M_{sx}$

$\therefore \phi M_{rx} = \phi M_{sx} = 182$ kN-m

$\phi M_{ry} = 47.6(1-n^2) = 47.54$ kN-m $> \phi M_{sy}$

$\therefore \phi M_{ry} = \phi M_{sy} = 40$ kN-m

Now $\gamma = 1.4 + \left(\frac{N^*}{\phi N_s}\right) = 1.435 < 2.0$ ok.

Then $\left(\frac{M_x^*}{\phi M_{sx}}\right)^{\gamma} + \left(\frac{M_y^*}{\phi M_{sy}}\right)^{\gamma} = \left(\frac{107}{182}\right)^{1.435} + \left(\frac{5}{40}\right)^{1.435}$

$= 0.466 + 0.050 = 0.5166 < 1.0$ ok.

ii) Member Capacity Checks (Section 8.3.3.2)

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{cy}}\right)^{1.4} \leq 1.0$$

From The Tables 6-5 (1) $\phi N_{cx} = 1185$ kN ($L_x = 6.5$)

Table 6-5 (2) $\phi N_{cy} = 287.5$ kN

Table 5.3-5 $\phi M_{bx} = 120.75$ kN.

$\alpha_m = 1.75$

$\phi M_{bx} = \min(\alpha_m(\phi M_{bx}, \phi M_{sx})$

$= \min(211.30, 182)$

$\phi M_{bx} = 182$ kN-m.

Calculate in-plane and out-of Plane capacity

a) $\phi M_{ix} = \phi M_{sx}\left(1 - \frac{N^*}{\phi N_{cx}}\right) = 182\left(1 - \frac{50}{1185}\right)$

$\phi M_{ix} = 174.32$ kN-m.

b) $\phi M_{ox} = \phi M_{cx}\left(1 - \frac{N^*}{\phi N_{cy}}\right) = 182\left(1 - \frac{50}{287.5}\right)$

$= 150.35$

$\phi M_{ox} = 150.34$ kN-m.

$\therefore \phi M_{cx} = \min(\phi M_{ix}, \phi M_{ox}) = \min(174.32, 150.34)$

$= 150.34$ kN-m

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

$$\phi M_{iy} = \phi M_{sy}\left(1 - \frac{N^*}{\phi N_{cy}}\right) = 40\left(1 - \frac{50}{2875}\right) = 38.04$$

$$\phi M_{iy} = 33.04 \text{ kN-m}$$

Thus $$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} \leq 1.0$$

$$\left(\frac{107}{150.34}\right)^{1.4} + \left(\frac{5}{33.04}\right)^{1.4} \leq 1.0$$

$$0.621 + 0.071 \leq 1.0$$

$$0.692 \leq 1.0$$

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

SUBJECT: Zone J, Level 2, External Col @ GL (SG) for 1.2G+1.6Q Comb. — PAGE NO: OF

BY: Jamil Khan — DATE: 26/5/04

DESIGN DATA

Enter trial section size here **310 UB 46.2**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	107	kNm
Factored bending moment about minor Y - axis	M_y^* =	5	kNm
Length between X axis restraints	L_x =	4.000	m
Length between Y axis restraints	L_y =	4.000	m
Strength reduction factor	∅ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	729000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	163000	mm^3
Moment of inertia about X axis (table)	I_x =	100000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9010000	mm^4
Torsion constant (table)	J =	233000	mm^4
Warping constant (table)	I_w =	1.97E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

There should be another parameter for buckling length, as the spreadsheet use Lx as buckling length. The buckling length can be reduced by by introdcuing Fly braces. In absence of Bukling lenth paramenter, we use Lx = Ly = buckling length

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	50	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	5930	mm^2
Radius of gyration about X axis (table)	r_x =	130	mm
Radius of gyration about Y axis (table)	r_y =	39	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	5930	mm^2
Tensile strength	f_u =	410	MPa

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

SUBJECT: Zone J, Level 2, External Col @ GL (SG) for 1.2G+1.6Q Comb. — PAGE NO: OF

BY: Jamil Khan — DATE: 26/5/04

8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1.4 Have you checked if axial force is significant?

8.2 DESIGN ACTIONS

Factored bending moment about major X - axis	$M_x^* =$	107 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	5 kNm
Length between X axis restraints	$L_x =$	4.000 m
Length between Y axis restraints	$L_y =$	4.000 m

8.3 SECTION CAPACITY

8.3.2 Uniaxial Bending About Major Principal X-Axis

$M_x^* \le \varnothing M_{rx}$ 196.83

All sections
$\le \varnothing M_{sx}(1-N^*/\varnothing N_s)$ $\varnothing M_{rx} =$ 190 kNm

M_x^*		$\varnothing M_{rx}$	
107 kNm	≤	190 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18 M_{sx}(1-N^*/\varnothing N_s) \le \varnothing Msx$ $\varnothing M_{rx} =$ 225 kNm

$\varnothing M_{rx}$		$\varnothing M_{sx}$			
225 kNm	≥	197 kNm	∴	Ø Msx =	197 kNm is critical

M_x^*		Ø Msx =	
107 kNm	≤	197 kNm	∴ O.K.

8.3.3 Uniaxial Bending About Minor Principal Y-Axis

$M_y^* \le \varnothing M_{ry}$

All sections
$\le \varnothing M_{sy}(1-N^*/\varnothing N_s)$ $\varnothing M_{ry} =$ 43 kNm

M_y^*		$\varnothing M_{ry}$	
5 kNm	≤	43 kNm	∴ O.K.

I sections to Cl. 8.1.5
$\le \varnothing 1.19 M_{sy}(1-(N^*/\varnothing N_s)^2) \le \varnothing Msy$ $\varnothing M_{ry} =$ 52 kNm

$\varnothing M_{ry}$		$\varnothing M_{sy}$			
52 kNm	≥	44 kNm	∴	Ø Msy =	44 kNm is critical

M_y^*		Ø Msy =	
5 kNm	≤	44 kNm	∴ O.K.

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SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18 M_{sy}(1-N^*/\varnothing N_s) \le \varnothing Msy$ $\varnothing M_{ry} =$ 50 kNm

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CALCULATION SHEET

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50 kNm	**≥**	**44 kNm**	**∴**	**Ø Msy =**	**44 kNm is critical**

M_y^*		Ø Msy =
5 kNm	**≤**	**44 kNm** ∴ **O.K.**

8.3.4 Biaxial Bending

Enter $ØM_{rx}$ of applicable section 190 kNm kNm (ie. all, I, or hollow section)

Enter $ØM_{ry}$ of applicable section 43 kNm kNm (ie. all, I, or hollow section)

All sections

$N^*/ØN_s + M_x^*/ØM_{sx} + M_y^*/ØM_{sy} \leq 1.0$ **= 0.690**

O.K.

I or SHS or RHS sections to Cl. 8.1.5

$(M_x^*/ØM_{rx})^\gamma + (M_y^*/ØM_{ry})^\gamma \leq 1.0$ **= 0.484**

where $\gamma = 1.4 + (N^*/ØN_s) \leq 2.0$ = 1.433

O.K.

8.4 MEMBER CAPACITY

8.4.2 IN PLANE CAPACITY

8.4.2.2 Compression Members

$M^* \leq ØM_i$

All sections

$\leq ØM_s(1-N^*/ØN_c) \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}(1-N^*/ØN_{cx}) \leq ØM_{rx}$ $ØM_{ix} =$ 183 kNm

$ØM_{ix}$		$ØM_{rx}$		
183 kNm	**≤**	**190 kNm**	**∴**	**Ø Mix1 = 183 kNm is critical**

M_x^*		Ø Mix1 =
107 kNm	**≤**	**183 kNm** ∴ **O.K.**

- Bending about minor principal y-axis

$\leq ØM_{sy}(1-N^*/ØN_{cy}) \leq ØM_{ry}$ $ØM_{iy} =$ 41 kNm

$ØM_{iy}$		$ØM_{ry}$		
41 kNm	**≤**	**43 kNm**	**∴**	**Ø Miy1 = 41 kNm is critical**

M_y^*		Ø Miy1 =
5 kNm	**≤**	**41 kNm** ∴ **O.K.**

I or SHS or RHS sections to Cl. 8.1.5

$\leq ØM_s\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_c) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_c))}\} \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_{cx}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_{cx}))}\} \leq ØM_{rx}$

$ØM_{ix} =$ 319 kNm

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$\varnothing M_{ix}$		$\varnothing M_{rx}$			
319 kNm	≥	190 kNm	∴	Ø Mrx =	190 kNm is critical

M_x^*		Ø Mrx =	
107 kNm	≤	190 kNm	∴ O.K.

- Bending about minor principal y-axis

$$\leq \varnothing M_{sy}\{[1-((1+\beta_m)/2)^3](1-N^*/\varnothing N_{cy}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/\varnothing N_{cy}))}\} \leq \varnothing M_{ry}$$

$\varnothing M_{iy}$ = 72 kNm

$\varnothing M_{iy}$		$\varnothing M_{ry}$			
72 kNm	≥	43 kNm	∴	Ø Mry =	43 kNm is critical

M_y^*		Ø Mry =	
5 kNm	≤	43 kNm	∴ O.K.

Enter $\varnothing M_{ix}$ of applicable section 190 kNm (ie. all, I, or hollow section)
Enter $\varnothing M_{iy}$ of applicable section 43 kNm (ie. all, I, or hollow section)

8.4.2.3 Tension Members

A member subject to a tensile force and B.M. shall satisfy 8.3 SECTION CAPACITY

8.4.4 OUT OF PLANE CAPACITY

8.4.4.1 Compression Members

$M_x^* \leq \varnothing M_{ox}$

All sections

$\leq \varnothing M_{bx}(1-N^*/\varnothing N_{cy})$ $\varnothing M_{ox}$ = 110 kNm

M_x^*		$\varnothing M_{ox}$	
107 kNm	≤	110 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5

$\leq \varnothing\alpha_{bc}M_{bxo}\sqrt{[(1-N^*/\varnothing N_{cy})(1-(N^*/\varnothing N_{oz})]} \leq \varnothing M_{rx}$ $\varnothing M_{ox}$ = 141 kNm

where $1/\alpha_{bc} = 1-\beta_m/2 + (1+\beta_m/2)^3(0.4 - 0.23\ N^*/\varnothing N_{cy})$ = 0.797

α_{bc} = 1.255

$M_{bxo} = M_{bx} / \alpha_m \times \alpha_m = 1.0$ = 131 kNm

$N_{oz} = GJ + (\Pi^2 E I_w/L_z^2) / (I_x + I_y)/A$ = 2336 kN

L_z = Dist between partial or full retraints which prevent twist about centriod = 4.000 m

$\varnothing M_{ox}^*$		$\varnothing M_{rx}$			
141 kNm	≤	190 kNm	∴	Ø Mox2 =	141 kNm is critical

M_x^*		Ø Mox2 =	
107 kNm	≤	141 kNm	∴ O.K.

8.4.4.2 Tension Members

$M_x^* \leq \varnothing M_{ox}$

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CALCULATION SHEET

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All sections

$\leq \varnothing M_{bx}(1+N^*/\varnothing N_t) \leq M_{rx}$ $\varnothing M_{ox} =$ 122 kNm

$\varnothing M_{ox}$		$\varnothing M_{rx}$		
122 kNm	**≤**	**190 kNm**	**∴**	**Ø Mox3 = 122 kNm is critical**

M_x^*		Ø Mox3 =	
107 kNm	**≤**	**122 kNm**	**∴ O.K.**

8.4.5 BIAXIAL BENDING CAPACITY

8.4.5.1 Compression Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{cx} = \varnothing M_{lx}$

For partial or no restraint **$\varnothing M_{cx}$ = Lesser of $\varnothing M_{lx}$ or $\varnothing M_{ox}$**

Enter $\varnothing M_{cx}$ for applicable restraint, $\varnothing M_{cx}$ = 190 kNm

$(M_x^*/\varnothing M_{cx})^{1.4} + (M_y^*/\varnothing M_{ly})^{1.4} \leq 1.0$ **= 0.496**

O.K.

8.4.5.2 Tension Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{tx} = \varnothing M_{rx}$

For partial or no restraint **$\varnothing M_{tx}$ = Lesser of $\varnothing M_{rx}$ or $\varnothing M_{ox}$**

Enter $\varnothing M_{tx}$ for applicable restraint, $\varnothing M_{tx}$ = 190 kNm

$(M_x^*/\varnothing M_{tx})^{1.4} + (M_y^*/\varnothing M_{ry})^{1.4} \leq 1.0$ **= 0.496**

O.K.

Calculation Sheet

Job name: Job No:

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i) Load combination $1.2G + 1.6Q$

$M_u = 107$ kN-m $\quad P_u = 50$ kN

ii) Load combination $0.9G + W_{WIND\ ULT}$

$M_u = 61$ kN-m $\quad P_u = -6$ kN

iii) Load combination $G + Q + E_u$

Q for Roof $= 0$

$G + E_u$

$M_u = 38$ kN-m $\quad P_u = 302$ kN

Section 310 UB 40.4

Effective Length

Flexural buckling (x-axis) = 6.5 m

Flexural buckling (y-axis) = 3.25 m

Lateral buckling = 3.25 m

4.0 m

The section need to revised for 310UB40.4 to 310UB46

Pl. section to revised Cols

Design action effects

$N^* = 302$ kN

$M_x^* = 38$ kN-m

$M_y^* = 3$ kN-m

$\left.\begin{matrix} \\ \\ \end{matrix}\right\} G + Q^{\circ} + E_u$

ii) Section capacity checks (Sec 8.3.3.1)

From table 8.1-5

$\phi N_s = 1430$ kN

$\phi M_{sx} = 182$ kN-m

$\phi M_{sy} = 40$ kN-m

$$n = \frac{N^*}{\phi N_s} = 0.211$$

Using Table 8.1-5

$\phi M_{rx} = 208(1-n) = 164.12 < \phi M_{sx}$

$\therefore \phi M_{rx} = 164.12$ kN-m

$\phi M_{ry} = 47.6(1-n^2) = 45.88$ kN-m $> \phi M_{sy}$

$\therefore \phi M_{ry} = \phi M_{sy} = 40$ kN-m.

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Calculation Sheet

Job name: Job No:

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Now $\gamma = 1.4 + \left(\frac{N^*}{\phi N_s}\right) = 1.611 < 2.0$ ok y.

Then $\left(\frac{M_x^*}{\phi M_{sx}}\right)^{\gamma} + \left(\frac{M_y^*}{\phi M_{sy}}\right)^{\gamma} = \left(\frac{107}{164}\right)^{1.611} + \left(\frac{5}{40}\right)^{1.611}$

$= 0.502 + 0.035 = 0.537 < 1.0$ ok y.

ii) Member Capacity Checks (Section 8.3.3.2)

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{cy}}\right)^{1.4} \leq 1.0$$

From the Tables 6-5(A) $\phi N_{cx} = 1185$ kN ($L_x = 6.5$)

Table 6-5(B) $\phi N_{cy} = 764.6$ kN

Table 5.3-5 $\phi M_{bx} = 120.75$ kN.

$\alpha_m = 1.75$

$\phi M_{bx} = \min(\alpha_m(\phi M_{bx}, \phi M_{sx})$

$= \min(211.30, 182)$

$\phi M_{bx} = 182$ kN-m.

Calculate in-plane and out-of Plane capacity.

a) $\phi M_{ix} = \phi M_{sx}\left(1 - \frac{N^*}{\phi N_{cx}}\right) = 182\left(1 - \frac{302}{1185}\right)$

$\phi M_{ix} = 135.61$ kN-m.

b) $\phi M_{ox} = \phi M_{sx}\left(1 - \frac{N^*}{\phi N_{cy}}\right) = 182\left(1 - \frac{302}{764}\right)$

$= 110.06$

$\phi M_{ox} = 110.06$ kN-m.

$\therefore \phi M_{cx} = \min(\phi M_{ix}, \phi M_{ox}) = \min(135.61, 110.06)$

$= 110.06$ kN-m

Calculation Sheet

Job name: Job No:

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$$\phi M_{iy} = \phi M_{sy}\left(1 - \frac{N^*}{\phi N_{cy}}\right) = 40\left(1 - \frac{302}{764}\right) = 24.18$$

$\phi M_{iy} = 24.18$ kN-m.

Thus $$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} < 1.0$$

$$\left(\frac{38}{110.4}\right)^{1.4} + \left(\frac{3}{24.18}\right)^{1.4} < 1.0$$

$$0.225 + 0.053 < 1.0$$

$$0.278 < 1.0$$

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CALCULATION SHEET

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DESIGN DATA

Enter trial section size here **310 UB 46.2**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	38	kNm
Factored bending moment about minor Y - axis	M_y^* =	5	kNm
Length between X axis restraints	L_x =	8.000	m
Length between Y axis restraints	L_y =	4.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	729000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	163000	mm^3
Moment of inertia about X axis (table)	I_x =	100000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9010000	mm^4
Torsion constant (table)	J =	233000	mm^4
Warping constant (table)	I_w =	1.97E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	302	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	5930	mm^2
Radius of gyration about X axis (table)	r_x =	130	mm
Radius of gyration about Y axis (table)	r_y =	39	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	302	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	5930	mm^2
Tensile strength	f_u =	410	MPa

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8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1.4 Have you checked if axial force is significant?

8.2 DESIGN ACTIONS

Factored bending moment about major X - axis	M_x^* =	38 kNm
Factored bending moment about minor Y - axis	M_y^* =	5 kNm
Length between X axis restraints	L_x =	8.000 m
Length between Y axis restraints	L_y =	4.000 m

8.3 SECTION CAPACITY

8.3.2 Uniaxial Bending About Major Principal X-Axis

$M_x^* \le \phi M_{rx}$

All sections
$\le \phi M_{sx}(1-N^*/\phi N_s)$ ϕM_{rx} = 158 kNm

M_x^*		ϕM_{rx}	
38 kNm	**≤**	**158 kNm**	**∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5
$\le \phi 1.18M_{sx}(1-N^*/\phi N_s) \le \phi Msx$ ϕM_{rx} = 186 kNm

ϕM_{rx}		ϕM_{sx}			
186 kNm	**≤**	**197 kNm**	**∴**	**Ø Mrx2 =**	**186 kNm is critical**

M_x^*		Ø Mrx2 =	
38 kNm	**≤**	**186 kNm**	**∴ O.K.**

8.3.3 Uniaxial Bending About Minor Principal Y-Axis

$M_y^* \le \phi M_{ry}$

All sections
$\le \phi M_{sy}(1-N^*/\phi N_s)$ ϕM_{ry} = 35 kNm

M_y^*		ϕM_{ry}	
5 kNm	**≤**	**35 kNm**	**∴ O.K.**

I sections to Cl. 8.1.5
$\le \phi 1.19M_{sy}(1-(N^*/\phi N_s)^2) \le \phi Msy$ ϕM_{ry} = 50 kNm

ϕM_{ry}		ϕM_{sy}			
50 kNm	**≥**	**44 kNm**	**∴**	**Ø Msy =**	**44 kNm is critical**

M_y^*		Ø Msy =	
5 kNm	**≤**	**44 kNm**	**∴ O.K.**

SHS or RHS sections to Cl. 8.1.5
$\le \phi 1.18M_{sy}(1-N^*/\phi N_s) \le \phi Msy$ ϕM_{ry} = 42 kNm

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42 kNm ≤ 44 kNm ∴ Ø Mry3 = 42 kNm is critical

M_y^*	Ø Mry3 =	
5 kNm	≤ 42 kNm	∴ O.K.

8.3.4 Biaxial Bending

Enter $ØM_{rx}$ of applicable section 158 kNm kNm (ie. all, I, or hollow section)
Enter $ØM_{ry}$ of applicable section 35 kNm kNm (ie. all, I, or hollow section)

All sections
$N^*/ØN_s + M_x^*/ØM_{sx} + M_y^*/ØM_{sy} \leq 1.0$ = 0.504

O.K.

I or SHS or RHS sections to Cl. 8.1.5
$(M_x^*/ØM_{rx})^\gamma + (M_y^*/ØM_{ry})^\gamma \leq 1.0$ = 0.147

where $\gamma = 1.4 + (N^*/ØN_s) \leq 2.0$ = 1.598

O.K.

8.4 MEMBER CAPACITY

8.4.2 IN PLANE CAPACITY

8.4.2.2 Compression Members

$M^* \leq ØM_i$

All sections
$\leq ØM_s(1-N^*/ØN_c) \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}(1-N^*/ØN_{cx}) \leq ØM_{rx}$ $ØM_{ix}$ = 116 kNm

$ØM_{ix}$	$ØM_{rx}$		
116 kNm	≤ 158 kNm	∴	Ø Mix1 = 116 kNm is critical

M_x^*	Ø Mix1 =
38 kNm	≤ 116 kNm ∴ O.K.

- Bending about minor principal y-axis

$\leq ØM_{sy}(1-N^*/ØN_{cy}) \leq ØM_{ry}$ $ØM_{iy}$ = 26 kNm

$ØM_{iy}$	$ØM_{ry}$		
26 kNm	≤ 35 kNm	∴	Ø Miy1 = 26 kNm is critical

M_y^*	Ø Miy1 =
5 kNm	≤ 26 kNm ∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5
$\leq ØM_s\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_c) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_c))}\} \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_{cx}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_{cx}))}\} \leq ØM_{rx}$
$ØM_{ix}$ = 328 kNm

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$\varnothing M_{lx}$		$\varnothing M_{rx}$			
328 kNm	≥	158 kNm	∴	Ø Mrx =	158 kNm is critical

M_x^*		Ø Mrx =	
38 kNm	≤	158 kNm	∴ O.K.

- Bending about minor principal y-axis

$$\leq \varnothing M_{sy}\{[1-((1+\beta_m)/2)^3](1-N^*/\varnothing N_{cy}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/\varnothing N_{cy}))}\} \leq \varnothing M_{ry}$$

$\varnothing M_{iy}$ = 73 kNm

$\varnothing M_{ly}$		$\varnothing M_{ry}$			
73 kNm	≥	35 kNm	∴	Ø Mry =	35 kNm is critical

M_y^*		Ø Mry =	
5 kNm	≤	35 kNm	∴ O.K.

Enter $\varnothing M_{ix}$ of applicable section 157.91371 kNm (ie. all, I, or hollow section)
Enter $\varnothing M_{iy}$ of applicable section 35.308553 kNm (ie. all, I, or hollow section)

8.4.2.3 Tension Members

A member subject to a tensile force and B.M. shall satisfy 8.3 SECTION CAPACITY

8.4.4 OUT OF PLANE CAPACITY

8.4.4.1 Compression Members

$M_x^* \leq \varnothing M_{ox}$

All sections

$\leq \varnothing M_{bx}(1-N^*/\varnothing N_{cy})$ $\varnothing M_{ox}$ = 36 kNm

M_x^*		$\varnothing M_{ox}$	
38 kNm	≥	36 kNm	∴ N.G. - Reselect trial section size

I or SHS or RHS sections to Cl. 8.1.5

$\leq \varnothing\alpha_{bc}M_{bxo}\sqrt{[(1-N^*/\varnothing N_{cy})(1-(N^*/\varnothing N_{oz})]} \leq \varnothing M_{rx}$ $\varnothing M_{ox}$ = 77 kNm

where $1/\alpha_{bc} = 1-\beta_m/2 + (1+\beta_m/2)^3(0.4 - 0.23\ N^*/\varnothing N_{cy})$ = 0.529

α_{bc} = 1.889

$M_{bxo} = M_{bx} / \alpha_m \times \alpha_m$=1.0 = 68 kNm

$N_{oz} = GJ + (\Pi^2 E I_w/L_z^2) / (I_x + I_y)/A$ = 1345 kN

L_z = Dist between partial or full retraints which prevent twist about centriod = 8.000 m

$\varnothing M_{ox}^*$		$\varnothing M_{rx}$			
77 kNm	≤	158 kNm	∴	Ø Mox2 =	77 kNm is critical

M_x^*		Ø Mox2 =	
38 kNm	≤	77 kNm	∴ O.K.

8.4.4.2 Tension Members

$M_x^* \leq \varnothing M_{ox}$

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All sections

$\leq \varnothing M_{bx}(1+N^*/\varnothing N_t) \leq M_{rx}$ $\varnothing M_{ox} =$ 73 kNm

$\varnothing M_{ox}$		$\varnothing M_{rx}$			
73 kNm	**≤**	**158 kNm**	**∴**	**Ø Mox3 =**	**73 kNm is critical**

M_x^*		Ø Mox3 =	
38 kNm	**≤**	**73 kNm**	**∴ O.K.**

8.4.5 BIAXIAL BENDING CAPACITY

8.4.5.1 Compression Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{cx} = \varnothing M_{lx}$

For partial or no restraint **$\varnothing M_{cx}$ = Lesser of $\varnothing M_{lx}$ or $\varnothing M_{ox}$**

Enter $\varnothing M_{cx}$ for applicable restraint, $\varnothing M_{cx}$ = 158 kNm

$(M_x^*/\varnothing M_{cx})^{1.4} + (M_y^*/\varnothing M_{ly})^{1.4} \leq 1.0$ **= 0.201**

O.K.

8.4.5.2 Tension Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{tx} = \varnothing M_{rx}$

For partial or no restraint **$\varnothing M_{tx}$ = Lesser of $\varnothing M_{rx}$ or $\varnothing M_{ox}$**

Enter $\varnothing M_{tx}$ for applicable restraint, $\varnothing M_{tx}$ = 158 kNm

$(M_x^*/\varnothing M_{tx})^{1.4} + (M_y^*/\varnothing M_{ry})^{1.4} \leq 1.0$ **= 0.201**

O.K.

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CALCULATION SHEET

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DESIGN DATA

Enter trial section size here 310 UB 46.2
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	55	kNm
Factored bending moment about minor Y - axis	M_y^* =	25	kNm
Length between X axis restraints	L_x =	4.000	m
Length between Y axis restraints	L_y =	4.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	729000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	163000	mm^3
Moment of inertia about X axis (table)	I_x =	100000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9010000	mm^4
Torsion constant (table)	J =	233000	mm^4
Warping constant (table)	I_w =	1.97E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

There should be another parameter for buckling length, as the spreadsheet use Lx as buckling length. Th buckling length can be reduced by by introdcuing Fly braces. In absence of Bukling lenth paramenter, we use Lx = Ly = buckling length

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	50	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	5930	mm^2
Radius of gyration about X axis (table)	r_x =	130	mm
Radius of gyration about Y axis (table)	r_y =	39	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	5930	mm^2
Tensile strength	f_u =	410	MPa

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8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1.4 Have you checked if axial force is significant?

8.2 DESIGN ACTIONS

Factored bending moment about major X - axis	M_x^* =	55 kNm
Factored bending moment about minor Y - axis	M_y^* =	25 kNm
Length between X axis restraints	L_x =	4.000 m
Length between Y axis restraints	L_y =	4.000 m

8.3 SECTION CAPACITY

8.3.2 Uniaxial Bending About Major Principal X-Axis

$M_x^* \le \varnothing M_{rx}$ 196.83

All sections
$\le \varnothing M_{sx}(1-N^*/\varnothing N_s)$ $\varnothing M_{rx}$ = 190 kNm

M_x^*		$\varnothing M_{rx}$	
55 kNm	**≤**	**190 kNm**	**∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sx}(1-N^*/\varnothing N_s) \le \varnothing Msx$ $\varnothing M_{rx}$ = 225 kNm

$\varnothing M_{rx}$		$\varnothing M_{sx}$			
225 kNm	**≥**	**197 kNm**	**∴**	**Ø Msx =**	**197 kNm is critical**

M_x^*		Ø Msx =	
55 kNm	**≤**	**197 kNm**	**∴ O.K.**

8.3.3 Uniaxial Bending About Minor Principal Y-Axis

$M_y^* \le \varnothing M_{ry}$

All sections
$\le \varnothing M_{sy}(1-N^*/\varnothing N_s)$ $\varnothing M_{ry}$ = 43 kNm

M_y^*		$\varnothing M_{ry}$	
25 kNm	**≤**	**43 kNm**	**∴ O.K.**

I sections to Cl. 8.1.5
$\le \varnothing 1.19M_{sy}(1-(N^*/\varnothing N_s)^2) \le \varnothing Msy$ $\varnothing M_{ry}$ = 52 kNm

$\varnothing M_{ry}$		$\varnothing M_{sy}$			
52 kNm	**≥**	**44 kNm**	**∴**	**Ø Msy =**	**44 kNm is critical**

M_y^*		Ø Msy =	
25 kNm	**≤**	**44 kNm**	**∴ O.K.**

SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sy}(1-N^*/\varnothing N_s) \le \varnothing Msy$ $\varnothing M_{ry}$ = 50 kNm

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CALCULATION SHEET

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50 kNm	**≥**	**44 kNm**	**∴**	**Ø Msy =**	**44 kNm is critical**

M_y^*		Ø Msy =	
25 kNm	**≤**	**44 kNm**	**∴ O.K.**

8.3.4 Biaxial Bending

Enter $ØM_{rx}$ of applicable section 190 kNm kNm (ie. all, I, or hollow section)
Enter $ØM_{ry}$ of applicable section 43 kNm kNm (ie. all, I, or hollow section)

All sections
$N^*/ØN_s + M_x^*/ØM_{sx} + M_y^*/ØM_{sy} \leq 1.0$ **= 0.880**

O.K.

I or SHS or RHS sections to Cl. 8.1.5
$(M_x^*/ØM_{rx})^\gamma + (M_y^*/ØM_{ry})^\gamma \leq 1.0$ **= 0.635**

where $\gamma = 1.4 + (N^*/ØN_s) \leq 2.0$ = 1.433

O.K.

8.4 MEMBER CAPACITY

8.4.2 IN PLANE CAPACITY

8.4.2.2 Compression Members

$M^* \leq ØM_i$

All sections
$\leq ØM_s(1-N^*/ØN_c) \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}(1-N^*/ØN_{cx}) \leq ØM_{rx}$ $ØM_{ix} =$ 183 kNm

$ØM_{ix}$		$ØM_{rx}$		
183 kNm	**≤**	**190 kNm**	**∴**	**Ø Mix1 = 183 kNm is critical**

M_x^*		Ø Mix1 =	
55 kNm	**≤**	**183 kNm**	**∴ O.K.**

- Bending about minor principal y-axis

$\leq ØM_{sy}(1-N^*/ØN_{cy}) \leq ØM_{ry}$ $ØM_{iy} =$ 41 kNm

$ØM_{iy}$		$ØM_{ry}$		
41 kNm	**≤**	**43 kNm**	**∴**	**Ø Miy1 = 41 kNm is critical**

M_y^*		Ø Miy1 =	
25 kNm	**≤**	**41 kNm**	**∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5
$\leq ØM_s\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_c) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_c))}\} \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_{cx}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_{cx}))}\} \leq ØM_{rx}$

$ØM_{ix} =$ 319 kNm

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$\varnothing M_{ix}$		$\varnothing M_{rx}$			
319 kNm	**≥**	**190 kNm**	**∴**	**Ø Mrx =**	**190 kNm is critical**

M_x^*		Ø Mrx =	
55 kNm	**≤**	**190 kNm**	**∴ O.K.**

- Bending about minor principal y-axis

$$\leq \varnothing M_{sy}\{[1-((1+\beta_m)/2)^3](1-N^*/\varnothing N_{cy}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/\varnothing N_{cy}))}\} \leq \varnothing M_{ry}$$

$\varnothing M_{iy}$ = 72 kNm

$\varnothing M_{iy}$		$\varnothing M_{ry}$			
72 kNm	**≥**	**43 kNm**	**∴**	**Ø Mry =**	**43 kNm is critical**

M_y^*		Ø Mry =	
25 kNm	**≤**	**43 kNm**	**∴ O.K.**

Enter $\varnothing M_{ix}$ of applicable section 190 kNm (ie. all, I, or hollow section)

Enter $\varnothing M_{iy}$ of applicable section 43 kNm (ie. all, I, or hollow section)

8.4.2.3 Tension Members

A member subject to a tensile force and B.M. shall satisfy 8.3 SECTION CAPACITY

8.4.4 OUT OF PLANE CAPACITY

8.4.4.1 Compression Members

$M_x^* \leq \varnothing M_{ox}$

All sections

$\leq \varnothing M_{bx}(1-N^*/\varnothing N_{cy})$ $\varnothing M_{ox}$ = 110 kNm

M_x^*		$\varnothing M_{ox}$	
55 kNm	**≤**	**110 kNm**	**∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5

$\leq \varnothing\alpha_{bc}M_{bxo}\sqrt{[(1-N^*/\varnothing N_{cy})(1-(N^*/\varnothing N_{oz})]} \leq \varnothing M_{rx}$ $\varnothing M_{ox}$ = 141 kNm

where $1/\alpha_{bc} = 1-\beta_m/2 + (1+\beta_m/2)^3(0.4 - 0.23\ N^*/\varnothing N_{cy})$ = 0.797

α_{bc} = 1.255

$M_{bxo} = M_{bx} / \alpha_m \times \alpha_m$=1.0 = 131 kNm

$N_{oz} = GJ + (\Pi^2 E I_w/L_z^2) / (I_x + I_y)/A$ = 2336 kN

L_z = Dist between partial or full retraints which prevent twist about centriod = 4.000 m

$\varnothing M_{ox}^*$		$\varnothing M_{rx}$			
141 kNm	**≤**	**190 kNm**	**∴**	**Ø Mox2 =**	**141 kNm is critical**

M_x^*		Ø Mox2 =	
55 kNm	**≤**	**141 kNm**	**∴ O.K.**

8.4.4.2 Tension Members

$M_x^* \leq \varnothing M_{ox}$

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All sections

$\leq \varnothing M_{bx}(1+N^*/\varnothing N_t) \leq M_{rx}$ $\varnothing M_{ox} =$ 122 kNm

$\varnothing M_{ox}$		$\varnothing M_{rx}$		
122 kNm	**≤**	**190 kNm**	**∴**	**Ø Mox3 = 122 kNm is critical**

M_x^*		Ø Mox3 =	
55 kNm	**≤**	**122 kNm**	**∴ O.K.**

8.4.5 BIAXIAL BENDING CAPACITY

8.4.5.1 Compression Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{cx} = \varnothing M_{lx}$

For partial or no restraint $\varnothing M_{cx}$ = **Lesser of** $\varnothing M_{lx}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{cx}$ for applicable restraint, $\varnothing M_{cx}$ = 190 kNm

$(M_x^*/\varnothing M_{cx})^{1.4} + (M_y^*/\varnothing M_{ly})^{1.4} \leq 1.0$ **= 0.650**

O.K.

8.4.5.2 Tension Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{tx} = \varnothing M_{rx}$

For partial or no restraint $\varnothing M_{tx}$ = **Lesser of** $\varnothing M_{rx}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{tx}$ for applicable restraint, $\varnothing M_{tx}$ = 190 kNm

$(M_x^*/\varnothing M_{tx})^{1.4} + (M_y^*/\varnothing M_{ry})^{1.4} \leq 1.0$ **= 0.650**

O.K.

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Calculation Sheet

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Internal columns.

Maximum Ultimate Downward load on column is 144 kN.

Maximum Ultimate upward tension on column is 64 kN.

From Design Charts 42-25

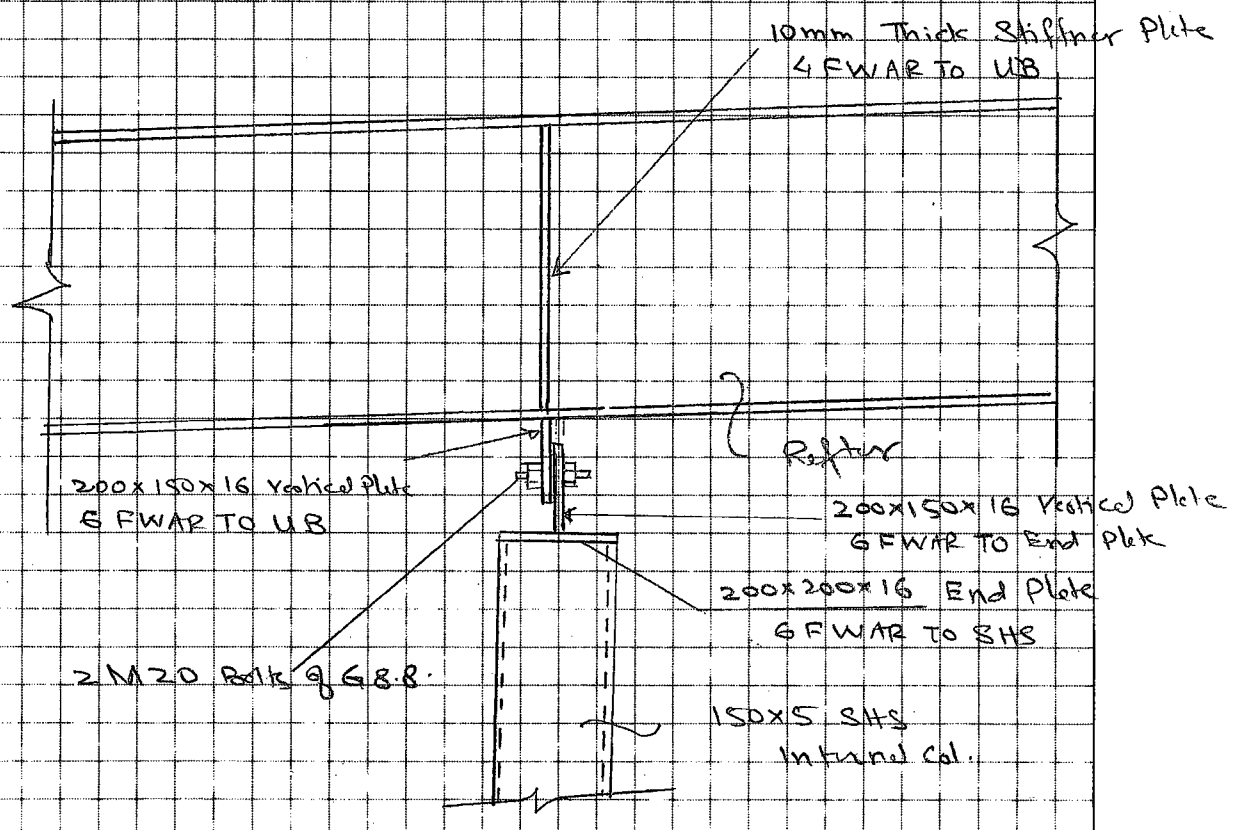
Design Capacity of 150x5 SHS

For L_e = 6m P_{ux} = 406 kN

For L_e = 8m P_{ux} = 245 kN.

For L_e = 10m P_{ux} = 161 kN.

∴ Section is okay

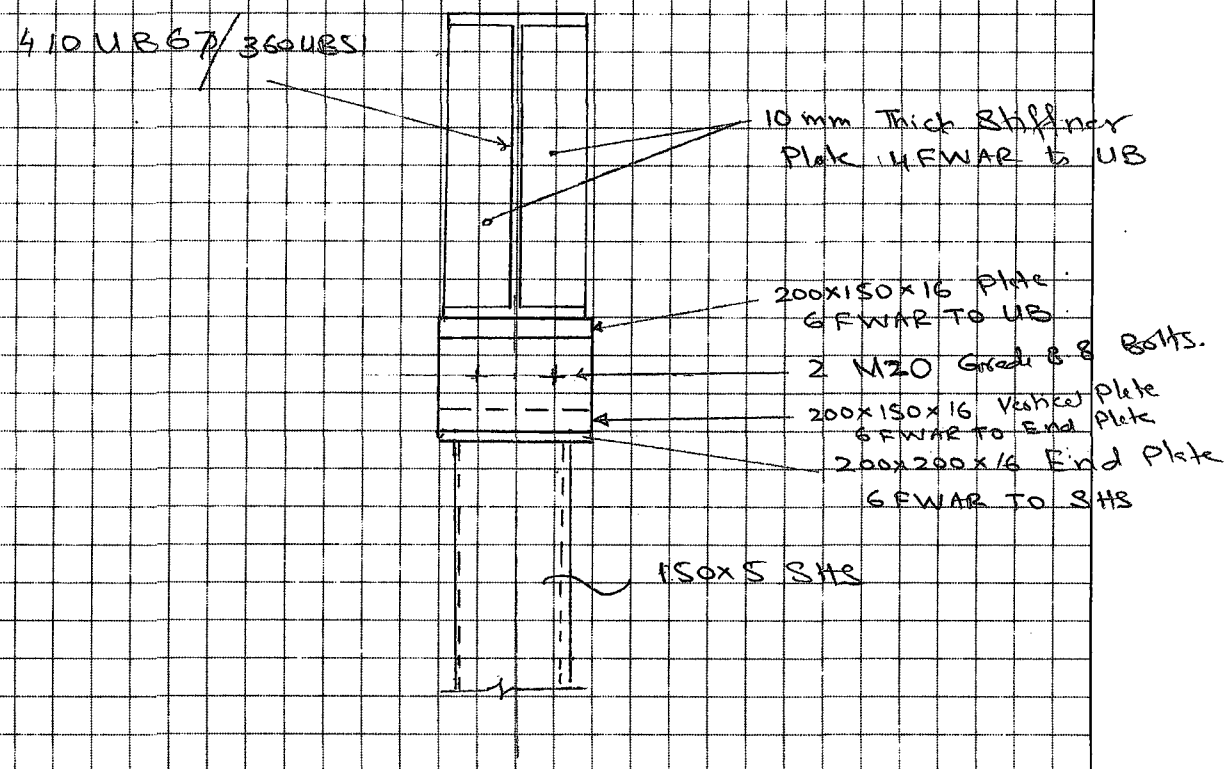


Calculation Sheet

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Connection Detail between Internal Column and Roof Rafter.

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CALCULATION SHEET

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DESIGN DATA

Enter trial section size here **150x150x5.0 SHS**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	0	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	8.350	m
Length between Y axis restraints	L_y =	8.350	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	144000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	144000	mm^3
Moment of inertia about X axis (table)	I_x =	9700000	mm^4
Moment of inertia about Y axis (table)	I_y =	9700000	mm^4
Torsion constant (table)	J =	15600	mm^4
Warping constant (table)	I_w =	0	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	144	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	2810	mm^2
Radius of gyration about X axis (table)	r_x =	58.7	mm
Radius of gyration about Y axis (table)	r_y =	58.7	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	2	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	64	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

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CALCULATION SHEET

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6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load $N^* =$ 144 kN

6.1 DESIGN FOR AXIAL COMPRESSION

$N^* \le \varnothing N_s$ and
$N^* \le \varnothing N_c$

6.2 NOMINAL SECTION CAPACITY

$N^* \le \varnothing N_s$
$\le \varnothing k_f A_n f_y$

Young's Modulus	$E =$	200 GPa
Minimum value of yield stress for any element	$f_y =$	250 MPa
Strength reduction factor	$\varnothing =$	0.9
Length between supports - X axis	$L_x =$	8.350 m
Length between supports - Y axis	$L_y =$	8.350 m
Effective length factor about X axis	$k_{ex} =$	1
Effective length factor about Y axis	$k_{ey} =$	1
Form factor (by calculation)	$k_f =$	0.954
Gross area of cross section (use nett if approp.) (table)	$A =$	2810 mm^2
	$\varnothing N_s =$	**603 kN**

N^*		$\varnothing N_s$	
144 kN	$\le$	**603 kN**	**$\therefore$ O.K.**

6.3 NOMINAL MEMBER CAPACITY

$N^* \le \varnothing N_c$
$\le \alpha_c \varnothing N_s$

Section constant, α_b from table 6.3.3(1)	$\alpha_b =$	2
Radius of gyration about X axis (table)	$r_x =$	58.7 mm
Radius of gyration about Y axis (table)	$r_y =$	58.7 mm
$k_{ex}L_x$	$L_{ex} =$	8.350 m
$k_{ey}L_y$	$L_{ey} =$	8.350 m
L_{ex}/r_x	$L_{ex}/r_x =$	142.2
L_{ey}/r_y	$L_{ey}/r_y =$	142.2

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About X-axis

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CALCULATION SHEET

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Modified slenderness, λ_n

$\lambda_n = (L_{ex}/r_x)(k_f * f_y/250)$ $\quad \lambda_{nx} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\quad \alpha_a =$ 13.70

$\lambda = \lambda_n + \alpha_a\alpha_b$ $\quad \lambda =$ 166.34

Calculate imperfection, η

$\eta = 0.00326(\lambda-13.5) \geq 0$ $\quad \eta =$ 0.498

$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\quad \xi =$ 0.719

$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)})$ $\quad \alpha_c =$ 0.245

$\varnothing N_{cx} =$ **148 kN**

N*		$\varnothing N_{cx}$	
144 kN	≤	**148 kN**	**∴ O.K.**

About Y-axis

Modified slenderness, λ_n

$\lambda_n = (L_{ey}/r_y)(k_f * f_y/250)$ $\quad \lambda_{ny} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\quad \alpha_a =$ 13.70

$\lambda = \lambda_n + \alpha_a\alpha_b$ $\quad \lambda =$ 166.34

Calculate imperfection, η

$\eta = 0.00326(\lambda-13.5) \geq 0$ $\quad \eta =$ 0.498

$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\quad \xi =$ 0.719

$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)})$ $\quad \alpha_c =$ 0.245

$\varnothing N_{cy} =$ **148 kN**

N*		$\varnothing N_{cy}$	
144 kN	≤	**148 kN**	**∴ O.K.**

Section size	150x150x5.0 SHS	O.K.

Use

150x150x5.0 SHS

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CALCULATION SHEET

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6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load $N^* =$ 144 kN

6.1 DESIGN FOR AXIAL COMPRESSION

$N^* \le \varnothing N_s$ and
$N^* \le \varnothing N_c$

6.2 NOMINAL SECTION CAPACITY

$N^* \le \varnothing N_s$
$\le \varnothing k_f A_n f_y$

Young's Modulus	$E =$	200 GPa
Minimum value of yield stress for any element	$f_y =$	250 MPa
Strength reduction factor	$\varnothing =$	0.9
Length between supports - X axis	$L_x =$	8.350 m
Length between supports - Y axis	$L_y =$	8.350 m
Effective length factor about X axis	$k_{ex} =$	1
Effective length factor about Y axis	$k_{ey} =$	1
Form factor (by calculation)	$k_f =$	0.954
Gross area of cross section (use nett if approp.) (table)	$A =$	2810 mm^2
	$\varnothing N_s =$	**603 kN**

N^*		$\varnothing N_s$	
144 kN	$\le$	603 kN	$\therefore$ O.K.

6.3 NOMINAL MEMBER CAPACITY

$N^* \le \varnothing N_c$
$\le \alpha_c \varnothing N_s$

Section constant, α_b from table 6.3.3(1)	$\alpha_b =$	-1
Radius of gyration about X axis (table)	$r_x =$	58.7 mm
Radius of gyration about Y axis (table)	$r_y =$	58.7 mm
$k_{ex}L_x$	$L_{ex} =$	8.350 m
$k_{ey}L_y$	$L_{ey} =$	8.350 m
L_{ex}/r_x	$L_{ex}/r_x =$	142.2
L_{ey}/r_y	$L_{ey}/r_y =$	142.2

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About X-axis

Beca Carter Hollings & Ferner

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Modified slenderness, λ_n

$\lambda_n = (L_{ex}/r_x)(k_f*f_y/250)$ $\lambda_{nx} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\alpha_a =$ 13.70
$\lambda = \lambda_n + \alpha_a\alpha_b$ $\lambda =$ 125.24

Calculate imperfection, η
$\eta = 0.00326(\lambda-13.5) \geq 0$ $\eta =$ 0.364
$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\xi =$ 0.852
$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)}$ $\alpha_c =$ 0.394

$\emptyset N_{cx} =$ **238 kN**

N*		$\emptyset N_{cx}$	
144 kN	≤	**238 kN**	**∴ O.K.**

About Y-axis

Modified slenderness, λ_n

$\lambda_n = (L_{ey}/r_y)(k_f*f_y/250)$ $\lambda_{ny} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\alpha_a =$ 13.70
$\lambda = \lambda_n + \alpha_a\alpha_b$ $\lambda =$ 125.24

Calculate imperfection, η
$\eta = 0.00326(\lambda-13.5) \geq 0$ $\eta =$ 0.364
$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\xi =$ 0.852
$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)}$ $\alpha_c =$ 0.394

$\emptyset N_{cy} =$ **238 kN**

N*		$\emptyset N_{cy}$	
144 kN	≤	**238 kN**	**∴ O.K.**

Section size	**150x150x5.0 SHS**	**O.K.**

Use
150x150x5.0 SHS

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6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load $N^* =$ 144 kN

6.1 DESIGN FOR AXIAL COMPRESSION

$N^* \le \varnothing N_s$ and
$N^* \le \varnothing N_c$

6.2 NOMINAL SECTION CAPACITY

$N^* \le \varnothing N_s$
$\le \varnothing k_f A_n f_y$

Young's Modulus	$E =$	200 GPa
Minimum value of yield stress for any element	$f_y =$	250 MPa
Strength reduction factor	$\varnothing =$	0.9
Length between supports - X axis	$L_x =$	8.350 m
Length between supports - Y axis	$L_y =$	8.350 m
Effective length factor about X axis	$k_{ex} =$	1
Effective length factor about Y axis	$k_{ey} =$	1
Form factor (by calculation)	$k_f =$	0.954
Gross area of cross section (use nett if approp.) (table)	$A =$	2810 mm^2
	$\varnothing N_s =$	**603 kN**

N^*		$\varnothing N_s$	
144 kN	$\le$	603 kN	$\therefore$ O.K.

6.3 NOMINAL MEMBER CAPACITY

$N^* \le \varnothing N_c$
$\le \alpha_c \varnothing N_s$

Section constant, α_b from table 6.3.3(1)	$\alpha_b =$	0
Radius of gyration about X axis (table)	$r_x =$	58.7 mm
Radius of gyration about Y axis (table)	$r_y =$	58.7 mm
$k_{ex}L_x$	$L_{ex} =$	8.350 m
$k_{ey}L_y$	$L_{ey} =$	8.350 m
L_{ex}/r_x	$L_{ex}/r_x =$	142.2
L_{ey}/r_y	$L_{ey}/r_y =$	142.2

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About X-axis

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Warehouse Building, Level Internal Columns
PAGE NO: OF
BY: Jamil Khan
DATE: 25/5/04

Modified slenderness, λ_n

$\lambda_n = (L_{ex}/r_x)(k_f * f_y/250)$ $\lambda_{nx} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\alpha_a =$ 13.70

$\lambda = \lambda_n + \alpha_a\alpha_b$ $\lambda =$ 138.94

Calculate imperfection, η

$\eta = 0.00326(\lambda-13.5) \geq 0$ $\eta =$ 0.409

$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\xi =$ 0.796

$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)})$ $\alpha_c =$ 0.334

$\varnothing N_{cx} =$ 201 kN

N*		$\varnothing N_{cx}$	
144 kN	**≤**	**201 kN**	**∴ O.K.**

About Y-axis

Modified slenderness, λ_n

$\lambda_n = (L_{ey}/r_y)(k_f * f_y/250)$ $\lambda_{ny} =$ 138.9

Calculate α_c, where α_c is from the modified slenderness λ_n and λ_n from the code formula Cl 6.3.3 as below.

$\alpha_a = 2100(\lambda_n-13.5)/(\lambda_n^2-15.3\lambda_n+2050)$ $\alpha_a =$ 13.70

$\lambda = \lambda_n + \alpha_a\alpha_b$ $\lambda =$ 138.94

Calculate imperfection, η

$\eta = 0.00326(\lambda-13.5) \geq 0$ $\eta =$ 0.409

$\xi = ((\lambda/90)^2+1+\eta)/(2*(\lambda/90)^2)$ $\xi =$ 0.796

$\alpha_c = \xi(1-\sqrt{(1-(90/(\xi\lambda))^2)})$ $\alpha_c =$ 0.334

$\varnothing N_{cy} =$ 201 kN

N*		$\varnothing N_{cy}$	
144 kN	**≤**	**201 kN**	**∴ O.K.**

Section size 150x150x5.0 SHS **O.K.**

Use
150x150x5.0 SHS

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

SUBJECT: Warehouse Building, Level Internal Columns — PAGE NO: OF

BY: Jamil Khan — DATE: 30/4/04

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load $N_t^* =$ 64 kN

7.1 DESIGN FOR AXIAL TENSION

$N^* \le \varnothing N_t$

7.2 NOMINAL SECTION CAPACITY

$N^* \le \varnothing A_g f_y$ (Yield) and
$\le \varnothing 0.85 k_{te} A_n f_u$ (Fracture)

Strength reduction factor	$\varnothing =$	0.9
Gross area of cross section (table)	$A =$	2810 mm^2
Minimum value of yield stress for any element	$f_y =$	250 MPa
Correction factor for distribution of forces (by calculation)	$k_{te} =$	0.85
Nett area of cross section (by calculation)	$A_n =$	12500 mm^2
Tensile strength	$f_u =$	410 MPa

$\varnothing N_{t\,(Yield)} =$ 632.25 kN

$\varnothing N_{t\,(Fracture)} =$ 3333 kN

$\varnothing N_{t\,(Yield)}$		$\varnothing N_{t\,(Fracture)}$		
632 kNm	**≤**	**3333 kNm**	**∴**	**$\varnothing$ Nty = 632 kNm is critical**

N_t^*		$\varnothing$ Nty =	
64 kN	**≤**	**632 kN**	**∴ O.K.**

Section size 150x150x5.0 SHS O.K.

Use
150x150x5.0 SHS

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Zone J, Mazz Col @ GL (S9)/(SA) for G+Q+Eu Comb.
PAGE NO: OF
BY: Jamil Khan
DATE: 3/6/04

DESIGN DATA

Enter trial section size here **530 UB 92.4**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	378	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	3.500	m
Length between Y axis restraints	L_y =	3.500	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	2370000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	342000	mm^3
Moment of inertia about X axis (table)	I_x =	554000000	mm^4
Moment of inertia about Y axis (table)	I_y =	23800000	mm^4
Torsion constant (table)	J =	775000	mm^4
Warping constant (table)	I_w =	1.59E+12	mm^6
Moment modification factor	αm =	1.75	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	744	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	11800	mm^2
Radius of gyration about X axis (table)	r_x =	217	mm
Radius of gyration about Y axis (table)	r_y =	44.9	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	11800	mm^2
Tensile strength	f_u =	410	MPa

8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1.4 Have you checked if axial force is significant?

8.2 DESIGN ACTIONS

Factored bending moment about major X - axis	$M_x^* =$	378 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	3.500 m
Length between Y axis restraints	$L_y =$	3.500 m

8.3 SECTION CAPACITY

8.3.2 Uniaxial Bending About Major Principal X-Axis

$M_x^* \le \varnothing M_{rx}$

All sections
$\le \varnothing M_{sx}(1-N^*/\varnothing N_s)$ $\varnothing M_{rx} =$ 483 kNm

M_x^*		$\varnothing M_{rx}$	
378 kNm	≤	483 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sx}(1-N^*/\varnothing N_s) \le \varnothing Msx$ $\varnothing M_{rx} =$ 570 kNm

$\varnothing M_{rx}$		$\varnothing M_{sx}$			
570 kNm	≤	640 kNm	∴	Ø Mrx2 =	570 kNm is critical

M_x^*		Ø Mrx2 =	
378 kNm	≤	570 kNm	∴ O.K.

8.3.3 Uniaxial Bending About Minor Principal Y-Axis

$M_y^* \le \varnothing M_{ry}$

All sections
$\le \varnothing M_{sy}(1-N^*/\varnothing N_s)$ $\varnothing M_{ry} =$ 70 kNm

M_y^*		$\varnothing M_{ry}$	
0 kNm	≤	70 kNm	∴ O.K.

I sections to Cl. 8.1.5
$\le \varnothing 1.19M_{sy}(1-(N^*/\varnothing N_s)^2) \le \varnothing Msy$ $\varnothing M_{ry} =$ 103 kNm

$\varnothing M_{ry}$		$\varnothing M_{sy}$			
103 kNm	≥	92 kNm	∴	Ø Msy =	92 kNm is critical

M_y^*		Ø Msy =	
0 kNm	≤	92 kNm	∴ O.K.

SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sy}(1-N^*/\varnothing N_s) \le \varnothing Msy$ $\varnothing M_{ry} =$ 82 kNm

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Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

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82 kNm	**≤**	**92 kNm**	**∴**	**Ø Mry3 =**	**82 kNm is critical**

M_y^*		Ø Mry3 =
0 kNm	**≤**	**82 kNm** **∴ O.K.**

8.3.4 Biaxial Bending

Enter $ØM_{rx}$ of applicable section 483 kNm kNm (ie. all, I, or hollow section)
Enter $ØM_{ry}$ of applicable section 70 kNm kNm (ie. all, I, or hollow section)

All sections
$N^*/ØN_s + M_x^*/ØM_{sx} + M_y^*/ØM_{sy} \leq 1.0$ **= 0.835**

O.K.

I or SHS or RHS sections to Cl. 8.1.5
$(M_x^*/ØM_{rx})^\gamma + (M_y^*/ØM_{ry})^\gamma \leq 1.0$ **= 0.668**

where $\gamma = 1.4 + (N^*/ØN_s) \leq 2.0$ = 1.645

O.K.

8.4 MEMBER CAPACITY

8.4.2 IN PLANE CAPACITY

8.4.2.2 Compression Members

$M^* \leq ØM_i$

All sections
$\leq ØM_s(1-N^*/ØN_c) \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}(1-N^*/ØN_{cx}) \leq ØM_{rx}$ $ØM_{ix} =$ 401 kNm

$ØM_{ix}$		$ØM_{rx}$		
401 kNm	**≤**	**483 kNm**	**∴**	**Ø Mix1 = 401 kNm is critical**

M_x^*		Ø Mix1 =
378 kNm	**≤**	**401 kNm** **∴ O.K.**

- Bending about minor principal y-axis

$\leq ØM_{sy}(1-N^*/ØN_{cy}) \leq ØM_{ry}$ $ØM_{iy} =$ 58 kNm

$ØM_{iy}$		$ØM_{ry}$		
58 kNm	**≤**	**70 kNm**	**∴**	**Ø Miy1 = 58 kNm is critical**

M_y^*		Ø Miy1 =
0 kNm	**≤**	**58 kNm** **∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5
$\leq ØM_s\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_c) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_c))}\} \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_{cx}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_{cx}))}\} \leq ØM_{rx}$

$ØM_{ix} =$ 1067 kNm

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$\emptyset M_{lx}$		$\emptyset M_{rx}$			
1067 kNm	**≥**	**483 kNm**	**∴**	**Ø Mrx =**	**483 kNm is critical**

M_x^*		Ø Mrx =	
378 kNm	**≤**	**483 kNm**	**∴ O.K.**

- Bending about minor principal y-axis

$$\leq \emptyset M_{sy}\{[1-((1+\beta_m)/2)^3](1-N^*/\emptyset N_{cy}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/\emptyset N_{cy}))}\} \leq \emptyset M_{ry}$$

$\emptyset M_{iy}$ = 154 kNm

$\emptyset M_{ly}$		$\emptyset M_{ry}$			
154 kNm	**≥**	**70 kNm**	**∴**	**Ø Mry =**	**70 kNm is critical**

M_y^*		Ø Mry =	
0 kNm	**≤**	**70 kNm**	**∴ O.K.**

Enter $\emptyset M_{ix}$ of applicable section 483.26425 kNm (ie. all, I, or hollow section)
Enter $\emptyset M_{iy}$ of applicable section 69.736866 kNm (ie. all, I, or hollow section)

8.4.2.3 Tension Members

A member subject to a tensile force and B.M. shall satisfy 8.3 SECTION CAPACITY

8.4.4 OUT OF PLANE CAPACITY

8.4.4.1 Compression Members

$M_x^* \leq \emptyset M_{ox}$

All sections

$\leq \emptyset M_{bx}(1-N^*/\emptyset N_{cy})$ $\emptyset M_{ox}$ = 507 kNm

M_x^*		$\emptyset M_{ox}$	
378 kNm	**≤**	**507 kNm**	**∴ O.K.**

I or SHS or RHS sections to Cl. 8.1.5

$\leq \emptyset\alpha_{bc}M_{bxo}\sqrt{[(1-N^*/\emptyset N_{cy})(1-(N^*/\emptyset N_{oz})]} \leq \emptyset M_{rx}$ $\emptyset M_{ox}$ = 610 kNm

where $1/\alpha_{bc} = 1-\beta_m/2 + (1+\beta_m/2)^3(0.4 - 0.23\ N^*/\emptyset N_{cy})$ = 0.561

α_{bc} = 1.784

$M_{bxo} = M_{bx} / \alpha_m \times \alpha_m$=1.0 = 514 kNm

$N_{oz} = GJ + (\Pi^2 E I_w/L_z^2) / (I_x + I_y)/A$ = 6499 kN

L_z = Dist between partial or full retraints which prevent twist about centriod = 3.500 m

$\emptyset M_{ox}^*$		$\emptyset M_{rx}$			
610 kNm	**≥**	**483 kNm**	**∴**	**Ø Mrx =**	**483 kNm is critical**

M_x^*		Ø Mrx =	
378 kNm	**≤**	**483 kNm**	**∴ O.K.**

8.4.4.2 Tension Members

$M_x^* \leq \emptyset M_{ox}$

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All sections

$\leq \varnothing M_{bx}(1+N^*/\varnothing N_t) \leq M_{rx}$ $\varnothing M_{ox} =$ 1000 kNm

$\varnothing M_{ox}$		$\varnothing M_{rx}$			
1000 kNm	**≥**	**483 kNm**	**∴**	**Ø Mrx =**	**483 kNm is critical**

M_x^*		Ø Mrx =	
378 kNm	**≤**	**483 kNm**	**∴ O.K.**

8.4.5 BIAXIAL BENDING CAPACITY

8.4.5.1 Compression Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{cx} = \varnothing M_{lx}$

For partial or no restraint $\varnothing M_{cx}$ = **Lesser of** $\varnothing M_{lx}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{cx}$ for applicable restraint, $\varnothing M_{cx}$ = 483 kNm

$(M_x^*/\varnothing M_{cx})^{1.4} + (M_y^*/\varnothing M_{ly})^{1.4} \leq 1.0$ = 0.709

O.K.

8.4.5.2 Tension Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{tx} = \varnothing M_{rx}$

For partial or no restraint $\varnothing M_{tx}$ = **Lesser of** $\varnothing M_{rx}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{tx}$ for applicable restraint, $\varnothing M_{tx}$ = 483 kNm

$(M_x^*/\varnothing M_{tx})^{1.4} + (M_y^*/\varnothing M_{ry})^{1.4} \leq 1.0$ = 0.709

O.K.

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Edge columns.

Applied Moment

1) From Portal Frame

case 1.2G + 1.6Q $M_u = 107$ KN-m
$P_u = 50$ KN-m ↑

case 0.9G + Wind Ultimate $M_u = -61$ KN-m
$P_u = -6$ KN ↓

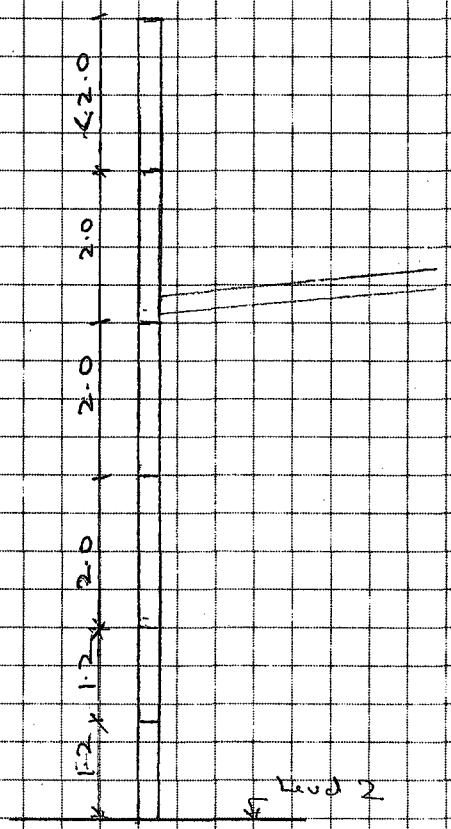
For Cantilever Portion $M_u = -61$ KN-m

The member we are using as external column is

310UB40.

$L_e = 2$ m

$M_{uk} = 180$ KN-m.



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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Warehouse Building, External Column @ GL (SG)
PAGE NO: OF
BY: Jamil Khan
DATE: 30/4/04

DESIGN DATA

Enter trial section size here 310 UB 40.4
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	107 kNm
Factored bending moment about minor Y - axis	M_y^* =	5 kNm
Length between X axis restraints	L_x =	2.000 m
Length between Y axis restraints	L_y =	2.000 m
Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	250 MPa
Young's modulus	E =	200 GPa
Shear modulus of elasticity	G =	80 GPa
Effective section modulus about X axis (table)	Z_{ex} =	633000 mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	139000 mm^3
Moment of inertia about X axis (table)	I_x =	86400000 mm^4
Moment of inertia about Y axis (table)	I_y =	7650000 mm^4
Torsion constant (table)	J =	157000 mm^4
Warping constant (table)	I_w =	1.65E+11 mm^6
Moment modification factor	αm =	1.00
Ratio of smaller to larger bending moment	βm =	2
Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	50 kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250 MPa
Gross area of cross section (use nett if approp.) (table)	A =	5210 mm^2
Radius of gyration about X axis (table)	r_x =	129 mm
Radius of gyration about Y axis (table)	r_y =	38.3 mm
Effective length factor about X axis	k_{ex} =	1
Effective length factor about Y axis	k_{ey} =	1
Form factor (by calculation)	k_f =	0.954
Section constant, α_b from table 6.3.3(1)	α_b =	2

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	50 kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85
Nett area of cross section (by calculation)	A_n =	12500 mm^2
Tensile strength	f_u =	410 MPa

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
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5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	107 kNm
Factored bending moment about minor Y - axis	M_y^* =	5 kNm
Length between X axis restraints	L_x =	2.000 m
Length between Y axis restraints	L_y =	2.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \phi M_{sx}$ and
$M_x^* \le \phi M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \phi M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \phi M_{sx}$
$\le \phi f_y Z_{ex}$

Strength reduction factor	ϕ =	0.9
Yield stress of flanges	f_y =	250 MPa
Effective section modulus about X axis (table)	Z_{ex} =	633000 mm^3
	ϕM_{sx} =	**142 kNm**

M_x^*		ϕM_{sx}		
107 kNm	≤	**142 kNm**	**∴ O.K.**	**310 UB 40.4**

$M_y^* \le \phi M_{sy}$
$\le \phi f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	139000 mm^3
	ϕM_{sy} =	**31 kNm**

M_y^*		ϕM_{sy}		
5 kNm	≤	**31 kNm**	**∴ O.K.**	**310 UB 40.4**

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

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5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 2

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	7650000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	157000 mm^4
Warping constant (table)	$I_w =$	1.65E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 2.000 m

$M_{oa} =$ 595 kNm

$\alpha s =$ 0.8918518

$\varnothing M_{bx} =$ **127 kNm**

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
127 kNm	≤	**142 kNm**	∴	**Ø Mbx = 127 kNm is critical**

M_x^*		Ø Mbx =		
107 kNm	≤	**127 kNm**	**∴ O.K.**	**310 UB 40.4**

			Use
Section size	**310 UB 40.4**	**O.K.**	**310 UB 40.4**

Design
Of
First Floor Beam

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Calculation Sheet
Job name: Q Gate – The Warehouse Job No:
Subject: Level 2 Slabs. (II) Page No: of
By: Date:

Summary

Use beams at 2100 ctrs max

Use 150 slabs on 0.75 hibond

Use –ve Reo 509 mm²/m

→ 665 mesh + HD12@300 c/c 1600 long.

+ Transverse Reo ~~above slab~~

HD12@400

+ BL steel HD12@600

Fire Steel

Under Fire $W = 0.5 + 2.9 + 0.6(16.3) = 13.1$ kPa

Use in effect yield stress of 400 mpa in lieu of 500

$M_u = 13.1(2.1)^2/8 = 7.6$ kN-m x2

$A_s = \frac{7.6}{12 \times 3^2 \times 2} = 84 \text{ mm}^2/\text{m} \times 2$

$= 9(.85)(400)(0.125)$

$= 168 \text{ mm}^2/\text{m}$

HD12@600 c/c. Temp steel

241059

Calculation Sheet

Job name: _____ Job No: _____

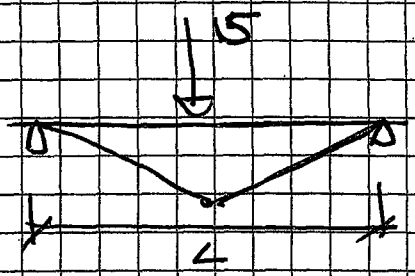
Subject: _____ Page No: 1 of _____

By: _____ Date: _____

Warehouse

Talking to Brian Wilson (09 4898900) of TWL Ex 9661S he advised that the racks are 900 wide with an Isle between racks ⇒ therefore we have 15 kN point loads in 1 direction @ 2.7 m c/c and have aisle width of say 1200 mm ie 15 kN point loads @ 1.2 & 900 c/c

Direction 1



$M = \frac{15L}{4}$

moment distributed over slab width of 0.6 L

So moment/m² = $\frac{15L}{4(.6L)}$ = 6.25 kN-m/m

= $\frac{WL^2}{8}$

if W = 8.5 (WH Spec) ⇒ L = 2.4 c/c.

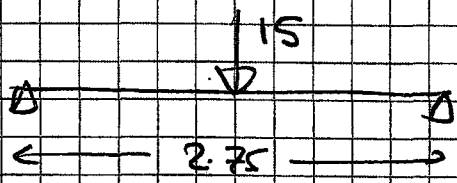
Calculation Sheet

Job name: Job No:

Subject: Page No: 2 of

By: Date:

Substruction 2 check for 2.7 span



Width = 0.6 × 2.75 = 1.65 m

$mom/m = 10.3/1.65 = 6.24$ KN-m

$6.24 = \frac{w(2.75)^2}{8}$

$w = 6.6$ Kpa — OK

For 2.4 Span M= 9.

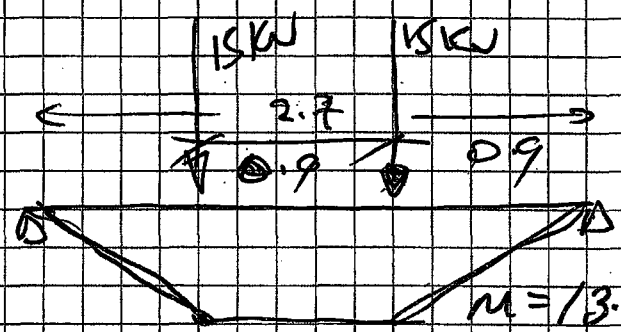
Width = 1.44 $mom/m = 6.25$

$6.25 = w(2.4)^2/8 =$

$w = 8.7$ Kpa — OK

⇒ WA Spec 8.5 Kpa.

Substruction 2



Width reduced by = 2.7 × 0.6 = 1.62

⇒ $mom/m = \frac{13.5}{1.6} = 8.4$ KN-M = $\frac{w(2.7)^2}{8}$

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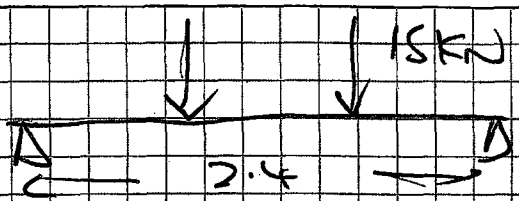
Calculation Sheet

Job name: Job No: 3

Subject: Page No: of

By: Date:

OR



$M = 0.75 \times 15 = 11.25$.

resisted by w.dll = 1.44

$M_{om}/_M = 7.81 = w\frac{(2.4)^2}{8}$

$w = \underline{10.8\ kPa}$

LOOK AT SLAB SPANNING = 10.5/4 = 2.625 m

Use Hubond Method for point load fee page 12 of hubond manual. - Try 150 slab

(a) Checked load portion/effective width

$b_M = 100 + 2(150-55) = 290mm$ $b_M = 290$

(b) Critical distance

$a = \frac{290}{2} + (150-25) + \frac{150}{2} = 345mm$

(c) effective slab width =

$b_{ei} = 290 + 245\left(1 - \frac{245}{2625}\right) = 512mm$

(d) $V = 1.2(.15+0.025)(235)\left(\frac{2.625}{2}\right) = 1.2(3.851) = 4.62$

$= 1.2(.5)(2.625/2) = 1.2(1.31) = 1.57$

$= 1.6(15) = 24.0$

30.19.

(e) Shear stress $v_c = 0.17\sqrt{40} = 1.075$

(f) shear capacity 1 rib = $156(150)(1.075) = 25.1\ kN$.

(g) have $512/302 = 1.69$ Ribs

(h) Shear capacity $= 1.69 \times 25 \times 0.8 = 33.8kN > 30.2kN$

$\Rightarrow$ shear capacity is OK.

Calculation Sheet

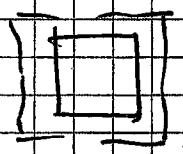
Job name: Job No:

Subject: Page No: 4 of

By: Date:

Check shear bond

Punching shear 15 kN



$U = 4(100 + 150 - 55) + 125) =$

$= 1280 \Rightarrow A = 0.102$

$V = 1.6(15) + (2.9 + 0.5)(0.1)(1.2)$

$= 24.4$ kN

$V_{cap} = 1280(155 - 55)(1075)$

$= 137$ kN $>>$ 24 — OK

Shear Bond

$$G_{Col+Q} = \frac{15(19000 - 3(3,625))}{0.00743(-0.393(3,625)^2 + 3200(3625) - 3.52\times10^6)}$$

$$= \frac{15(11125)10^3}{0.00743(2,171,984)} = 10.3 \text{ kPa}$$

$G_{SDL} = 10.8 + 0.5 = 11.3$ kPa

$\Rightarrow$ From table 10 for $L = 2.6$ & $D_t = 150$

$SL = 14.9$ kPa > 10.3

$+0.5$

> 10.8 kPa

& Allow SL = 0.5

$\Rightarrow$ Bond shear OK.

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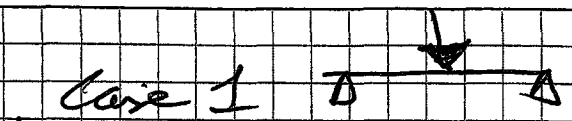
Calculation Sheet

Job name: Job No:

Subject: Page No: 5 of

By: Date:

Negative Bending Case 1



$b_{eb} = 290 + 2(345)\left(1 - \frac{345}{2600}\right) = 888$ mm single

$= 290 + 1.33(345)\left(1 - \frac{345}{2600}\right) = 704$ continuous

Max bending occurs at midspan

$b_{eb} = 290 + 1.33\left(\frac{2600}{2}\right)\left(1 - \frac{1300}{2600}\right)$

$= 940$ mm.

Applied moment due to point load over support is

$1.6(.094(2.8 \times 15)/.940 = 6.24$ kN-m.

$\text{equiv UDL} = \frac{6.24 \times 10^3}{.063 \times 1.6\ 2600^2}$

$G_k + Q = 9.2$ kPa.

$SDL = 9.2 + 0.5 = 9.7$ kPa

~~Cf pressure gauge we were~~ Calculated 6.6 kPa.

-ve Case 2



$b_{eb} = 290 + 1.33(850)\left(1 - \frac{850}{2600}\right)$

$= 470$ mm.

-ve moment over support $= \frac{2}{3} \times (15 \times 0.85) = 8.5$ kN-m (R.

$M_{ult} = 1.6(8.5) = 13.6$

$\text{equiv UDL} = \frac{wL^2}{10} = 13.6$

$w = 19.7$ kPa.

Beca **Calculation Sheet**

Job name: _____ Job No: _____

Subject: _____ Page No: 6 of _____

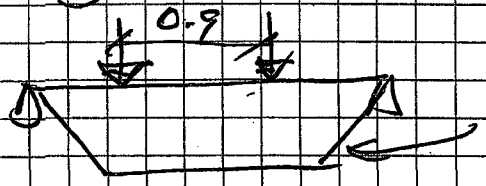
By: _____ Date: _____

→ total SDL = 14.8 Kpa + 0.5 = 15.3 Kpa.

from table 12

2600 # 150 Max SL = 11.9 KPa

Using Wang → Reduce span to 2.1



$M = 0.6(15) = 9\ KN\text{-}m$

$= \frac{W(2.1)^2}{8}$

$W = 16.3\ Kpa.$

16.3 + 0.5 Kpa = 16.8 Kpa

for 2200 # 150 slab MAX SL = 17.4 Kpa — OK.

⇒ -ve moment for case 2 govern

$f_s = 57 + 27(17.8) = 510\ MPa$

⇒ H12 @ 225

OR 666 mesh + H10 @ 300 — 1.65 kg/m² over support.

OR 665 w + H8 @ 300 — 1.65 kg/m² over support

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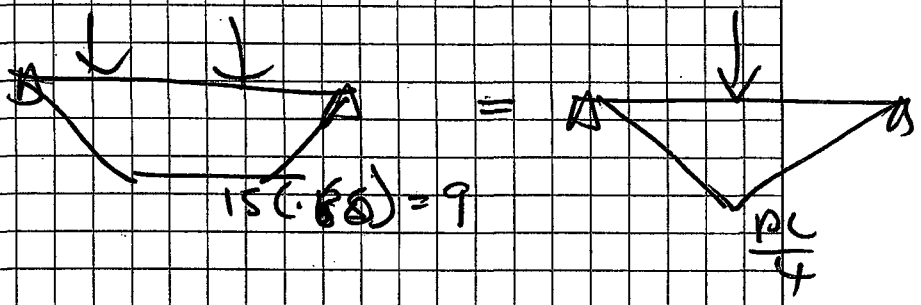
check +ve

CASE 1

$$\sigma_{SOIL} + Q\,(P) = \frac{1000\,(15)}{0.00247\,(2100)^2 - 14.65\,(2100) + 27100}$$

$$= \frac{15000}{7227} = 2.07\ \text{kPa} - \text{OK}$$

CASE 2



$$P_e = 17.1\ \text{kN}$$

$$\sigma_{SOIL} + Q = \frac{1000\,(17.1)}{.00247\,(2,100)^2 - 1465\,(2100) + 27100}$$

$$= 4.6\ \text{kPa} \Rightarrow \text{OK}$$

Require Additional Transverse Reo

$$2\,(150 - 58)\ \text{mm}^2/\text{m} = 190\ \text{mm}^2/\text{m}$$

$\Rightarrow$ H20 @ 400 ctrs centred in slab

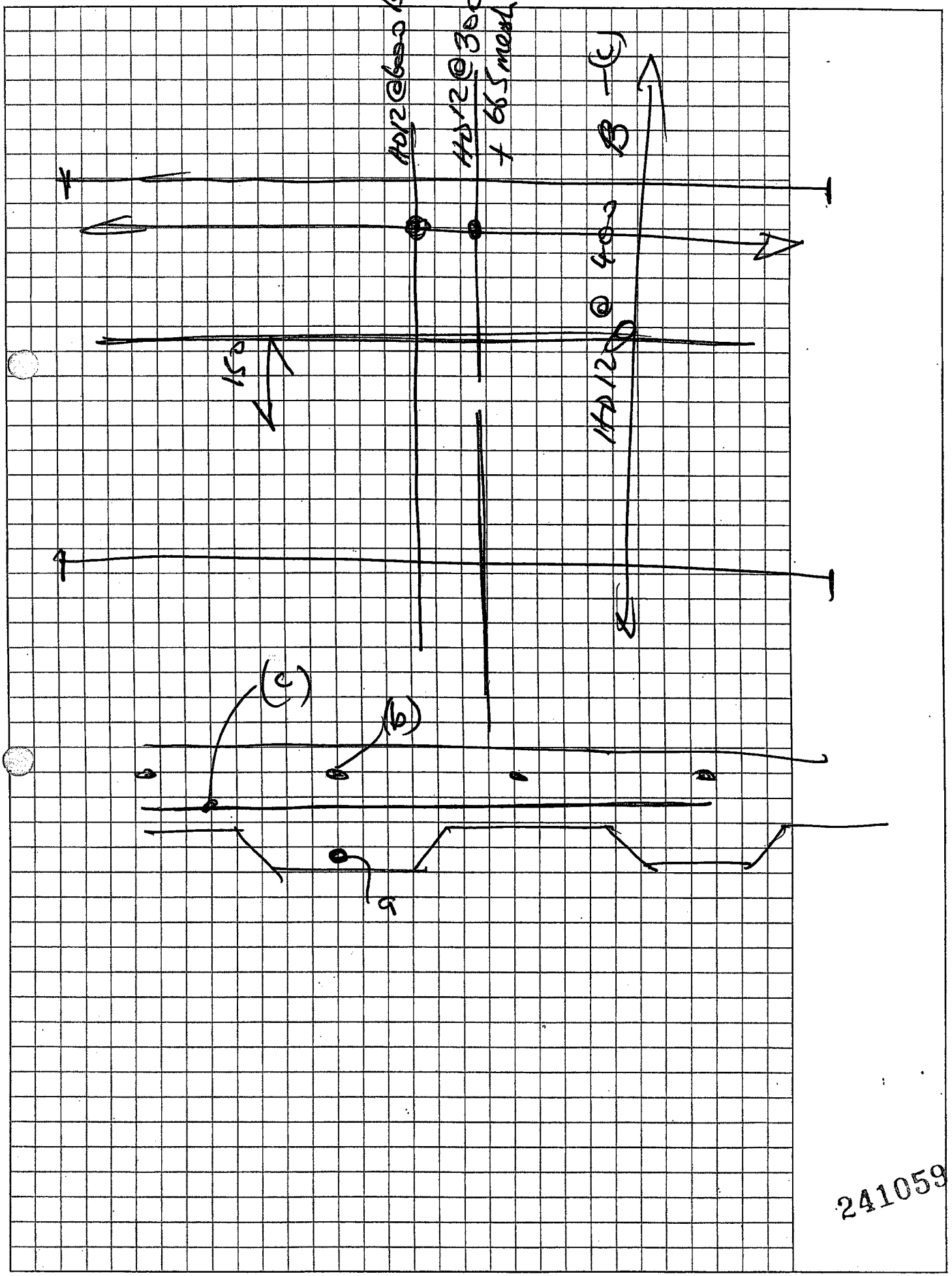
Beca

Calculation Sheet

Job name: _____ Job No: _____

Subject: _____ Page No: ____ of ____

By: _____ Date: _____



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Calculation Sheet

Job name: Queensgate wall Redevelopment
Subject: Zone 23 LVL2 floor beam design
By: DMS

Job No: 2711265/109
Page No: 1 of
Date: 20-5-04

160mm thick Libbed (0.75 m bearing) Hollow Core Type.
Pt. Loads.

SDL = 0.5kPa
LL = 10.0kPa

Storage load factors

Strength & Serviceability.

Sketch LL = 1.6kPa
4.0% dead-man

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams SG to SK 15m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	14779 mm
Adj. Beam Centres :	2100 mm
	2100 mm
Beam Size	750LB84
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	49 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	70 KNm
Depend. Bending Strength	98 KNm

Stage II : Concrete Placement

Factored Bending Moment :	332 KNm
Depend. Bending Strength	688 KNm

% Composite Action 72 %

Bending Strength

Bending Strength Required :	1201 KNm
Dependable Bending Strength :	1307 KNm

Shear Strength

Shear Strength Required:	325 kN
Dependable Shear Strength:	654 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	240 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-0.2 mm
SDL+φs LL+Shrink+Creep:	44.8 mm
Span/DF : Steel Immediate:	-62123
Span/DF : Top of Concrete:	330
Span/DF : Steel Total:	331

Vibration

Natural Frequency :	4.96 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.12 mm
Initial Amplitude from Heel Drop	0.056 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating 15.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.71 KN/mm
Weld Leg Reqd: SP	Single side	4 mm
	Double side	2 mm
Bottom Flange:vwb: VdepQb/Is		0.71 KN/mm
Weld Leg Reqd: SP	Single side	4 mm
	Double side	2 mm

Comparative Cost:

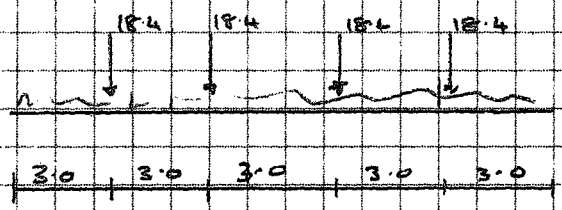
Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$1,383

Calculation Sheet

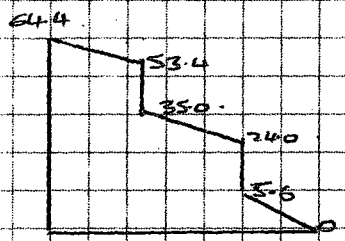
Job name: Queensgate
Subject: S8 SK to SG 2J LVL Beam
By: DBS.

Job No: 2711265
Page No: 1 of 2.
Date: 11-6-04

Canopy loads



$R = 2 \times 18.4 + 3.675 \times 7.5$
$= 64.4$ kN.



$M_u = \frac{64.4 + 53.4}{2} \times 3 + \frac{35 + 24}{2} \times 3 + \frac{5.6}{2} \times 1.5$

$= 176.7 + 88.5 + 4.2$
$= 269.4$ kNm

equiv udl for Cabana design.

$\frac{wl^2}{8} = M_u$.

$w = \frac{8 M_u}{l^2}$.

$= \frac{8 \times 269.4}{(14.779)^2}$.

$= 9.87$ kN/m.

use 700NB173 ≈ 60% M Capacity gives allowance for axial loads from wall bracing & Canopy bracing off reg.

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE : 2
SECTION : 14.8m span with canopy
JOB No.: 2711265 DESIGN: dbs
DATE : 11-Jun-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	14779 mm	
Overhang:(Beam cc):	100 mm	II Cl 13.4.1.3
Adj. Beam Centres :	2100 mm	
Stud Edge Distance	160 mm	
Beam Size	700WB173	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 HI-Bond	
Deck Span Type :	Triple	
Deck Span :	2100 mm	
Slab Thickness :	160 mm	
Slab Mass :	3.17 KPa	
Concrete Strength :	40.0 MPa	
Concrete Density :	2350 kg/m3	

Shear Studs

Stud Diameter :	19 mm	
Weld.Stud Height :	95 mm	
No. of Studs /Beam :	49 @300	
Studs /Flange Width:	1	
Stud UTS :	415 MPa	

Loading

Super. Dead Load :	0.50 KPa	
Other Super. DL :	9.87 KN/m	Canopy loads
Super Live Load :	10.00 KPa	
Other Super. LL :	0.00 KN/m	
Construction LL:		
i) Deck Placement:	0.30 KPa	
ii) Conc. Placement:	0.90 KPa	

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	74 KNm	
Depend. Bending Strength	532 KNm	

Stage II : Concrete Placement

Factored Bending Moment :	226 KNm	
Depend. Bending Strength	1725 KNm	
% Composite Action	56 %	

Bending Strength

Bending Strength Required :	1026 KNm	
Composite Bending Strength :	2217 KNm	II Cl 13.4.1.3
Non-Composite Bending Strength:	1725 KNm	

Shear Strength

Shear Strength Required:	278 kN	
Dependable Shear Strength:	1160 kN	

Unshored Bottom Flange Tension

Serviceability Flange Stress:	107 MPa	
Allow. Bottom Flange Stress:	270 MPa	

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	8.1 mm	
SDL+φs LL+Shrink+Creep:	32.7 mm	
Span/DF : Steel Immediate:	1823	
Span/DF : Top of Concrete:	451	
Span/DF : Steel Total:	362	

Vibration

Natural Frequency :	5.04 Hz	
Deflection Due to 1.0 kN Point Load at Midspan :	0.08 mm	
Initial Amplitude from Heel Drop	0.021 mm	
Damping Required :	2.6 %	

FRR: Fire Resistance Rating

Composite Section Rating:	25.6 min	
Non-Composite Section Rating:	23.3 min	

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.66 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.66 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$4,134

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams SG to SK 7.5m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	7499 mm
Adj. Beam Centres :	2100 mm
	2200 mm
Beam Size	400LB38
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2200 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	25 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	14 KNm
Depend. Bending Strength	60 KNm

Stage II : Concrete Placement

Factored Bending Moment :	86 KNm
Depend. Bending Strength	190 KNm

% Composite Action 81 %

Bending Strength

Bending Strength Required :	316 KNm
Dependable Bending Strength :	398 KNm

Shear Strength

Shear Strength Required:	168 kN
Dependable Shear Strength:	308 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	212 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	11.3 mm
SDL+φs LL+Shrink+Creep:	16.4 mm
Span/DF : Steel Immediate:	663
Span/DF : Top of Concrete:	457
Span/DF : Steel Total:	271

Vibration

Natural Frequency :	8.60 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.071 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating 16.0 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.72 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.72 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$355

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary Beams SG to SK 7.5m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 28-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	7499 mm
Adj. Beam Centres :	2100 mm
	2200 mm
Beam Size	410UB54
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2200 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	25
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	16 KNm
Depend. Bending Strength	91 KNm

Stage II : Concrete Placement

Factored Bending Moment :	87 KNm
Depend. Bending Strength	284 KNm

% Composite Action 57 %

Bending Strength

Bending Strength Required :	316 KNm
Dependable Bending Strength :	499 KNm

Shear Strength

Shear Strength Required:	169 kN
Dependable Shear Strength:	496 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	155 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	8.3 mm
SDL+φs LL+Shrink+Creep:	14.6 mm
Span/DF : Steel Immediate:	908
Span/DF : Top of Concrete:	513
Span/DF : Steel Total:	328

Vibration

Natural Frequency :	9.86 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.06 mm
Initial Amplitude from Heel Drop	0.059 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating 19.7 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.11 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.11 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$473

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams SG to SK 7.5m span end beam
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	7499	mm
Overhang:(Beam cc):	0	mm
Adj. Beam Centres :	2200	mm
Stud Edge Distance	160	mm
Beam Size	400LB38	
Beam Yield Stress	300	MPa
Precamber	0	mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2200	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	25	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	0.00	KN/m
Super Live Load :	10.00	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
i) Deck Placement:	0.30	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	8	KNm
Depend. Bending Strength	60	KNm

Stage II : Concrete Placement

Factored Bending Moment :	45	KNm
Depend. Bending Strength	190	KNm

% Composite Action 81 %

Bending Strength

Bending Strength Required :	162	KNm
Composite Bending Strength :	386	KNm
Non-Composite Bending Strength:	190	KNm

Shear Strength

Shear Strength Required:	87	kN
Dependable Shear Strength:	308	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	113	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	5.9 mm
SDL+φs LL+Shrink+Creep:	12.1 mm
Span/DF : Steel Immediate:	1263
Span/DF : Top of Concrete:	619
Span/DF : Steel Total:	415

Vibration

Natural Frequency :	11.18 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.08 mm
Initial Amplitude from Heel Drop	0.049 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating

Composite Section Rating:	20.0 min
Non-Composite Section Rating:	15.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.72 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.72 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$530

241059

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Licence No.: 890702

USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary Beams SG to SK 7.5m span edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 28-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	7499	mm	
Overhang:(Beam cc):	100	mm	II Cl 13.4.1.3
Adj. Beam Centres :	2200	mm	
Stud Edge Distance	160	mm	
Beam Size	410UB54		
Beam Yield Stress	300	MPa	
Precamber	0	mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2200	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	25	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	0.00	KN/m
Super Live Load :	10.00	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
I) Deck Placement:	0.30	KPa
II) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	9	KNm
Depend. Bending Strength	91	KNm

Stage II : Concrete Placement

Factored Bending Moment :	50	KNm
Depend. Bending Strength	284	KNm

% Composite Action 57 %

Bending Strength

Bending Strength Required :	178	KNm
Composite Bending Strength :	490	KNm II Cl 13.4.1.3
Non-Composite Bending Strength:	284	KNm

Shear Strength

Shear Strength Required:	95	kN
Dependable Shear Strength:	496	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	91	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	4.8 mm
SDL+φs LL+Shrink+Creep:	11.5 mm
Span/DF : Steel Immediate:	1561
Span/DF : Top of Concrete:	653
Span/DF : Steel Total:	460

Vibration

Natural Frequency :	12.30 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.06 mm
Initial Amplitude from Heel Drop	0.042 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating

Composite Section Rating:	22.8 min
Non-Composite Section Rating:	19.7 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.11 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.11 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$712

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams SG to SK 3.2m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	3200 mm
Adj. Beam Centres :	1900 mm
	1900 mm
Beam Size	180UB16
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	1900 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	10 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	2 KNm
Depend. Bending Strength	18 KNm

Stage II : Concrete Placement

Factored Bending Moment :	13 KNm
Depend. Bending Strength	37 KNm

% Composite Action	77 %

Bending Strength

Bending Strength Required :	50 KNm
Dependable Bending Strength :	105 KNm

Shear Strength

Shear Strength Required:	63 kN
Dependable Shear Strength:	126 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	140 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	4.0 mm
SDL+φs LL+Shrink+Creep:	4.0 mm
Span/DF : Steel Immediate:	800
Span/DF : Top of Concrete:	807
Span/DF : Steel Total:	402

Vibration

Natural Frequency :	24.70 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.033 mm
Damping Required :	3.6 %

FRR: Fire Resistance Rating	19.3 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.69 KN/mm
Weld Leg Reqd: SP	Single side	4 mm
	Double side	2 mm
Bottom Flange:vwb: VdepQb/Is		0.69 KN/mm
Weld Leg Reqd: SP	Single side	4 mm
	Double side	2 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$80

241059

Calculation Sheet

Job name: Queenagate

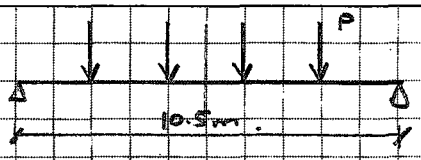
Subject: Zone 23 LVL 2 floor beam design

By: DWS

Job No: 2711265/104

Page No: 7 of

Date: 20-5-04

① 

P = load from secondary beam marks. 750UB84

$P = 84kg \times \frac{9.81}{1000} \times \frac{14.779}{2}$

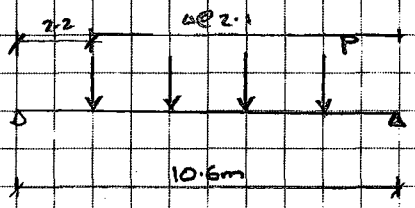
$= 6.09\ kN$

$M = 6.09 \times 2.1 + 2 \times 6.09 \times 2.1$

$= 38.4\ kNm$

$\frac{wL^2}{8} = 38.4\ kNm$

$w = 2.78\ kN/m$

② 

P from 400UB38.

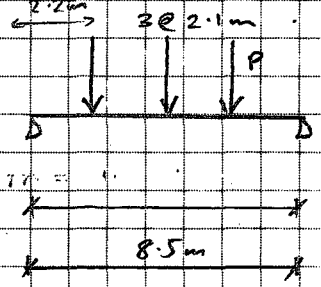
$P = 38kg \times \frac{9.81}{1000} \times \frac{7.499}{2}$

$= 1.4\ kN$

$M = 2 \times 1.4 \times 2.2 + 1.4 \times 2.1$

$= 9.1\ kNm$

$w = 0.648\ kN/m$

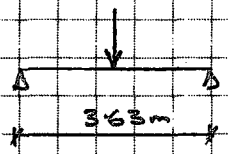
③ 

P from 400UB38

$m = 1.5 \times 1.4 \times 2.2 + 0.5 \times 1.4 \times 2.1$

$= 6.09\ kNm$

$w = 0.674\ kN/m$

④ 

P from 180UB16.

$P = 16 \times 9.81/1000 \times 3.2/2$

$= 0.251\ kN$

$m = 0.23\ kNm$

$w = 0.128\ kN/m$

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams SK 10.5m span edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500 mm	
Overhang:(Beam cc):	100 mm	II Cl 13.4.1.3
Adj. Beam Centres :	14779 mm	
Stud Edge Distance	160 mm	
Beam Size	800WB192	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2100 mm	
Slab Thickness :	160 mm	
Slab Mass :	3.17 KPa	
Concrete Strength :	40.0 MPa	
Concrete Density :	2350 kg/m3	

Shear Studs

Stud Diameter :	19 mm	
Weld.Stud Height :	95 mm	
No. of Studs /Beam :	35 @300	
Studs /Flange Width:	1	
Stud UTS :	415 MPa	

Loading

Super. Dead Load :	0.50 KPa	
Other Super. DL :	2.78 KN/m	2nd ary beams .
Super Live Load :	10.00 KPa	
Other Super. LL :	0.00 KN/m	
Construction LL:		
i) Deck Placement:	0.30 KPa	
ii) Conc. Placement:	0.90 KPa	

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	91 KNm
Depend. Bending Strength	991 KNm

Stage II : Concrete Placement

Factored Bending Moment :	591 KNm
Depend. Bending Strength	2160 KNm

% Composite Action 35 %

Bending Strength

Bending Strength Required :	2202 KNm
Composite Bending Strength :	2633 KNm II ~~Cl 13.4.1.3~~
Non-Composite Bending Strength:	2160 KNm

Shear Strength

Shear Strength Required:	839 kN
Dependable Shear Strength:	1282 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	173 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	7.0 mm
SDL+φs LL+Shrink+Creep:	19.9 mm
Span/DF : Steel Immediate:	1509
Span/DF : Top of Concrete:	527
Span/DF : Steel Total:	390

Vibration

Natural Frequency :	8.70 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.01 mm
Initial Amplitude from Heel Drop	0.043 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.8 min
Non-Composite Section Rating:	19.2 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.59 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.59 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$3,247

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary Beams SK 10.5m Span edge
JOB No.: 2711265/109 DESIGN: dws
DATE : 28-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500	mm	
Overhang:(Beam cc):	100	mm	ll Cl 13.4.1.3
Adj. Beam Centres :	14779	mm	
Stud Edge Distance	160	mm	
Beam Size	900WB175		
Beam Yield Stress	300	MPa	
Precamber	0	mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2100	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	35	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	2.78	KN/m
Super Live Load :	10.00	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
I) Deck Placement:	0.30	KPa
Ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	89	KNm
Depend. Bending Strength	730	KNm

Stage II : Concrete Placement

Factored Bending Moment :	589	KNm
Depend. Bending Strength	2025	KNm

% Composite Action 35 %

Bending Strength

Bending Strength Required :	2199	KNm
Composite Bending Strength :	2579	KNm ll Cl 13.4.1.3
Non-Composite Bending Strength:	2025	KNm

Shear Strength

Shear Strength Required:	838	kN
Dependable Shear Strength:	1750	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	184	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.9	mm
SDL+φs LL+Shrink+Creep:	19.3	mm
Span/DF : Steel Immediate:	1513	
Span/DF : Top of Concrete:	544	
Span/DF : Steel Total:	400	

Vibration

Natural Frequency :	8.94	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.01	mm
Initial Amplitude from Heel Drop	0.041	mm
Damping Required :	3.0	%

FRR: Fire Resistance Rating

Composite Section Rating:	20.1	min
Non-Composite Section Rating:	17.0	min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is			1.73 KN/mm
Weld Leg Reqd: SP	Single side	10	mm
	Double side	5	mm
Bottom Flange:vwb: VdepQb/Is			1.73 KN/mm
Weld Leg Reqd: SP	Single side	10	mm
	Double side	5	mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$2,971

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams SK 10.6m span edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10600 mm	
Overhang:(Beam cc):	100 mm	~~Cl 13.4.1.3~~
Adj. Beam Centres :	7499 mm	
Stud Edge Distance	160 mm	
Beam Size	800WB122	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2200 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	35 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.65 KN/m 2nd edge beam
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	52 KNm
Depend. Bending Strength	320 KNm

Stage II : Concrete Placement

Factored Bending Moment :	314 KNm
Depend. Bending Strength	1215 KNm

% Composite Action 36 %

Bending Strength

Bending Strength Required :	1145 KNm	
Composite Bending Strength :	1720 KNm	~~II Cl 13.4.1.3~~
Non-Composite Bending Strength:	1215 KNm	

Shear Strength

Shear Strength Required:	432 kN
Dependable Shear Strength:	1245 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	153 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	7.2 mm
SDL+φs LL+Shrink+Creep:	18.9 mm
Span/DF : Steel Immediate:	1478
Span/DF : Top of Concrete:	560
Span/DF : Steel Total:	406

Vibration

Natural Frequency :	9.05 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.03 mm
Initial Amplitude from Heel Drop	0.041 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating

Composite Section Rating:	20.9 min
Non-Composite Section Rating:	17.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$2,127

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams SH 8.5m span stair wall edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	8500	mm
Overhang:(Beam cc):	100	mm
Adj. Beam Centres :	7499	mm
Stud Edge Distance	160	mm
Beam Size	530UB92	
Beam Yield Stress	300	MPa
Precamber	0	mm

!! ~~Cl 13.4.1.3~~

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2200	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	28	@300
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	0.67	KN/m 2 way beams
Super Live Load :	10.00	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
i) Deck Placement:	0.30	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	30	KNm
Depend. Bending Strength	219	KNm

Stage II : Concrete Placement

Factored Bending Moment :	200	KNm
Depend. Bending Strength	640	KNm

% Composite Action 37 %

Bending Strength

Bending Strength Required :	734	KNm
Composite Bending Strength :	939	KNm
Non-Composite Bending Strength:	640	KNm

!! ~~Cl 13.4.1.3~~

Shear Strength

Shear Strength Required:	345	kN
Dependable Shear Strength:	881	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	178	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	8.3 mm
SDL+φs LL+Shrink+Creep:	19.4 mm
Span/DF : Steel Immediate:	1028
Span/DF : Top of Concrete:	439
Span/DF : Steel Total:	308

Vibration

Natural Frequency :	9.12	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.03	mm
Initial Amplitude from Heel Drop	0.054	mm
Damping Required :	3.2	%

FRR: Fire Resistance Rating

Composite Section Rating:	19.9 min
Non-Composite Section Rating:	15.2 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.49 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.49 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,315

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams between SH & SG 3.6m span stair wall edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	3630 mm	
Overhang:(Beam cc):	100 mm	~~Cl 13.4.1.3~~
Adj. Beam Centres :	3200 mm	
Stud Edge Distance	160 mm	
Beam Size	250UB26	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	1900 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	12 @ 300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.14 KN/m 2-day beam
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	2 KNm
Depend. Bending Strength	47 KNm

Stage II : Concrete Placement

Factored Bending Moment :	15 KNm
Depend. Bending Strength	86 KNm
% Composite Action	58 %

Bending Strength

Bending Strength Required :	58 KNm
Composite Bending Strength :	172 KNm ~~Cl 13.4.1.3~~
Non-Composite Bending Strength:	86 KNm

Shear Strength

Shear Strength Required:	64 kN
Dependable Shear Strength:	201 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	85 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	1.8 mm
SDL+φs LL+Shrink+Creep:	3.8 mm
Span/DF : Steel Immediate:	2024
Span/DF : Top of Concrete:	957
Span/DF : Steel Total:	650

Vibration

Natural Frequency :	26.15 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.025 mm
Damping Required :	3.4 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.3 min
Non-Composite Section Rating:	17.7 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.75 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.75 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$187

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beam between S4 & S5 3.2m span stair wall edge
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	3200 mm	
Overhang:(Beam cc):	100 mm	~~I.C.I 13.4.1.3~~
Adj. Beam Centres :	1900 mm	
Stud Edge Distance	160 mm	
Beam Size	250UB26	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	1900 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	10 @ 300 ,
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.60 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	1 KNm
Depend. Bending Strength	~~52 KNm~~

Stage II : Concrete Placement

Factored Bending Moment :	8 KNm
Depend. Bending Strength	~~86 KNm~~

% Composite Action 48 %

Bending Strength

Bending Strength Required :	28 KNm	
Composite Bending Strength :	~~160 KNm~~	~~II.C.I 13.4.1.3~~
Non-Composite Bending Strength:	86 KNm	

Shear Strength

Shear Strength Required:	35 kN
Dependable Shear Strength:	201 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	41 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	0.7 mm
SDL+φs LL+Shrink+Creep:	2.3 mm
Span/DF : Steel Immediate:	4672
Span/DF : Top of Concrete:	1392
Span/DF : Steel Total:	1073

Vibration

Natural Frequency :	40.05 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.013 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating

Composite Section Rating:	23.1 min
Non-Composite Section Rating:	21.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.75 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.75 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$162

Calculation Sheet

Job name: Queensgate — Level Redevelopment Job No: 2711265/10a

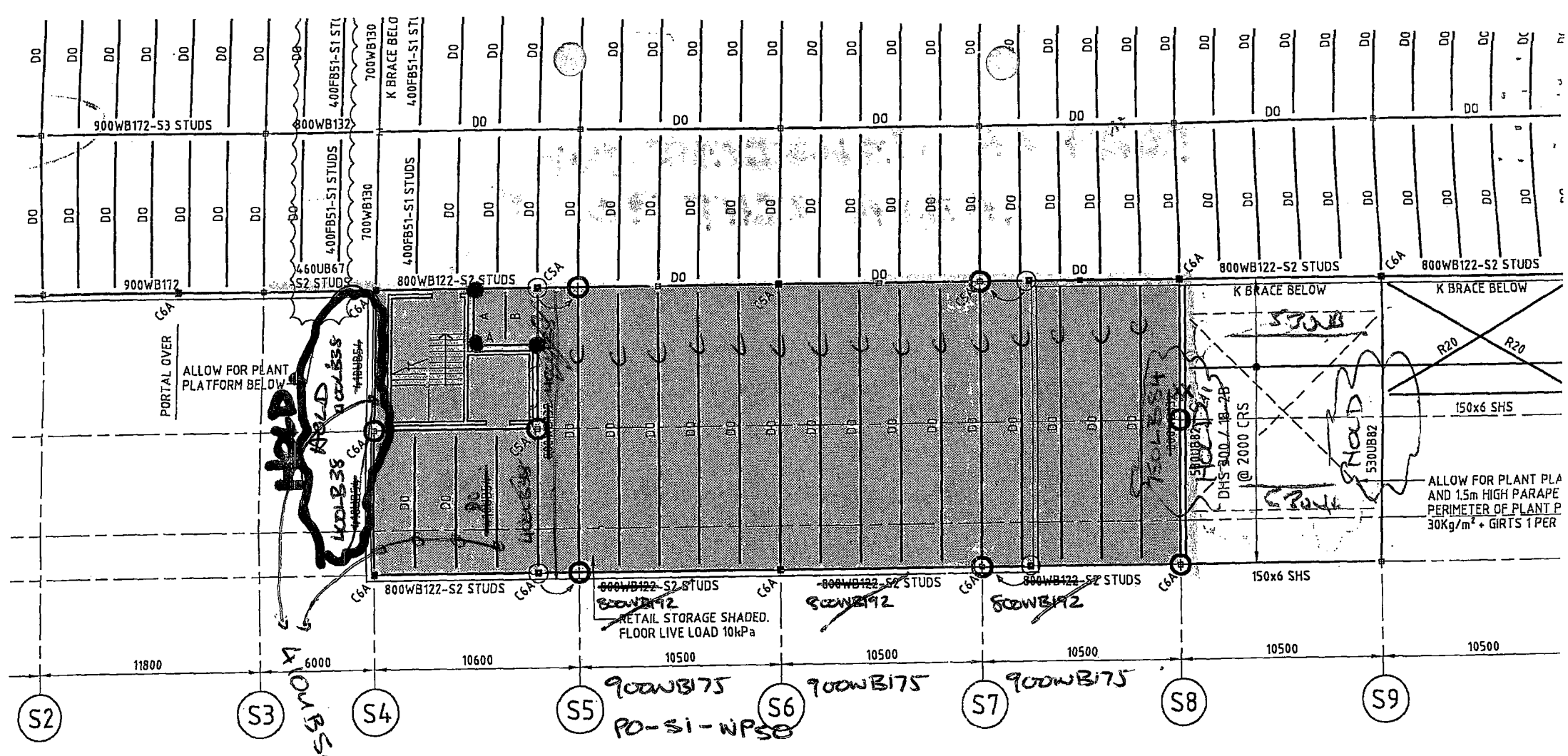
Subject: Zone B3 LVL2 floor beam design Page No: of

By: DWS Date: 20-5-04

Connections:

750LB84	to	Primary	WP60 DWC	750LB84	
450LB38	to	Primary	WP60 SWC	450LB46.	Desg check (same as)
180UB16	to	Primary	WP50 SWC	180UB16.	
750LB84	to	Column	WP60 NC	750LB84	
800WB192	to	Column.	~~AC 1-2 70 NC 800WB192~~		Desig
800WB122	to	Column	WP40 NC	800WB122	
450LB38	to	Column.	WP60 NC	450LB46	Desg check. (same as)
530UB92	to	Column.	WP50 NC	530UB92	
250UB26	to	Column	WP40 NC	250UB26	

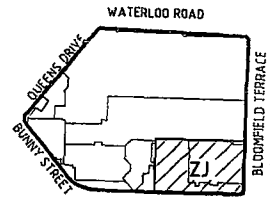
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- Columns.
- A = 250UB26
- B = 180UB16
- C = 750LB84

all beams have studs @ 300 Centres.

750LB84 only have 25mm Precamber.



NOTE
ALL DIMENSIONS TO BE CHECKED ON SITE.
WRITTEN DIMENSIONS ONLY TO BE USED.
REFER TO ALL DETAIL DWGS, STRUCTURAL MECHANICAL & SERVICE DWGS BEFORE COMMENCING WORK.
REFER ANY DISCREPANCIES TO ARCHITECTS.

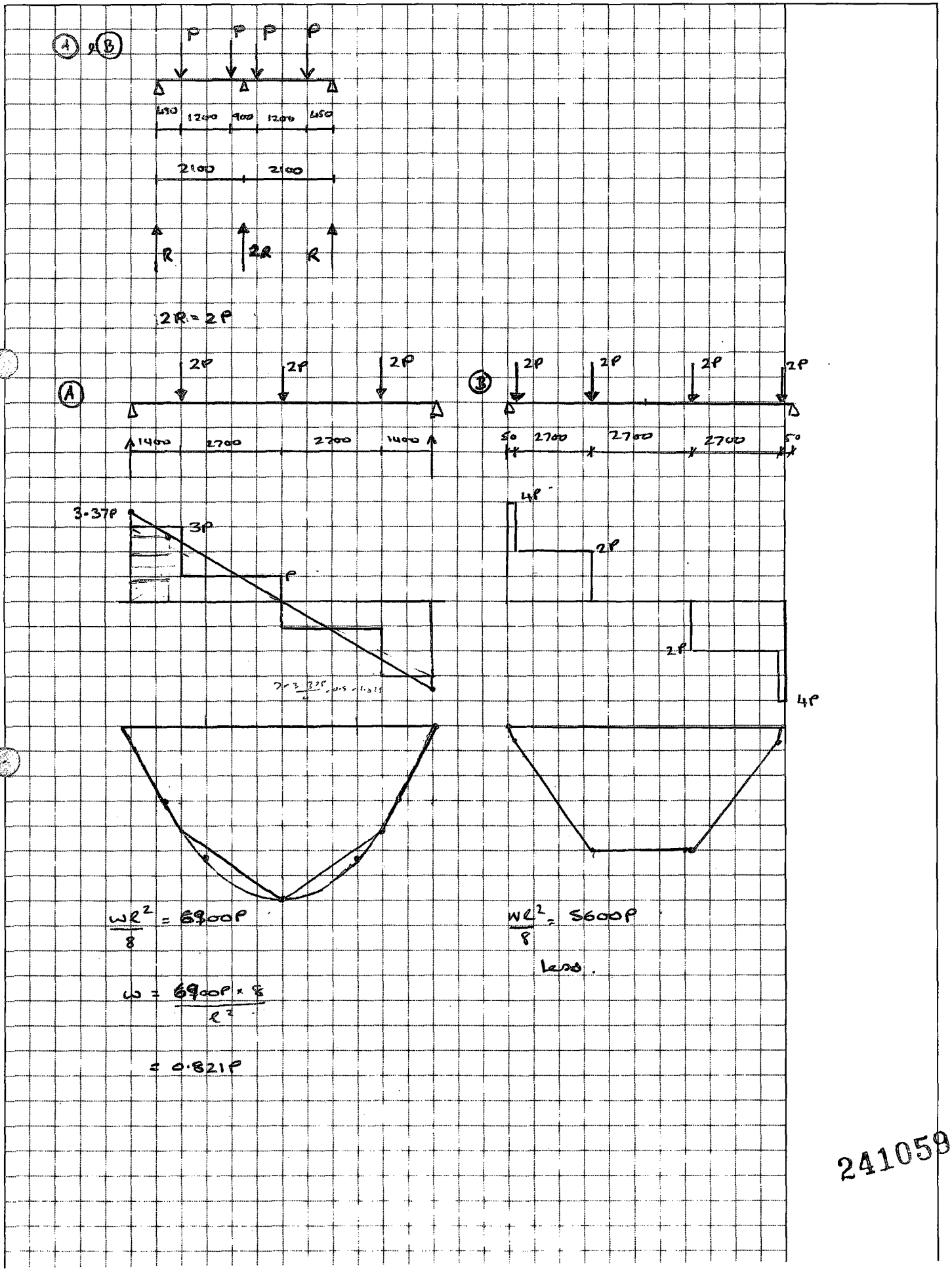
Description	Rev.	Date	By	Description

TE REDEVELOPMENT
LEVEL 2

Scale	Project/Job Number	Computer Ref.	Drawing Number
1:200	2711265	S102--ZJ	S102-

Calculation Sheet

Job name: Queensgate mall Redevelopment
Job No: 2711265/109
Subject: Warehouse floor design LVL2 PI Main Span
Page No: 1 of
By: DJOS
Date: 18-5-04



$\frac{wL^2}{8} = 6900P$

$w = \frac{6900P \times 8}{L^2}$

$= 0.821P$

$\frac{wL^2}{8} = 5600P$

less.

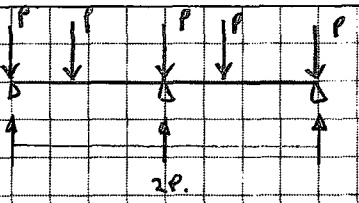
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Calculation Sheet

Job name: Queensgate mall Redevelopment. Job No: 2711265/109

Subject: Warehouse floor slab Zone B5 Mac floor LVL2 Page No: 2 of

By: DJS. Date: 18-5-04



as per (A).



(D) $R_2 = P\left(\frac{0.9+2.1+3.0+4.2+5.1+6.3+7.2}{8.2}\right)$.

$= 3.512P$

$R_1 = 3.488P$

(E) $R_2 = P\left(\frac{1.2+2.1+3.3+4.2+5.4+6.3+7.5}{8.2}\right)$.

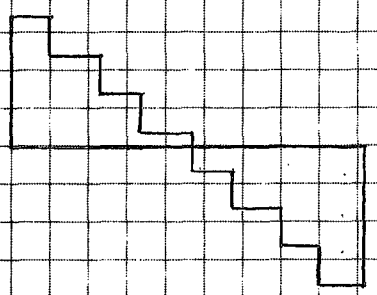
$= 3.659P$

$R_1 = 3.341P$

(F) $R_2 = P\left(\frac{0.6+2.0+2.9+4.1+5.0+6.2+7.1}{8.2}\right)$.

$= 3.402P$.

$R_1 = 3.598P$.

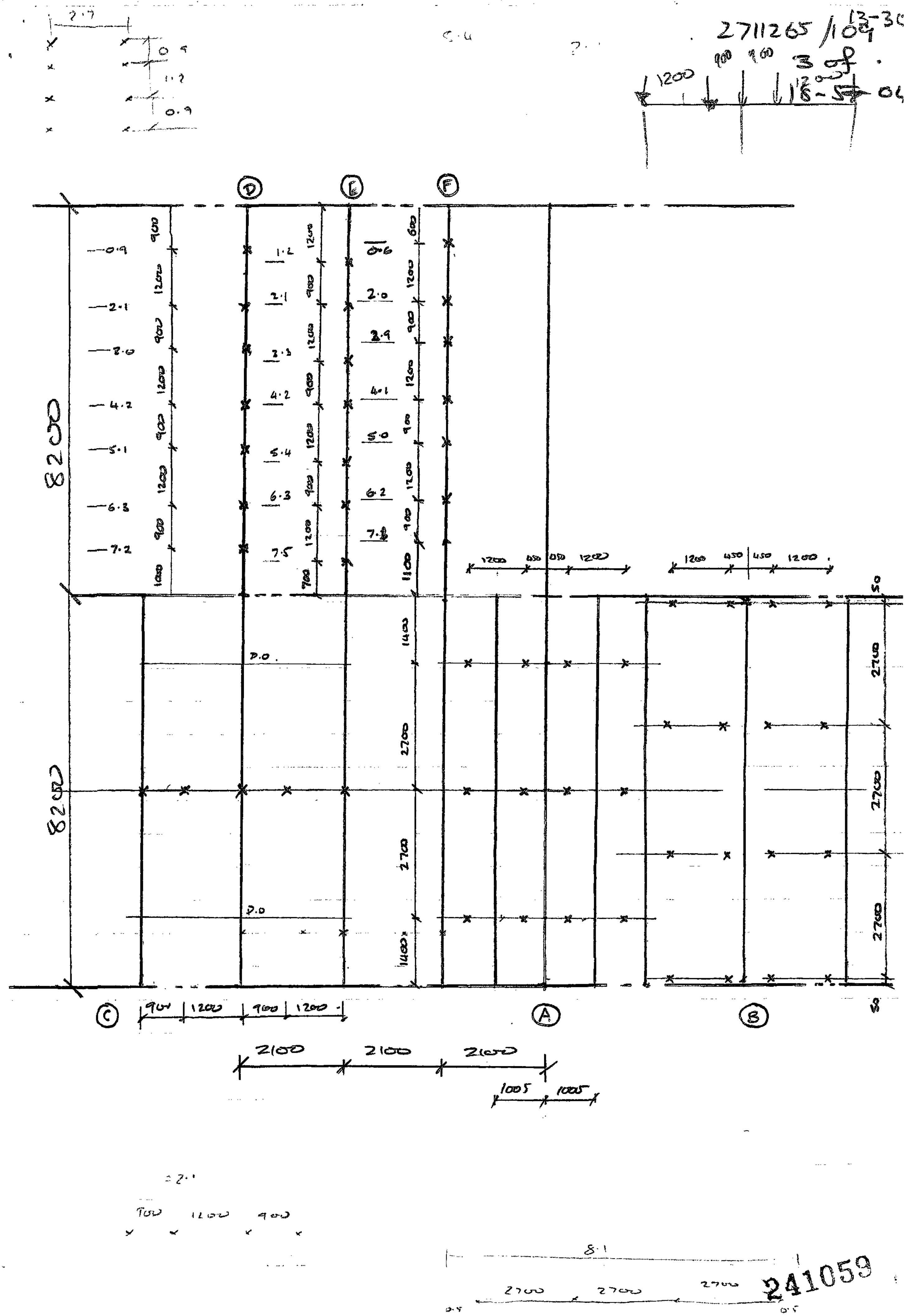


3.488	0.9	3.341	1.2	3.598	0.6
2.488	1.2	2.341	0.9	2.598	1.2
1.488	0.9	1.341	1.2	1.598	0.9
0.488	1.2	.341	0.9	.598	1.2
M = 8.05P		M = 8.03P		M = 7.43P	

$W = \frac{8.05P \times 8}{8.2^2}$.

$= 0.958P$

$L_u = 0.456P$ (kPa)



2711265 /10
3 of
D
E
F
8200
8200
C
A
B
900
1200
900
1200
2100
2100
2100
1005
1005
1200
450
450
1200
2700
2700
2700
1400
1400
8·1
2700
2700
2700

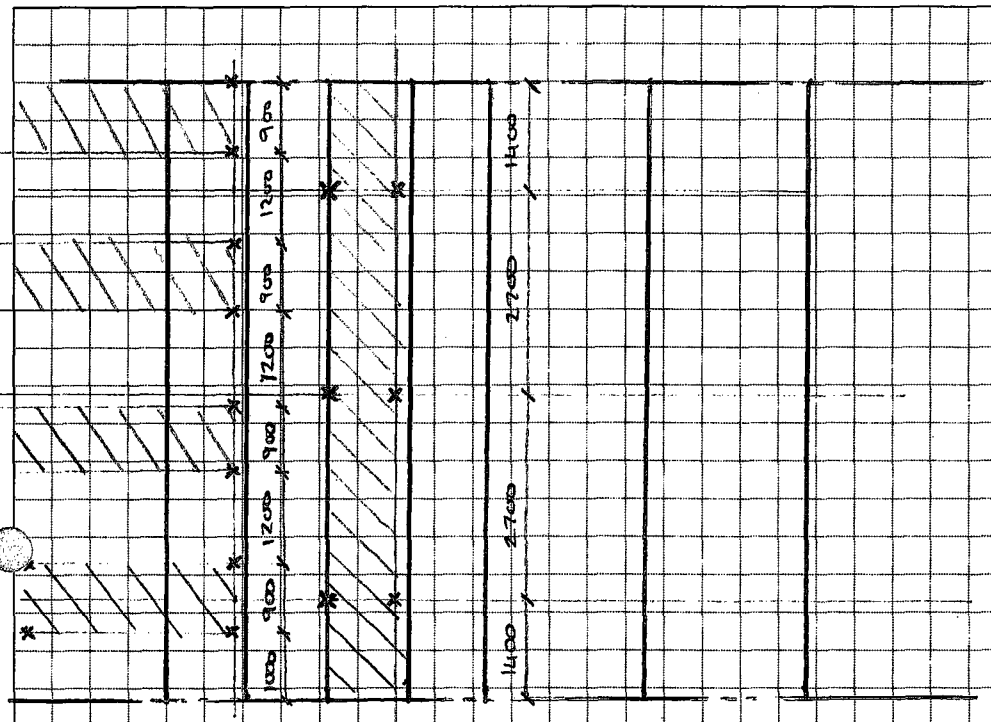
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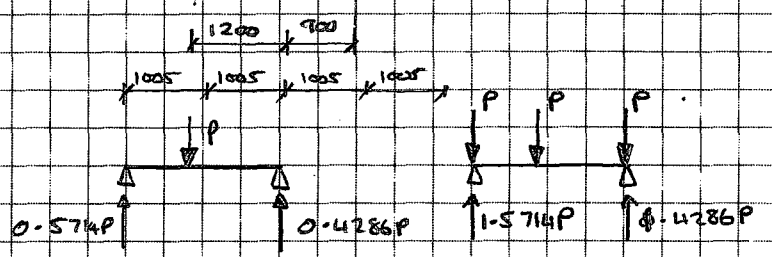
Calculation Sheet

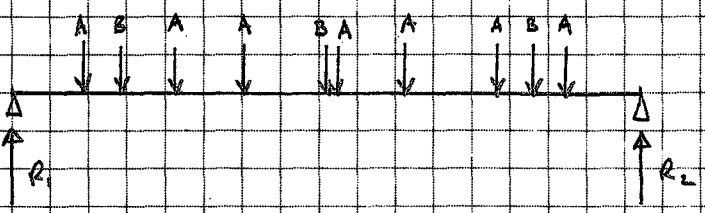
Job name: Queensgate mall Redevelopment **Job No:** 2711265/109

Subject: Warehouse floor slab Zone ZJ LVL2 Mainfloor **Page No:** 4 of

By: Dias. **Date:** 19-5-04







A = 0.4286P

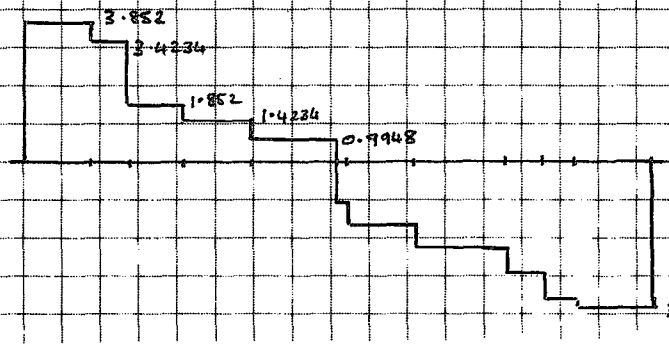
B = 1.5714P

$$R_2 = \frac{0.4286P(0.9+2.1+3.0+4.2+5.1+6.3+7.2) + 1.5714P(1.4+4.1+6.8)}{8.2}$$

$$= \frac{12.3437P + 19.3262P}{8.2}$$

$$= 3.8624P$$

$$R_1 = 3.8520P$$



$M = 8.8502P$

$$w = \frac{8 \times 8.8502P}{8.2^2}$$

$$= 1.053P$$

Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Warehouse floor design LVL2 ZJ Mezz. floor
By: DWS

Job No: 2711265/109
Page No: 5 of
Date: 19-5-04

load Summary for racking System: for secondary beams.

$W_{max} = 1.053P$ (kN/m)
$= 0.5014P$ (kPa)

$V_{max} = 4.648P$ (kN) at support

EQ loading Requires P = 29kN.

$W_{max} = 0.5014 \times 29kN$
$= 14.54$ kPa. or 13.22kPa for straight beam only.

$V_{max} = 4.648 \times 29kN$
$= 135$ kN.

USE 4255B53

$V_{max} = 135$

$\phi V_b = 413kN$ from Table. ✓

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE #6
SECTION : Secondary beams SB to SG 8.2m span
JOB No.: 2711265/109 DESIGN: dws
DATE : 20-May-04 CHECK :

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	8200 mm
Adj. Beam Centres :	2100 mm
	2100 mm
Beam Size	425SB53
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	27 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	14.54 KPa Rack loads
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	18 KNm
Depend. Bending Strength	95 KNm

Stage II : Concrete Placement

Factored Bending Moment :	102 KNm
Depend. Bending Strength	310 KNm

% Composite Action 62 %

Bending Strength

Bending Strength Required :	498 KNm
Dependable Bending Strength :	529 KNm

Shear Strength

Shear Strength Required:	243 kN
Dependable Shear Strength:	413 kN

Shear

Unshored Bottom Flange Tension

Serviceability Flange Stress:	216 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	9.9 mm
SDL+φs LL+Shrink+Creep:	21.8 mm
Span/DF : Steel Immediate:	829
Span/DF : Top of Concrete:	376
Span/DF : Steel Total:	259

Vibration

Natural Frequency :	8.07 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.060 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating 16.3 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$510

COBENZ 97 1.2 a

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Licence No.: 990702

USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams between SB & SG S1 & S4
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	8200 mm
Adj. Beam Centres :	2000 mm
	2000 mm
Beam Size	425SB53
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	27 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	14.20 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	18 KNm
Depend. Bending Strength	95 KNm

Stage II : Concrete Placement

Factored Bending Moment :	97 KNm
Depend. Bending Strength	310 KNm

% Composite Action	62 %

Bending Strength

Bending Strength Required :	465 KNm
Dependable Bending Strength :	529 KNm

Shear Strength

Shear Strength Required:	227 kN
Dependable Shear Strength:	413 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	202 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	9.4 mm
SDL+φs LL+Shrink+Creep:	20.9 mm
Span/DF : Steel Immediate:	872
Span/DF : Top of Concrete:	393
Span/DF : Steel Total:	271

Vibration

Natural Frequency :	8.56 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.060 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating	17.0 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$510

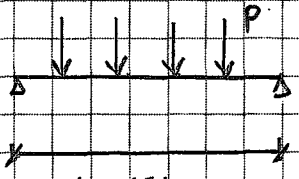
241059

Calculation Sheet

Job name: Queensgate mall Redevelopment.
Subject: Zone 25 LVL2 floor beam design.
By: DOS

Job No: 2711265/109
Page No: 7 of
Date: 20-5-04

Internal Primary beams. S4 to S11 x S6 to S5.



P from 4255B53

$P = 53 \times \frac{9.81}{1000} \times \frac{8.2}{2}$.

$= 2.13\,kN \times 2$

$M_1 = 13.04\,kNm \times 2$ $M_2 = 13.43\,kNm$ $M_3 = 13.86\,kNm$

$w_1 = 1.004\,kN/m$ $w_2 = 0.975\,kN/m$ $w_3 = 0.987\,kN/m$.
$= 2.008\,kN/m$. $\times 2$ $\times 2$

Design = 11 for $w_1 = 1.004\,kN/m \times 2 = 2.008\,kN/m$.

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Licence No.: 990702

USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams between SC to SF S4 to S11
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	800WB122
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 HI-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	52 @200.
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	2.01 KN/m 2-dry beams.
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	104 KNm
Depend. Bending Strength	325 KNm

Stage II : Concrete Placement

Factored Bending Moment :	646 KNm
Depend. Bending Strength	1215 KNm

% Composite Action 52 %

Bending Strength

Bending Strength Required :	1760 KNm
Dependable Bending Strength :	1866 KNm

Shear Strength

Shear Strength Required:	671 kN
Dependable Shear Strength:	1245 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	241 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	14.4 mm
SDL+φs LL+Shrink+Creep:	20.8 mm
Span/DF : Steel Immediate:	730
Span/DF : Top of Concrete:	504
Span/DF : Steel Total:	298

Vibration

Natural Frequency :	6.98 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.064 mm
Damping Required :	3.1 %

FRR: Fire Resistance Rating 16.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

Comparative Cost:

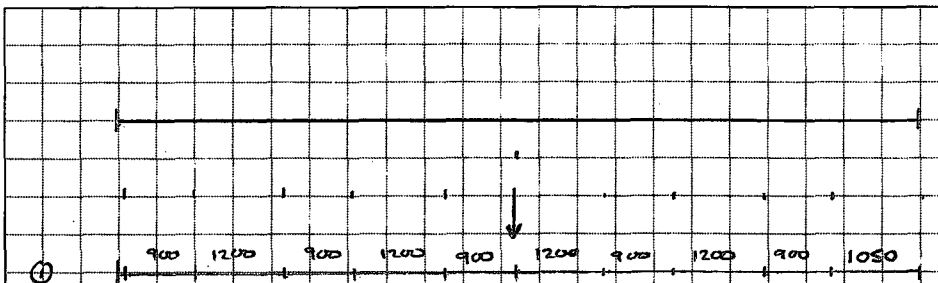
Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$1,427

241059

Calculation Sheet

Job name: Queensgate
Subject:
By: DWS

Job No: 2711265 }109
Page No: 9 of
Date: 21-5-04.



① $R_1 = 5.43P$ kN $R_2 = 4.57P$ kN

② $R_1 = 4.57P$ kN $R_2 = 4.43P$ kN

③ $R_1 = 5P$ kN $R_2 = 5P$ kN

①	
5.43	150
4.43	900
3.43	1200
2.43	900
1.43	1200
0.43	900

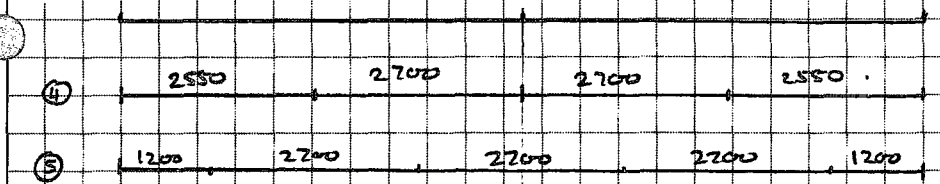
$M = 13.21P$ kNm

②	
4.57	900
3.57	1200
2.57	900
1.57	1200
0.57	900

$M = 13.11P$ kNm

③	
5	600
4	900
3	1200
2	900
1	1200

$M = 13.2$ kNm



④ $R_1 = 3P$ kN $= R_2$

⑤ $R_1 = 4P$ kN $= R_2$ 232 kN.

$M_4 = 3P \times 2.55 + P \times 2.7$
$= 10.35P$ kNm

$M_5 = 4P \times 1200 + 2P \times 2700$
$= 10.2P$ kNm

Governing load $M = 13.21P$ kNm for 10.5m. 13.45 for 10.6m.

$w = 0.959P$ kN/m $w = 0.976$ kN/m

$P = 29$ kN for DD worst case $w = 27.8$ kN/m

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams between SA to SB S4 to S11
JOB No.: 2711265/109 DESIGN: dws
DATE : 20-May-04 CHECK :

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10600 mm
Adj. Beam Centres :	2067 mm
	2067 mm
Beam Size	525SB77
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	35 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	34 KNm
Depend. Bending Strength	134 KNm

Stage II : Concrete Placement

Factored Bending Moment :	161 KNm
Depend. Bending Strength	539 KNm

% Composite Action 56 %

Bending Strength

Bending Strength Required :	764 KNm
Dependable Bending Strength :	862 KNm

Shear Strength

Shear Strength Required:	288 kN
Dependable Shear Strength:	680 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	202 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-12.8 mm
SDL+φs LL+Shrink+Creep:	30.8 mm
Span/DF : Steel Immediate:	-829
Span/DF : Top of Concrete:	345
Span/DF : Steel Total:	590

Vibration

Natural Frequency :	6.97 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.08 mm
Initial Amplitude from Heel Drop	0.060 mm
Damping Required :	3.1 %

FRR: Fire Resistance Rating 17.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.22 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.22 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$914

241059

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Licence No.: 990702

USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams between SA to SB S4 to S11
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500 mm
Adj. Beam Centres :	2067 mm
	2067 mm
Beam Size	500SB68
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	35 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	32 KNm
Depend. Bending Strength	128 KNm

Stage II : Concrete Placement

Factored Bending Moment :	158 KNm
Depend. Bending Strength	472 KNm

% Composite Action 63 %

Bending Strength

Bending Strength Required :	749 KNm
Dependable Bending Strength :	760 KNm

Shear Strength

Shear Strength Required:	285 kN
Dependable Shear Strength:	440 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	223 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-11.4 mm
SDL+φs LL+Shrink+Creep:	32.7 mm
Span/DF : Steel Immediate:	-921
Span/DF : Top of Concrete:	321
Span/DF : Steel Total:	492

Vibration

Natural Frequency :	6.58 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.09 mm
Initial Amplitude from Heel Drop	0.066 mm
Damping Required :	3.1 %

FRR: Fire Resistance Rating 16.3 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.87 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.87 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$816

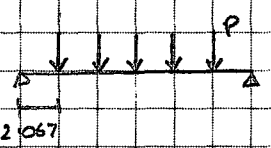
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

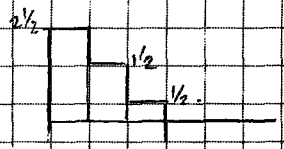
Primary beams SA to SB. (S4) to (S11)



P from S255B77 max.

$P = \frac{77 \times 9.81}{1000} \times 10.6$

$P = 8.01\ kN$



$= (2\frac{1}{2} + 1\frac{1}{2} + \frac{1}{2}) \times 8.01 \times 2.067$

$M = 74.5\ kNm$

$w = 3.87\ kN/m$

241059

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams between SA to SB S5 to S10
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400	mm
Adj. Beam Centres :	10600	mm
	10600	mm
Beam Size	800SB208	
Beam Yield Stress	300	MPa
Precamber	0	mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span :	2100	mm
Deck Span Type :	Triple	
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	62 @ 200	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	3.87	KN/m
Super Live Load :	6.50	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
i) Deck Placement:	0.40	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fu	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	198	KNm
Depend. Bending Strength	1040	KNm

Stage II : Concrete Placement

Factored Bending Moment :	1179	KNm
Depend. Bending Strength	2483	KNm

% Composite Action	37 %

Bending Strength

Bending Strength Required :	3216	KNm
Dependable Bending Strength :	3359	KNm

Shear Strength

Shear Strength Required:	1037	kN
Dependable Shear Strength:	1750	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	222	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Steel Beam Immed.:	15.3	mm
DL+φs LL+Shrink+Creep:	26.1	mm
Span/DF : Steel Immediate:	809	
Span/DF : Top of Concrete:	475	
Span/DF : Steel Total:	299	

Vibration

Natural Frequency :	6.29	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02	mm
Initial Amplitude from Heel Drop	0.056	mm
Damping Required :	3.0	%

FRR: Fire Resistance Rating	18.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.84 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm
Bottom Flange:vwb: VdepQb/Is		1.84 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,758

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SB between S4 and S11
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10600 mm
Adj. Beam Centres :	2067 mm
	8200 mm
Beam Size	800WB122
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	70 @ 150.
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	1.00 KN/m 2 edge beam.
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	74 KNm
Depend. Bending Strength	320 KNm

Stage II : Concrete Placement

Factored Bending Moment :	414 KNm
Depend. Bending Strength	1215 KNm

% Composite Action 70 %

Bending Strength

Bending Strength Required :	1928 KNm
Dependable Bending Strength :	1967 KNm

Shear Strength

Shear Strength Required:	728 kN
Dependable Shear Strength:	1245 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	240 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	9.4 mm
SDL+φs LL+Shrink+Creep:	23.9 mm
Span/DF : Steel Immediate:	1127
Span/DF : Top of Concrete:	444
Span/DF : Steel Total:	318

Vibration

Natural Frequency :	8.45 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.050 mm
Damping Required :	3.1 %

FRR: Fire Resistance Rating 17.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$1,489

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Stair beams at S11 SF
JOB No.: 2711265/109 DESIGN: dws
DATE : 21-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	5200 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	2000 mm	
Stud Edge Distance	160 mm	
Beam Size	250UB31	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	single
Deck Span :	2000 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	17 @ 300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.22 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	3 KNm	
Depend. Bending Strength	48 KNm	

Stage II : Concrete Placement

Factored Bending Moment :	21 KNm	
Depend. Bending Strength	107 KNm	
% Composite Action	67 %	

Bending Strength

Bending Strength Required :	97 KNm	
Composite Bending Strength :	225 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	107 KNm	

Shear Strength

Shear Strength Required:	75 kN
Dependable Shear Strength:	248 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	109 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	4.1 mm
SDL+φs LL+Shrink+Creep:	8.8 mm
Span/DF : Steel Immediate:	1255
Span/DF : Top of Concrete:	589
Span/DF : Steel Total:	401

Vibration

Natural Frequency :	14.88 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.06 mm
Initial Amplitude from Heel Drop	0.042 mm
Damping Required :	3.4 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.1 min
Non-Composite Section Rating:	16.0 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$313

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Secondary beams G.A to S5 S1 to S3.



$R_1 = 6P$ kN

6P	102.5
5	1200
4	900
3	1200
2	900
1P	1200

M: 16.82P kNm

w = 0.949P kN/m

= use Prelim values as Greater.

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams between SA to SB S1 to S3
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	2067 mm
	2067 mm
Beam Size	530UB82
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 HI-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	59 @200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	44 KNm
Depend. Bending Strength	115 KNm

Stage II : Concrete Placement

Factored Bending Moment :	209 KNm
Depend. Bending Strength	556 KNm

% Composite Action	89 %

Bending Strength

Bending Strength Required :	969 KNm
Dependable Bending Strength :	1031 KNm

Shear Strength

Shear Strength Required:	326 kN
Dependable Shear Strength:	817 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	248 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-5.4 mm
SDL+φs LL+Shrink+Creep:	41.8 mm
Span/DF : Steel Immediate:	-2220
Span/DF : Top of Concrete:	285
Span/DF : Steel Total:	326

Vibration

Natural Frequency :	5.37 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.11 mm
Initial Amplitude from Heel Drop	0.062 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating	17.8 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.35 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.35 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

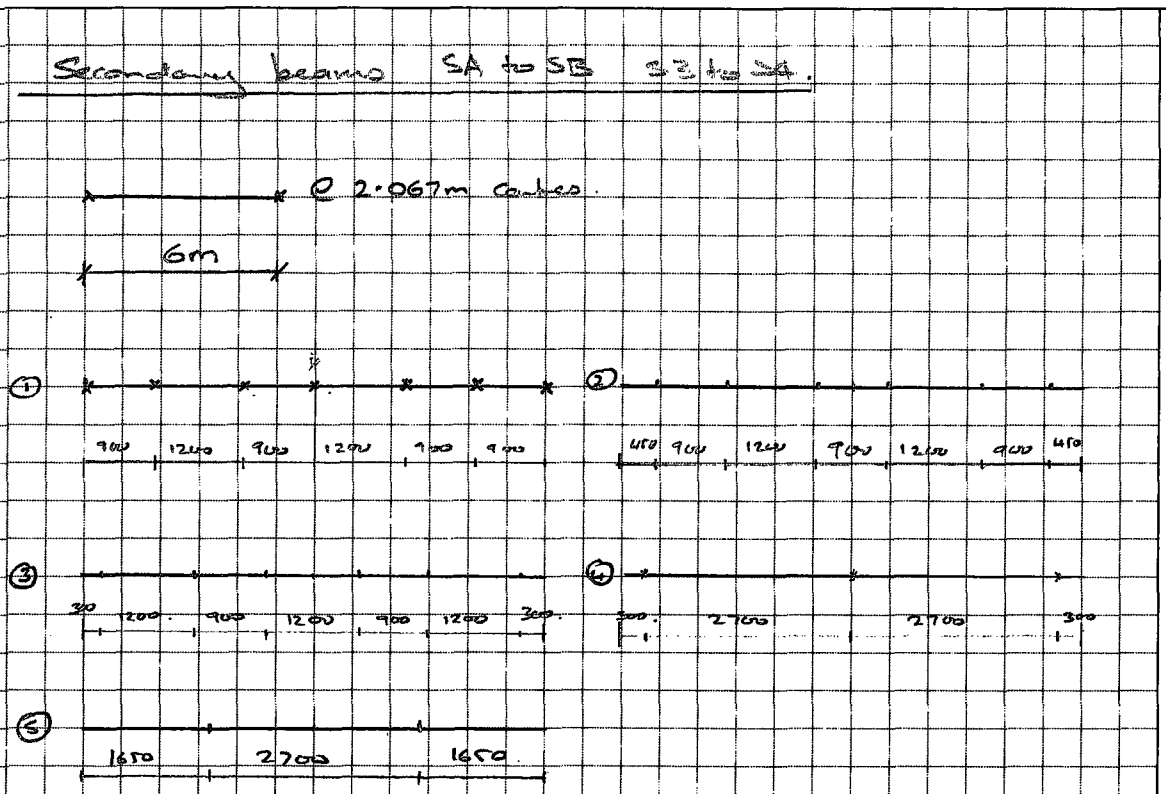
Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$1,141

Calculation Sheet

Job name: Queensgate mall Redevelopment
Job No: 2711265/109
Subject: W/H Zone 25 floor design LVL2
Page No: of
By: DJS
Date: 24-5-04

Secondary beams 5A to 5B 3.3 to 3.4.



	R_1	R_2	M
1	2.45P	2.55P	4.35P
2	3P	3P	4.35P
3	3P	3P	4.2P
4	3P	3P	3.6P
5	2P	2P	3.3P

Governing load 4.35P kNm

$w = 0.9667P$ kN/m
$= 28$ kN/m
$= 13.56$ kPa

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beams between SA & SB S3 to S4
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6000 mm
Adj. Beam Centres :	2067 mm
	2067 mm
Beam Size	310UB40
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2067 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	20 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.56 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	9 KNm
Depend. Bending Strength	74 KNm

Stage II : Concrete Placement

Factored Bending Moment :	53 KNm
Depend. Bending Strength	168 KNm

% Composite Action 61 %

Bending Strength

Bending Strength Required :	247 KNm
Dependable Bending Strength :	319 KNm

Shear Strength

Shear Strength Required:	164 kN
Dependable Shear Strength:	300 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	180 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.9 mm
SDL+φs LL+Shrink+Creep:	13.4 mm
Span/DF : Steel Immediate:	868
Span/DF : Top of Concrete:	449
Span/DF : Steel Total:	296

Vibration

Natural Frequency :	11.91 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.05 mm
Initial Amplitude from Heel Drop	0.061 mm
Damping Required :	3.5 %

FRR: Fire Resistance Rating 18.0 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.96 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.96 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$298

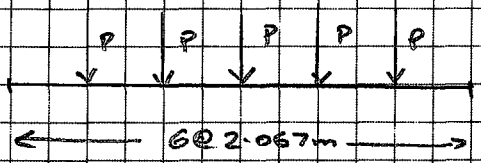
Calculation Sheet

Job name: Queensgate – Level Redevelopment
Job No: 2711265/109
Subject: W/H Zone 23 LVL2 floor beam design
Page No: of
By: DWS
Date: 24-5-04

Primary beams on S3 x S4 between S4 x S8.

$$P = 82 \cdot \frac{9.81}{1000} \times \frac{11.8}{2} + 40 \times \frac{9.81}{1000} \times \frac{5.0}{2}$$

$$= 5.923 \text{ kN}.$$



$m = 55.1 \text{ kNm}$
$w = 2.87 \text{ kN/m}$.

Primary Beam on S2 between S4 x S3

$$P = 82 \times \frac{9.81}{1000} \times \frac{11.8 + 11.905}{2}$$

$$= 9.534.$$

$m = 88.65 \text{ kNm}$.
$w = 4.61 \text{ kN/m}$.

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams between SA & SB on S3 & S4
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm
Adj. Beam Centres :	6000 mm
	11800 mm
Beam Size	800WB192
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2067 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	62 @200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	2.87 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	170 KNm
Depend. Bending Strength	823 KNm

Stage II : Concrete Placement

Factored Bending Moment :	996 KNm
Depend. Bending Strength	2160 KNm

% Composite Action	40 %

Bending Strength

Bending Strength Required :	2697 KNm
Dependable Bending Strength :	2856 KNm

Shear Strength

Shear Strength Required:	870 kN
Dependable Shear Strength:	1282 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	215 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	16.4 mm
SDL+φs LL+Shrink+Creep:	27.3 mm
Span/DF : Steel Immediate:	755
Span/DF : Top of Concrete:	454
Span/DF : Steel Total:	283

Vibration

Natural Frequency :	5.81 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.056 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating	19.1 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.59 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.59 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,554

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams between SA & SB on S2
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm
Adj. Beam Centres :	11905 mm
	11800 mm
Beam Size	900WB218
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2067 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	82 @ 150.
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	4.61 KN/m 2 poly beams
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	218 KNm
Depend. Bending Strength	1082 KNm

Stage II : Concrete Placement

Factored Bending Moment :	1316 KNm
Depend. Bending Strength	2689 KNm

% Composite Action 46 %

Bending Strength

Bending Strength Required :	3600 KNm
Dependable Bending Strength :	3650 KNm

Shear Strength

Shear Strength Required:	1161 kN
Dependable Shear Strength:	1769 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	234 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	15.8 mm
SDL+φs LL+Shrink+Creep:	26.3 mm
Span/DF : Steel Immediate:	782
Span/DF : Top of Concrete:	471
Span/DF : Steel Total:	294

Vibration

Natural Frequency :	6.16 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.057 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating 18.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.87 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm
Bottom Flange:vwb: VdepQb/Is		1.87 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,933

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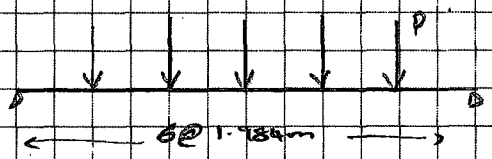
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Primary beam along SB Gridline 3



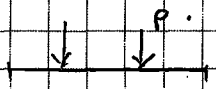
P = 2.13kN.

m = (2½ + 1½ + ½) × 1.784 × 2.13.

= 19.0kNm

w = 1.07kN/m.

Primary Beam along SB Gridline 84.



P = 2.13kN

m = 4.26kNm

w = 0.947kN/m.

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SB between S1 & S3
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	2067 mm
	8200 mm
Beam Size	800Wb168
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	59 @ 200.
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	1.09 KN/m 2 edge beams
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	103 KNm
Depend. Bending Strength	611 KNm

Stage II : Concrete Placement

Factored Bending Moment :	532 KNm
Depend. Bending Strength	1831 KNm

% Composite Action 43 %

Bending Strength

Bending Strength Required :	2443 KNm
Dependable Bending Strength :	2521 KNm

Shear Strength

Shear Strength Required:	821 kN
Dependable Shear Strength:	1273 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	208 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	9.7 mm
SDL+φs LL+Shrink+Creep:	28.5 mm
Span/DF : Steel Immediate:	1226
Span/DF : Top of Concrete:	417
Span/DF : Steel Total:	311

Vibration

Natural Frequency :	7.89 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.046 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating 19.7 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.54 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.54 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,165

241059

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary Beams along SB between S1 & S3
JOB No.: 2711265/109 DESIGN: dws
DATE : 28-May-04 CHECK :

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905	mm
Adj. Beam Centres :	2067	mm
	8200	mm
Beam Size	900WB175	
Beam Yield Stress	300	MPa
Precamber	0	mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span :	2100	mm
Deck Span Type :	Triple	
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	59	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	1.09	KN/m
Super Live Load :	13.50	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
i) Deck Placement:	0.40	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fu	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	104	KNm
Depend. Bending Strength	613	KNm

Stage II : Concrete Placement

Factored Bending Moment :	531	KNm
Depend. Bending Strength	2025	KNm

% Composite Action	42 %

Bending Strength

Bending Strength Required :	2443	KNm
Dependable Bending Strength :	2863	KNm

Shear Strength

Shear Strength Required:	821	kN
Dependable Shear Strength:	1750	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	190	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	8.1	mm
SDL+φs LL+Shrink+Creep:	24.6	mm
Span/DF : Steel Immediate:	1466	
Span/DF : Top of Concrete:	484	
Span/DF : Steel Total:	364	

Vibration

Natural Frequency :	8.69	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02	mm
Initial Amplitude from Heel Drop	0.040	mm
Damping Required :	3.0	%

FRR: Fire Resistance Rating	20.7	min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73	KN/mm
Weld Leg Reqd: SP	Single side	10	mm
	Double side	5	mm
Bottom Flange:vwb: VdepQb/Is		1.73	KN/mm
Weld Leg Reqd: SP	Single side	10	mm
	Double side	5	mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,249

13-54

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SB between S3 & S4
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 24-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6000 mm
Adj. Beam Centres :	2067 mm
	8200 mm
Beam Size	460UB67
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	30
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.95 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	21 KNm
Depend. Bending Strength	181 KNm

Stage II : Concrete Placement

Factored Bending Moment :	128 KNm
Depend. Bending Strength	397 KNm

% Composite Action	55 %

Bending Strength

Bending Strength Required :	613 KNm
Dependable Bending Strength :	665 KNm

Shear Strength

Shear Strength Required:	409 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	213 MPa
Yow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	4.9 mm
SDL+φs LL+Shrink+Creep:	11.9 mm
Span/DF : Steel Immediate:	1227
Span/DF : Top of Concrete:	504
Span/DF : Steel Total:	357

Vibration

Natural Frequency :	13.54 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.054 mm
Damping Required :	3.5 %

FRR: Fire Resistance Rating	17.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$487

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary Beams along SA,SB between S3 to S4
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 28-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6000 mm
Adj. Beam Centres :	2067 mm
	8200 mm
Beam Size	625LB71
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	30
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.95 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	21 KNm
Depend. Bending Strength	227 KNm

Stage II : Concrete Placement

Factored Bending Moment :	128 KNm
Depend. Bending Strength	508 KNm

% Composite Action 52 %

Bending Strength

Bending Strength Required :	612 KNm
Dependable Bending Strength :	890 KNm

Shear Strength

Shear Strength Required:	408 kN
Dependable Shear Strength:	794 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	163 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	2.6 mm
SDL+φs LL+Shrink+Creep:	7.7 mm
Span/DF : Steel Immediate:	2274
Span/DF : Top of Concrete:	783
Span/DF : Steel Total:	583

Vibration

Natural Frequency :	17.35 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.01 mm
Initial Amplitude from Heel Drop	0.034 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating 18.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.05 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		1.05 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$510

Calculation Sheet

Job name:

Subject:

By:

Job No: 2711265/109

Page No: of

Date: 25-4-05

Beam (S11) SF to SG.

$0.250 \times 2.0\,m \times 23.5 = 11.75\ kN/m$

$+ 5.0\,kPa \times 2.0\,m = 10\ kN/m$

$w = 21.75\ kN/m$

$m = \frac{wL^2}{8}$

$= \frac{21.75 \times 8.2^2}{8}$

$= 183\ kNm$

$L_e = 8.2\,m$

$\phi M_b = 203\ kNm$ 530UB92.4

$M^* \leq \phi M_b$

$183 \leq 203\ kNm$

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : ~~Stair beams at S11 SF~~ Beams on S4 S4 to S5
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :** S8 to S10.

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties		
Beam Span :	10600 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	2067 mm	
Stud Edge Distance	160 mm	
Beam Size	525SB77	
Beam Yield Stress	300 MPa	
Precamber	25 mm	
Decking and Slab Properties		
Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2100 mm	
Slab Thickness :	160 mm	
Slab Mass :	3.17 KPa	
Concrete Strength :	40.0 MPa	
Concrete Density :	2350 kg/m3	
Shear Studs		
Stud Diameter :	19 mm	
Weld.Stud Height :	95 mm	
No. of Studs /Beam :	35	
Studs /Flange Width:	1	
Stud UTS :	415 MPa	
Loading		
Super. Dead Load :	0.50 KPa	
Other Super. DL :	0.00 KN/m	
Super Live Load :	13.50 KPa	
Other Super. LL :	0.00 KN/m	
Construction LL:		
i) Deck Placement:	0.30 KPa	
ii) Conc. Placement:	0.90 KPa	
Load Factors		
φu: Ultimate :	0.60	
φl: Long Term:	0.70	
φs: Short Term :	1.00	
Deflections		
Shrinkage Strain :	0.0003	
Vibration		
Long Term LL:	2.00 KPa	
Damping Available :	4.0 %	
Welded Flange / Web		
Minimum Plate UTS: Fup	440 MPa	
Weld Metal UTS: Fuw	530 MPa	

RESULTS

Stage I : Deck Placement		
Factored Bending Moment :	22 KNm	
Depend. Bending Strength	134 KNm	
Stage II : Concrete Placement		
Factored Bending Moment :	93 KNm	
Depend. Bending Strength	539 KNm	
% Composite Action	56 %	
Bending Strength		
Bending Strength Required :	425 KNm	
Composite Bending Strength :	848 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	539 KNm	
Shear Strength		
Shear Strength Required:	160 kN	
Dependable Shear Strength:	680 kN	
Unshored Bottom Flange Tension		
Serviceability Flange Stress:	116 MPa	
Allow. Bottom Flange Stress:	270 MPa	
Deflections (Assuming Level Screeding)		
Net Steel Beam Immed.:	-17.8 mm	
SDL+φs LL+Shrink+Creep:	22.5 mm	
Span/DF : Steel Immediate:	-595	
Span/DF : Top of Concrete:	472	
Span/DF : Steel Total:	2280	
Vibration		
Natural Frequency :	8.69 Hz	
Deflection Due to 1.0 kN Point Load at Midspan :	0.09 mm	
Initial Amplitude from Heel Drop	0.044 mm	
Damping Required :	3.0 %	
FRR: Fire Resistance Rating		
Composite Section Rating:	22.3 min	
Non-Composite Section Rating:	19.0 min	
Welded Flange / Web		
Top Flange:vwt: VdepQt/Is	1.22 KN/mm	
Weld Leg Reqd: SP Single side	7 mm	
Double side	4 mm	
Bottom Flange:vwb: VdepQb/Is	1.22 KN/mm	
Weld Leg Reqd: SP Single side	7 mm	
Double side	4 mm	
Comparative Cost		
Steel/kg :	$1.55	
Shear Stud :	$3.50	
Comparative Cost Factor:	$1,388	

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Beam along S11 between SA & SB
JOB No.: 2711265/109 DESIGN: dws
DATE : 21-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm	
Overhang:(Beam cc):	100 mm	ll Cl 13.4.1.3
Adj. Beam Centres :	10194 mm	
Stud Edge Distance	160 mm	
Beam Size	800SB208	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2067 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	62 @ 200 c/c
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	2.70 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	1.11 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	105 KNm
Depend. Bending Strength	1040 KNm

Stage II : Concrete Placement

Factored Bending Moment :	589 KNm
Depend. Bending Strength	2483 KNm

% Composite Action 53 %

Bending Strength

Bending Strength Required :	2760 KNm	
Composite Bending Strength :	3324 KNm	ll Cl 13.4.1.3
Non-Composite Bending Strength:	2483 KNm	

Shear Strength

Shear Strength Required:	890 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	182 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	7.7 mm
SDL+φs LL+Shrink+Creep:	26.8 mm
Span/DF : Steel Immediate:	1609
Span/DF : Top of Concrete:	463
Span/DF : Steel Total:	359

Vibration

Natural Frequency :	7.77 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.036 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.8 min
Non-Composite Section Rating:	18.0 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.84 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm
Bottom Flange:vwb: VdepQb/Is		1.84 KN/mm
Weld Leg Reqd: SP	Single side	11 mm
	Double side	6 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$4,223

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : ~~Stair beams at S11 SE~~ Beam at SA Sie to Sh
JOB No.: 2711265/109 DESIGN: dws
DATE : 21-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	2067 mm	
Stud Edge Distance	160 mm	
Beam Size	500SB68	
Beam Yield Stress	300 MPa	
Precamber	25 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2100 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	35
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	13.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	20 KNm	
Depend. Bending Strength	128 KNm	

Stage II : Concrete Placement

Factored Bending Moment :	90 KNm	
Depend. Bending Strength	472 KNm	
% Composite Action	63 %	

Bending Strength

Bending Strength Required :	415 KNm	
Composite Bending Strength :	746 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	472 KNm	

Shear Strength

Shear Strength Required:	158 kN
Dependable Shear Strength:	440 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	128 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-17.1 mm
SDL+φs LL+Shrink+Creep:	23.7 mm
Span/DF : Steel Immediate:	-615
Span/DF : Top of Concrete:	444
Span/DF : Steel Total:	1594

Vibration

Natural Frequency :	8.25 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.10 mm
Initial Amplitude from Heel Drop	0.048 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.0 min
Non-Composite Section Rating:	17.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.87 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.87 KN/mm
Weld Leg Reqd: SP	Single side	5 mm
	Double side	3 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,236

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along ~~SB between S4 and S11~~ S6.
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :** S4 to S8.

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6300 mm
Adj. Beam Centres :	14779 mm
	8200 mm
Beam Size	800WB122
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	21
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	3.50 KN/m
Super Live Load :	10.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	49 KNm
Depend. Bending Strength	650 KNm

Stage II : Concrete Placement

Factored Bending Moment :	312 KNm
Depend. Bending Strength	1215 KNm

% Composite Action 21 %

Bending Strength

Bending Strength Required :	1197 KNm
Dependable Bending Strength :	1550 KNm

Shear Strength

Shear Strength Required:	760 kN
Dependable Shear Strength:	1245 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	157 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	2.5 mm
SDL+φs LL+Shrink+Creep:	7.3 mm
Span/DF : Steel Immediate:	2555
Span/DF : Top of Concrete:	868
Span/DF : Steel Total:	648

Vibration

Natural Frequency :	15.46 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.00 mm
Initial Amplitude from Heel Drop	0.033 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating 19.6 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.37 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$827

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COBENZ 97 1.2a

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SP between S1 and S3
JOB No.: 2711265/109 DESIGN: dws
DATE : ~~21-May-04~~ 25-5-04 CHECK :

14-6-04 due to change in profile - Corrus

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	800WB168
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	59 @ 200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	2.08 KN/m 2 deg
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	143 KNm
Depend. Bending Strength	611 KNm

Stage II : Concrete Placement

Factored Bending Moment :	842 KNm
Depend. Bending Strength	1831 KNm

% Composite Action	43 %

Bending Strength

Bending Strength Required :	2275 KNm
Dependable Bending Strength :	2521 KNm

Shear Strength

Shear Strength Required:	764 kN
Dependable Shear Strength:	1273 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	212 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	15.3 mm
SDL+φs LL+Shrink+Creep:	24.7 mm
Span/DF : Steel Immediate:	777
Span/DF : Top of Concrete:	482
Span/DF : Steel Total:	297

Vibration

Natural Frequency :	6.42 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.058 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating	18.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.54 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.54 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,165

COBENZ 97 1.2 a
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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SF,SE,SD,SC between S3
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6000 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	460UB67
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	30 Q200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	2.08 KN/m 2 dary.
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	31 KNm
Depend. Bending Strength	181 KNm

Stage II : Concrete Placement

Factored Bending Moment :	204 KNm
Depend. Bending Strength	397 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	568 KNm
Dependable Bending Strength :	665 KNm

Shear Strength

Shear Strength Required:	379 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	221 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	7.8 mm
SDL+φs LL+Shrink+Creep:	10.4 mm
Span/DF : Steel Immediate:	770
Span/DF : Top of Concrete:	578
Span/DF : Steel Total:	330

Vibration

Natural Frequency :	11.20 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.071 mm
Damping Required :	3.6 %

FRR: Fire Resistance Rating 16.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$487

241059

13-65

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	6000	mm
Adj. Beam Centres :	8200	mm
	8200	mm
Beam Size	625LB71	
Beam Yield Stress	300	MPa
Precamber	0	mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span :	2100	mm
Deck Span Type :	Triple	
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	30	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	2.08	KN/m
Super Live Load :	6.50	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
i) Deck Placement:	0.40	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fu	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	31	KNm
Depend. Bending Strength	227	KNm

Stage II : Concrete Placement

Factored Bending Moment :	202	KNm
Depend. Bending Strength	508	KNm

% Composite Action 52 %

Bending Strength

Bending Strength Required :	566	KNm
Dependable Bending Strength :	890	KNm

Shear Strength

Shear Strength Required:	378	kN
Dependable Shear Strength:	794	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	168	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	4.2	mm
SDL+φs LL+Shrink+Creep:	6.8	mm
Span/DF : Steel Immediate:	1436	
Span/DF : Top of Concrete:	888	
Span/DF : Steel Total:	549	

Vibration

Natural Frequency :	14.16	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.01	mm
Initial Amplitude from Heel Drop	0.050	mm
Damping Required :	3.5	%

FRR: Fire Resistance Rating 18.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is			1.05 KN/mm
Weld Leg Reqd: SP	Single side		6 mm
	Double side		3 mm
Bottom Flange:vwb: VdepQb/Is			1.05 KN/mm
Weld Leg Reqd: SP	Single side		6 mm
	Double side		3 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$510

Calculation Sheet

Job name: Queensgate — M

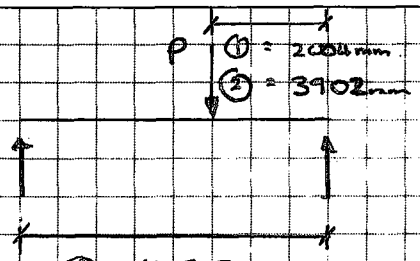
Subject: W/H 25 LVL2 floor beam design.

By: DWS

Job No: 2711265/109

Page No: of

Date: 25-5-04



P = 141kN Grids SE, SC

Secondary beam loads.

$P_{sec} = 53 \times \frac{9.81}{1000} \times 8.2$

$= 4.26kN$

$M_{max} = 423.6 kNm$

$w = \frac{423.6 \times 8}{11.8^2}$

$= 24.34 kN/m$

OR Exterior beams P = 70kN + 183kN from wall bracing.

$P_{sec} = 2.13kN$

$M_{max} = 715.4kNm$.

Grids SA, SG.

$w = 41.1 kN/m$.

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SPACE GASS 9.03 (Win) - BECA CARTER HOLLINGS & FERNER LTD
Job: Unnamed
Designer: Date: 25 May 2004, 5:15 pm Page: 1

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	11.905	0.000	0.000
3	15.000	0.000	0.000
4	26.800	0.000	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	11.905
2	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	11.800

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDR						
2	RFDDDR						
3	FFDDDR						
4	RFDDDR						

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
1	1	1	G	16.667%	0.000 0.000	-4.260 0.000	0.000 0.000
	1	2	G	33.333%	0.000 0.000	-4.260 0.000	0.000 0.000
	1	3	G	50.000%	0.000 0.000	-4.260 0.000	0.000 0.000
	1	4	G	66.667%	0.000 0.000	-4.260 0.000	0.000 0.000
	1	5	G	83.333%	0.000 0.000	-4.260 0.000	0.000 0.000
	2	1	G	16.667%	0.000 0.000	-4.260 0.000	0.000 0.000
	2	2	G	33.333%	0.000 0.000	-4.260 0.000	0.000 0.000
	2	3	G	50.000%	0.000 0.000	-4.260 0.000	0.000 0.000
	2	4	G	66.667%	0.000 0.000	-4.260 0.000	0.000 0.000
	2	5	G	83.333%	0.000 0.000	-4.260 0.000	0.000 0.000
2	1	1	G	8.800	0.000 0.000	-141.000 0.000	0.000 0.000
	2	1	G	7.400	0.000 0.000	-141.000 0.000	0.000 0.000

COMBINATION LOAD CASES

Load case 3:
1.000 * Load case 1:
1.000 * Load case 2:

NODE REACTIONS (kN,kNm)

Load case 1 (Not analysed):

Load case 2 (Not analysed):

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	47.425	0.000	0.000	0.000	0.000
2	0.000	114.875	0.000	0.000	0.000	0.000
3	0.000	63.226	0.000	0.000	0.000	0.000
4	0.000	99.074	0.000	0.000	0.000	0.000
Load	0.000	-324.600	0.000			
Reac	0.000	324.600	0.000			

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum) 11.905 m

Memb 1, Case 3 (Linear):

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	47.425*	0.000	0.000	0.000	0.000
1.191	0.000	47.425*	0.000	0.000	0.000	56.459
1.984	0.000	47.425*	0.000	0.000	0.000	94.101
1.984	0.000	43.165	0.000	0.000	0.000	94.101
2.381	0.000	43.165	0.000	0.000	0.000	111.228
3.572	0.000	43.165	0.000	0.000	0.000	162.616
3.968	0.000	43.165	0.000	0.000	0.000	179.744
3.968	0.000	38.905	0.000	0.000	0.000	179.744
4.762	0.000	38.905	0.000	0.000	0.000	210.623
5.952	0.000	38.905	0.000	0.000	0.000	256.939
5.952	0.000	34.645	0.000	0.000	0.000	256.939
5.953	0.000	34.645	0.000	0.000	0.000	256.939
7.143	0.000	34.645	0.000	0.000	0.000	298.184
7.937	0.000	34.645	0.000	0.000	0.000	325.682
7.937	0.000	30.385	0.000	0.000	0.000	325.682
8.334	0.000	30.385	0.000	0.000	0.000	337.738
8.800	0.000	30.385	0.000	0.000	0.000	351.913*
8.800	0.000	-110.615	0.000	0.000	0.000	351.913*
9.524	0.000	-110.615	0.000	0.000	0.000	271.827
9.921	0.000	-110.615	0.000	0.000	0.000	227.936
9.921	0.000	-114.875#	0.000	0.000	0.000	227.936
10.715	0.000	-114.875#	0.000	0.000	0.000	136.759

11.8 m

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
11.905	0.000	-114.875#	0.000	0.000	0.000	0.000#

Memb 2, Case 3 (Linear):

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	63.226*	0.000	0.000	0.000	0.000#
1.180	0.000	63.226*	0.000	0.000	0.000	74.607
1.967	0.000	63.226*	0.000	0.000	0.000	124.347
1.967	0.000	58.966	0.000	0.000	0.000	124.347
2.360	0.000	58.966	0.000	0.000	0.000	147.539
3.540	0.000	58.966	0.000	0.000	0.000	217.119
3.933	0.000	58.966	0.000	0.000	0.000	240.310
3.933	0.000	54.706	0.000	0.000	0.000	240.310
4.720	0.000	54.706	0.000	0.000	0.000	283.348
5.900	0.000	54.706	0.000	0.000	0.000	347.901
5.900	0.000	50.446	0.000	0.000	0.000	347.901
7.080	0.000	50.446	0.000	0.000	0.000	407.428
7.400	0.000	50.446	0.000	0.000	0.000	423.570*
7.400	0.000	-90.554	0.000	0.000	0.000	423.570*
7.867	0.000	-90.554	0.000	0.000	0.000	381.308
7.867	0.000	-94.814	0.000	0.000	0.000	381.308
8.260	0.000	-94.814	0.000	0.000	0.000	344.019
9.440	0.000	-94.814	0.000	0.000	0.000	232.139
9.833	0.000	-94.814	0.000	0.000	0.000	194.849
9.833	0.000	-99.074#	0.000	0.000	0.000	194.849
10.620	0.000	-99.074#	0.000	0.000	0.000	116.907
11.800	0.000	-99.074#	0.000	0.000	0.000	0.000

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SE,SC 11.9m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78 @150
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	19.86 KN/m 2nd leg + Column.
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	144 KNm
Depend. Bending Strength	613 KNm

Stage II : Concrete Placement

Factored Bending Moment :	838 KNm
Depend. Bending Strength	2025 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	2649 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	890 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	227 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.7 mm
SDL+φs LL+Shrink+Creep:	25.1 mm
Span/DF : Steel Immediate:	934
Span/DF : Top of Concrete:	475
Span/DF : Steel Total:	315

Vibration

Natural Frequency :	5.98 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.046 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating 17.6 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,302

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Licence No.: 990702

USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary beams along SE,SC 11.8m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11800 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 HI-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78 @150
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	24.34 KN/m 2 edge + Column,
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	142 KNm
Depend. Bending Strength	621 KNm

Stage II : Concrete Placement

Factored Bending Moment :	822 KNm
Depend. Bending Strength	2025 KNm

% Composite Action	55 %

Bending Strength

Bending Strength Required :	2695 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	914 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	232 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.3 mm
SDL+φs LL+Shrink+Creep:	25.4 mm
Span/DF : Steel Immediate:	961
Span/DF : Top of Concrete:	465
Span/DF : Steel Total:	313

Vibration

Natural Frequency :	5.88 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.044 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating	17.1 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,283

SPACE GASS 9.03 (Win) - BECA CARTER HOLLINGS & FERNER LTD
Job: Unnamed
Designer: Date: 25 May 2004, 5:40 pm Page: 1

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	11.905	0.000	0.000
3	15.000	0.000	0.000
4	26.800	0.000	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	11.905
2	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	11.800

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDR						
2	RFDDDR						
3	FFDDDR						
4	RFDDDR						

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
1	1	1	G	16.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	2	G	33.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	3	G	50.000%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	4	G	66.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	5	G	83.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	1	G	16.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	2	G	33.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	3	G	50.000%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	4	G	66.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	5	G	83.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
2	1	1	G	8.800	0.000	-70.000	0.000
					0.000	0.000	0.000
	2	1	G	7.400	0.000	-253.000	0.000
					0.000	0.000	0.000

COMBINATION LOAD CASES

Load case 3:

1.000 * Load case 1:
1.000 * Load case 2:

NODE REACTIONS (kN,kNm)

Load case 1 (Not analysed):

Load case 2 (Not analysed):

Load case 3 (Linear):

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	23.582	0.000	0.000	0.000	0.000
2	0.000	57.068	0.000	0.000	0.000	0.000
3	0.000	99.664	0.000	0.000	0.000	0.000
4	0.000	163.986	0.000	0.000	0.000	0.000
Load	0.000	-344.300	0.000			
Reac	0.000	344.300	0.000			

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

Memb 1, Case 3 (Linear):

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	23.582*	0.000	0.000	0.000	0.000#
1.191	0.000	23.582*	0.000	0.000	0.000	28.074
1.984	0.000	23.582*	0.000	0.000	0.000	46.792
1.984	0.000	21.452	0.000	0.000	0.000	46.792
2.381	0.000	21.452	0.000	0.000	0.000	55.304
3.572	0.000	21.452	0.000	0.000	0.000	80.842
3.968	0.000	21.452	0.000	0.000	0.000	89.354
3.968	0.000	19.322	0.000	0.000	0.000	89.354
4.762	0.000	19.322	0.000	0.000	0.000	104.690
5.952	0.000	19.322	0.000	0.000	0.000	127.693
5.952	0.000	17.192	0.000	0.000	0.000	127.693
5.953	0.000	17.192	0.000	0.000	0.000	127.693
7.143	0.000	17.192	0.000	0.000	0.000	148.160
7.937	0.000	17.192	0.000	0.000	0.000	161.806
7.937	0.000	15.062	0.000	0.000	0.000	161.806
8.334	0.000	15.062	0.000	0.000	0.000	167.782
8.800	0.000	15.062	0.000	0.000	0.000	174.809*
8.800	0.000	-54.938	0.000	0.000	0.000	174.809*
9.524	0.000	-54.938	0.000	0.000	0.000	135.034
9.921	0.000	-54.938	0.000	0.000	0.000	113.235
9.921	0.000	-57.068#	0.000	0.000	0.000	113.235
10.715	0.000	-57.068#	0.000	0.000	0.000	67.939
11.905	0.000	-57.068#	0.000	0.000	0.000	0.000

Memb 2, Case 3 (Linear):

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	99.664*	0.000	0.000	0.000	0.000
1.180	0.000	99.664*	0.000	0.000	0.000	117.603
1.967	0.000	99.664*	0.000	0.000	0.000	196.010
1.967	0.000	97.534	0.000	0.000	0.000	196.010
2.360	0.000	97.534	0.000	0.000	0.000	234.369
3.540	0.000	97.534	0.000	0.000	0.000	349.459
3.933	0.000	97.534	0.000	0.000	0.000	387.819
3.933	0.000	95.404	0.000	0.000	0.000	387.819
4.720	0.000	95.404	0.000	0.000	0.000	462.874
5.900	0.000	95.404	0.000	0.000	0.000	575.450
5.900	0.000	93.274	0.000	0.000	0.000	575.450
7.080	0.000	93.274	0.000	0.000	0.000	685.514
7.400	0.000	93.274	0.000	0.000	0.000	715.361*
7.400	0.000	-159.726	0.000	0.000	0.000	715.361*
7.867	0.000	-159.726	0.000	0.000	0.000	640.816
7.867	0.000	-161.856	0.000	0.000	0.000	640.816
8.260	0.000	-161.856	0.000	0.000	0.000	577.159
9.440	0.000	-161.856	0.000	0.000	0.000	386.169
9.833	0.000	-161.856	0.000	0.000	0.000	322.512
9.833	0.000	-163.986#	0.000	0.000	0.000	322.512
10.620	0.000	-163.986#	0.000	0.000	0.000	193.504
11.800	0.000	-163.986#	0.000	0.000	0.000	0.000#

241059

COBENZ 97 1.2 a
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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Primary Beams on SA & SG between S1 & S3
JOB No.: 2711265/109 DESIGN: dws
DATE : 21-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11800	mm	
Overhang:(Beam cc):	100	mm	II Cl 13.4.1.3
Adj. Beam Centres :	8200	mm	
Stud Edge Distance	160	mm	
Beam Size	900WB175		
Beam Yield Stress	300	MPa	
Precamber	0	mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2067	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	78 @150.	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	41.10	KN/m Zudey + Columns.
Super Live Load :	6.50	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
I) Deck Placement:	0.30	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	78	KNm
Depend. Bending Strength	621	KNm

Stage II : Concrete Placement

Factored Bending Moment :	431	KNm
Depend. Bending Strength	2025	KNm

% Composite Action 71 %

Bending Strength

Bending Strength Required :	1989	KNm
Composite Bending Strength :	2919	KNm II Cl 13.4.1.3
Non-Composite Bending Strength:	2025	KNm

Shear Strength

Shear Strength Required:	674	kN
Dependable Shear Strength:	1750	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	181	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.5	mm
SDL+φs LL+Shrink+Creep:	24.8	mm
Span/DF : Steel Immediate:	1816	
Span/DF : Top of Concrete:	477	
Span/DF : Steel Total:	378	

Vibration

Natural Frequency :	5.77	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02	mm
Initial Amplitude from Heel Drop	0.025	mm
Damping Required :	2.7	%

FRR: Fire Resistance Rating

Composite Section Rating:	19.3	min
Non-Composite Section Rating:	13.8	min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$3,474

Calculation Sheet

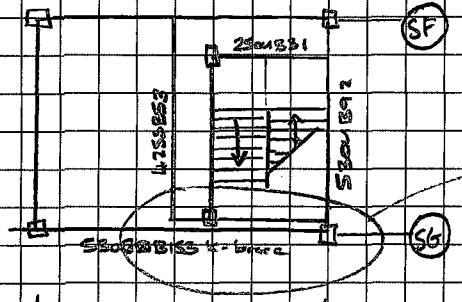
Job name: Queensgate Mall Redevelopment. Job No: 2711265/109

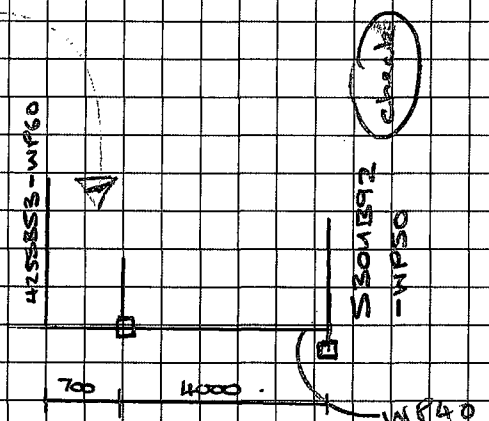
Subject: Zone J LVL2 floor beams. Page No: 1 of 2

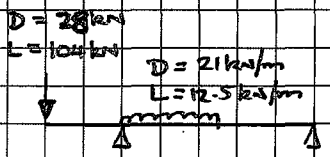
By: Dean Saunders. Date: 31/5/04

Analysis — 1 floor beam at staircase

(S11) (SD) to (S7)







$M^* = 140\ kNm$ $L_e = 4.0m$

$V^* = 102\ kN$

use 410UB54. $\phi M_{bx} = 169\ kNm$ ✓

$\phi V_v = 529\ kN$ ✓

241059

SPACE GASS 9.03 (Win) - BECA CARTER HOLLINGS & FERNER LTD
Job: Unnamed
Designer: Date: 31 May 2004, 6:36 pm Page: 1

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	0.700	0.000	0.000
3	4.700	0.000	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	0.700
2	0.00			Norm	2	3	1	1	FFFFFF	FFFFFF	4.000

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	1	0.000	-28.000	0.000	0.000	0.000	0.000
2	1	0.000	-104.000	0.000	0.000	0.000	0.000
4	1	0.000	84.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
1	2	1	GI	0.000%	50.000%	0.000	-21.000	0.000
						0.000	-21.000	0.000
2	2	1	GI	0.000%	50.000%	0.000	-12.500	0.000
						0.000	-12.500	0.000

COMBINATION LOAD CASES

Load case 3: 1.2G+1.6Q
1.200 * Load case 1: Dead
1.600 * Load case 2: Live

Load case 5: 1.2G+1.6Q without rack loads
1.000 * Load case 3: 1.2G+1.6Q
1.600 * Load case 4:

LOAD CASE TITLES

Load Case	Title
1	Dead
2	Live
3	1.2G+1.6Q
5	1.2G+1.6Q without rack loads

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 3 (Linear): 1.2G+1.6Q

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	0.000	-200.000	0.000	0.000	0.000	0.000
	2	0.000	-200.000	0.000	0.000	0.000	-140.000
2	2	0.000	102.800	0.000	0.000	0.000	-140.000
	3	0.000	12.400	0.000	0.000	0.000	0.000

Load case 5 (Linear): 1.2G+1.6Q without rack loads

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	0.000	-65.600	0.000	0.000	0.000	0.000
	2	0.000	-65.600	0.000	0.000	0.000	-45.920
2	2	0.000	79.280	0.000	0.000	0.000	-45.920
	3	0.000	-11.120	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 3 (Linear): 1.2G+1.6Q

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
2	0.000	302.800	0.000	0.000	0.000	0.000
3	0.000	-12.400	0.000	0.000	0.000	0.000
Load	0.000	-290.400	0.000			
Reac	0.000	290.400	0.000			

Load case 5 (Linear): 1.2G+1.6Q without rack loads

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
2	0.000	144.880	0.000	0.000	0.000	0.000
3	0.000	11.120	0.000	0.000	0.000	0.000
Load	0.000	-156.000	0.000			
Reac	0.000	156.000	0.000			

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, @=Minimum)

Memb 1, Case 3 (Linear): 1.2G+1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	-200.000	0.000	0.000	0.000	0.000*
0.070	0.000	-200.000	0.000	0.000	0.000	-14.000
0.140	0.000	-200.000	0.000	0.000	0.000	-28.000
0.210	0.000	-200.000	0.000	0.000	0.000	-42.000
0.280	0.000	-200.000	0.000	0.000	0.000	-56.000
0.350	0.000	-200.000	0.000	0.000	0.000	-70.000
0.420	0.000	-200.000	0.000	0.000	0.000	-84.000
0.490	0.000	-200.000	0.000	0.000	0.000	-98.000
0.560	0.000	-200.000	0.000	0.000	0.000	-112.000
0.630	0.000	-200.000	0.000	0.000	0.000	-126.000
0.700	0.000	-200.000	0.000	0.000	0.000	-140.000@

Memb 2, Case 3 (Linear): 1.2G+1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	102.800*	0.000	0.000	0.000	-140.000@
0.400	0.000	84.720	0.000	0.000	0.000	-102.496
0.800	0.000	66.640	0.000	0.000	0.000	-72.224
1.200	0.000	48.560	0.000	0.000	0.000	-49.184
1.600	0.000	30.480	0.000	0.000	0.000	-33.376
2.000	0.000	12.400	0.000	0.000	0.000	-24.800
2.400	0.000	12.400@	0.000	0.000	0.000	-19.840
2.800	0.000	12.400	0.000	0.000	0.000	-14.880
3.200	0.000	12.400	0.000	0.000	0.000	-9.920
3.600	0.000	12.400	0.000	0.000	0.000	-4.960
4.000	0.000	12.400	0.000	0.000	0.000	0.000*

Memb 1, Case 5 (Linear): 1.2G+1.6Q without rack loads

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	-65.600	0.000	0.000	0.000	0.000*
0.070	0.000	-65.600	0.000	0.000	0.000	-4.592
0.140	0.000	-65.600	0.000	0.000	0.000	-9.184
0.210	0.000	-65.600	0.000	0.000	0.000	-13.776
0.280	0.000	-65.600	0.000	0.000	0.000	-18.368
0.350	0.000	-65.600	0.000	0.000	0.000	-22.960
0.420	0.000	-65.600	0.000	0.000	0.000	-27.552
0.490	0.000	-65.600	0.000	0.000	0.000	-32.144
0.560	0.000	-65.600	0.000	0.000	0.000	-36.736
0.630	0.000	-65.600	0.000	0.000	0.000	-41.328
0.700	0.000	-65.600	0.000	0.000	0.000	-45.920@

Memb 2, Case 5 (Linear): 1.2G+1.6Q without rack loads

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	79.280*	0.000	0.000	0.000	-45.920@
0.400	0.000	61.200	0.000	0.000	0.000	-17.824
0.800	0.000	43.120	0.000	0.000	0.000	3.040
1.200	0.000	25.040	0.000	0.000	0.000	16.672
1.600	0.000	6.960	0.000	0.000	0.000	23.072*
2.000	0.000	-11.120@	0.000	0.000	0.000	22.240
2.400	0.000	-11.120	0.000	0.000	0.000	17.792
2.800	0.000	-11.120@	0.000	0.000	0.000	13.344
3.200	0.000	-11.120@	0.000	0.000	0.000	8.896
3.600	0.000	-11.120	0.000	0.000	0.000	4.448
4.000	0.000	-11.120@	0.000	0.000	0.000	0.000

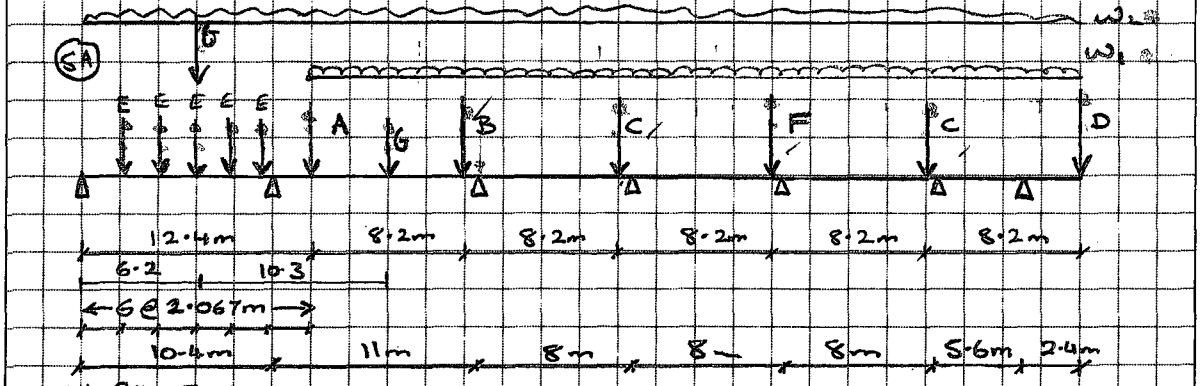
Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: W/H LVL2 Z3 floor beam design
By: DWS

Job No: 2711265/109
Page No: of
Date: 25-5-04

beams along (S1)

Revised 14-6-04 for ones below (S6)



0.5kPa SDL
6.5kPa L.L on beam spans.

C Dead = $8.2\text{m} \times \frac{11.905\text{m}}{2} \times 3.67\text{kPa} + 53 \times \frac{9.81}{1000} \times 8.2 \times 3 + 168 \times \frac{9.81}{1000} \times \frac{11.905}{2}$

= ~~52.0~~ 179 kN + 12.79 kN + 9.81 kN

= 74.6 kN ~~209~~

Live = $8.2 \times \frac{11.905}{2} \times 6.5\text{kPa}$

= 317.3 kN

C D = 74.6kN
L = 317kN

B Dead = ~~52.0~~ 179 kN + 12.79kN + $175 \times \frac{9.81}{1000} \times \frac{11.905}{2}$ + 47kN

= 122kN

Live = $8.2 \times \frac{11.905}{2} \times 6.5\text{kPa} + 53\text{kN}$

= 370.3 kN

B D = 122kN
L = 370kN

F Dead = ~~52.0~~ 179 kN + 12.79kN + 10.22kN + 35kN

= 110kN ~~209~~

Live = 317.3kN + 42kN

= 359.3 kN

F D = 110kN
L = 359kN

D Dead = $(\text{~~52.0~~ 179 kN} + 12.79\text{kN})/2 + 10.22\text{kN} + 21\text{kN}$

= 63.6kN

Live = $\frac{317.3\text{kN}}{2} + 20$

= 178.7 kN

D D = 64kN
L = 179kN

241059

Calculation Sheet

Job name: Queensgate mall redevelopment
Subject: w/h LVL2 EJ floor beam design.
By: DWS

Job No: 2711265/109
Page No: of
Date: 26-5-04

E Dead = $\frac{11.905}{2} \times 2.067 \times 3.67 + 82 \times \frac{9.81}{1000} \times \frac{11.905}{2}$

= 45.2 + 4.79kN

= 500 kN

Live = $\frac{11.905}{2} \times 2.067 \times 6.5$

= 80 kN

E_D = 50kN
L = 80kN.

A Dead = $\frac{11.905}{2} \times \left(\frac{2.067}{2} + \frac{8.2}{2}\right) \times 3.67 + \frac{12.79kN}{2} + 9.81kN$.

= 112.1kN + 6.395 + 9.81

= 128.3kN

Live = $\frac{11.905}{2} \times \left(\frac{2.067}{2} + \frac{8.2}{2}\right) \times 6.5$

= 198.5 kN

A_D = 128kN
L = 199kN.

G Dead = 16.7kN
Live = 6.9kN

G_D = 17kN
L = 7kN

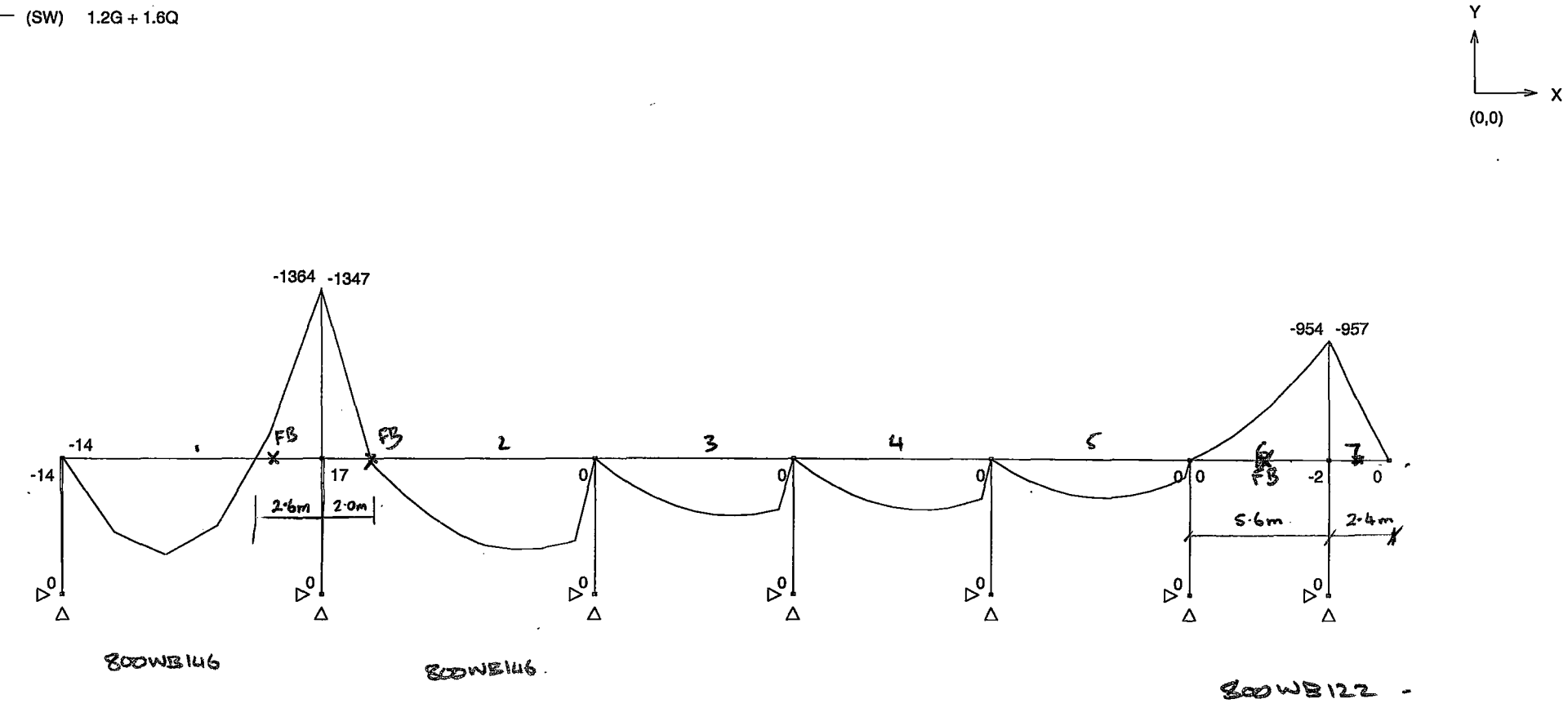
W_1 Dead = 0.5kN/m
Live = 14.4 kN/m ✓

W_1D = 0.5kN/m
L = 14.4 kN/m

Cladding W_2 Dead = 10.0m × 0.35kPa.
= 3.5kN/m ✓

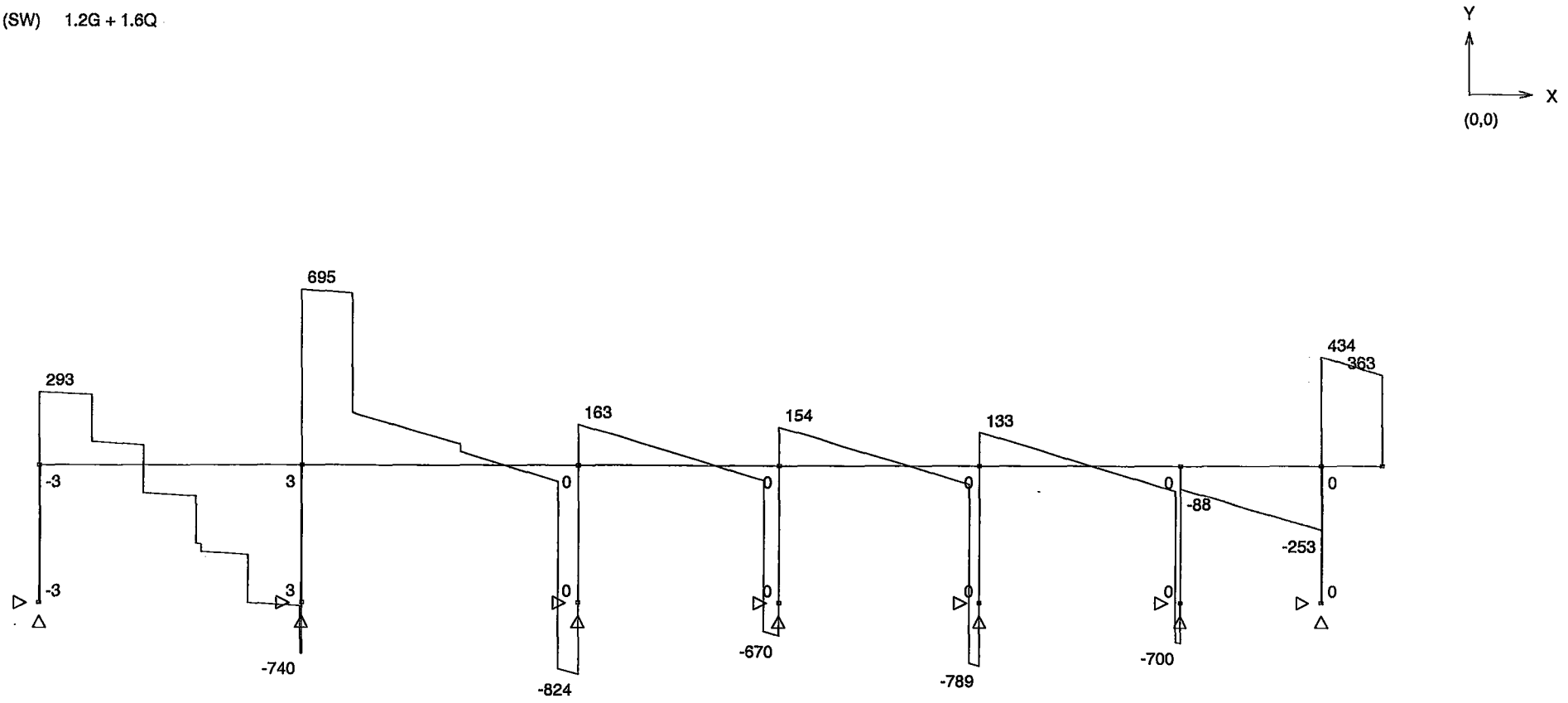
W_2D = 3.5kN/m

Load cases:
■ 4 ——— (SW) 1.2G + 1.6Q



Moment
No general restraint

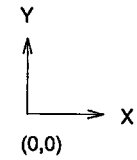
Load cases:
■ 4 —— (SW) 1.2G + 1.6Q

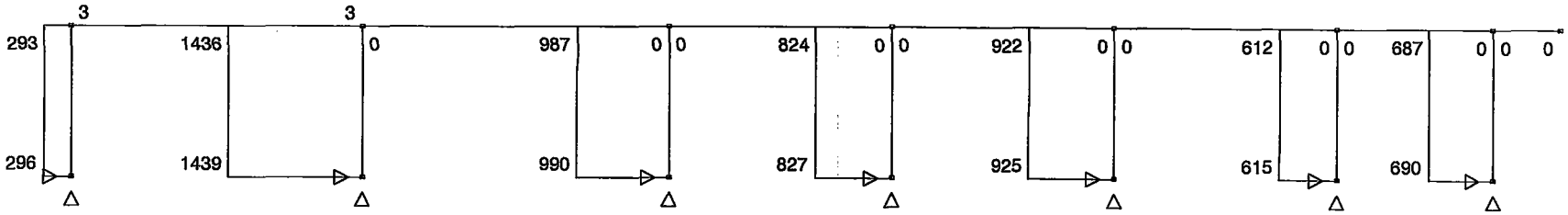


Shear
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: P:\27\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:250, Load: None, Disp: None, Moment: None, Shear: 25, Axial: None, Torsion: None

Load cases:
■ 4 ——— (SW) 1.2G + 1.6Q





281059

Axial
No general restraint

ANALYSIS STATUS REPORT

Job name ..... Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Location ..... P:\27\2711265\SGass

Length units	m	
Section property units	mm	
Material strength units	MPa	
Mass density units	T/m^3	
Temperature units	Celsius	
Force units	kN	
Moment units	kNm	
Mass units	T	
Acceleration units	g's	
Translation units	mm	
Stress units	MPa	
Nodes	15	(2500)
Members	14	(5000)
Restrained nodes	7	(2500)
Nodes with spring restraints	0	(2500)
Section properties	2	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	26	(10000)
Member distributed forces	19	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(200)
Combination load cases	1	(200)
Load cases with titles	4	(200)
Lumped masses	0	(10000)
Spectral load cases	0	(200)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	13	
Total degrees of freedom	55	
Primary load cases	3	(200)
Mass load cases	0	(200)

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.400	0.000	0.000
3	21.400	0.000	0.000
4	29.400	0.000	0.000
5	37.400	0.000	0.000
6	45.400	0.000	0.000
7	51.000	0.000	0.000
8	53.400	0.000	0.000
9	0.000	-5.400	0.000
10	10.400	-5.400	0.000
11	21.400	-5.400	0.000
12	29.400	-5.400	0.000
13	37.400	-5.400	0.000
14	45.400	-5.400	0.000
15	51.000	-5.400	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	2	1	FFFFFF	FFFFFF	10.400
2	0.00			Norm	2	3	2	1	FFFFFF	FFFFFR	11.000
3	0.00			Norm	3	4	2	1	FFFFFR	FFFFFR	8.000
4	0.00			Norm	4	5	2	1	FFFFFR	FFFFFR	8.000
5	0.00			Norm	5	6	2	1	FFFFFR	FFFFFR	8.000
6	0.00			Norm	6	7	2	1	FFFFFR	FFFFFF	5.600
7	0.00			Norm	7	8	2	1	FFFFFF	FFFFFF	2.400
8	0.00			Norm	9	1	1	1	FFFFFF	FFFFFF	5.400
9	0.00			Norm	10	2	1	1	FFFFFF	FFFFFF	5.400
10	0.00			Norm	11	3	1	1	FFFFFF	FFFFFF	5.400
11	0.00			Norm	12	4	1	1	FFFFFF	FFFFFF	5.400
12	0.00			Norm	13	5	1	1	FFFFFF	FFFFFF	5.400
13	0.00			Norm	14	6	1	1	FFFFFF	FFFFFF	5.400
14	0.00			Norm	15	7	1	1	FFFFFF	FFFFFF	5.400

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
9	FFDDDR						
10	FFDDDR						
11	FFDDDR						
12	FFDDDR						
13	FFDDDR						
14	FFDDDR						
15	FFDDDR						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Section Name	Mark	Angle	Type	Flipped	Source
1	250 X 6 SHS	S3	0	Not applicable	No	Manual
2	800 WB 146	S7	0	Not applicable	No	Manual

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.7300E+03	6.8100E+05	5.6200E+07	5.6200E+07	INFINITE	INFINITE	0.00
2	1.8600E+04	1.7200E+06	6.9400E+07	2.0400E+09	INFINITE	INFINITE	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+05	0.25	7.8500E+00	1.1700E-05	

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
1	1	1	G	2.067	0.000	-50.000	0.000
					0.000	0.000	0.000
	1	2	G	4.134	0.000	-50.000	0.000
					0.000	0.000	0.000
	1	3	G	6.201	0.000	-50.000	0.000
					0.000	0.000	0.000
	1	4	G	8.268	0.000	-50.000	0.000
					0.000	0.000	0.000
	1	5	G	10.335	0.000	-50.000	0.000
					0.000	0.000	0.000
	1	6	G	6.400	0.000	-17.000	0.000
					0.000	0.000	0.000
	2	1	G	2.000	0.000	-128.000	0.000
					0.000	0.000	0.000
	2	2	G	6.300	0.000	-17.000	0.000
					0.000	0.000	0.000
	2	3	G	10.200	0.000	-122.000	0.000
					0.000	0.000	0.000
	3	1	G	7.400	0.000	-74.600	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-110.000	0.000
					0.000	0.000	0.000
	5	1	G	7.800	0.000	-74.600	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-64.000	0.000
					0.000	0.000	0.000
2	1	1	G	2.067	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	2	G	4.134	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	3	G	6.201	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	4	G	8.268	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	5	G	10.335	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	6	G	6.400	0.000	-7.000	0.000
					0.000	0.000	0.000
	2	1	G	2.000	0.000	-199.000	0.000
					0.000	0.000	0.000
	2	2	G	6.300	0.000	-7.000	0.000
					0.000	0.000	0.000
	2	3	G	10.200	0.000	-370.000	0.000
					0.000	0.000	0.000
	3	1	G	7.400	0.000	-317.000	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-359.000	0.000
					0.000	0.000	0.000
	5	1	G	7.800	0.000	-317.000	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-179.000	0.000
					0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
1	1	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	2	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	2	2	GI	2.000	11.000	0.000	-0.500	0.000
						0.000	-0.500	0.000
	3	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	3	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	4	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	4	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	5	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	5	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	6	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	6	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	7	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	7	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
2	2	1	GI	2.000	11.000	0.000	-14.400	0.000
						0.000	-14.400	0.000
	3	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000

SPACE GASS 9.03 (Win) - BECA CARTER HOLLINGS & FERNER LTD
Job: P:\27\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Designer: RBW Date: 26 May 2004, 12:18 pm Page: 2
Queensgate
Roof Rafter for Warehouse

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
	4	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	5	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	6	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	7	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000

SELF WEIGHT (g's)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
3	0.000	-1.000	0.000

COMBINATION LOAD CASES

Load case 4: 1.2G + 1.6Q

1.200 * Load case 1: Dead Loads (SDL, Hibond floor, Clad, sw, Cols)
1.600 * Load case 2: Live loads (floor, cols)
1.200 * Load case 3: selfweight

LOAD CASE TITLES

Load Case	Title
1	Dead Loads (SDL, Hibond floor, Clad, sw, Cols) does not include actual beam
2	Live loads (floor, cols)
3	selfweight estimate of size
4	1.2G + 1.6Q

NODE REACTIONS (kN,kNm)

Load case 4 (Linear): 1.2G + 1.6Q

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
9	2.636	295.539	0.000	0.000	0.000	0.000
10	-3.073	1438.781	0.000	0.000	0.000	0.000
11	0.000	990.253	0.000	0.000	0.000	0.000
12	0.000	826.611	0.000	0.000	0.000	0.000
13	0.000	925.322	0.000	0.000	0.000	0.000
14	0.000	615.220	0.000	0.000	0.000	0.000
15	0.438	690.198	0.000	0.000	0.000	0.000
Load Reac	0.000	-5781.923	0.000			
	0.000	5781.923	0.000			

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, @=Minimum)

Memb 1, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	2.636	292.681*	0.000	0.000	0.000	-14.233
1.040	2.636	286.526	0.000	0.000	0.000	286.955
2.067	2.636	280.448	0.000	0.000	0.000	578.096
2.067	2.636	92.448	0.000	0.000	0.000	578.096
2.080	2.636	92.371	0.000	0.000	0.000	579.297
3.120	2.636	86.216	0.000	0.000	0.000	672.163
4.134	2.636	80.215	0.000	0.000	0.000	756.543*
4.134	2.636	-107.785	0.000	0.000	0.000	756.543@
4.160	2.636	-107.939	0.000	0.000	0.000	753.739
5.200	2.636	-114.094	0.000	0.000	0.000	638.282
6.201	2.636	-120.018	0.000	0.000	0.000	521.109
6.201	2.636	-308.018	0.000	0.000	0.000	521.109
6.240	2.636	-308.249	0.000	0.000	0.000	509.092
6.400	2.636	-309.196	0.000	0.000	0.000	459.696
6.400	2.636	-340.796	0.000	0.000	0.000	459.696
7.280	2.636	-346.004	0.000	0.000	0.000	157.504
8.268	2.636	-351.851	0.000	0.000	0.000	-187.236
8.268	2.636	-539.851	0.000	0.000	0.000	-187.236
8.320	2.636	-540.159	0.000	0.000	0.000	-215.317
9.360	2.636	-546.314	0.000	0.000	0.000	-780.283
10.335	2.636	-552.084	0.000	0.000	0.000	-1315.751
10.335	2.636	-740.084	0.000	0.000	0.000	-1315.751
10.400	2.636	-740.469@	0.000	0.000	0.000	-1363.869@

7.78~

Memb 2, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.438	695.453*	0.000	0.000	0.000	-1347.273@
1.100	-0.438	688.943	0.000	0.000	0.000	-585.855
2.000	-0.438	683.617	0.000	0.000	0.000	31.797
2.000	-0.438	211.617	0.000	0.000	0.000	31.797
2.200	-0.438	205.705	0.000	0.000	0.000	73.529
3.300	-0.438	173.191	0.000	0.000	0.000	281.922
4.400	-0.438	140.677	0.000	0.000	0.000	454.550
5.500	-0.438	108.163	0.000	0.000	0.000	591.412
6.300	-0.438	84.516	0.000	0.000	0.000	668.484
6.300	-0.438	52.916	0.000	0.000	0.000	668.484
6.600	-0.438	44.049	0.000	0.000	0.000	683.029
7.700	-0.438	11.535	0.000	0.000	0.000	713.599*
8.800	-0.438	-20.979	0.000	0.000	0.000	708.406
9.900	-0.438	-53.493	0.000	0.000	0.000	667.446
10.200	-0.438	-62.361	0.000	0.000	0.000	650.067
10.200	-0.438	-800.761	0.000	0.000	0.000	650.067
11.000	-0.438	-824.407@	0.000	0.000	0.000	0.000

Memb 3, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.438	162.987*	0.000	0.000	0.000	0.000
0.800	-0.438	139.340	0.000	0.000	0.000	120.931
1.600	-0.438	115.694	0.000	0.000	0.000	222.945
2.400	-0.438	92.047	0.000	0.000	0.000	306.041
3.200	-0.438	68.401	0.000	0.000	0.000	370.220
4.000	-0.438	44.754	0.000	0.000	0.000	415.482
4.800	-0.438	21.107	0.000	0.000	0.000	441.826
5.600	-0.438	-2.539	0.000	0.000	0.000	449.254*
6.400	-0.438	-26.186	0.000	0.000	0.000	437.764
7.200	-0.438	-49.832	0.000	0.000	0.000	407.357
7.400	-0.438	-55.744	0.000	0.000	0.000	396.799
7.400	-0.438	-652.464	0.000	0.000	0.000	396.799
8.000	-0.438	-670.199@	0.000	0.000	0.000	0.000@

Memb 4, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.438	153.553*	0.000	0.000	0.000	0.000
0.800	-0.438	129.907	0.000	0.000	0.000	113.384
1.600	-0.438	106.260	0.000	0.000	0.000	207.851
2.400	-0.438	82.613	0.000	0.000	0.000	283.400
3.200	-0.438	58.967	0.000	0.000	0.000	340.032
4.000	-0.438	35.320	0.000	0.000	0.000	377.747
4.800	-0.438	11.674	0.000	0.000	0.000	396.544*
5.600	-0.438	-11.973	0.000	0.000	0.000	396.424
6.400	-0.438	-35.620	0.000	0.000	0.000	377.387
7.200	-0.438	-59.266	0.000	0.000	0.000	339.433
7.600	-0.438	-71.089	0.000	0.000	0.000	313.362
7.600	-0.438	-777.490	0.000	0.000	0.000	313.362
8.000	-0.438	-789.313@	0.000	0.000	0.000	0.000@

Memb 5, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.438	133.151*	0.000	0.000	0.000	0.000@
0.800	-0.438	109.504	0.000	0.000	0.000	97.062
1.600	-0.438	85.858	0.000	0.000	0.000	175.207
2.400	-0.438	62.211	0.000	0.000	0.000	234.435
3.200	-0.438	38.565	0.000	0.000	0.000	274.745
4.000	-0.438	14.918	0.000	0.000	0.000	296.138
4.800	-0.438	-8.729	0.000	0.000	0.000	298.614*
5.600	-0.438	-32.375	0.000	0.000	0.000	282.172
6.400	-0.438	-56.022	0.000	0.000	0.000	246.813
7.200	-0.438	-79.668	0.000	0.000	0.000	192.537
7.800	-0.438	-97.403	0.000	0.000	0.000	139.416
7.800	-0.438	-694.123	0.000	0.000	0.000	139.416
8.000	-0.438	-700.035@	0.000	0.000	0.000	0.000

Memb 6, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.438	-87.674*	0.000	0.000	0.000	0.000*
0.560	-0.438	-104.226	0.000	0.000	0.000	-53.732
1.120	-0.438	-120.779	0.000	0.000	0.000	-116.733
1.680	-0.438	-137.331	0.000	0.000	0.000	-189.004
2.240	-0.438	-153.884	0.000	0.000	0.000	-270.545
2.800	-0.438	-170.437	0.000	0.000	0.000	-361.354
3.360	-0.438	-186.989	0.000	0.000	0.000	-461.434
3.920	-0.438	-203.542	0.000	0.000	0.000	-570.782
4.480	-0.438	-220.095	0.000	0.000	0.000	-689.401
5.040	-0.438	-236.647	0.000	0.000	0.000	-817.288
5.600	-0.438	-253.200@	0.000	0.000	0.000	-954.445@

Memb 7, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	434.140*	0.000	0.000	0.000	-956.808@
0.240	0.000	427.046	0.000	0.000	0.000	-853.466
0.480	0.000	419.952	0.000	0.000	0.000	-751.826
0.720	0.000	412.858	0.000	0.000	0.000	-651.889
0.960	0.000	405.764	0.000	0.000	0.000	-553.654
1.200	0.000	398.670	0.000	0.000	0.000	-457.122
1.440	0.000	391.576	0.000	0.000	0.000	-362.293
1.680	0.000	384.482	0.000	0.000	0.000	-269.166
1.920	0.000	377.388	0.000	0.000	0.000	-177.741
2.160	0.000	370.294	0.000	0.000	0.000	-88.019
2.400	0.000	363.200@	0.000	0.000	0.000	0.000*

Memb 8, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	295.539*	-2.636	0.000	0.000	0.000	0.000*
0.540	295.254	-2.636	0.000	0.000	0.000	-1.423
1.080	294.968	-2.636	0.000	0.000	0.000	-2.847
1.620	294.682	-2.636	0.000	0.000	0.000	-4.270
2.160	294.396	-2.636	0.000	0.000	0.000	-5.693
2.700	294.110	-2.636	0.000	0.000	0.000	-7.116
3.240	293.824	-2.636	0.000	0.000	0.000	-8.540
3.780	293.538	-2.636	0.000	0.000	0.000	-9.963
4.320	293.253	-2.636	0.000	0.000	0.000	-11.386
4.860	292.967	-2.636	0.000	0.000	0.000	-12.809
5.400	292.681@	-2.636	0.000	0.000	0.000	-14.233@

Memb 9, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	1438.781*	3.073	0.000	0.000	0.000	0.000@
0.540	1438.495	3.073	0.000	0.000	0.000	1.660
1.080	1438.209	3.073	0.000	0.000	0.000	3.319
1.620	1437.923	3.073	0.000	0.000	0.000	4.979
2.160	1437.637	3.073	0.000	0.000	0.000	6.638
2.700	1437.351	3.073	0.000	0.000	0.000	8.298
3.240	1437.065	3.073	0.000	0.000	0.000	9.958
3.780	1436.780	3.073	0.000	0.000	0.000	11.617
4.320	1436.494	3.073	0.000	0.000	0.000	13.277
4.860	1436.208	3.073	0.000	0.000	0.000	14.936
5.400	1435.922@	3.073	0.000	0.000	0.000	16.596*

Memb 10, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	990.253*	0.000	0.000	0.000	0.000	0.000*
0.540	989.967	0.000	0.000	0.000	0.000	0.000
1.080	989.681	0.000	0.000	0.000	0.000	0.000
1.620	989.395	0.000	0.000	0.000	0.000	0.000
2.160	989.109	0.000	0.000	0.000	0.000	0.000
2.700	988.823	0.000	0.000	0.000	0.000	0.000
3.240	988.538	0.000	0.000	0.000	0.000	0.000
3.780	988.252	0.000	0.000	0.000	0.000	0.000
4.320	987.966	0.000	0.000	0.000	0.000	0.000
4.860	987.680	0.000	0.000	0.000	0.000	0.000
5.400	987.394@	0.000	0.000	0.000	0.000	0.000@

241058

Memb 11, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	826.611*	0.000	0.000	0.000	0.000	0.000*
0.540	826.325	0.000	0.000	0.000	0.000	0.000
1.080	826.039	0.000	0.000	0.000	0.000	0.000
1.620	825.753	0.000	0.000	0.000	0.000	0.000
2.160	825.467	0.000	0.000	0.000	0.000	0.000
2.700	825.181	0.000	0.000	0.000	0.000	0.000
3.240	824.896	0.000	0.000	0.000	0.000	0.000
3.780	824.610	0.000	0.000	0.000	0.000	0.000
4.320	824.324	0.000	0.000	0.000	0.000	0.000
4.860	824.038	0.000	0.000	0.000	0.000	0.000
5.400	823.752#	0.000	0.000	0.000	0.000	0.000#

Memb 12, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	925.322*	0.000	0.000	0.000	0.000	0.000#
0.540	925.036	0.000	0.000	0.000	0.000	0.000
1.080	924.751	0.000	0.000	0.000	0.000	0.000
1.620	924.465	0.000	0.000	0.000	0.000	0.000
2.160	924.179	0.000	0.000	0.000	0.000	0.000
2.700	923.893	0.000	0.000	0.000	0.000	0.000
3.240	923.607	0.000	0.000	0.000	0.000	0.000
3.780	923.321	0.000	0.000	0.000	0.000	0.000
4.320	923.036	0.000	0.000	0.000	0.000	0.000
4.860	922.750	0.000	0.000	0.000	0.000	0.000
5.400	922.464#	0.000	0.000	0.000	0.000	0.000*

Memb 13, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	615.220*	0.000	0.000	0.000	0.000	0.000*
0.540	614.934	0.000	0.000	0.000	0.000	0.000
1.080	614.648	0.000	0.000	0.000	0.000	0.000
1.620	614.362	0.000	0.000	0.000	0.000	0.000
2.160	614.076	0.000	0.000	0.000	0.000	0.000
2.700	613.791	0.000	0.000	0.000	0.000	0.000
3.240	613.505	0.000	0.000	0.000	0.000	0.000
3.780	613.219	0.000	0.000	0.000	0.000	0.000
4.320	612.933	0.000	0.000	0.000	0.000	0.000
4.860	612.647	0.000	0.000	0.000	0.000	0.000
5.400	612.361#	0.000	0.000	0.000	0.000	0.000#

Memb 14, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	690.198*	-0.438	0.000	0.000	0.000	0.000*
0.540	689.912	-0.438	0.000	0.000	0.000	-0.236
1.080	689.626	-0.438	0.000	0.000	0.000	-0.473
1.620	689.340	-0.438	0.000	0.000	0.000	-0.709
2.160	689.055	-0.438	0.000	0.000	0.000	-0.945
2.700	688.769	-0.438	0.000	0.000	0.000	-1.182
3.240	688.483	-0.438	0.000	0.000	0.000	-1.418
3.780	688.197	-0.438	0.000	0.000	0.000	-1.654
4.320	687.911	-0.438	0.000	0.000	0.000	-1.891
4.860	687.625	-0.438	0.000	0.000	0.000	-2.127
5.400	687.340#	-0.438	0.000	0.000	0.000	-2.363#

Calculation Sheet

Job name:
Subject:
By:

Job No: 2711265/109
Page No: of
Date: 26-5-04

Revision 14/6/04

(A) to (B)

$M^* = -1364$ kNm -1383

$L_e = 2.6$ m

use 800WB146.

ϕM_b with $\alpha_m = 1.0$.

$= 1412$ kNm

$1364 \leq 1412$kNm ✓ ✓

check Composite action for positive bending section. ✓

check shear

$V^* = 740$kN 742kN

$\alpha_{vm} = \left[2.2 - \frac{1.6 \times 1364}{1412}\right]$

$= 0.64$

$\phi V_v = 1190$ kN $\times \alpha_{vm}$

$= 779$ kN

740kN ≤ 779kN ✓ ✓

800WB146 ok
studs @ 200

(B) to (C)

$M^* = -1347$ kNm 1366kN ✓

$V^* = 824$ kN 698kN or 695kN 703. for shear interaction.

$\phi V_v = 1190$ ✓ ✓

$\alpha_m \phi V_v \approx 780$kN ✓ ✓

use 800WB146 studs @ 200 as above.

Composite ✓ ✓

Place Rybrace 2m from Column ℄ over Continuous support.

241059

COBENZ 97 1.2a
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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Beams along S1 10.4m span
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 21-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10400 mm	
Overhang:(Beam cc):	100 mm	ll Cl 13.4.1.3
Adj. Beam Centres :	11905 mm	
Stud Edge Distance	160 mm	
Beam Size	800WB146	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2087 mm
Slab Thickness :	160 mm
Slab Mass :	0.00 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	52 @200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.00 KPa
Other Super. DL :	56.00 KN/m Equiv loading.
Super Live Load :	0.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	71 KNm
Depend. Bending Strength	540 KNm

Stage II : Concrete Placement

Factored Bending Moment :	141 KNm
Depend. Bending Strength	1534 KNm

% Composite Action 53 %

Bending Strength

Bending Strength Required :	1087 KNm	750KNm
Composite Bending Strength :	2746 KNm	ll Cl 13.4.1.3
Non-Composite Bending Strength:	1534 KNm	

Shear Strength

Shear Strength Required:	418 kN
Dependable Shear Strength:	1257 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	115 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	0.5 mm
SDL+φs LL+Shrink+Creep:	18.8 mm
Span/DF : Steel Immediate:	19926
Span/DF : Top of Concrete:	552
Span/DF : Steel Total:	537

Vibration

Natural Frequency :	3.52 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.06 mm
Initial Amplitude from Heel Drop	0.171 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.0 min
Non-Composite Section Rating:	17.1 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.47 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.47 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$2,536

COBENZ 97 1.2 a

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Beams along S1 11.0m span
JOB No.: 2711265/109 DESIGN: dws
DATE : 21-May-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11000 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	2067 mm	
Stud Edge Distance	160 mm	
Beam Size	800WB146	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2067 mm
Slab Thickness :	160 mm
Slab Mass :	0.00 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	52 @200
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.00 KPa
Other Super. DL :	47.20 KN/m equiv loads .
Super Live Load :	0.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	36 KNm
Depend. Bending Strength	501 KNm

Stage II : Concrete Placement

Factored Bending Moment :	51 KNm
Depend. Bending Strength	1534 KNm

% Composite Action 60 %

Bending Strength

Bending Strength Required :	1030 KNm	
Composite Bending Strength :	2139 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	1534 KNm	

Shear Strength

Shear Strength Required:	374 kN
Dependable Shear Strength:	1257 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	110 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	0.7 mm
SDL+φs LL+Shrink+Creep:	20.2 mm
Span/DF : Steel Immediate:	16840
Span/DF : Top of Concrete:	544
Span/DF : Steel Total:	527

Vibration

Natural Frequency :	3.68 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.041 mm
Damping Required :	2.7 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.5 min
Non-Composite Section Rating:	17.8 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.47 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.47 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$2,671

241059

Calculation Sheet

Job name:
Subject:
By:

Job No: 2711265/109
Page No: of
Date:

SF to SG Span = 5.6m

$M^* = -954$ kNm 1041 kNm
$V^* = 253$ kN 263 kN.

Place midspan flybrace.

$L_e = 2.8$m.

$\phi M_b = 1030$kNm for 800WB122 800WB146 $\phi M_b = 1360$ ✓

$\phi V_v = 1190$kN ✓ $\alpha_m = 0.97$ ✓

use 800WB122 Studs @ 300.
800WB146.

SF to SG Cantilever span = 2.4m

$M^* = -957$ kNm 1045 kN.
$V^* = 434$ kN. 471 kN.

use 800WB122 as above. use 800WB146. as above.

(SC) to SF

8.0m span design as composite section.

$V^* = 790$kN 1004 kN. use 800WB168 $\phi M_b = 2528$kNm
$M^* = 450$kNm 539 kN $\phi V = 1190$kN.

$w_{equiv} = 56.3$ kN/m includes factors.

$M^*_D = 58.2$kNm $M^*_L = 230$kNm } no factors
$w_D = 7.28$kN/m $w_L = 28.75$kN/m }

Shear governs use 530UB82 $\phi V_v = 876$kN.

790 < 876kN ✓

800WB168
530UB82
Studs @ 300.

Calculation Sheet

Job name: Queensgate mall Redevelopment. **Job No:** 2711265
Subject: Zone J LVL2 floor beam checks **Page No:** of
By: DBS **Date:** 14-6-04

Checking loads for open roof area:

SDL = 0.35 kPa.

Column on SG.

G_{SDL} = 4.1m x 10.4m x 0.35kPa.
= 14.9kN.

Column on SF.

G_{SDL} = (4.1 + 2.1) x 10.4 x 0.35
= 22.6kN

150 x 5 SHS between SF & SE

G_{SDL} = (2.1 + 2.1) x 10.4 x 0.35
= 15.3kN.

Column on SE.

G_{SDL} = (2.2 + 2.1) x 10.4 x 0.35
= 15.7kN.

Column on SE at S1

G_{SDL} = (2.3 + 4.1) x 10.4 x 0.35
= 23.2kN.

150 x 5 SHS to SE.

G_{SDL} = (2.3 + 2.2) x 10.4 x 0.35
= 16.4kN.

241052

Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Zone 3 LVL2 floor beam checks.
By: DBS

Job No: 2711265
Page No: of
Date: 14-6-04

Full Column Loads

Location		Gen CLAD	selfweight.	G	Q	Brace
SG	S2 to S3	37.7	8	11	21	183
SG	S1 to S2	50.2.	8	11	21	
SF	S1 to S2	22.6	5			
150 SHS	SE to SF	15.3	2.3			
SE	S1 to S2	15.7	9	36	42	
150 SHS	SE	16.4	2.3.			
SE	S1	23.2	6	12	11	88.0.
SE	S2 to S3	0	9	36	42	
	S1 to S2	0	14	53	52	
SC	S2 to S3	0	14	53	52.	

Use spacegass to determine moments from column & 2nd storey beam loads he convert to equiv udls for Cobenz composite design

SG 1-2.

G = 126.011 kNm. Q = 35.014 kN

udl = $\frac{126.011 \times 8}{11.905^2}$. udl = $\frac{35.014 \times 8}{11.905^2}$.

= 7.11 kN/m = 2.0 kN/m

SG 2-3

G = 542.5 kN Q = 54.8 kN

udl = 36.9 kN/m. udl = 3.2 kN/m.

SC 1-2.

G = 133 kN Q = 86.7 kN

udl = 7.5 kN/m. udl = 4.9 kN/m.

SC & SG 2-3

G = 208.3 kN Q = 135.8 kN.

udl = 12.0 kN/m udl = 7.8 kN/m.

Calculation Sheet

Job name: Queensgate mall — Job No: 2711265

Subject: Zone 3 LVL 2 floor beam checks. — Page No: of

By: DBS — Date: 14-6-04

SF 1-2

G = 82.5 kNm

udl = 4.7 kN/m

SE 1-2

G = 154.1 kNm Q = 70.0 kNm

udl = 8.7 kN/m udl = 4.0 kN/m

150 SHS POST CHECK 2nd beam SE to SF S1-S2

G = 36.1 kNm

udl = 4.3 kN/m

241059

ANALYSIS STATUS REPORT

Job name ..... Queensgate Zone J - Warehouse Level 2 open roof area check
Location ..... P:\27\2711265\SGass

Length units	m
Section property units	mm
Material strength units	MPa
Mass density units	T/m^3
Temperature units	Celsius
Force units	kN
Moment units	kNm
Mass units	T
Acceleration units	g's
Translation units	mm
Stress units	MPa

Nodes	6	(2500)
Members	3	(5000)
Restrained nodes	6	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	43	(10000)
Member distributed forces	0	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	0	(200)
Combination load cases	0	(200)
Load cases with titles	7	(200)
Lumped masses	0	(10000)
Spectral load cases	0	(200)

Static analysis	Y
Dynamic analysis	N
Response analysis	N
Buckling analysis	N
Ill-conditioned	N
Non-linear convergence	Y
Frontwidth	2
Total degrees of freedom	6
Primary load cases	7 (200)
Mass load cases	0 (200)

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	11.905	0.000	0.000
3	20.000	0.000	0.000
4	31.800	0.000	0.000
5	35.000	0.000	0.000
6	43.200	0.000	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	11.905
2	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	11.800
3	0.00			Norm	5	6	1	1	FFFFFF	FFFFFF	8.200

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
1	1	1	G	16.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	2	G	33.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	3	G	50.000%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	4	G	66.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	5	G	83.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	1	6	G	9.900	0.000	-69.200	0.000
					0.000	0.000	0.000
	2	1	G	16.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	2	G	33.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	3	G	50.000%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	4	G	66.667%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	5	G	83.333%	0.000	-2.130	0.000
					0.000	0.000	0.000
	2	6	G	7.900	0.000	-239.700	0.000
					0.000	0.000	0.000
	3	1	G	4.100	0.000	-17.600	0.000
					0.000	0.000	0.000
2	1	1	G	9.900	0.000	-21.000	0.000
					0.000	0.000	0.000
	2	1	G	7.900	0.000	-21.000	0.000
					0.000	0.000	0.000
3	1	1	G	16.667%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	2	G	33.333%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	3	G	50.000%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	4	G	66.667%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	5	G	83.333%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	6	G	9.900	0.000	-36.400	0.000
					0.000	0.000	0.000
6	1	1	G	16.667%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	2	G	33.333%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	3	G	50.000%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	4	G	66.667%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	5	G	83.333%	0.000	-4.260	0.000
					0.000	0.000	0.000
	1	6	G	9.900	0.000	-69.500	0.000
					0.000	0.000	0.000
	1	7	G	4.400	0.000	-18.700	0.000
					0.000	0.000	0.000
7	1	1	G	9.900	0.000	-42.000	0.000
					0.000	0.000	0.000

LOAD CASE TITLES

Load Case	Title
1	G SG 1-2, SG2-3, 150SHS E-F Cladding+roof+2ndary beams
2	Q SG 1-2, SG2-3, 150SHS E-F Roof
3	G SC 1-2, SC&SE2-3 roof+2ndary beams
4	Q SC 1-2, SC&SE2-3 roof
5	G SF 1-2 Cladding+roof+2ndary beams
6	G SE 1-2 Cladding+roof+2ndary beams
7	Q SE 1-2 roof

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): G

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	16.979	0.000	0.000	0.000	0.000
2	0.000	62.871	0.000	0.000	0.000	0.000
3	0.000	84.548	0.000	0.000	0.000	0.000
4	0.000	165.802	0.000	0.000	0.000	0.000
5	0.000	8.800	0.000	0.000	0.000	0.000
6	0.000	8.800	0.000	0.000	0.000	0.000
Load	0.000	-347.800	0.000			
Reac	0.000	347.800	0.000			

Load case 2 (Linear): Q

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	3.537	0.000	0.000	0.000	0.000
2	0.000	17.463	0.000	0.000	0.000	0.000
3	0.000	6.941	0.000	0.000	0.000	0.000
4	0.000	14.059	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-42.000	0.000			
Reac	0.000	42.000	0.000			

Load case 3 (Linear): G

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	21.934	0.000	0.000	0.000	0.000
2	0.000	66.366	0.000	0.000	0.000	0.000
3	0.000	32.794	0.000	0.000	0.000	0.000
4	0.000	55.506	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-176.600	0.000			
Reac	0.000	176.600	0.000			

Load case 4 (Linear): Q

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	8.758	0.000	0.000	0.000	0.000
2	0.000	43.242	0.000	0.000	0.000	0.000
3	0.000	17.186	0.000	0.000	0.000	0.000
4	0.000	34.814	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-104.000	0.000			
Reac	0.000	104.000	0.000			

Load case 5 (Linear): G

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	16.780	0.000	0.000	0.000	0.000
2	0.000	40.920	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-57.700	0.000			
Reac	0.000	57.700	0.000			

Load case 6 (Linear): G

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	34.144	0.000	0.000	0.000	0.000
2	0.000	75.356	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-109.500	0.000			
Reac	0.000	109.500	0.000			

Load case 7 (Linear): Q

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	7.073	0.000	0.000	0.000	0.000
2	0.000	34.927	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000
Load	0.000	-42.000	0.000			
Reac	0.000	42.000	0.000			

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, @=Minimum)

Memb 1, Case 1 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	16.979*	0.000	0.000	0.000	0.000
1.191	0.000	16.979*	0.000	0.000	0.000	20.214
1.984	0.000	16.979*	0.000	0.000	0.000	33.691
1.984	0.000	14.849	0.000	0.000	0.000	33.691
2.381	0.000	14.849	0.000	0.000	0.000	39.583
3.572	0.000	14.849	0.000	0.000	0.000	57.261
3.968	0.000	14.849	0.000	0.000	0.000	63.153
3.968	0.000	12.719	0.000	0.000	0.000	63.153
4.400	0.000	12.719	0.000	0.000	0.000	68.644
4.400	0.000	12.719	0.000	0.000	0.000	68.644
4.762	0.000	12.719	0.000	0.000	0.000	73.249
5.952	0.000	12.719	0.000	0.000	0.000	88.391
5.952	0.000	10.589	0.000	0.000	0.000	88.391
5.953	0.000	10.589	0.000	0.000	0.000	88.391
7.143	0.000	10.589	0.000	0.000	0.000	100.998
7.937	0.000	10.589	0.000	0.000	0.000	109.403
7.937	0.000	8.459	0.000	0.000	0.000	109.403
8.334	0.000	8.459	0.000	0.000	0.000	112.760
9.524	0.000	8.459	0.000	0.000	0.000	122.830
9.900	0.000	8.459	0.000	0.000	0.000	126.011*
9.900	0.000	-60.741	0.000	0.000	0.000	126.011*
9.921	0.000	-60.741	0.000	0.000	0.000	124.748
9.921	0.000	-62.871@	0.000	0.000	0.000	124.748
10.715	0.000	-62.871@	0.000	0.000	0.000	74.847
11.905	0.000	-62.871@	0.000	0.000	0.000	0.000@

Memb 2, Case 1 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	84.548*	0.000	0.000	0.000	0.000
1.180	0.000	84.548*	0.000	0.000	0.000	99.766
1.967	0.000	84.548*	0.000	0.000	0.000	166.281
1.967	0.000	82.418	0.000	0.000	0.000	166.281
2.360	0.000	82.418	0.000	0.000	0.000	198.695
3.540	0.000	82.418	0.000	0.000	0.000	295.948
3.933	0.000	82.418	0.000	0.000	0.000	328.363
3.933	0.000	80.288	0.000	0.000	0.000	328.363
4.720	0.000	80.288	0.000	0.000	0.000	391.526
5.900	0.000	80.288	0.000	0.000	0.000	486.265
5.900	0.000	78.158	0.000	0.000	0.000	486.265
7.080	0.000	78.158	0.000	0.000	0.000	578.492
7.867	0.000	78.158	0.000	0.000	0.000	639.979
7.867	0.000	76.028	0.000	0.000	0.000	639.979
7.900	0.000	76.028	0.000	0.000	0.000	642.510*
7.900	0.000	-163.672	0.000	0.000	0.000	642.510*
8.260	0.000	-163.672	0.000	0.000	0.000	583.588
9.440	0.000	-163.672	0.000	0.000	0.000	390.455
9.833	0.000	-163.672	0.000	0.000	0.000	326.084
9.833	0.000	-165.802@	0.000	0.000	0.000	326.084
10.620	0.000	-165.802@	0.000	0.000	0.000	195.646
11.800	0.000	-165.802@	0.000	0.000	0.000	0.000@

Memb 3, Case 1 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	8.800*	0.000	0.000	0.000	0.000@
0.820	0.000	8.800*	0.000	0.000	0.000	7.216
1.640	0.000	8.800*	0.000	0.000	0.000	14.432
2.460	0.000	8.800*	0.000	0.000	0.000	21.648
3.280	0.000	8.800*	0.000	0.000	0.000	28.864
4.100	0.000	8.800*	0.000	0.000	0.000	36.080*
4.100	0.000	-8.800@	0.000	0.000	0.000	36.080*
4.100	0.000	-8.800@	0.000	0.000	0.000	36.080
4.920	0.000	-8.800@	0.000	0.000	0.000	28.864
5.740	0.000	-8.800@	0.000	0.000	0.000	21.648
6.560	0.000	-8.800@	0.000	0.000	0.000	14.432
7.380	0.000	-8.800@	0.000	0.000	0.000	7.216
8.200	0.000	-8.800@	0.000	0.000	0.000	0.000

Memb 1, Case 2 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	3.537*	0.000	0.000	0.000	0.000@
1.191	0.000	3.537*	0.000	0.000	0.000	4.211
1.984	0.000	3.537*	0.000	0.000	0.000	7.018
1.984	0.000	3.537*	0.000	0.000	0.000	7.018
2.381	0.000	3.537*	0.000	0.000	0.000	8.421
3.572	0.000	3.537*	0.000	0.000	0.000	12.632
3.968	0.000	3.537*	0.000	0.000	0.000	14.035
3.968	0.000	3.537*	0.000	0.000	0.000	14.035
4.400	0.000	3.537*	0.000	0.000	0.000	15.562
4.400	0.000	3.537*	0.000	0.000	0.000	15.562
4.762	0.000	3.537*	0.000	0.000	0.000	16.842
5.952	0.000	3.537*	0.000	0.000	0.000	21.053
5.952	0.000	3.537*	0.000	0.000	0.000	21.053
5.953	0.000	3.537*	0.000	0.000	0.000	21.053
7.143	0.000	3.537*	0.000	0.000	0.000	25.263
7.937	0.000	3.537*	0.000	0.000	0.000	28.070
7.937	0.000	3.537*	0.000	0.000	0.000	28.070
8.334	0.000	3.537*	0.000	0.000	0.000	29.474
9.524	0.000	3.537*	0.000	0.000	0.000	33.684
9.900	0.000	3.537*	0.000	0.000	0.000	35.014*
9.900	0.000	-17.463@	0.000	0.000	0.000	35.014*
9.921	0.000	-17.463@	0.000	0.000	0.000	34.651
9.921	0.000	-17.463@	0.000	0.000	0.000	34.651
10.715	0.000	-17.463@	0.000	0.000	0.000	20.790
11.905	0.000	-17.463@	0.000	0.000	0.000	0.000

Memb 2, Case 2 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	6.941*	0.000	0.000	0.000	0.000
1.180	0.000	6.941*	0.000	0.000	0.000	8.190
1.967	0.000	6.941*	0.000	0.000	0.000	13.650
1.967	0.000	6.941*	0.000	0.000	0.000	13.650
2.360	0.000	6.941*	0.000	0.000	0.000	16.380
3.540	0.000	6.941*	0.000	0.000	0.000	24.570
3.933	0.000	6.941*	0.000	0.000	0.000	27.300
3.933	0.000	6.941*	0.000	0.000	0.000	27.300
4.720	0.000	6.941*	0.000	0.000	0.000	32.760
5.900	0.000	6.941*	0.000	0.000	0.000	40.950
5.900	0.000	6.941*	0.000	0.000	0.000	40.950
7.080	0.000	6.941*	0.000	0.000	0.000	49.140
7.867	0.000	6.941*	0.000	0.000	0.000	54.600
7.867	0.000	6.941*	0.000	0.000	0.000	54.600
7.900	0.000	6.941*	0.000	0.000	0.000	54.831*
7.900	0.000	-14.059@	0.000	0.000	0.000	54.831*
8.260	0.000	-14.059@	0.000	0.000	0.000	49.770
9.440	0.000	-14.059@	0.000	0.000	0.000	33.180
9.833	0.000	-14.059@	0.000	0.000	0.000	27.651
9.833	0.000	-14.059@	0.000	0.000	0.000	27.651
10.620	0.000	-14.059@	0.000	0.000	0.000	16.590
11.800	0.000	-14.059@	0.000	0.000	0.000	0.000@

Memb 3, Case 2 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

Memb 1, Case 3 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	21.934*	0.000	0.000	0.000	0.000
1.191	0.000	21.934*	0.000	0.000	0.000	26.112
1.984	0.000	21.934*	0.000	0.000	0.000	43.521
1.984	0.000	17.674	0.000	0.000	0.000	43.521
2.381	0.000	17.674	0.000	0.000	0.000	50.534
3.572	0.000	17.674	0.000	0.000	0.000	71.575
3.968	0.000	17.674	0.000	0.000	0.000	78.588
3.968	0.000	13.414	0.000	0.000	0.000	78.588
4.400	0.000	13.414	0.000	0.000	0.000	84.379
4.400	0.000	13.414	0.000	0.000	0.000	84.379
4.762	0.000	13.414	0.000	0.000	0.000	89.235
5.952	0.000	13.414	0.000	0.000	0.000	105.204
5.952	0.000	9.154	0.000	0.000	0.000	105.204
5.953	0.000	9.154	0.000	0.000	0.000	105.204
7.143	0.000	9.154	0.000	0.000	0.000	116.102
7.937	0.000	9.154	0.000	0.000	0.000	123.367
7.937	0.000	4.894	0.000	0.000	0.000	123.367
8.334	0.000	4.894	0.000	0.000	0.000	125.309
9.524	0.000	4.894	0.000	0.000	0.000	131.135
9.900	0.000	4.894	0.000	0.000	0.000	132.975*
9.900	0.000	-62.106	0.000	0.000	0.000	132.975*
9.921	0.000	-62.106	0.000	0.000	0.000	131.684
9.921	0.000	-66.366@	0.000	0.000	0.000	131.684
10.715	0.000	-66.366@	0.000	0.000	0.000	79.009
11.905	0.000	-66.366@	0.000	0.000	0.000	0.000@

Memb 2, Case 3 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	32.794*	0.000	0.000	0.000	0.000
1.180	0.000	32.794*	0.000	0.000	0.000	38.697
1.967	0.000	32.794*	0.000	0.000	0.000	64.496
1.967	0.000	28.534	0.000	0.000	0.000	64.496

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
2.360	0.000	28.534	0.000	0.000	0.000	75.719
3.540	0.000	28.534	0.000	0.000	0.000	109.389
3.933	0.000	28.534	0.000	0.000	0.000	120.611
3.933	0.000	24.274	0.000	0.000	0.000	120.611
4.720	0.000	24.274	0.000	0.000	0.000	139.708
5.900	0.000	24.274	0.000	0.000	0.000	168.351
5.900	0.000	20.014	0.000	0.000	0.000	168.351
7.080	0.000	20.014	0.000	0.000	0.000	191.968
7.867	0.000	20.014	0.000	0.000	0.000	207.713
7.867	0.000	15.754	0.000	0.000	0.000	207.713
7.900	0.000	15.754	0.000	0.000	0.000	208.237#
7.900	0.000	-51.246	0.000	0.000	0.000	208.237*
8.260	0.000	-51.246	0.000	0.000	0.000	189.789
9.440	0.000	-51.246	0.000	0.000	0.000	129.319
9.833	0.000	-51.246	0.000	0.000	0.000	109.164
9.833	0.000	-55.506#	0.000	0.000	0.000	109.164
10.620	0.000	-55.506#	0.000	0.000	0.000	65.497
11.800	0.000	-55.506#	0.000	0.000	0.000	0.000#

Memb 3, Case 3 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

Memb 1, Case 4 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	8.758*	0.000	0.000	0.000	0.000#
1.191	0.000	8.758*	0.000	0.000	0.000	10.426
1.984	0.000	8.758*	0.000	0.000	0.000	17.377
1.984	0.000	8.758*	0.000	0.000	0.000	17.377
2.381	0.000	8.758*	0.000	0.000	0.000	20.852
3.572	0.000	8.758*	0.000	0.000	0.000	31.278
3.968	0.000	8.758*	0.000	0.000	0.000	34.753
3.968	0.000	8.758*	0.000	0.000	0.000	34.753
4.400	0.000	8.758*	0.000	0.000	0.000	38.534
4.400	0.000	8.758*	0.000	0.000	0.000	38.534
4.762	0.000	8.758*	0.000	0.000	0.000	41.704
5.952	0.000	8.758*	0.000	0.000	0.000	52.130
5.952	0.000	8.758*	0.000	0.000	0.000	52.130
5.953	0.000	8.758*	0.000	0.000	0.000	52.130
7.143	0.000	8.758*	0.000	0.000	0.000	62.556
7.937	0.000	8.758*	0.000	0.000	0.000	69.507
7.937	0.000	8.758*	0.000	0.000	0.000	69.507
8.334	0.000	8.758*	0.000	0.000	0.000	72.982
9.524	0.000	8.758*	0.000	0.000	0.000	83.408
9.900	0.000	8.758*	0.000	0.000	0.000	86.701*
9.900	0.000	-43.242#	0.000	0.000	0.000	86.701*
9.921	0.000	-43.242#	0.000	0.000	0.000	85.802
9.921	0.000	-43.242#	0.000	0.000	0.000	85.802
10.715	0.000	-43.242#	0.000	0.000	0.000	51.480
11.905	0.000	-43.242#	0.000	0.000	0.000	0.000

Memb 2, Case 4 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	17.186*	0.000	0.000	0.000	0.000#
1.180	0.000	17.186*	0.000	0.000	0.000	20.280
1.967	0.000	17.186*	0.000	0.000	0.000	33.801
1.967	0.000	17.186*	0.000	0.000	0.000	33.801
2.360	0.000	17.186*	0.000	0.000	0.000	40.560
3.540	0.000	17.186*	0.000	0.000	0.000	60.840
3.933	0.000	17.186*	0.000	0.000	0.000	67.599
3.933	0.000	17.186*	0.000	0.000	0.000	67.599
4.720	0.000	17.186*	0.000	0.000	0.000	81.120
5.900	0.000	17.186*	0.000	0.000	0.000	101.400
5.900	0.000	17.186*	0.000	0.000	0.000	101.400
7.080	0.000	17.186*	0.000	0.000	0.000	121.680
7.867	0.000	17.186*	0.000	0.000	0.000	135.201
7.867	0.000	17.186*	0.000	0.000	0.000	135.201
7.900	0.000	17.186*	0.000	0.000	0.000	135.773*
7.900	0.000	-34.814#	0.000	0.000	0.000	135.773*
8.260	0.000	-34.814#	0.000	0.000	0.000	123.240
9.440	0.000	-34.814#	0.000	0.000	0.000	82.160
9.833	0.000	-34.814#	0.000	0.000	0.000	68.468
9.833	0.000	-34.814#	0.000	0.000	0.000	68.468
10.620	0.000	-34.814#	0.000	0.000	0.000	41.080
11.800	0.000	-34.814#	0.000	0.000	0.000	0.000

Memb 3, Case 4 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

Memb 1, Case 5 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	16.780*	0.000	0.000	0.000	0.000
1.191	0.000	16.780*	0.000	0.000	0.000	19.977
1.984	0.000	16.780*	0.000	0.000	0.000	33.296
1.984	0.000	12.520	0.000	0.000	0.000	33.296
2.381	0.000	12.520	0.000	0.000	0.000	38.264
3.572	0.000	12.520	0.000	0.000	0.000	53.169
3.968	0.000	12.520	0.000	0.000	0.000	58.137
3.968	0.000	8.260	0.000	0.000	0.000	58.137
4.400	0.000	8.260	0.000	0.000	0.000	61.703
4.400	0.000	8.260	0.000	0.000	0.000	61.703
4.762	0.000	8.260	0.000	0.000	0.000	64.694
5.952	0.000	8.260	0.000	0.000	0.000	74.527
5.952	0.000	4.000	0.000	0.000	0.000	74.527
5.953	0.000	4.000	0.000	0.000	0.000	74.527
7.143	0.000	4.000	0.000	0.000	0.000	79.290
7.937	0.000	4.000	0.000	0.000	0.000	82.465*
7.937	0.000	-0.260	0.000	0.000	0.000	82.465*
8.334	0.000	-0.260	0.000	0.000	0.000	82.362
9.524	0.000	-0.260	0.000	0.000	0.000	82.053
9.900	0.000	-0.260	0.000	0.000	0.000	81.955
9.900	0.000	-36.660	0.000	0.000	0.000	81.955
9.921	0.000	-36.660	0.000	0.000	0.000	81.193
9.921	0.000	-40.920#	0.000	0.000	0.000	81.193
10.715	0.000	-40.920#	0.000	0.000	0.000	48.715
11.905	0.000	-40.920#	0.000	0.000	0.000	0.000#

Memb 2, Case 5 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
1.180	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
2.360	0.000	0.000	0.000	0.000	0.000	0.000
3.540	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
4.720	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
7.080	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
8.260	0.000	0.000	0.000	0.000	0.000	0.000
9.440	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
10.620	0.000	0.000	0.000	0.000	0.000	0.000
11.800	0.000	0.000	0.000	0.000	0.000	0.000

Memb 3, Case 5 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

Memb 1, Case 6 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	34.144*	0.000	0.000	0.000	0.000#
1.191	0.000	34.144*	0.000	0.000	0.000	40.648
1.984	0.000	34.144*	0.000	0.000	0.000	67.748
1.984	0.000	29.884	0.000	0.000	0.000	67.748
2.381	0.000	29.884	0.000	0.000	0.000	79.606
3.572	0.000	29.884	0.000	0.000	0.000	115.182
3.968	0.000	29.884	0.000	0.000	0.000	127.040
3.968	0.000	25.624	0.000	0.000	0.000	127.040
4.400	0.000	25.624	0.000	0.000	0.000	138.101
4.400	0.000	6.924	0.000	0.000	0.000	138.101
4.762	0.000	6.924	0.000	0.000	0.000	140.608
5.952	0.000	6.924	0.000	0.000	0.000	148.850
5.952	0.000	2.664	0.000	0.000	0.000	148.850
5.953	0.000	2.664	0.000	0.000	0.000	148.850
7.143	0.000	2.664	0.000	0.000	0.000	152.021
7.937	0.000	2.664	0.000	0.000	0.000	154.135*
7.937	0.000	-1.596	0.000	0.000	0.000	154.135*
8.334	0.000	-1.596	0.000	0.000	0.000	153.502
9.524	0.000	-1.596	0.000	0.000	0.000	151.601
9.900	0.000	-1.596	0.000	0.000	0.000	151.001
9.900	0.000	-71.096	0.000	0.000	0.000	151.001
9.921	0.000	-71.096	0.000	0.000	0.000	149.523
9.921	0.000	-75.356#	0.000	0.000	0.000	149.523
10.715	0.000	-75.356#	0.000	0.000	0.000	89.712
11.905	0.000	-75.356#	0.000	0.000	0.000	0.000

Memb 2, Case 6 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
1.180	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
2.360	0.000	0.000	0.000	0.000	0.000	0.000
3.540	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
4.720	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
7.080	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
8.260	0.000	0.000	0.000	0.000	0.000	0.000
9.440	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
10.620	0.000	0.000	0.000	0.000	0.000	0.000
11.800	0.000	0.000	0.000	0.000	0.000	0.000

Memb 3, Case 6 (Linear): G

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

Memb 1, Case 7 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	7.073*	0.000	0.000	0.000	0.000#
1.191	0.000	7.073*	0.000	0.000	0.000	8.421
1.984	0.000	7.073*	0.000	0.000	0.000	14.035
1.984	0.000	7.073*	0.000	0.000	0.000	14.035
2.381	0.000	7.073*	0.000	0.000	0.000	16.842
3.572	0.000	7.073*	0.000	0.000	0.000	25.263
3.968	0.000	7.073*	0.000	0.000	0.000	28.070
3.968	0.000	7.073*	0.000	0.000	0.000	28.070
4.400	0.000	7.073*	0.000	0.000	0.000	31.123
4.400	0.000	7.073*	0.000	0.000	0.000	31.123
4.762	0.000	7.073*	0.000	0.000	0.000	33.684
5.952	0.000	7.073*	0.000	0.000	0.000	42.105
5.952	0.000	7.073*	0.000	0.000	0.000	42.105
5.953	0.000	7.073*	0.000	0.000	0.000	42.105
7.143	0.000	7.073*	0.000	0.000	0.000	50.526
7.937	0.000	7.073*	0.000	0.000	0.000	56.140
7.937	0.000	7.073*	0.000	0.000	0.000	56.140
8.334	0.000	7.073*	0.000	0.000	0.000	58.947
9.524	0.000	7.073*	0.000	0.000	0.000	67.368
9.900	0.000	7.073*	0.000	0.000	0.000	70.028*
9.900	0.000	-34.927#	0.000	0.000	0.000	70.028*
9.921	0.000	-34.927#	0.000	0.000	0.000	69.301
9.921	0.000	-34.927#	0.000	0.000	0.000	69.301
10.715	0.000	-34.927#	0.000	0.000	0.000	41.580
11.905	0.000	-34.927#	0.000	0.000	0.000	0.000

Memb 2, Case 7 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
1.180	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
1.967	0.000	0.000	0.000	0.000	0.000	0.000
2.360	0.000	0.000	0.000	0.000	0.000	0.000
3.540	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
3.933	0.000	0.000	0.000	0.000	0.000	0.000
4.720	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
5.900	0.000	0.000	0.000	0.000	0.000	0.000
7.080	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.867	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
7.900	0.000	0.000	0.000	0.000	0.000	0.000
8.260	0.000	0.000	0.000	0.000	0.000	0.000
9.440	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
9.833	0.000	0.000	0.000	0.000	0.000	0.000
10.620	0.000	0.000	0.000	0.000	0.000	0.000
11.800	0.000	0.000	0.000	0.000	0.000	0.000

Memb 3, Case 7 (Linear): Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	0.000	0.000	0.000	0.000	0.000	0.000
1.640	0.000	0.000	0.000	0.000	0.000	0.000
2.460	0.000	0.000	0.000	0.000	0.000	0.000
3.280	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.100	0.000	0.000	0.000	0.000	0.000	0.000
4.920	0.000	0.000	0.000	0.000	0.000	0.000
5.740	0.000	0.000	0.000	0.000	0.000	0.000
6.560	0.000	0.000	0.000	0.000	0.000	0.000
7.380	0.000	0.000	0.000	0.000	0.000	0.000
8.200	0.000	0.000	0.000	0.000	0.000	0.000

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	8200 mm	
Stud Edge Distance	160 mm	
Beam Size	900WB175	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2000 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	9.11 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	2.00 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	80 KNm
Depend. Bending Strength	613 KNm

Stage II : Concrete Placement

Factored Bending Moment :	439 KNm
Depend. Bending Strength	2025 KNm

% Composite Action 71 %

Bending Strength

Bending Strength Required :	1401 KNm	
Composite Bending Strength :	2920 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	2025 KNm	

Shear Strength

Shear Strength Required:	471 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	123 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.7 mm
SDL+φs LL+Shrink+Creep:	16.9 mm
Span/DF : Steel Immediate:	1771
Span/DF : Top of Concrete:	705
Span/DF : Steel Total:	504

Vibration

Natural Frequency :	8.09 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.033 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating

Composite Section Rating:	24.2 min
Non-Composite Section Rating:	20.9 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$3,502

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : SG 2-3
JOB No.: 2711265/109 DESIGN: dbs
DATE : 14-Jun-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11800 mm	
Overhang:(Beam cc):	100 mm	!! Cl 13.4.1.3
Adj. Beam Centres :	8200 mm	
Stud Edge Distance	160 mm	
Beam Size	900WB175	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2000 mm
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	36.90 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	3.20 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	78 KNm	
Depend. Bending Strength	621 KNm	

Stage II : Concrete Placement

Factored Bending Moment :	431 KNm	
Depend. Bending Strength	2025 KNm	
% Composite Action	71 %	

Bending Strength

Bending Strength Required :	1990 KNm	
Composite Bending Strength :	2919 KNm	!! Cl 13.4.1.3
Non-Composite Bending Strength:	2025 KNm	

Shear Strength

Shear Strength Required:	674 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	179 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.5 mm
SDL+φs LL+Shrink+Creep:	24.4 mm
Span/DF : Steel Immediate:	1820
Span/DF : Top of Concrete:	483
Span/DF : Steel Total:	382

Vibration

Natural Frequency :	5.98 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.026 mm
Damping Required :	2.7 %

FRR: Fire Resistance Rating

Composite Section Rating:	19.6 min
Non-Composite Section Rating:	14.3 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$3,474

241059

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : SC&SE 2-3
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 14-Jun-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11800 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	12.00 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	7.80 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	142 KNm
Depend. Bending Strength	621 KNm

Stage II : Concrete Placement

Factored Bending Moment :	820 KNm
Depend. Bending Strength	2025 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	2653 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	899 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	223 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.2 mm
SDL+φs LL+Shrink+Creep:	24.2 mm
Span/DF : Steel immediate:	963
Span/DF : Top of Concrete:	487
Span/DF : Steel Total:	323

Vibration

Natural Frequency :	6.52 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.048 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating 18.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,283

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	7.50 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	4.90 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	144 KNm
Depend. Bending Strength	613 KNm

Stage II : Concrete Placement

Factored Bending Moment :	836 KNm
Depend. Bending Strength	2025 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	2523 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	848 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	213 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.7 mm
SDL+φs LL+Shrink+Creep:	23.2 mm
Span/DF : Steel Immediate:	937
Span/DF : Top of Concrete:	513
Span/DF : Steel Total:	332

Vibration

Natural Frequency :	6.69 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.050 mm
Damping Required :	3.0 %

FRR: Fire Resistance Rating 19.2 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,302

241059

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	1.50 KPa
Other Super. DL :	4.70 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	144 KNm
Depend. Bending Strength	613 KNm

Stage II : Concrete Placement

Factored Bending Moment :	836 KNm
Depend. Bending Strength	2025 KNm

% Composite Action	55 %

Bending Strength

Bending Strength Required :	2499 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	840 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	214 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.7 mm
SDL+φs LL+Shrink+Creep:	23.4 mm
Span/DF : Steel Immediate:	937
Span/DF : Top of Concrete:	509
Span/DF : Steel Total:	330

Vibration

Natural Frequency :	6.35 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.048 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating	18.8 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,302

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : SE 1-2
JOB No.: 2711265/109 **DESIGN:** dbs
DATE : 14-Jun-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	11905 mm
Adj. Beam Centres :	8200 mm
	8200 mm
Beam Size	900WB175
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	78
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	1.50 KPa Superimposed
Other Super. DL :	8.70 KN/m
Super Live Load :	6.50 KPa
Other Super. LL :	4.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	144 KNm
Depend. Bending Strength	613 KNm

Stage II : Concrete Placement

Factored Bending Moment :	836 KNm
Depend. Bending Strength	2025 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	2698 KNm
Dependable Bending Strength :	2980 KNm

Shear Strength

Shear Strength Required:	906 kN
Dependable Shear Strength:	1750 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	229 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.7 mm
SDL+φs LL+Shrink+Creep:	25.3 mm
Span/DF : Steel Immediate:	937
Span/DF : Top of Concrete:	471
Span/DF : Steel Total:	313

Vibration

Natural Frequency :	6.13 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.02 mm
Initial Amplitude from Heel Drop	0.047 mm
Damping Required :	2.9 %

FRR: Fire Resistance Rating 17.7 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.73 KN/mm
Weld Leg Reqd: SP	Single side	10 mm
	Double side	5 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$2,302

241059

COBENZ 97 1.2 a
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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : 2ndary with SHS post over SE-SF, S1-S2
JOB No.: 2711265/109 **DESIGN:** dbs
DATE : 14-Jun-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY
Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	8200 mm
Adj. Beam Centres :	2000 mm
	2000 mm
Beam Size	425SB53
Beam Yield Stress	300 MPa
Precamber	0 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2000 mm
Deck Span Type :	Triple
Slab Thickness :	160 mm
Slab Mass :	3.17 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	27
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	4.30 KN/m
Super Live Load :	14.54 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	2.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	18 KNm
Depend. Bending Strength	95 KNm

Stage II : Concrete Placement

Factored Bending Moment :	97 KNm
Depend. Bending Strength	310 KNm

% Composite Action 62 %

Bending Strength

Bending Strength Required :	517 KNm
Dependable Bending Strength :	529 KNm

Shear Strength

Shear Strength Required:	252 kN
Dependable Shear Strength:	413 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	227 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	9.4 mm
SDL+φs LL+Shrink+Creep:	23.3 mm
Span/DF : Steel Immediate:	872
Span/DF : Top of Concrete:	351
Span/DF : Steel Total:	250

Vibration

Natural Frequency :	7.15 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.053 mm
Damping Required :	3.0 %

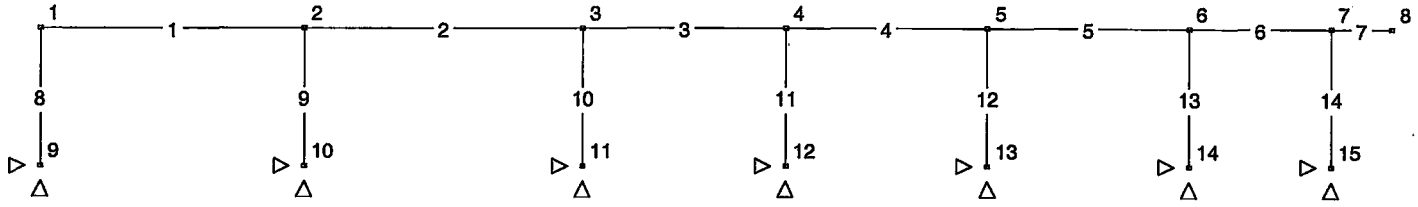
FRR: Fire Resistance Rating 15.2 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm
Bottom Flange:vwb: VdepQb/Is		0.95 KN/mm
Weld Leg Reqd: SP	Single side	6 mm
	Double side	3 mm

Comparative Cost:

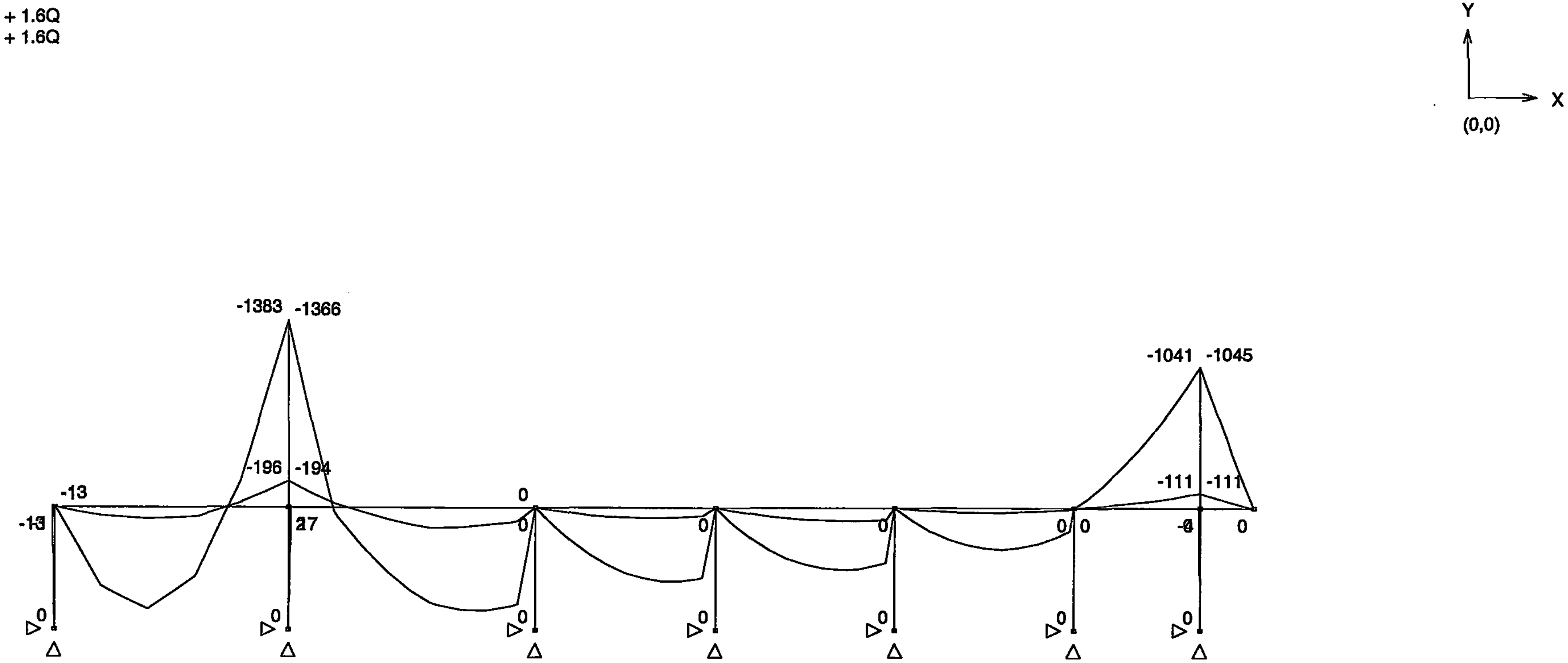
Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$510



241059

Nos
No general restraint

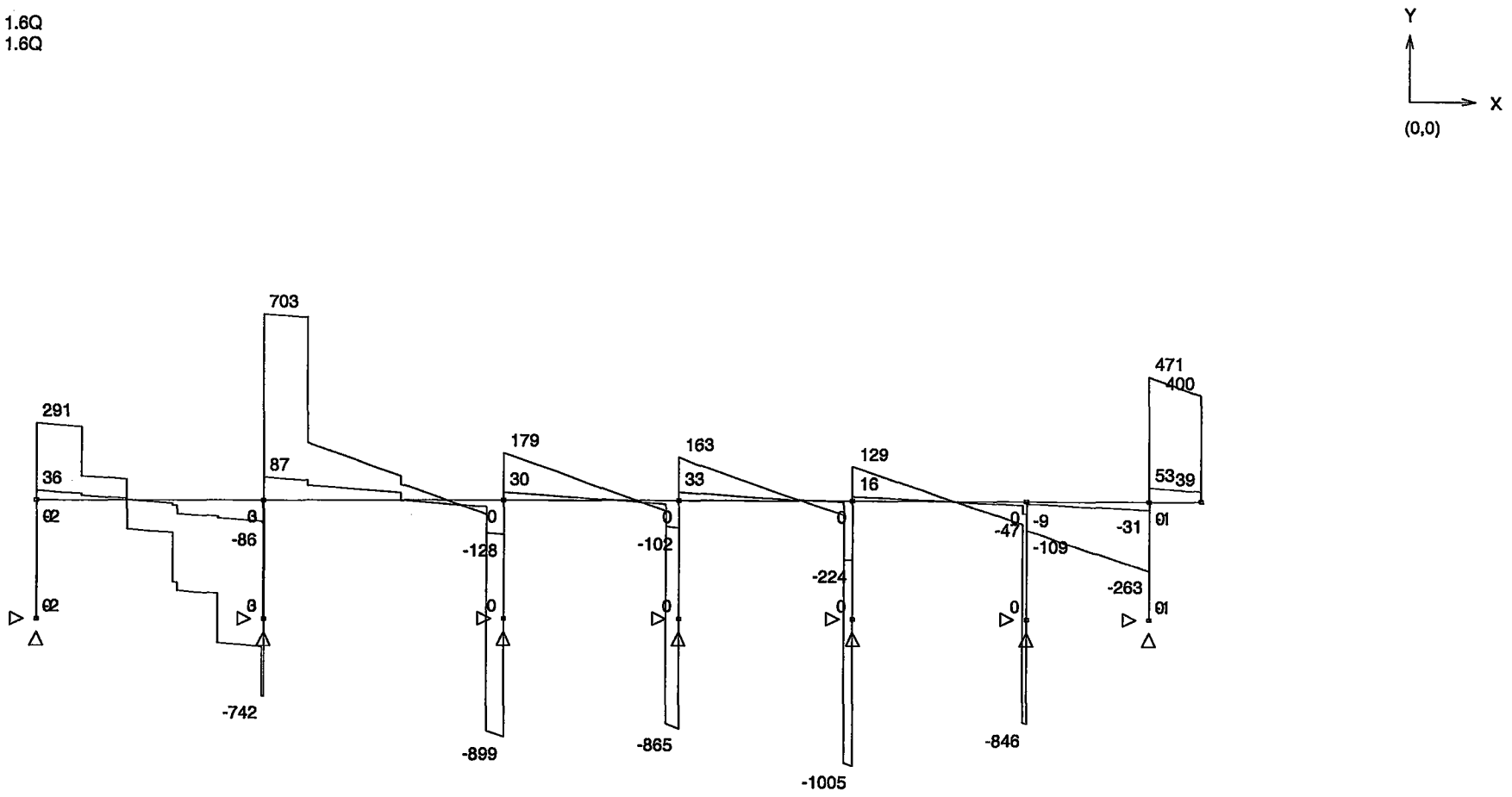
All combination load cases:
4 (SW) 1.2G + 1.6Q
7 (SW) 1.2G + 1.6Q



bending
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: P:\27\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: None, Disp: None, Moment: 50, Shear: None, Axial: None, Torsion: None

All combination load cases:
■ 4 —— (SW) 1.2G + 1.6Q
■ 7 —— (SW) 1.2G + 1.6Q



241059

shear
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: P:\27\...\Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: None, Disp: None, Moment: None, Shear: 25, Axial: None, Torsion: None

ANALYSIS STATUS REPORT

Job name ..... Queensgate Zone J - Warehouse Level 2 beams @ GL S1 Rev 02
Location ..... P:\27\2711265\SGass

Length units	m	
Section property units	mm	
Material strength units	MPa	
Mass density units	T/m^3	
Temperature units	Celsius	
Force units	kN	
Moment units	kNm	
Mass units	T	
Acceleration units	g's	
Translation units	mm	
Stress units	MPa	
Nodes	15	(2500)
Members	14	(5000)
Restrained nodes	7	(2500)
Nodes with spring restraints	0	(2500)
Section properties	2	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	40	(10000)
Member distributed forces	20	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(200)
Combination load cases	2	(200)
Load cases with titles	7	(200)
Lumped masses	0	(10000)
Spectral load cases	0	(200)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	13	
Total degrees of freedom	55	
Primary load cases	5	(200)
Mass load cases	0	(200)

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	10.400	0.000	0.000
3	21.400	0.000	0.000
4	29.400	0.000	0.000
5	37.400	0.000	0.000
6	45.400	0.000	0.000
7	51.000	0.000	0.000
8	53.400	0.000	0.000
9	0.000	-5.400	0.000
10	10.400	-5.400	0.000
11	21.400	-5.400	0.000
12	29.400	-5.400	0.000
13	37.400	-5.400	0.000
14	45.400	-5.400	0.000
15	51.000	-5.400	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	2	1	FFFFFF	FFFFFF	10.400
2	0.00			Norm	2	3	2	1	FFFFFF	FFFFFR	11.000
3	0.00			Norm	3	4	2	1	FFFFFR	FFFFFR	8.000
4	0.00			Norm	4	5	2	1	FFFFFR	FFFFFR	8.000
5	0.00			Norm	5	6	2	1	FFFFFR	FFFFFR	8.000
6	0.00			Norm	6	7	2	1	FFFFFR	FFFFFF	5.600
7	0.00			Norm	7	8	2	1	FFFFFF	FFFFFF	2.400
8	0.00			Norm	9	1	1	1	FFFFFF	FFFFFF	5.400
9	0.00			Norm	10	2	1	1	FFFFFF	FFFFFF	5.400
10	0.00			Norm	11	3	1	1	FFFFFF	FFFFFF	5.400
11	0.00			Norm	12	4	1	1	FFFFFF	FFFFFF	5.400
12	0.00			Norm	13	5	1	1	FFFFFF	FFFFFF	5.400
13	0.00			Norm	14	6	1	1	FFFFFF	FFFFFF	5.400
14	0.00			Norm	15	7	1	1	FFFFFF	FFFFFF	5.400

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
9	FFDDDR						
10	FFDDDR						
11	FFDDDR						
12	FFDDDR						
13	FFDDDR						
14	FFDDDR						
15	FFDDDR						

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
1	1	1	G	2.067	0.000	-4.800	0.000
					0.000	0.000	0.000
	1	2	G	4.134	0.000	-4.800	0.000
					0.000	0.000	0.000
	1	3	G	6.201	0.000	-4.800	0.000
					0.000	0.000	0.000
	1	4	G	8.268	0.000	-4.800	0.000
					0.000	0.000	0.000
	1	5	G	10.335	0.000	-4.800	0.000
					0.000	0.000	0.000
	1	6	G	6.400	0.000	-17.000	0.000
					0.000	0.000	0.000
	2	1	G	2.000	0.000	-16.000	0.000
					0.000	0.000	0.000
	2	2	G	6.300	0.000	-17.000	0.000
					0.000	0.000	0.000
	2	3	G	10.200	0.000	-70.000	0.000
					0.000	0.000	0.000
	3	1	G	7.400	0.000	-71.000	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-157.400	0.000
					0.000	0.000	0.000
	5	1	G	7.800	0.000	-27.000	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-27.200	0.000
					0.000	0.000	0.000
2	1	6	G	6.400	0.000	-7.000	0.000
					0.000	0.000	0.000
	2	2	G	6.300	0.000	-7.000	0.000
					0.000	0.000	0.000
	2	3	G	10.200	0.000	-9.000	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-18.000	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-4.000	0.000
					0.000	0.000	0.000
5	1	1	G	2.067	0.000	-45.200	0.000
					0.000	0.000	0.000
	1	2	G	4.134	0.000	-45.200	0.000
					0.000	0.000	0.000
	1	3	G	6.201	0.000	-45.200	0.000
					0.000	0.000	0.000
	1	4	G	8.268	0.000	-45.200	0.000
					0.000	0.000	0.000
	1	5	G	10.335	0.000	-45.200	0.000
					0.000	0.000	0.000
	2	1	G	2.000	0.000	-112.000	0.000
					0.000	0.000	0.000
	2	2	G	10.200	0.000	-179.000	0.000
					0.000	0.000	0.000
	3	1	G	7.400	0.000	-179.000	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-179.000	0.000
					0.000	0.000	0.000
	5	1	G	7.800	0.000	-179.000	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-90.000	0.000
					0.000	0.000	0.000
6	1	1	G	2.067	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	2	G	4.134	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	3	G	6.201	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	4	G	8.268	0.000	-80.000	0.000
					0.000	0.000	0.000
	1	5	G	10.335	0.000	-80.000	0.000
					0.000	0.000	0.000
	2	1	G	2.000	0.000	-199.000	0.000
					0.000	0.000	0.000
	2	3	G	10.200	0.000	-317.000	0.000
					0.000	0.000	0.000
	3	1	G	7.400	0.000	-317.000	0.000
					0.000	0.000	0.000
	4	1	G	7.600	0.000	-317.000	0.000
					0.000	0.000	0.000
	5	1	G	7.800	0.000	-317.000	0.000
					0.000	0.000	0.000
	7	1	G	100.000%	0.000	-158.000	0.000
					0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
1	1	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	2	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	3	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	4	1	GI	0.000%	50.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
	4	3	GI	50.000%	100.000%	0.000	-1.800	0.000
						0.000	-1.800	0.000
	5	1	GI	0.000%	100.000%	0.000	-1.800	0.000
						0.000	-1.800	0.000
	6	1	GI	0.000%	100.000%	0.000	-1.800	0.000
						0.000	-1.800	0.000
	7	1	GI	0.000%	100.000%	0.000	-3.500	0.000
						0.000	-3.500	0.000
5	2	2	GI	2.000	11.000	0.000	-0.500	0.000
						0.000	-0.500	0.000

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
	3	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	4	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	5	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	6	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
	7	2	GI	0.000%	100.000%	0.000	-0.500	0.000
						0.000	-0.500	0.000
6	2	1	GI	2.000	11.000	0.000	-14.400	0.000
						0.000	-14.400	0.000
	3	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	4	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	5	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	6	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000
	7	1	GI	0.000%	100.000%	0.000	-14.400	0.000
						0.000	-14.400	0.000

SELF WEIGHT (g's)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
3	0.000	-1.000	0.000

COMBINATION LOAD CASES

Load case 4: 1.2G + 1.6Q

1.200 * Load case 1: Dead Loads (Clad, roof, sw, Cols)
1.600 * Load case 2: Live loads (roof, cols)
1.200 * Load case 3: selfweight

Load case 7: 1.2G + 1.6Q

1.200 * Load case 1: Dead Loads (Clad, roof, sw, Cols)
1.600 * Load case 2: Live loads (roof, cols)
1.200 * Load case 3: selfweight
1.200 * Load case 5: G
1.600 * Load case 6: Q

LOAD CASE TITLES

Load Case	Title
1	Dead Loads (Clad, roof, sw, Cols) does not include actual beam
2	Live loads (roof, cols)
3	selfweight estimate of size
4	1.2G + 1.6Q note inclu hibond loads
5	G hibond loads to remove for cobenz
6	Q hibond loads to remove for cobenz
7	1.2G + 1.6Q all inclu

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, @=Minimum)

Memb 1, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.250	35.873*	0.000	0.000	0.000	-1.348
1.040	0.250	29.718	0.000	0.000	0.000	32.759
2.067	0.250	23.640	0.000	0.000	0.000	60.158
2.067	0.250	17.880	0.000	0.000	0.000	60.158
2.080	0.250	17.803	0.000	0.000	0.000	60.390
3.120	0.250	11.648	0.000	0.000	0.000	75.705
4.134	0.250	5.647	0.000	0.000	0.000	84.473*
4.134	0.250	-0.113	0.000	0.000	0.000	84.473*
4.160	0.250	-0.267	0.000	0.000	0.000	84.468
5.200	0.250	-6.422	0.000	0.000	0.000	80.990
6.201	0.250	-12.346	0.000	0.000	0.000	71.596
6.201	0.250	-18.106	0.000	0.000	0.000	71.596
6.240	0.250	-18.337	0.000	0.000	0.000	70.886
6.400	0.250	-19.284	0.000	0.000	0.000	67.876
6.400	0.250	-50.884	0.000	0.000	0.000	67.876
7.280	0.250	-56.092	0.000	0.000	0.000	20.807
8.268	0.250	-61.939	0.000	0.000	0.000	-37.501
8.268	0.250	-67.699	0.000	0.000	0.000	-37.501
8.320	0.250	-68.007	0.000	0.000	0.000	-41.029
9.360	0.250	-74.162	0.000	0.000	0.000	-114.957
10.335	0.250	-79.932	0.000	0.000	0.000	-190.078
10.335	0.250	-85.692	0.000	0.000	0.000	-190.078
10.400	0.250	-86.077@	0.000	0.000	0.000	-195.661@

Memb 2, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.030	86.568*	0.000	0.000	0.000	-194.152@
1.100	-0.030	80.058	0.000	0.000	0.000	-102.508
2.000	-0.030	74.731	0.000	0.000	0.000	-32.853
2.000	-0.030	55.531	0.000	0.000	0.000	-32.853
2.200	-0.030	54.348	0.000	0.000	0.000	-21.865
3.300	-0.030	47.838	0.000	0.000	0.000	34.337
4.400	-0.030	41.327	0.000	0.000	0.000	83.378
5.500	-0.030	34.817	0.000	0.000	0.000	125.258
6.300	-0.030	30.083	0.000	0.000	0.000	151.218*
6.300	-0.030	-1.517	0.000	0.000	0.000	151.218*
6.600	-0.030	-3.293	0.000	0.000	0.000	150.496
7.700	-0.030	-9.803	0.000	0.000	0.000	143.294
8.800	-0.030	-16.313	0.000	0.000	0.000	128.930
9.900	-0.030	-22.823	0.000	0.000	0.000	107.406
10.200	-0.030	-24.598	0.000	0.000	0.000	100.293
10.200	-0.030	-122.998	0.000	0.000	0.000	100.293
11.000	-0.030	-127.733@	0.000	0.000	0.000	0.000

Memb 3, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.030	30.063*	0.000	0.000	0.000	0.000
0.800	-0.030	25.328	0.000	0.000	0.000	22.157
1.600	-0.030	20.594	0.000	0.000	0.000	40.525
2.400	-0.030	15.859	0.000	0.000	0.000	55.107
3.200	-0.030	11.125	0.000	0.000	0.000	65.900
4.000	-0.030	6.390	0.000	0.000	0.000	72.906
4.800	-0.030	1.655	0.000	0.000	0.000	76.124*
5.600	-0.030	-3.079	0.000	0.000	0.000	75.555
6.400	-0.030	-7.814	0.000	0.000	0.000	71.197
7.200	-0.030	-12.548	0.000	0.000	0.000	63.053
7.400	-0.030	-13.732	0.000	0.000	0.000	60.424
7.400	-0.030	-98.932	0.000	0.000	0.000	60.424
8.000	-0.030	-102.483@	0.000	0.000	0.000	0.000@

Memb 4, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.030	32.517*	0.000	0.000	0.000	0.000
0.800	-0.030	27.782	0.000	0.000	0.000	24.120
1.600	-0.030	23.048	0.000	0.000	0.000	44.452
2.400	-0.030	18.313	0.000	0.000	0.000	60.996
3.200	-0.030	13.579	0.000	0.000	0.000	73.753
4.000	-0.030	8.844	0.000	0.000	0.000	82.722
4.800	-0.030	5.741	0.000	0.000	0.000	88.556
5.600	-0.030	2.639	0.000	0.000	0.000	91.908
6.400	-0.030	-0.464	0.000	0.000	0.000	92.779*
7.200	-0.030	-3.566	0.000	0.000	0.000	91.167
7.600	-0.030	-5.118	0.000	0.000	0.000	89.430
7.600	-0.030	-222.798	0.000	0.000	0.000	89.430
8.000	-0.030	-224.349@	0.000	0.000	0.000	0.000@

Memb 5, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.030	16.323*	0.000	0.000	0.000	0.000@
0.800	-0.030	13.220	0.000	0.000	0.000	11.817
1.600	-0.030	10.118	0.000	0.000	0.000	21.153
2.400	-0.030	7.015	0.000	0.000	0.000	28.006
3.200	-0.030	3.913	0.000	0.000	0.000	32.377
4.000	-0.030	0.810	0.000	0.000	0.000	34.266*
4.800	-0.030	-2.293	0.000	0.000	0.000	33.673
5.600	-0.030	-5.395	0.000	0.000	0.000	30.598
6.400	-0.030	-8.498	0.000	0.000	0.000	25.041
7.200	-0.030	-11.600	0.000	0.000	0.000	17.001
7.800	-0.030	-13.927	0.000	0.000	0.000	9.343
7.800	-0.030	-46.327	0.000	0.000	0.000	9.343
8.000	-0.030	-47.103@	0.000	0.000	0.000	0.000@

Memb 6, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.030	-8.887*	0.000	0.000	0.000	0.000*
0.560	-0.030	-11.059	0.000	0.000	0.000	-5.585
1.120	-0.030	-13.231	0.000	0.000	0.000	-12.386
1.680	-0.030	-15.403	0.000	0.000	0.000	-20.404
2.240	-0.030	-17.575	0.000	0.000	0.000	-29.637
2.800	-0.030	-19.746	0.000	0.000	0.000	-40.087
3.360	-0.030	-21.918	0.000	0.000	0.000	-51.753
3.920	-0.030	-24.090	0.000	0.000	0.000	-64.636
4.480	-0.030	-26.262	0.000	0.000	0.000	-78.734
5.040	-0.030	-28.434	0.000	0.000	0.000	-94.049
5.600	-0.030	-30.605@	0.000	0.000	0.000	-110.580@

Memb 7, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	53.244*	0.000	0.000	0.000	-110.741@
0.240	0.000	51.823	0.000	0.000	0.000	-98.133
0.480	0.000	50.403	0.000	0.000	0.000	-85.865
0.720	0.000	48.983	0.000	0.000	0.000	-73.939
0.960	0.000	47.562	0.000	0.000	0.000	-62.354
1.200	0.000	46.142	0.000	0.000	0.000	-51.109
1.440	0.000	44.722	0.000	0.000	0.000	-40.206
1.680	0.000	43.301	0.000	0.000	0.000	-29.643
1.920	0.000	41.881	0.000	0.000	0.000	-19.421
2.160	0.000	40.460	0.000	0.000	0.000	-9.540
2.400	0.000	39.040@	0.000	0.000	0.000	0.000*

Memb 8, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	38.731*	-0.250	0.000	0.000	0.000	0.000*
0.540	38.445	-0.250	0.000	0.000	0.000	-0.135
1.080	38.160	-0.250	0.000	0.000	0.000	-0.270
1.620	37.874	-0.250	0.000	0.000	0.000	-0.404
2.160	37.588	-0.250	0.000	0.000	0.000	-0.539
2.700	37.302	-0.250	0.000	0.000	0.000	-0.674
3.240	37.016	-0.250	0.000	0.000	0.000	-0.809
3.780	36.730	-0.250	0.000	0.000	0.000	-0.944
4.320	36.444	-0.250	0.000	0.000	0.000	-1.078
4.860	36.159	-0.250	0.000	0.000	0.000	-1.213
5.400	35.873@	-0.250	0.000	0.000	0.000	-1.348@

Memb 9, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	175.503*	0.279	0.000	0.000	0.000	0.000@
0.540	175.217	0.279	0.000	0.000	0.000	0.151
1.080	174.931	0.279	0.000	0.000	0.000	0.302
1.620	174.646	0.279	0.000	0.000	0.000	0.453
2.160	174.360	0.279	0.000	0.000	0.000	0.604
2.700	174.074	0.279	0.000	0.000	0.000	0.754
3.240	173.788	0.279	0.000	0.000	0.000	0.905
3.780	173.502	0.279	0.000	0.000	0.000	1.056
4.320	173.216	0.279	0.000	0.000	0.000	1.207
4.860	172.931	0.279	0.000	0.000	0.000	1.358
5.400	172.645@	0.279	0.000	0.000	0.000	1.509*

Memb 10, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	160.654*	0.000	0.000	0.000	0.000	0.000*
0.540	160.368	0.000	0.000	0.000	0.000	0.000
1.080	160.083	0.000	0.000	0.000	0.000	0.000
1.620	159.797	0.000	0.000	0.000	0.000	0.000
2.160	159.511	0.000	0.000	0.000	0.000	0.000
2.700	159.225	0.000	0.000	0.000	0.000	0.000
3.240	158.939	0.000	0.000	0.000	0.000	0.000
3.780	158.653	0.000	0.000	0.000	0.000	0.000
4.320	158.368	0.000	0.000	0.000	0.000	0.000
4.860	158.082	0.000	0.000	0.000	0.000	0.000
5.400	157.796#	0.000	0.000	0.000	0.000	0.000#

Memb 11, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	137.858*	0.000	0.000	0.000	0.000	0.000*
0.540	137.573	0.000	0.000	0.000	0.000	0.000
1.080	137.287	0.000	0.000	0.000	0.000	0.000
1.620	137.001	0.000	0.000	0.000	0.000	0.000
2.160	136.715	0.000	0.000	0.000	0.000	0.000
2.700	136.429	0.000	0.000	0.000	0.000	0.000
3.240	136.143	0.000	0.000	0.000	0.000	0.000
3.780	135.858	0.000	0.000	0.000	0.000	0.000
4.320	135.572	0.000	0.000	0.000	0.000	0.000
4.860	135.286	0.000	0.000	0.000	0.000	0.000
5.400	135.000#	0.000	0.000	0.000	0.000	0.000#

Memb 12, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	243.530*	0.000	0.000	0.000	0.000	0.000#
0.540	243.244	0.000	0.000	0.000	0.000	0.000
1.080	242.959	0.000	0.000	0.000	0.000	0.000
1.620	242.673	0.000	0.000	0.000	0.000	0.000
2.160	242.387	0.000	0.000	0.000	0.000	0.000
2.700	242.101	0.000	0.000	0.000	0.000	0.000
3.240	241.815	0.000	0.000	0.000	0.000	0.000
3.780	241.529	0.000	0.000	0.000	0.000	0.000
4.320	241.244	0.000	0.000	0.000	0.000	0.000
4.860	240.958	0.000	0.000	0.000	0.000	0.000
5.400	240.672#	0.000	0.000	0.000	0.000	0.000*

Memb 13, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	41.074*	0.000	0.000	0.000	0.000	0.000*
0.540	40.788	0.000	0.000	0.000	0.000	0.000
1.080	40.502	0.000	0.000	0.000	0.000	0.000
1.620	40.217	0.000	0.000	0.000	0.000	0.000
2.160	39.931	0.000	0.000	0.000	0.000	0.000
2.700	39.645	0.000	0.000	0.000	0.000	0.000
3.240	39.359	0.000	0.000	0.000	0.000	0.000
3.780	39.073	0.000	0.000	0.000	0.000	0.000
4.320	38.787	0.000	0.000	0.000	0.000	0.000
4.860	38.501	0.000	0.000	0.000	0.000	0.000
5.400	38.216#	0.000	0.000	0.000	0.000	0.000#

Memb 14, Case 4 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	86.708*	-0.030	0.000	0.000	0.000	0.000*
0.540	86.422	-0.030	0.000	0.000	0.000	-0.016
1.080	86.136	-0.030	0.000	0.000	0.000	-0.032
1.620	85.850	-0.030	0.000	0.000	0.000	-0.048
2.160	85.564	-0.030	0.000	0.000	0.000	-0.064
2.700	85.278	-0.030	0.000	0.000	0.000	-0.080
3.240	84.993	-0.030	0.000	0.000	0.000	-0.096
3.780	84.707	-0.030	0.000	0.000	0.000	-0.113
4.320	84.421	-0.030	0.000	0.000	0.000	-0.129
4.860	84.135	-0.030	0.000	0.000	0.000	-0.145
5.400	83.849#	-0.030	0.000	0.000	0.000	-0.161#

Memb 1, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	2.385	290.747*	0.000	0.000	0.000	-12.877
1.040	2.385	284.592	0.000	0.000	0.000	286.299
2.067	2.385	278.514	0.000	0.000	0.000	575.454
2.067	2.385	90.514	0.000	0.000	0.000	575.454
2.080	2.385	90.437	0.000	0.000	0.000	576.630
3.120	2.385	84.282	0.000	0.000	0.000	667.484
4.134	2.385	78.281	0.000	0.000	0.000	749.903*
4.134	2.385	-109.719	0.000	0.000	0.000	749.903#
4.160	2.385	-109.873	0.000	0.000	0.000	747.049
5.200	2.385	-116.028	0.000	0.000	0.000	629.580
6.201	2.385	-121.952	0.000	0.000	0.000	510.471
6.201	2.385	-309.952	0.000	0.000	0.000	510.471
6.240	2.385	-310.183	0.000	0.000	0.000	498.378
6.400	2.385	-311.130	0.000	0.000	0.000	448.673
6.400	2.385	-342.730	0.000	0.000	0.000	448.673
7.280	2.385	-347.938	0.000	0.000	0.000	144.780
8.268	2.385	-353.785	0.000	0.000	0.000	-201.871
8.268	2.385	-541.785	0.000	0.000	0.000	-201.871
8.320	2.385	-542.093	0.000	0.000	0.000	-230.052
9.360	2.385	-548.248	0.000	0.000	0.000	-797.029
10.335	2.385	-554.018	0.000	0.000	0.000	-1334.384
10.335	2.385	-742.018	0.000	0.000	0.000	-1334.384
10.400	2.385	-742.408#	0.000	0.000	0.000	-1382.628#

Memb 2, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.683	703.125*	0.000	0.000	0.000	-1366.061#
1.100	-0.683	696.615	0.000	0.000	0.000	-596.204
2.000	-0.683	691.288	0.000	0.000	0.000	28.353
2.000	-0.683	219.288	0.000	0.000	0.000	28.353
2.200	-0.683	213.377	0.000	0.000	0.000	71.619
3.300	-0.683	180.863	0.000	0.000	0.000	288.451
4.400	-0.683	148.349	0.000	0.000	0.000	469.517
5.500	-0.683	115.835	0.000	0.000	0.000	614.818
6.300	-0.683	92.188	0.000	0.000	0.000	698.027
6.300	-0.683	60.588	0.000	0.000	0.000	698.027
6.600	-0.683	51.720	0.000	0.000	0.000	714.873
7.700	-0.683	19.206	0.000	0.000	0.000	753.883
8.800	-0.683	-13.308	0.000	0.000	0.000	757.128*
9.900	-0.683	-45.822	0.000	0.000	0.000	724.606
10.200	-0.683	-54.689	0.000	0.000	0.000	709.530
10.200	-0.683	-875.089	0.000	0.000	0.000	709.530
11.000	-0.683	-898.736#	0.000	0.000	0.000	0.000

Memb 3, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.683	178.773*	0.000	0.000	0.000	0.000
0.800	-0.683	155.126	0.000	0.000	0.000	133.560
1.600	-0.683	131.480	0.000	0.000	0.000	248.202
2.400	-0.683	107.833	0.000	0.000	0.000	343.927
3.200	-0.683	84.187	0.000	0.000	0.000	420.735
4.000	-0.683	60.540	0.000	0.000	0.000	478.626
4.800	-0.683	36.893	0.000	0.000	0.000	517.599
5.600	-0.683	13.247	0.000	0.000	0.000	537.655
6.400	-0.683	-10.400	0.000	0.000	0.000	538.794*
7.200	-0.683	-34.046	0.000	0.000	0.000	521.016
7.400	-0.683	-39.958	0.000	0.000	0.000	513.615
7.400	-0.683	-847.158	0.000	0.000	0.000	513.615
8.000	-0.683	-864.893#	0.000	0.000	0.000	0.000#

Memb 4, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.683	163.177*	0.000	0.000	0.000	0.000#
0.800	-0.683	139.531	0.000	0.000	0.000	121.083
1.600	-0.683	115.884	0.000	0.000	0.000	223.249
2.400	-0.683	92.237	0.000	0.000	0.000	306.498
3.200	-0.683	68.591	0.000	0.000	0.000	370.829
4.000	-0.683	44.944	0.000	0.000	0.000	416.243
4.800	-0.683	22.930	0.000	0.000	0.000	443.393
5.600	-0.683	0.915	0.000	0.000	0.000	452.930*
6.400	-0.683	-21.100	0.000	0.000	0.000	444.857
7.200	-0.683	-43.114	0.000	0.000	0.000	419.171
7.600	-0.683	-54.121	0.000	0.000	0.000	399.724
7.600	-0.683	-993.801	0.000	0.000	0.000	399.724
8.000	-0.683	-1007.809#	0.000	0.000	0.000	0.000

Memb 5, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.683	128.933*	0.000	0.000	0.000	0.000#
0.800	-0.683	106.918	0.000	0.000	0.000	94.341
1.600	-0.683	84.904	0.000	0.000	0.000	171.069
2.400	-0.683	62.889	0.000	0.000	0.000	230.187
3.200	-0.683	40.875	0.000	0.000	0.000	271.692
4.000	-0.683	18.860	0.000	0.000	0.000	295.586
4.800	-0.683	-3.155	0.000	0.000	0.000	301.868*
5.600	-0.683	-25.169	0.000	0.000	0.000	290.538
6.400	-0.683	-47.184	0.000	0.000	0.000	261.597
7.200	-0.683	-69.198	0.000	0.000	0.000	215.044
7.800	-0.683	-85.709	0.000	0.000	0.000	168.572
7.800	-0.683	-840.109	0.000	0.000	0.000	168.572
8.000	-0.683	-845.613#	0.000	0.000	0.000	0.000

Memb 6, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	-0.683	-108.852*	0.000	0.000	0.000	0.000*
0.560	-0.683	-124.262	0.000	0.000	0.000	-65.272
1.120	-0.683	-139.672	0.000	0.000	0.000	-139.173
1.680	-0.683	-155.082	0.000	0.000	0.000	-221.704
2.240	-0.683	-170.492	0.000	0.000	0.000	-312.865
2.800	-0.683	-185.903	0.000	0.000	0.000	-412.656
3.360	-0.683	-201.313	0.000	0.000	0.000	-521.076
3.920	-0.683	-216.723	0.000	0.000	0.000	-638.126
4.480	-0.683	-232.133	0.000	0.000	0.000	-763.806
5.040	-0.683	-247.544	0.000	0.000	0.000	-898.115
5.600	-0.683	-262.954#	0.000	0.000	0.000	-1041.055#

Memb 7, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	0.000	470.780*	0.000	0.000	0.000	-1044.945#
0.240	0.000	463.686	0.000	0.000	0.000	-932.609
0.480	0.000	456.592	0.000	0.000	0.000	-822.175
0.720	0.000	449.498	0.000	0.000	0.000	-713.444
0.960	0.000	442.404	0.000	0.000	0.000	-606.416
1.200	0.000	435.310	0.000	0.000	0.000	-501.090
1.440	0.000	428.216	0.000	0.000	0.000	-397.467
1.680	0.000	421.122	0.000	0.000	0.000	-295.547
1.920	0.000	414.028	0.000	0.000	0.000	-195.328
2.160	0.000	406.934	0.000	0.000	0.000	-96.813
2.400	0.000	399.840#	0.000	0.000	0.000	0.000*

Memb 8, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	293.605*	-2.385	0.000	0.000	0.000	0.000*
0.540	293.319	-2.385	0.000	0.000	0.000	-1.288
1.080	293.034	-2.385	0.000	0.000	0.000	-2.575
1.620	292.748	-2.385	0.000	0.000	0.000	-3.863
2.160	292.462	-2.385	0.000	0.000	0.000	-5.151
2.700	292.176	-2.385	0.000	0.000	0.000	-6.439
3.240	291.890	-2.385	0.000	0.000	0.000	-7.726
3.780	291.604	-2.385	0.000	0.000	0.000	-9.014
4.320	291.319	-2.385	0.000	0.000	0.000	-10.302
4.860	291.033	-2.385	0.000	0.000	0.000	-11.589
5.400	290.747#	-2.385	0.000	0.000	0.000	-12.877#

Memb 9, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	1448.386*	3.068	0.000	0.000	0.000	0.000#
0.540	1448.100	3.068	0.000	0.000	0.000	1.657
1.080	1447.815	3.068	0.000	0.000	0.000	3.313
1.620	1447.529	3.068	0.000	0.000	0.000	4.970
2.160	1447.243	3.068	0.000	0.000	0.000	6.627
2.700	1446.957	3.068	0.000	0.000	0.000	8.284
3.240	1446.671	3.068	0.000	0.000	0.000	9.940
3.780	1446.385	3.068	0.000	0.000	0.000	11.597
4.320	1446.099	3.068	0.000	0.000	0.000	13.254
4.860	1445.814	3.068	0.000	0.000	0.000	14.910
5.400	1445.528#	3.068	0.000	0.000	0.000	16.567*

Memb 10, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	1080.367*	0.000	0.000	0.000	0.000	0.000*
0.540	1080.081	0.000	0.000	0.000	0.000	0.000
1.080	1079.795	0.000	0.000	0.000	0.000	0.000
1.620	1079.510	0.000	0.000	0.000	0.000	0.000
2.160	1079.224	0.000	0.000	0.000	0.000	0.000
2.700	1078.938	0.000	0.000	0.000	0.000	0.000
3.240	1078.652	0.000	0.000	0.000	0.000	0.000

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
3.780	1078.366	0.000	0.000	0.000	0.000	0.000
4.320	1078.080	0.000	0.000	0.000	0.000	0.000
4.860	1077.795	0.000	0.000	0.000	0.000	0.000
5.400	1077.509#	0.000	0.000	0.000	0.000	0.000#

Memb 11, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	1030.929*	0.000	0.000	0.000	0.000	0.000*
0.540	1030.643	0.000	0.000	0.000	0.000	0.000
1.080	1030.357	0.000	0.000	0.000	0.000	0.000
1.620	1030.071	0.000	0.000	0.000	0.000	0.000
2.160	1029.785	0.000	0.000	0.000	0.000	0.000
2.700	1029.499	0.000	0.000	0.000	0.000	0.000
3.240	1029.214	0.000	0.000	0.000	0.000	0.000
3.780	1028.928	0.000	0.000	0.000	0.000	0.000
4.320	1028.642	0.000	0.000	0.000	0.000	0.000
4.860	1028.356	0.000	0.000	0.000	0.000	0.000
5.400	1028.070#	0.000	0.000	0.000	0.000	0.000#

Memb 12, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	1136.600*	0.000	0.000	0.000	0.000	0.000*
0.540	1136.314	0.000	0.000	0.000	0.000	0.000
1.080	1136.028	0.000	0.000	0.000	0.000	0.000
1.620	1135.743	0.000	0.000	0.000	0.000	0.000
2.160	1135.457	0.000	0.000	0.000	0.000	0.000
2.700	1135.171	0.000	0.000	0.000	0.000	0.000
3.240	1134.885	0.000	0.000	0.000	0.000	0.000
3.780	1134.599	0.000	0.000	0.000	0.000	0.000
4.320	1134.313	0.000	0.000	0.000	0.000	0.000
4.860	1134.028	0.000	0.000	0.000	0.000	0.000
5.400	1133.742#	0.000	0.000	0.000	0.000	0.000#

Memb 13, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	739.620*	0.000	0.000	0.000	0.000	0.000#
0.540	739.334	0.000	0.000	0.000	0.000	0.000
1.080	739.048	0.000	0.000	0.000	0.000	0.000
1.620	738.762	0.000	0.000	0.000	0.000	0.000
2.160	738.476	0.000	0.000	0.000	0.000	0.000
2.700	738.191	0.000	0.000	0.000	0.000	0.000
3.240	737.905	0.000	0.000	0.000	0.000	0.000
3.780	737.619	0.000	0.000	0.000	0.000	0.000
4.320	737.333	0.000	0.000	0.000	0.000	0.000
4.860	737.047	0.000	0.000	0.000	0.000	0.000
5.400	736.761#	0.000	0.000	0.000	0.000	0.000*

Memb 14, Case 7 (Linear): 1.2G + 1.6Q

Station Location	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
0.000	736.592*	-0.683	0.000	0.000	0.000	0.000*
0.540	736.306	-0.683	0.000	0.000	0.000	-0.369
1.080	736.020	-0.683	0.000	0.000	0.000	-0.738
1.620	735.734	-0.683	0.000	0.000	0.000	-1.107
2.160	735.449	-0.683	0.000	0.000	0.000	-1.476
2.700	735.163	-0.683	0.000	0.000	0.000	-1.845
3.240	734.877	-0.683	0.000	0.000	0.000	-2.214
3.780	734.591	-0.683	0.000	0.000	0.000	-2.583
4.320	734.305	-0.683	0.000	0.000	0.000	-2.952
4.860	734.019	-0.683	0.000	0.000	0.000	-3.321
5.400	733.734#	-0.683	0.000	0.000	0.000	-3.690#

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	8000	mm	
Overhang:(Beam cc):	100	mm	ll Cl 13.4.1.3
Adj. Beam Centres :	2000	mm	
Stud Edge Distance	160	mm	
Beam Size	800WB168		
Beam Yield Stress	300	MPa	
Precamber	0	mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	Triple	
Deck Span :	2000	mm
Slab Thickness :	160	mm
Slab Mass :	3.17	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	78	
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	130.00	KN/m
Super Live Load :	0.00	KPa
Other Super. LL :	1.00	KN/m
Construction LL:		
i) Deck Placement:	0.30	KPa
ii) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	1.00	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	21	KNm
Depend. Bending Strength	971	KNm

Stage II : Concrete Placement

Factored Bending Moment :	63	KNm
Depend. Bending Strength	1831	KNm

% Composite Action 98 %

Bending Strength

Bending Strength Required :	1520	KNm	
Composite Bending Strength :	2528	KNm	ll Cl 13.4.1.3
Non-Composite Bending Strength:	1831	KNm	

Shear Strength

Shear Strength Required:	760	kN
Dependable Shear Strength:	1273	kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	141	MPa
Allow. Bottom Flange Stress:	270	MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	0.5 mm
SDL+φs LL+Shrink+Creep:	12.2 mm
Span/DF : Steel Immediate:	14659
Span/DF : Top of Concrete:	657
Span/DF : Steel Total:	629

Vibration

Natural Frequency :	6.75	Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.01	mm
Initial Amplitude from Heel Drop	0.004	mm
Damping Required :	2.5	%

FRR: Fire Resistance Rating

Composite Section Rating:	20.2 min
Non-Composite Section Rating:	15.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is			1.54 KN/mm
Weld Leg Reqd: SP	Single side	9	mm
	Double side	5	mm
Bottom Flange:vwb: VdepQb/Is			1.54 KN/mm
Weld Leg Reqd: SP	Single side	9	mm
	Double side	5	mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$2,356

Calculation Sheet

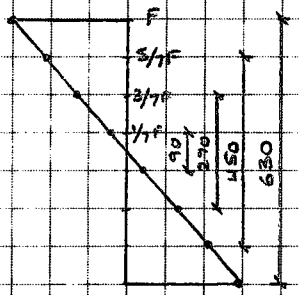
Job name: Queensgate mall
Subject: Zone J LVL2 floor beam
By: DBS

Job No: 2711265
Page No: of
Date: 15-6-04

Revised Connection for 800WB168 left hand end only

$V^* = 1005\,kN$

Try 8 M24



$M = 0.63F + 0.45 \times \frac{5}{7}F + 0.27 \times \frac{3}{7}F + 0.09 \times \frac{1}{7}F$
$= 1.08F$

$M^* = V^* e$
$= 1005\,kN \times 0.07$
$= 70.35\,kN\,m$

$F = 65.1\,kN$

$R^*_{bolt} = \sqrt{\left(\frac{1005}{8}\right)^2 + (65.1)^2}$



$= 141.5\,kN$

use 8M24 8.8 XS

8M24 8.8 XS.

$\phi V_{fn} = 186\,kN$

$141.5 \le 186\,kN$ ✓

$\phi V_b = 0.9 \times 2.2 \times 10 \times 24 \times 440$
$= 304\,kN$

$\phi V_p = 40 \times 10 \times 440 \times 0.9$
$= 158.4\,kN$

$141.5\,kN \le 158\,kN$ ✓

Cleat shear

$\phi V_{cleat} = 0.9 \times 0.6 \times 300 \times 16 \times (720 - 8 \times 26)$
$= 1327\,kN$

16mm 720 x 120 PLATE

$1005 \le 1327$ ✓

Weld continuation to col insert cleat.

$\phi V_w = 0.85 \times 2 \times 720$.
$= 1202\,kN$ ✓

6 241059 each side.

Calculation Sheet

Job name: Queensgate mall
Subject: Zone 3 LVL2 floor beam
By: DBS

Job No: 271265
Page No: of
Date: 15-6-04

web shear:

$\phi V_{web} = 0.9 \times 0.6 \times 310 \times 10 \times (760 - 6 \times 26)$
$= 924\ kN$.

Loads $\leq 924kN$ must use Doubler plate 10mm with 6— fillet around.

check web buckling.

$\phi V_{web,buck} = \alpha_v \times 0.9 \times 0.6 \times 310 \times 10 \times 760$

$= 0.939 \times 1272\ kN$
$= 1194\ kN$.

Loads $\leq 1194\ kN$ ✓

$\alpha_v = \left(\frac{82 \times t_w}{d_p k_f}\right)^2$

$= \left(\frac{82 \times 10}{760 \times \sqrt{\frac{300}{250}}}\right)^2$

$= 0.939$

check first bolt.

$F_{bolt} = 0.6 \times 300 \times 37 \times 10 + 10 \times 37 \times 440$

$= 229kN$

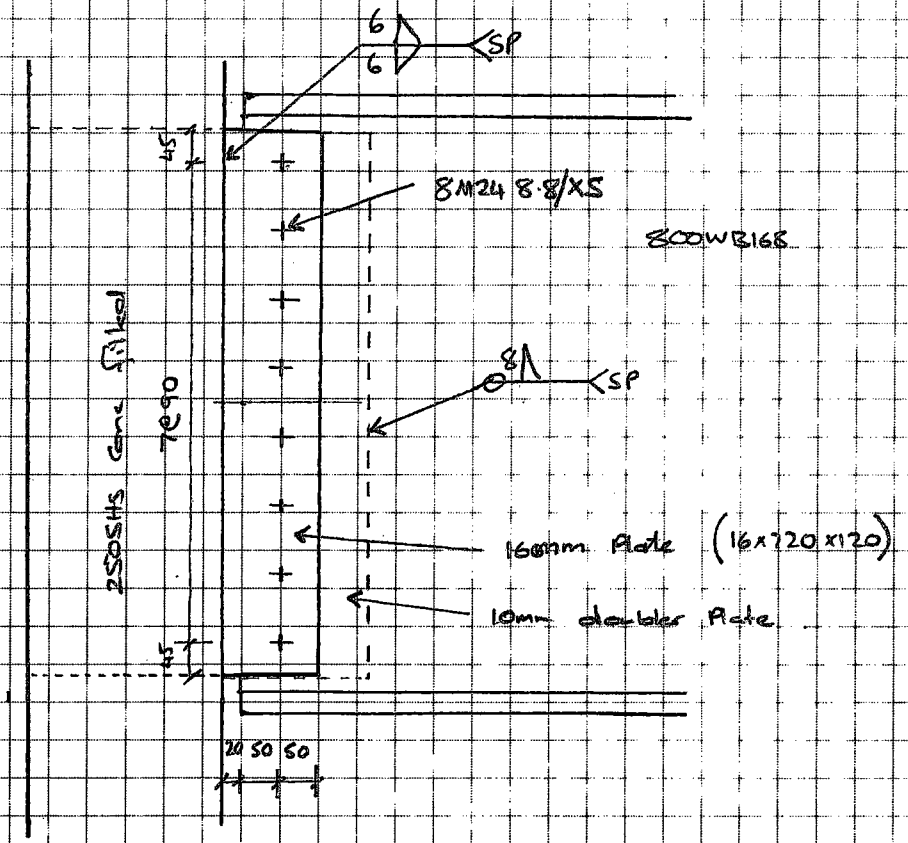
$V^*_{bolt} \leq \phi F_{bolt}$.

$141.5kN \leq 0.8 \times 229$
$\leq 163\ kN$ ✓

Calculation Sheet

Job name: Queensgate inf.
Subject: Zone 3 LVL2 floor beams.
By: DBS

Job No: 2711265
Page No: of
Date: 15-6-04



241059

Calculation Sheet

Job name: Queensgate mall Redevelopment
Job No: 2711265/109
Subject: Zone J LVL2 floor beam Connections
Page No: 1 of 18
By: Dean Saunders.
Date: 28-5-04

Beams on grids SA, SC, SE, SF, SG between S1 & S3

Pt. Point loads:

(A) S1 to S2 SG 63kN
S1 to S3 SA, SC, SE, SF 115kN

(B) S2 to S3 SG 100kN
164kN

udls Dead (10.5 + 3.17) kPa × 8.2m = 30.09 kN/m

$V = \frac{30.09 \times 11.905}{2}$

= 179kN

Live. 6.5kPa × 8.2m = 53.3kN/m

V = 317kN

Actual Shears:

(A) $V_{S1} = 1.2 \times (63 + 179) + 1.6 \times 317$ kN.

= 798kN

$V_{S2,S3} = 1.2 (115 + 179) + 1.6 \times 317$ kN.

= 860 kN

(B) $V = 1.2 \left(164 + \frac{179}{2}\right) + 1.6 \times \frac{317}{2}$.

= 558 kN

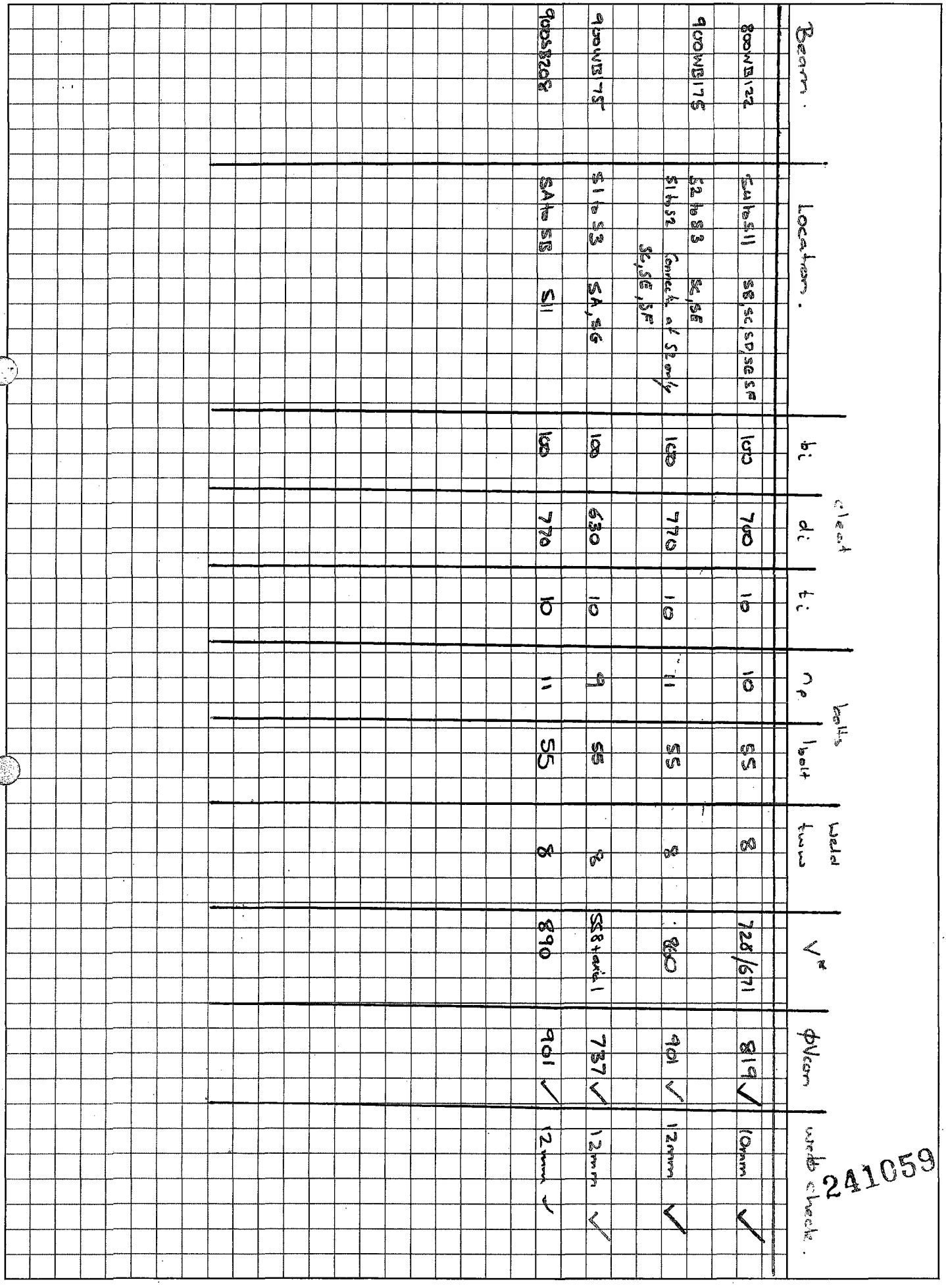
Methodology:

(1) Using Hera connectra quick select geometries to satisfy shear loads (plus axial if required) and ensure weld thickness is greater than cleat thickness.

(2) Design the remaining connections.

Calculation Sheet

Job name: Queensgate Mall Redevelopment
Job No: 2711265/109
Subject: Zone J LVL2 floor beam Connections.
Page No: 2 of 13
By: Dean Saunders.
Date: 28-5-04



241059

Calculation Sheet

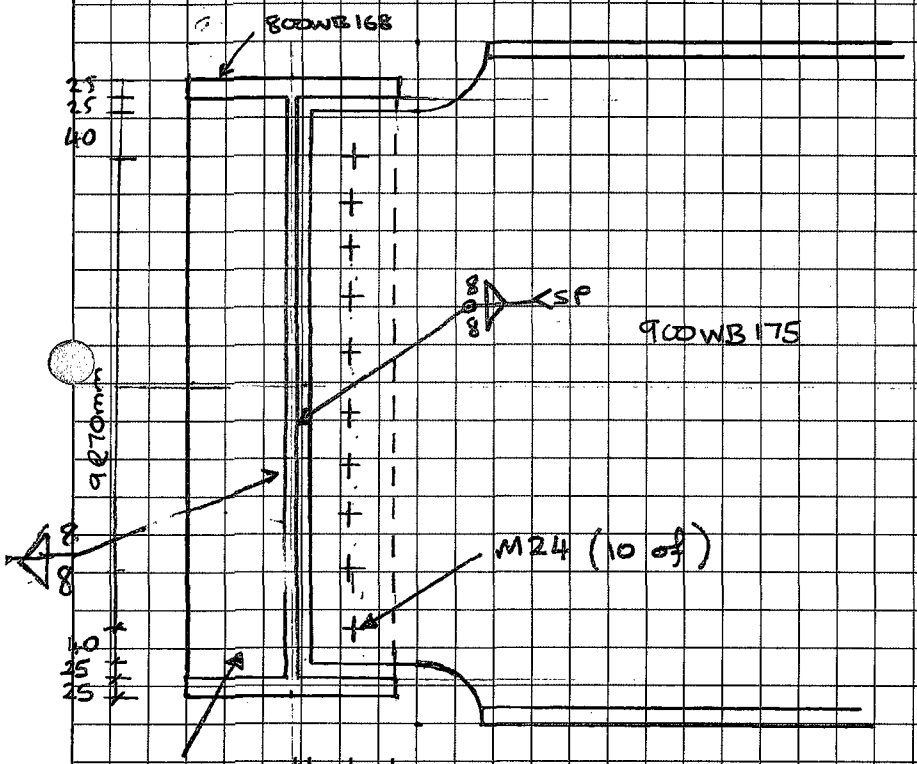
Job name: Queensgate mall Redevelopment. Job No: 2711265/109

Subject: Zone J LVL 2 floor beam Connection Page No: 3 of 15

By: DJOS Date: 28-5-04

Connections

900WB175 to 800WB168 $V^* = 798$kN



Cleat & web shear

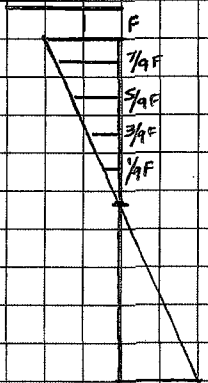
$V^* \leq \phi V_w$.

$\leq 0.9 \times 0.6 \times f_y A_{vg}$

$\leq 0.9 \times 0.6 \times 300 \times 12 \times (590 - 10 \times 26)$ M24 used

$798\text{kN} \leq 836\text{ kN}$ ✓

Bolts



$M = F \times 0.63 + 7/9 \times 0.49 + 5/9 F \times 0.35 + 3/9 F \times 0.21 + 1/9 \times 0.07$

$= 1.28F$

$F = 49.7$ kN

$f_{bolt} = \frac{798}{10} + 49.7$

$= 129.5$ kN

$\phi f_n = 133$ ✓

10 M24 bolts

Calculation Sheet

Job name: Queensgate mall Redevelopment — Job No: 2711265/109

Subject: Zone J LVL2 Down beam Connection — Page No: 4 of 13

By: DwS. — Date: 28-5-04

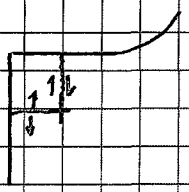
$\phi V_b = 3.2 \times 12 \times 24 \times 440$

$= 405 kN$

$\phi V_p = 40 \times 12 \times 440$

$= 211 kN \quad \times 10 = 2110 kN \geq 798$ ✓ ok.

Check first bolt.



$F_{web} = 0.6 \times 300 \times 12 \times 27 + 12 \times 37 \times 440$

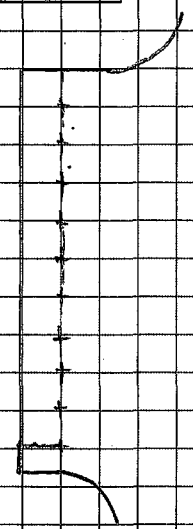
$= 253 kN$

$V^*_{bolt} \leq \phi F_{web}$

$127.5 kN \leq 0.9 \times 253 kN$ ✓

$\leq 228 kN$

Check Cope block.



$F_{web} = 0.6 \times 300 \times 12 \times (27 + 9 \times 44) + 12 \times 37 \times 440$

$= 1109 kN$

$798 \leq 0.9 \times 1109$ ✓ Plate o.k.

Cleat / stiffener welds:

$V^* = 798 kN$

$\phi V_{ws\ 8mm} = 1.11 kN/mm$

$\phi V_{ws} = 1.11 kN/mm \times 2 \times 710 mm$

$= 1576 kN$

$798 kN \leq 1576 kN$ ✓

241059

Calculation Sheet

Job name: Queensgate Job No: 2711265/109

Subject: Zone J LVL2 floor beam Connections Page No: 5 of

By: DWS Date: 31-5-04

① $V^* = 1037$kN on 900UB320

$d_1 = 856$mm $t_w = 12$mm

Shear on cleat use 16mm

$V^* \le \phi V_{w,w}$

$1037{,}000 \le 0.9 \times 0.6 \times 300 \times A_w$

$A_w \ge 6401$

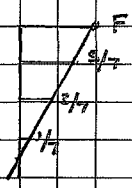
$L \ge 400$mm.

Try 8 M24 @90 with $g_2 = 45$.

$M^* = eV^*$

$= 0.7 \times 1037$

$= 72.6$ kNm



$M = F \times 0.63 + 5/7 F \times 0.45 + 3/7 \times 0.27 + 1/7 \times 0.09$

$= 1.081F$

$M^* \le 1.08F$

$F \ge 67.2$ kN

$V^* \le V_{bolt} \times n$

$V_{bolt} \ge \frac{1037}{8}$

≥ 130kN

$V_{bolt\ resul} = \sqrt{130^2 + 67.2^2}$

$= 146$ kN

$\phi V_{n\ c/s} = 186$kN $146 \le 186$kN ✓

8M24 8.8 c/s

Calculation Sheet

Job name: [illegible] Job No: 3711365/109

Subject: Beam 3 Level 6 [illegible] Page No: 6 of

By: DWS Date: 31-5-04

V_{bf} = 3.2 × 12 × 24 × 440 SB web governs.

= 405 kN ✓

V_p = 445 × 12 × 440

= 237 kN ✓ 213 ≥ 160.6 kN.

ϕV_p = 213 kN.

Shear on cleat

$A_{nv\,reduce}$ = 16 × (720 − 8 × 26)

= 8192 mm²

ϕV_{nv} = 0.9 × 0.6 × 300 × 8192

= 1327 kN

1037 kN ≤ 1327 kN ✓

Cleat
720 deep
16mm thick

check tearout Global

1st bolt F_{web} = 0.6 × 300 × 12 × 32 + 12 × 37 × 440. web governs.

= 264 kN

All bolts F_{web} = 0.6 × 300 × 12 × (32 + 7 × 64) + 12 × 37 × 440

= 1232 kN

$F_{web\,1st} \times 0.9 \geq 160.6$ kN.

237 kN ≥ 160.6 kN ✓

$F_{web\,all} \times 0.9 \geq 1037$

1109 kN ≥ 1037 ✓

Shear on web.

1037 kN ≤ 0.9 × 0.6 × 300 × 12 (L − 8 × 26).

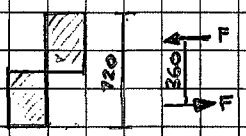
L ≥ 741 mm

web L ≥ 742 mm

welds:

V^* = 1037 kN M^* = 726 kNm

F = 202 kN



weld load = $\sqrt{1037^2 + 202^2}$

= 1056 kN

1056 kN ≤ 2 × 720 mm × 0.85 kN/m

≤ 1202 kN

6mm 241059
both sides

(2) $V^* = 870kN$ on 800WB192-PO-S4

$d_1 = 760mm$ $t_w = 10mm$

(3) $V^* = 821kN$ on 800WB156-PO-S4.

$d_1 = 760mm$ $t_w = 10mm$.

Weld.

$V^* = \sqrt{870^2 + 169^2}$

$= 886kN$

$M^* = 870 \times 0.07$

$= 60.9kNm$

$F = 169kN$

$886kN \leq 2 \times 720 \times 0.855$ f_{uw} 6mm

$\leq 1202kN$ ✓

6mm SP WP both sides.

Bolts.

Try 8M24 @ 90 with $g_c = 45$

$M^* = 60.9kNm \leq 1.08F$

$F \geq 56kN$

$V^*_{bolt} = V^*/n$

$= \frac{870kN}{8}$

$= 109kN$

$F^*_{bolt} = \sqrt{109^2 + 56^2}$

$= 122kN$

$\phi V_{n\times 1/3} = 186kN$ $122 \leq 186kN$ ✓

8M24 8.8 x/s

$V_b = 8.7 \times 10 \times 24 \times 440$ web t_w 10 governs.

$= 338kN$

$V_p = 45 \times 10 \times 440$.

$= 198kN$.

$\phi V_p = 178kN$.

$178 \geq 122kN$ ✓

Calculation Sheet

Job name: [illegible] Job No: 241059/1W

Subject: Zone J LVL 2 floor beam Connections Page No: [illegible] of

By: DLOS. Date: 3-6-24

Shear on cleat. Try 12mm

$\phi V_{ws} = 0.9 \times 0.6 \times 300 \times 12 \times (720 - 8 \times 26)$

$= 995\,kN$

$870kN \leq 995kN$ ✓

Shear on web

$\phi V_{ww} = 0.9 \times 0.6 \times 300 \times 10 \times (720 - 6 \times 26)$

$= 830\,kN$

$870 \not\leq 830kN$ Provide 10mm double Plate to Rivet cleat.

check bearing.

1st bolt $\phi F_{brg} = 0.9[0.6 \times 300 \times 10 \times 32 + 10 \times 37.4 \times 410]$

$= 198\,kN$

$146kN \leq 198kN$ ✓

All bolts $\phi F_{brg} = 0.9[0.6 \times 300 \times 10 \times [32 + 7 \times 64] + 10 \times 37.4 \times 410]$

$= 924\,kN$

$870kN \leq 924kN$ ✓

(4) $V^* = 1161kN$ — 900WB215 : PO.S2

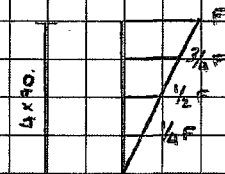
$t_1 = 850mm$ $t_2 = 12mm$

Try 9M24 @90 with $g_e = 45mm$

cleat. 890 deep

$M^* = 1161 \times 0.07$

$= 81.27\,kN.m$



$81.27kN \leq 1.35F$

$F \geq 60.2kN$

$M = 0.72F + \frac{3}{4}F \times 0.54 + \frac{1}{2}F \times 0.36 + \frac{1}{4}F \times 0.18$

$= 1.35F$

$V^*_{bolt} = 1161/9$

$= 129kN.$

$F^*_{bolt} = \sqrt{129^2 + 60.2^2}$

$= 142kN.$

$\phi V_{fn\,x/s} = 186kN$

241059
9M24 8.8 x/s

Calculation Sheet

Job name: Shortland Job No: 2711265/109

Subject: Zone 1 LVL 2 Podium Connection Page No: 9 of

By: [illegible] Date: 3-6-04

V_b = 405kN web t_w = 12mm Governs.

V_p = 237kN

ϕV_p = 213

142 ≤ 213kN ✓

Shear on cleat.

$\phi V_{wy} = 0.9 \times 0.6 \times 300 \times t_w \times (810 - 9 \times 26)$

1161kN ≤ 93.3t_w.

$t_w \geq 12.5$mm use 16mm

cleat t_i = 16mm

Shear on web. t_w=12

ϕV_{wy} = 1120kN so must use Doubler Plt

10mm doubler plate 8mm fillet.

bearing.

1st bolt $\phi F_{wb} = 0.9 [0.6 \times 300 \times 12 \times 32 + 12 \times 37 \times 440]$

= 238kN

142kN ≤ 238kN ✓

All bolts $\phi F_{wb} = 0.9 [0.6 \times 300 \times 12 \times (32 + 8 \times 64) + 12 \times 37 \times 440]$

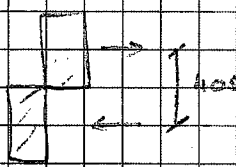
= 1233kN.

1161kN ≤ 1233kN ✓ (Does not Doubler plate)

weld.

ϕV_w = 2 × 810 × 0.835

= 1352kN



$V^* \leq \phi V_w$.

M^* = 51.2kNm.

F = 201kN.

$V^* = \sqrt{51^2 + 201^2} \times 2$

= 1230kN.

1230kN ≤ 1352kN ✓

6mm fillet both sides.

Design
Of
Mezzanine Floor Beams
& Bracing

241059

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Warehouse LVL2 PAGE 2 of 19
SECTION : Secondary beams for mezzanine
JOB No.: 2711265/109 DESIGN: dws
DATE : 19-May-04 CHECK :

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm
Adj. Beam Centres :	2100 mm
	2100 mm
Beam Size	460UB67
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2100 mm
Deck Span Type :	Triple
Slab Thickness :	130 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	41 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	3.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	45 KNm
Depend. Bending Strength	76 KNm

Stage II : Concrete Placement

Factored Bending Moment :	198 KNm
Depend. Bending Strength	397 KNm

% Composite Action 75 %

Bending Strength

Bending Strength Required :	358 KNm
Dependable Bending Strength :	684 KNm

Shear Strength

Shear Strength Required:	116 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	159 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	5.9 mm
SDL+φs LL+Shrink+Creep:	30.2 mm
Span/DF : Steel Immediate:	2115
Span/DF : Top of Concrete:	411
Span/DF : Steel Total:	344

Vibration

Natural Frequency :	4.92 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.20 mm
Initial Amplitude from Heel Drop	0.117 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating 20.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$947

241059

14-2

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beam in mezz work large span of Level
JOB No.: 2711265 **DESIGN:** dbs
DATE : 15-Jun-04 **CHECK :**

COMPOSITE INTERIOR BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an interior beam, equal flange I or Channel, simply supported and unpropped during construction.
Decking spans perpendicular to beam span. Refer NZS3404:Part1:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm
Adj. Beam Centres :	2150 mm
	2150 mm
Beam Size	460UB67
Beam Yield Stress	300 MPa
Precamber	25 mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span :	2150 mm
Deck Span Type :	Triple
Slab Thickness :	130 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	41 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.00 KN/m
Super Live Load :	3.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
i) Deck Placement:	0.40 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fu	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	46 KNm
Depend. Bending Strength	76 KNm

Stage II : Concrete Placement

Factored Bending Moment :	203 KNm
Depend. Bending Strength	397 KNm

% Composite Action 75 %

Bending Strength

Bending Strength Required :	367 KNm
Dependable Bending Strength :	685 KNm

Shear Strength

Shear Strength Required:	118 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	163 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	6.7 mm
SDL+φs LL+Shrink+Creep:	30.5 mm
Span/DF : Steel Immediate:	1860
Span/DF : Top of Concrete:	407
Span/DF : Steel Total:	334

Vibration

Natural Frequency :	4.87 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.20 mm
Initial Amplitude from Heel Drop	0.118 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating 20.3 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost:

Steel/kg :	$1.00
Shear Stud :	$2.80
Comparative Cost Factor :	$947

Calculation Sheet

Job name: Queensgate mall. **Job No:** 2711265

Subject: Mezzanine floor Zone 3 LVL2 **Page No:** of

By: DBS **Date:** 15-6-04

130mm Hi-bond slab.

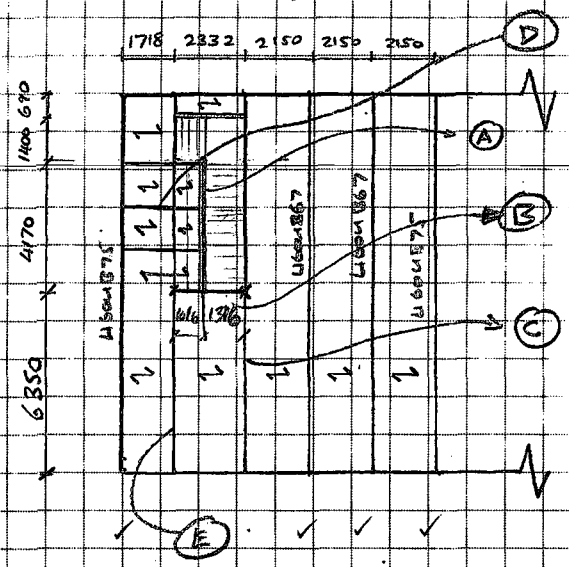
SDL = 0.5kPa f'_c = 40MPa ψ_s = 1.0 ψ_c = 0.7 ψ_l = 0.6
LL = 3.0kPa strength

664 mesh

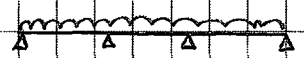
Max. span = 2500mm

Span varies 1718 to 2332mm

Secondary beams @ 12.4m



(A) Continuous PFC



w G = 1.52kN/m
Q = 1.52kN/m

M^* = 0.66kNm
M_s = 0.82kNm

ϕV_v = 3.6kN

use 150PFC ϕM_b = 30.2kNm ϕV_v = 156

(B)



w = 22kN/m Cantilever

M^* = 15kNm
ϕV_v = 26kN

use 200UB22 - WP30
ϕM_b = 39.6kNm

(A)
150PFC - WP30
stud @ 300

241059

(B)
200UB22 - PO - @300
- WP30

Calculation Sheet

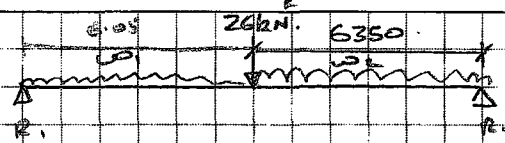
Job name: Job No:

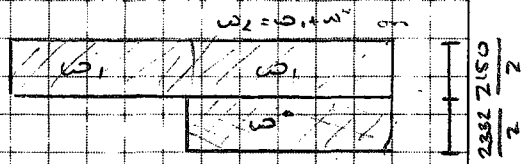
Subject: Page No: of

By: Date:

assume Q

(b).





G = 0.5 kPa SDL
= 2.47 kPa Hibeam
Q = 3.0 kPa.

w_1 G = 0.538 kN/m SDL.
Q = 3.225 kN/m

w_2 G = 1.12 kN/m
Q = 6.72 kN/m.

R_2 $G = \left(0.538 \times \frac{6.05^2}{2} + 1.12 \times 6.35 \times 9.225\right) / 12.4$ R_1

= 6.08 kN $G = 4.28$ kN.

$Q = \left(3.225 \times \frac{6.05^2}{2} + 26 \times 6.05 + 6.72 \times 6.35 \times 9.225\right) / 12.4$

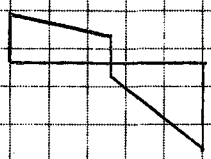
= 49.2 kN. Q = 39 kN.

M^* G



SFD $M_G^* = 16.6$ kNm

Q



SFD $M_Q^* = 177$ kNm

equiv udl $G = \frac{16.6 \times 8}{12.4^2}$ $Q = \frac{177 \times 8}{12.4^2}$

= 0.86 kN/m = 9.21 kN/m.

Cobeam used 460UB67 - PB5 @ 300 o/k

$V^* = 128$ kN. - WP30.

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beam in mezz stair edge
JOB No.: 2711265 **DESIGN:** dbs
DATE : 15-Jun-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm	
Overhang:(Beam cc):	100 mm	ll Cl 13.4.1.3
Adj. Beam Centres :	2150 mm	
Stud Edge Distance	130 mm	
Beam Size	460UB67	
Beam Yield Stress	300 MPa	
Precamber	25 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	double
Deck Span :	2150 mm
Slab Thickness :	130 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	41 @ 300 c
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.86 KN/m
Super Live Load :	0.00 KPa
Other Super. LL :	9.21 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	28 KNm
Depend. Bending Strength	76 KNm

Stage II : Concrete Placement

Factored Bending Moment :	114 KNm
Depend. Bending Strength	397 KNm

% Composite Action 75 %

Bending Strength

Bending Strength Required :	398 KNm	
Composite Bending Strength :	667 KNm	ll Cl 13.4.1.3
Non-Composite Bending Strength:	397 KNm	

Shear Strength

Shear Strength Required:	128 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	156 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-7.2 mm
SDL+φs LL+Shrink+Creep:	40.3 mm
Span/DF : Steel Immediate:	-1731
Span/DF : Top of Concrete:	307
Span/DF : Steel Total:	374

Vibration

Natural Frequency :	5.64 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.23 mm
Initial Amplitude from Heel Drop	0.088 mm
Damping Required :	3.2 %

FRR: Fire Resistance Rating

Composite Section Rating:	20.5 min
Non-Composite Section Rating:	15.4 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,433

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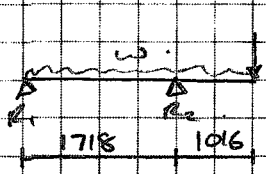
Calculation Sheet

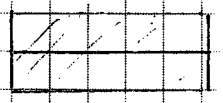
Job name: Job No:

Subject: Page No: of

By: Date:

(D)





$G_{SDL} = 0.5\ kPa$

$Q = 3.0\ kPa$

$G_{Slab} = 2.47\ kPa$

$w = 1.2(0.5 + 2.47) + 1.6 \times 3.0$

$= 8.36\ kN/m$

$R_2 = \left(8.36 \times \frac{(1.718+1.016)^2}{2} + 6.5 \times (1.718+1.016)\right) / 1.718$

$= 28.5\ kN$

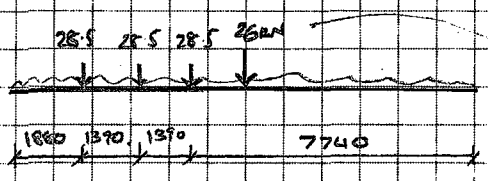
$R_1 = 0.87\ kN$

$M^*_- = 10.9\ kNm$

$M^*_+ = 3.05\ kNm$

use 200UB22 - PO - @300 - WP30

(E)



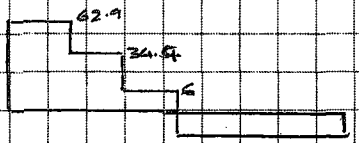
w's as per (C)

equiv udl $G = 0.56\ kN/m$

$Q = 9.21\ kN/m$

$R_1 = 28.5\left(\frac{7740 + 9130 + 10520}{12450}\right)$

$= 62.9\ kN$



$M^* = 174\ kNm$

equiv udl = 9.07 kN/m. put as G because includes factors already.

460UB67 - P25 - @300 -

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 PAGE :5
SECTION : Secondary beam in mezz stair edge
JOB No.: 2711265 **DESIGN:** dbs
DATE : 15-Jun-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm	
Overhang:(Beam cc):	100 mm	ll Cl 13.4.1.3
Adj. Beam Centres :	1718 mm	
Stud Edge Distance	130 mm	
Beam Size	460UB67	
Beam Yield Stress	300 MPa	
Precamber	25 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	double
Deck Span :	2332 mm
Slab Thickness :	130 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	41
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	9.93 KN/m
Super Live Load :	0.00 KPa
Other Super. LL :	9.21 KN/m
Construction LL:	
i) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	26 KNm
Depend. Bending Strength	76 KNm

Stage II : Concrete Placement

Factored Bending Moment :	95 KNm
Depend. Bending Strength	397 KNm

% Composite Action 78 %

Bending Strength

Bending Strength Required :	592 KNm	
Composite Bending Strength :	658 KNm	ll Cl 13.4.1.3
Non-Composite Bending Strength:	397 KNm	

Shear Strength

Shear Strength Required:	191 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	239 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-9.9 mm
SDL+φs LL+Shrink+Creep:	67.1 mm
Span/DF : Steel Immediate:	-1255
Span/DF : Top of Concrete:	185
Span/DF : Steel Total:	217

Vibration

Natural Frequency :	3.48 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.24 mm
Initial Amplitude from Heel Drop	0.050 mm
Damping Required :	2.7 %

FRR: Fire Resistance Rating

Composite Section Rating:	14.3 min
Non-Composite Section Rating:	5.2 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,433

Q300

Ignore as profile has greater width

Ignore as profile has greater width

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queensgate Mall Redevelopment
JOB NO: 2711265/109
SUBJECT: LVL2 Mezzanine beam design Zone ZJ
PAGE NO: 4 OF
BY: Dean Saunders
DATE: 19/5/04

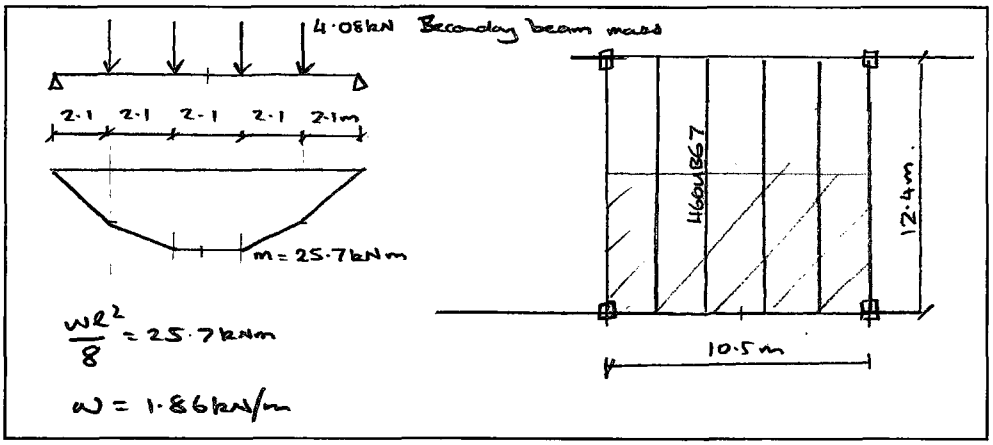
Primary Beams Span = 10.5 m

410UB67 secondary beams supported @ 2.1m centres 4 of

Point load = 67kg*9.81/1000*$Span_{2nd}$/2
= 4.08 kN

Moment = 25.7 kNm

Equiv udl = 1.86 kN/m



COBENZ 97 1.2 a

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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Warehouse LVL2 PAGE :5
SECTION : Primary beams for mezzanine
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 19-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500	mm
Overhang:(Beam cc):	0	mm
Adj. Beam Centres :	12400	mm
Stud Edge Distance	130	mm
Beam Size	530UB82	
Beam Yield Stress	300	MPa
Precamber	25	mm

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond	
Deck Span Type :	triple	
Deck Span :	2100	mm
Slab Thickness :	130	mm
Slab Mass :	2.47	KPa
Concrete Strength :	40.0	MPa
Concrete Density :	2350	kg/m3

Shear Studs

Stud Diameter :	19	mm
Weld.Stud Height :	95	mm
No. of Studs /Beam :	35	@300
Studs /Flange Width:	1	
Stud UTS :	415	MPa

Loading

Super. Dead Load :	0.50	KPa
Other Super. DL :	1.86	KN/m 2ndary beams
Super Live Load :	3.00	KPa
Other Super. LL :	0.00	KN/m
Construction LL:		
I) Deck Placement:	0.40	KPa
II) Conc. Placement:	0.90	KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50	KPa
Damping Available :	4.0	%

Welded Flange / Web

Minimum Plate UTS: Fup	440	MPa
Weld Metal UTS: Fuw	530	MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	77 KNm
Depend. Bending Strength	134 KNm

Stage II : Concrete Placement

Factored Bending Moment :	397 KNm
Depend. Bending Strength	556 KNm

% Composite Action 55 %

Bending Strength

Bending Strength Required :	766 KNm
Composite Bending Strength :	845 KNm
Non-Composite Bending Strength:	536 KNm

Parallel. — II Cl 13.4.1.3

Shear Strength

Shear Strength Required:	292 kN
Dependable Shear Strength:	817 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	250 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	1.8 mm
SDL+φs LL+Shrink+Creep:	30.1 mm
Span/DF : Steel Immediate:	5944
Span/DF : Top of Concrete:	348
Span/DF : Steel Total:	329

Vibration

Natural Frequency :	5.23 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.07 mm
Initial Amplitude from Heel Drop	0.138 mm
Damping Required :	3.5 %

FRR: Fire Resistance Rating

Composite Section Rating:	15.2 min
Non-Composite Section Rating:	8.1 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.35 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.35 KN/mm
Weld Leg Reqd: SP	Single side	8 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,457

241059

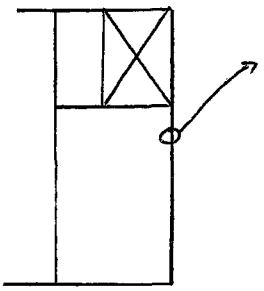
Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queensgate Mall Redevelopment
JOB NO: 2711265/109
SUBJECT: LVL2 Mezzanine beam design Zone ZJ
PAGE NO: 9 OF
BY: Dean Saunders
DATE: 19/5/04

External Secondary beam assume non-composite action due to stair void

span = 12.4 m



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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Warehouse LVL2 PAGE 3 10 of
SECTION : External Secondary beams with stair support beam
JOB No.: 2711265/109 **DESIGN:** dws
DATE : 20-May-04 **CHECK :**

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	12400 mm	
Overhang:(Beam cc):	0 mm	II Cl 13.4.1.3
Adj. Beam Centres :	2600 mm	
Stud Edge Distance	130 mm	
Beam Size	460UB67	
Beam Yield Stress	300 MPa	
Precamber	25 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2600 mm
Slab Thickness :	130 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	24 @300 c/s 7.2m portion.
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	0.15 KN/m Stair beams.
Super Live Load :	3.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
I) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	30 KNm
Depend. Bending Strength	76 KNm

Stage II : Concrete Placement

Factored Bending Moment :	127 KNm
Depend. Bending Strength	397 KNm

% Composite Action 44 %

Bending Strength

Bending Strength Required :	229 KNm	
Composite Bending Strength :	~~569 KNm~~	II Cl 13.4.1.3
Non-Composite Bending Strength:	397 KNm	

Shear Strength

Shear Strength Required:	74 kN
Dependable Shear Strength:	624 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	104 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	-5.0 mm
SDL+φs LL+Shrink+Creep:	26.9 mm
Span/DF : Steel Immediate:	-2460
Span/DF : Top of Concrete:	461
Span/DF : Steel Total:	567

Vibration

Natural Frequency :	5.79 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.22 mm
Initial Amplitude from Heel Drop	0.095 mm
Damping Required :	3.3 %

FRR: Fire Resistance Rating

Composite Section Rating:	22.4 min
Non-Composite Section Rating:	19.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm
Bottom Flange:vwb: VdepQb/Is		1.25 KN/mm
Weld Leg Reqd: SP	Single side	7 mm
	Double side	4 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,374

241059

Calculation Sheet

Job name: Queensgate mall Redevelopment — Job No: 2711265/109
Subject: Mezzanine floor design LVL2 Zone 3 — Page No: 11 of
By: DJS — Date: 20-5-04

Parts & Portions for crossbrace.

$R_p = 1.1$

W_p = Area × (Func + LL + Cdx) + steelwork. = 2057 kN (0.60)

$C_{fi} = C_{fm}$

$C_{fm} = \frac{C_h(T_1, \mu_0)}{C_L(T_1, \mu)} \times \frac{F_n}{W_n}$ $\quad \mu_0 = 3$, $\mu = 6$, $T = 0.1$ sec

$= \frac{0.37}{0.49} \times \frac{7493\text{kN}}{48,976\text{kN}}$

$= 0.298$

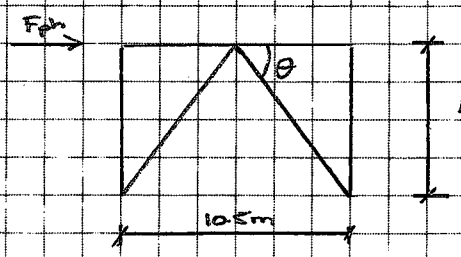
$C_{pi} = C_h(T_{pc}, \mu_p) \cdot C_{fi} / 0.4$

$= 0596.$

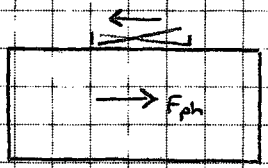
$F_{ph} = C_{ph} W_p R_p$

$= 0.596 \times 2057\text{kN} \times 1.1$

$= 1349\text{kN}$



$\theta = 37.6°$ Say $40°$



$N^* = \frac{F_{ph}}{2} \times \frac{1}{\cos\theta}$

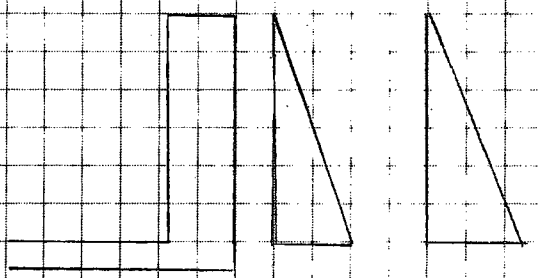
$= 880\text{kN}$

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Soil pr. at rest = $0.5 h \gamma_{soil}$

$\gamma_{soil} = 18\ kPa/m^3$

$\therefore$ Soil pr at rest $= 0.5 \times 1.5 \times 18 = 13.5\ kPa$

Water pr $= h\gamma_{water} = 1.5 \times 10 = 15\ kPa$

Ultimate Pr $= 1.2 \times 13.5 + 1.4 \times 15 = 37.2\ kPa$

Force $= \frac{1.5 \times 37.2}{2} = 27.9\ kN \Rightarrow$

$M_u = 27.9 \times \frac{1.5}{3} = 13.95\ kN\text{-}m$

$b = 1000 \quad h = 400 \quad d = 400 - 75 - 10/2 = 320$

$R_u = \frac{M_u}{\phi b d^2} = \frac{14 \times 10^6}{.85 \times 1000 \times 320^2} = 0.16084$

$m = \frac{f_y}{.85 f'_c} = \frac{250}{.85 \times 25} = 11.76$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 R_u m}{f_y}}\right) = 0.000645$

$\rho_{min} = \frac{\sqrt{f'_c}}{4 f_y} = 0.005$

$A_{s\,req} = \rho_{min} b d = 1600$

241059

Use 2 R12-125 links

Th A_s for External legs of 2 links

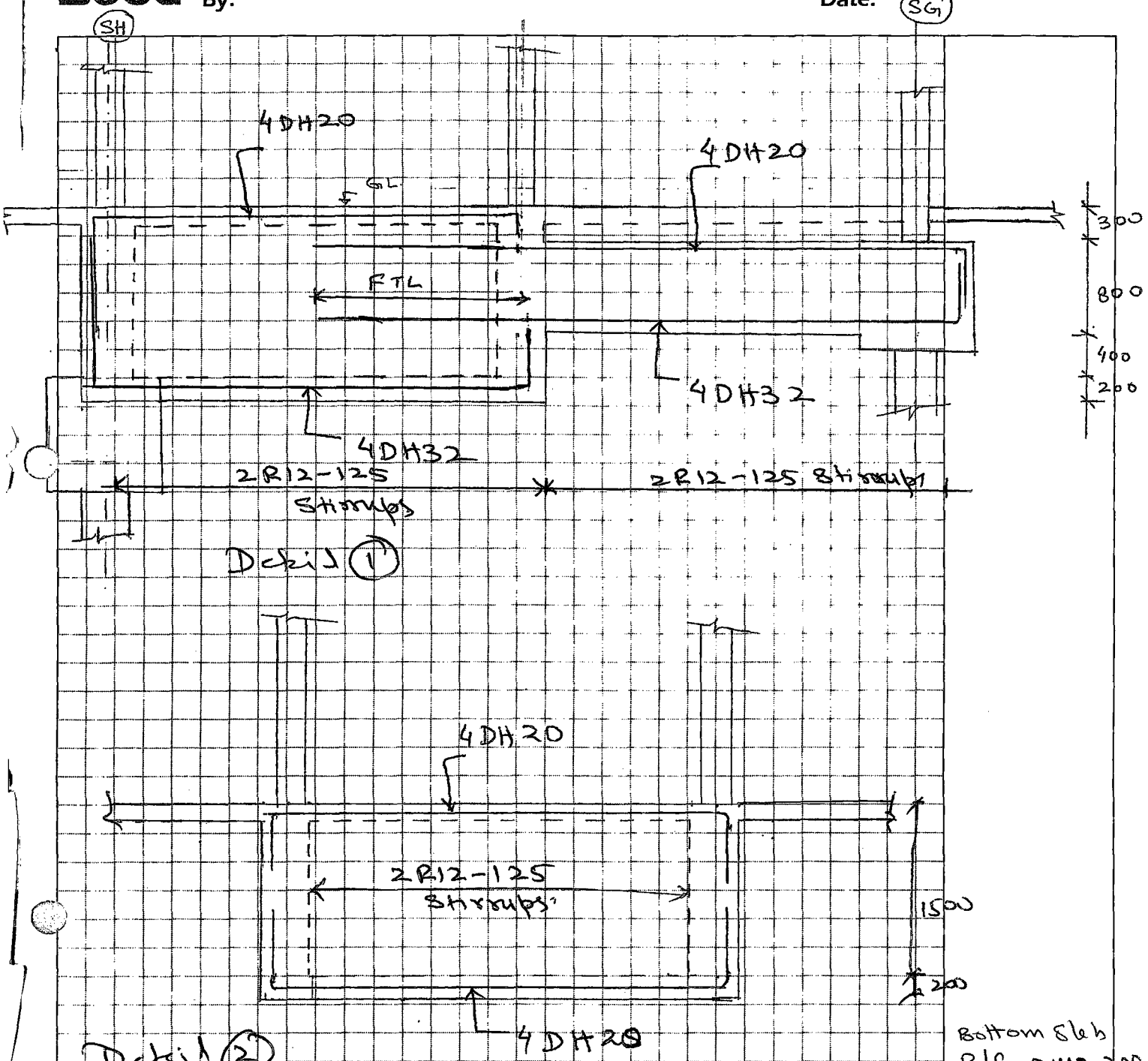
$= \frac{113 \times 2 \times 1000}{125} = 1808 > 1600$ OK

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Bottom Slab R/f DH12-200 Both Ways Top & Bott

Bottom Slab Design:

$w_u = 15 \times 1.4 = 21$ kPa $\quad l_x = 3.5 \quad l_y = 3.5$

$M_{ux} = M_{uy} = 0.056\, w_u l_x^2 = 14.406$ kN-m

$b = 1000 \quad h = 200 \quad d = 200 - 50 - 12/2 = 145$

$R_u = \frac{M_u}{\phi b d^2} = 0.806 \quad \rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2Rum}{f_y}}\right)$

$\rho = 0.00164 \quad \rho_{min} = \frac{0.7}{f_y} > 0.0014$

$A_s = \rho b d = 237.8$ mm^2/m $\quad$ Use DH12-200 Top & Bott

$A_{s,prov} = 390$ mm^2/m. ok

Calculation Sheet

Job name: Queenstown mall redevelopment
Subject: Mezzanine floor LVL2 Zone 23
By: DOS

Job No: 2711265/109
Page No: 12 of
Date: 20-5-04

$N^* \leq \phi N_s$
$\leq \phi N_c$

Assume pin-ended $k_e = 1$

$L_e = L$
$= 6.63m$

roof bracing shift 25/4/05

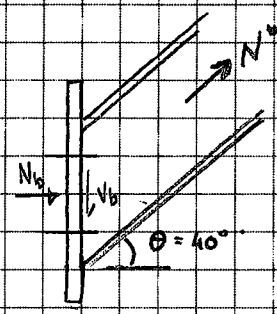
Say $L_e = 7m$

1024kN

Require 250UC 72.9 for $N^* = 880kN$ $L_e = 7m$ 250UC 72.9

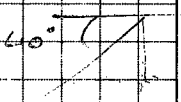
$\phi N_c = 1070kN$

Connection.



$V_b^* = \sin 40 N^*$
$= 566kN$ 65%

$N_b^* = \cos 40 N^*$
$= 674kN$ 78%



$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_f^*}{\phi N_{sf}}\right)^2 \leq 1.0$$

Say 4 bolts

Try M20. $\left(\frac{566}{4\times 92.6}\right)^2 + \left(\frac{674}{4\times 163}\right)^2 \leq 1.0$ ✗

Try M30 $\left(\frac{566}{4\times 214}\right)^2 + \left(\frac{674}{4\times 373}\right)^2 \leq 1.0$ $\left(\frac{655}{4\times 214}\right)^2 + \left(\frac{784}{4\times 373}\right)^2 \leq 1.0$

$0.4372 + 0.2041 \leq 1.0$ $0.867 \leq 1.0$ ✓

$0.6413 \leq 1.0$ ✓

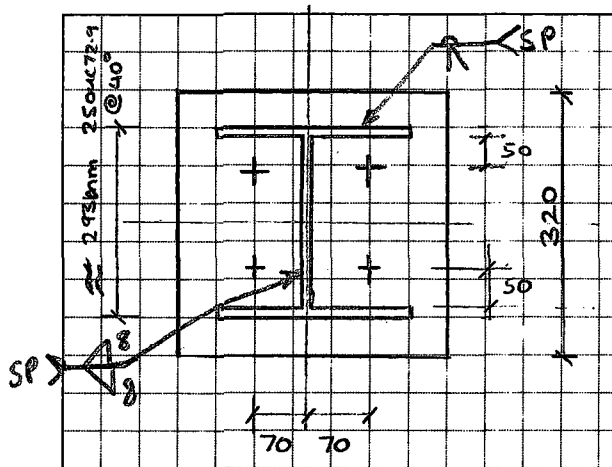
4M30.

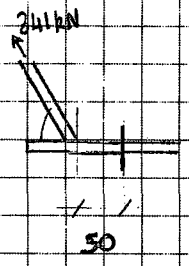
241059

Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Mezzanine floor LVL7 Zone 25
By: DWS.

Job No: 2711265/109
Page No: 13 of
Date: 24-5-04.





$M = 341kN \times 0.05m$
$= 17.05kNm$

Checks 25/4/05

$M = 396 \times \cos 40°$
$= 303kN.$

ok.

303.

$M_{p^*} \leq \phi z_e f_y$

$\leq 0.9 \times \frac{bd^2}{6} \times f_{yb}$

$17.05kNm \leq 0.9 \times \frac{275 \times 40^2}{6} \times 280$

Plate 320 x 275 x 40

$17.05 \leq 18.48kNm$ ✓

$V_b = 3.2 t_p d f_{up}$

$= 3.2 \times t_p \times 30 \times 440.$

$V^* \leq \phi V_b$

$142kN \leq 4224 t_p \times 0.9$

$t_p \geq 3.8mm$ ok. ✓

$V_p = 9 e t_p f_p$

$V^* \leq \phi V_p$

$141.5kN \leq 0.9 \times 45 \times t_p \times 440$

$t_p \geq 7.9mm$

t_p of Column $\geq 8mm$

BWM weld flanges

$f_w = 0.6 \times 14.2 \times 410.$
$= 3.49 kN/mm$

$L = 254mm$

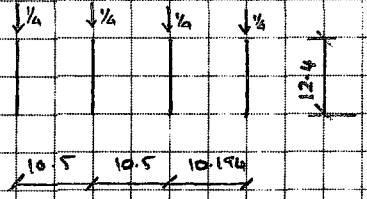
$\phi N_w = 0.8 \times 3.49 \times 254.$
$= 709kN.$

Calculation Sheet

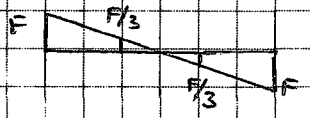
Job name: Queensgate mall Redevelopment
Subject: Mez2 LVL2 2I
By: DWS

Job No: 2711265 /109
Page No: 14 of
Date: 25-8-04

$F_{ph} = 1349$ for $\mu_p = 1$



$M_e = 31.196 \times 0.1 \times 1349\text{kN}$
$= 4208\text{kNm}$



$F \times 31.196 + F/3 \times 10.5 = 4208\text{kNm}$

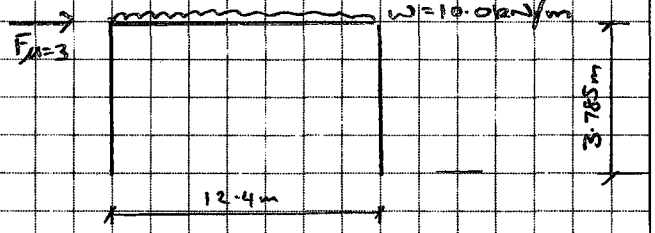
$F = 121\text{kN}$

$\frac{F_{ph}}{4} = 337\text{kN}$

$F_{parallel} = 337\text{kN} + 121\text{kN}$
$= 458\text{kN}$

use $\mu = 3$

$F_{\mu=3} = 153\text{kN}$.



$M^* = 378\text{kNm}$
$N^* = 153\text{kN}$

241059

2711265/109

150f19

14-16

NODE COORDINATES (m)

Node	X Coord	Y Coord	Z Coord
1	0.000	0.000	0.000
2	0.000	3.785	0.000
3	12.400	3.785	0.000
4	12.400	0.000	0.000

MEMBER DATA (deg,kNm/rad,m)
(F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	2	1	FFFFFF	FFFFFF	3.785
2	0.00			Norm	2	3	1	1	FFFFFF	FFFFFF	12.400
3	0.00			Norm	3	4	2	1	FFFFFF	FFFFFF	3.785

NODE RESTRAINTS (kN/m,kNm/rad)
(F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFDDDR						
4	FFDDDR						

NODE LOADS (kN,kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	2	76.300	0.000	0.000	0.000	0.000	0.000
	3	76.300	0.000	0.000	0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	2	1	GI	0.000%	100.000%	0.000	-10.000	0.000
						0.000	-10.000	0.000

COMBINATION LOAD CASES

Load case 3: G+Qu+E

1.000 * Load case 1: EQ
1.000 * Load case 2: G+Qu

LOAD CASE TITLES

Load Case	Title
1	EQ
2	G+Qu
3	G+Qu+E

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Linear): EQ

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	-46.580	76.300	0.000	0.000	0.000	0.000
	2	-46.580	76.300	0.000	0.000	0.000	288.796
2	2	0.000	-46.580	0.000	0.000	0.000	288.796
	3	0.000	-46.580	0.000	0.000	0.000	-288.796
3	3	46.580	76.300	0.000	0.000	0.000	-288.796
	4	46.580	76.300	0.000	0.000	0.000	0.000

Load case 2 (Linear): G+Qu

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	62.000	-23.534	0.000	0.000	0.000	0.000
	2	62.000	-23.534	0.000	0.000	0.000	-89.077
2	2	23.534	62.000	0.000	0.000	0.000	-89.077
	3	23.534	-62.000	0.000	0.000	0.000	-89.077
3	3	62.000	23.534	0.000	0.000	0.000	-89.077
	4	62.000	23.534	0.000	0.000	0.000	0.000

Load case 3 (Linear): G+Qu+E

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	15.420	52.766	0.000	0.000	0.000	0.000
	2	15.420	52.766	0.000	0.000	0.000	199.718
2	2	23.534	15.420	0.000	0.000	0.000	199.718
	3	23.534	-108.580	0.000	0.000	0.000	-377.873
3	3	108.580	99.834	0.000	0.000	0.000	-377.873
	4	108.580	99.834	0.000	0.000	0.000	0.000

NODE REACTIONS (kN,kNm)

Load case 1 (Linear): EQ

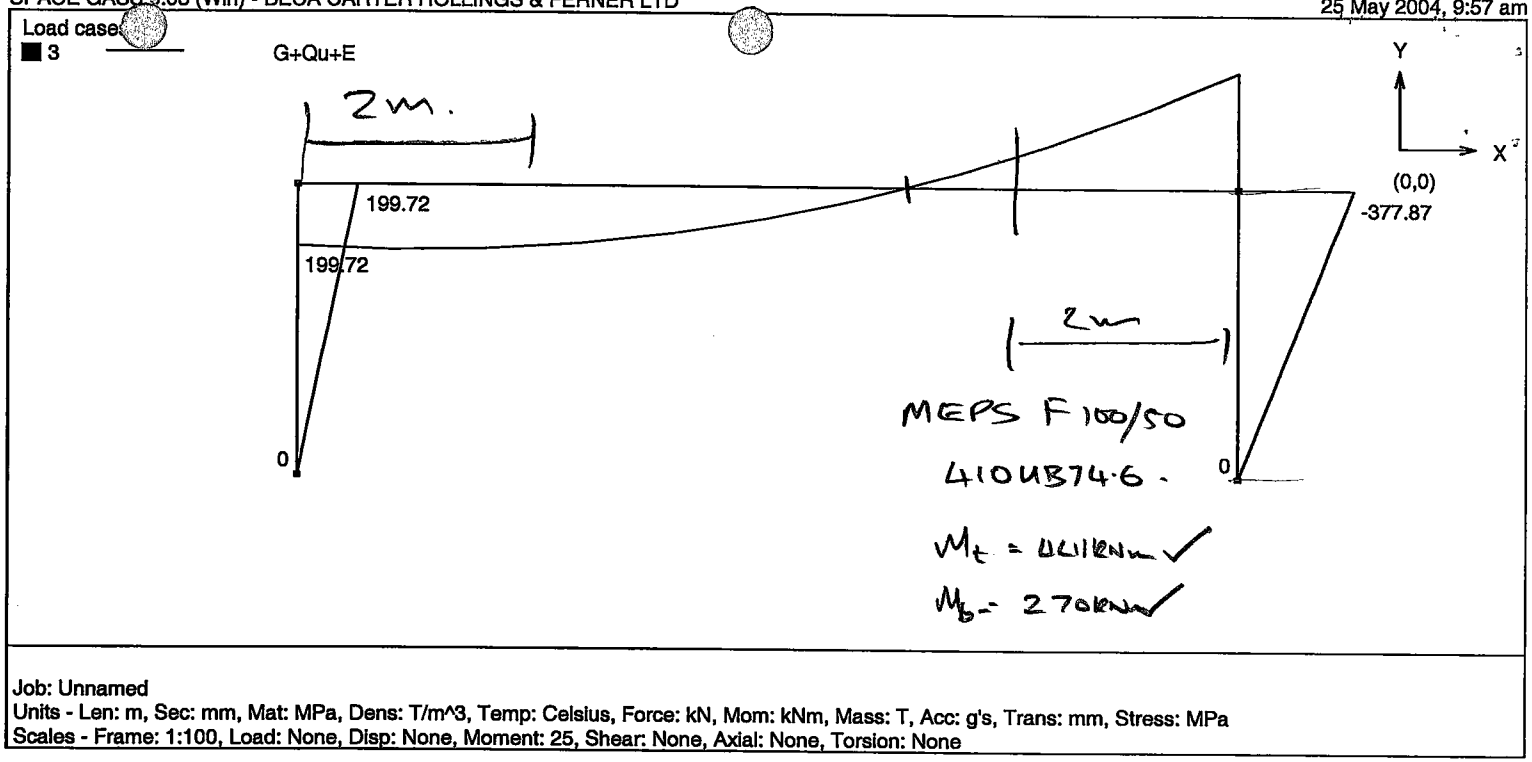
Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-76.300	-46.580	0.000	0.000	0.000	0.000
4	-76.300	46.580	0.000	0.000	0.000	0.000
Load	152.600	0.000	0.000			
Reac	-152.600	0.000	0.000			

Load case 2 (Linear): G+Qu

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	23.534	62.000	0.000	0.000	0.000	0.000
4	-23.534	62.000	0.000	0.000	0.000	0.000
Load	0.000	-124.000	0.000			
Reac	0.000	124.000	0.000			

Load case 3 (Linear): G+Qu+E

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	-52.766	15.420	0.000	0.000	0.000	0.000
4	-99.834	108.580	0.000	0.000	0.000	0.000
Load	152.600	-124.000	0.000			
Reac	-152.600	124.000	0.000			



Job: Unnamed
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:100, Load: None, Disp: None, Moment: 25, Shear: None, Axial: None, Torsion: None

378 kN-m

241059

271265 | 109
16/11/9 14-17

Calculation Sheet

Job name: Queensgate.
Subject: 25 Mezz w/H.
By: DWS

Job No: 2711265/109
Page No: 17 of
Date: 26-5-04

Engua $L_e = 2m$ max

$\phi M_b = 401 kNm$ for 460UB 74.6

$M^* \leq \phi M_b$.

$378 \leq 401 kNm$ ✓

$\phi V_v = 719 kN$.

$V^* = 108 kN$ /

no studs over $1.5 d_b = 700mm$.

$wt \geq \frac{d_1}{82}$ ok. /

Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Mezzanine LVL2. Zone J
By: DWS

Job No: 2711265/109
Page No: 18 of 19
Date: 27-5-04

Axial load = 1000 kN Compression.

L_e = 2.1m

Try 800WB122

ϕN_{cy} = 2650kN

$\left(1 - \frac{N^*}{\phi N_{cy}}\right)$

$= 1 - \frac{1000}{2650}$

$= 0.623$

Try 700WB115

ϕN_c = 2650

$\left(1 - \frac{N^*}{\phi N_{cy}}\right)$

$= 0.623$

Cohesive compositk along M^* = 797 kNm

$\phi M_y = 1457 \times 0.623$

$= 908.1$ kNm ok. ✓

use 700WB115

V^* = 332 kN
N^* = 1000 kN

241059

COBENZ 97 1.2 a
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USER : Beca Carter Hollings & Ferner Ltd: Wellington
JOB NAME : Queensgate Zone ZJ LVL2 Mezz PAGE :5
SECTION : Primary beams on SA
JOB No.: 2711265/109 DESIGN: dws
DATE : 1-Jun-04 CHECK :

COMPOSITE SPANDREL BEAM LIMIT STATE DESIGN : UNPROPPED

15.) DESIGN SUMMARY

Beam is an spandrel beam , equal flange I or Channel ,simply supported and unpropped during construction.
Decking spans perpendicular to beam span .If decking parallel set stud edge distance equal to slab thickness.
Where decking spans perpendicular to the beam and cantilevers clear over it by less than 550mm, then bending strength must be provided by the non-composite steel beam only. Deflection may be calculated using the composite section with a minimum of 25% composite action.
Refer cl 13.4.1.3 NZS3404:Part:1997

INPUT DATA

Beam Layout and Properties

Beam Span :	10500 mm	
Overhang:(Beam cc):	100 mm	II Cl 13.4.1.3
Adj. Beam Centres :	12400 mm	
Stud Edge Distance	160 mm	
Beam Size	700WB115	
Beam Yield Stress	300 MPa	
Precamber	0 mm	

Decking and Slab Properties

Deck Type :	0.75 Hi-Bond
Deck Span Type :	Triple
Deck Span :	2100 mm
Slab Thickness :	160 mm
Slab Mass :	2.47 KPa
Concrete Strength :	40.0 MPa
Concrete Density :	2350 kg/m3

Shear Studs

Stud Diameter :	19 mm
Weld.Stud Height :	95 mm
No. of Studs /Beam :	35 @300
Studs /Flange Width:	1
Stud UTS :	415 MPa

Loading

Super. Dead Load :	0.50 KPa
Other Super. DL :	1.86 KN/m 2 rocky beams .
Super Live Load :	3.00 KPa
Other Super. LL :	0.00 KN/m
Construction LL:	
I) Deck Placement:	0.30 KPa
ii) Conc. Placement:	0.90 KPa

Load Factors

φu: Ultimate :	0.60
φl: Long Term:	0.70
φs: Short Term :	1.00

Deflections

Shrinkage Strain :	0.0003

Vibration

Long Term LL:	0.50 KPa
Damping Available :	4.0 %

Welded Flange / Web

Minimum Plate UTS: Fup	440 MPa
Weld Metal UTS: Fuw	530 MPa

RESULTS

Stage I : Deck Placement

Factored Bending Moment :	69 KNm
Depend. Bending Strength	295 KNm

Stage II : Concrete Placement

Factored Bending Moment :	422 KNm
Depend. Bending Strength	1023 KNm

% Composite Action 38 %

Bending Strength

Bending Strength Required :	797 KNm	x0.623 = 908 ✓
Composite Bending Strength :	1457 KNm	II Cl 13.4.1.3
Non-Composite Bending Strength:	1023 KNm	

Shear Strength

Shear Strength Required:	304 kN
Dependable Shear Strength:	1121 kN

Unshored Bottom Flange Tension

Serviceability Flange Stress:	144 MPa
Allow. Bottom Flange Stress:	270 MPa

Deflections (Assuming Level Screeding)

Net Steel Beam Immed.:	12.0 mm
SDL+φs LL+Shrink+Creep:	16.3 mm
Span/DF : Steel Immediate:	873
Span/DF : Top of Concrete:	643
Span/DF : Steel Total:	370

Vibration

Natural Frequency :	7.74 Hz
Deflection Due to 1.0 kN Point Load at Midspan :	0.03 mm
Initial Amplitude from Heel Drop	0.083 mm
Damping Required :	3.4 %

FRR: Fire Resistance Rating

Composite Section Rating:	21.1 min
Non-Composite Section Rating:	17.5 min

Welded Flange / Web

Top Flange:vwt: VdepQt/Is		1.46 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm
Bottom Flange:vwb: VdepQb/Is		1.46 KN/mm
Weld Leg Reqd: SP	Single side	9 mm
	Double side	5 mm

Comparative Cost

Steel/kg :	$1.55
Shear Stud :	$3.50
Comparative Cost Factor:	$1,994

Calculation Sheet

Job name: Queensgate mall — Job No: 2711265/109

Subject: Zone 7 LVL 2 Floor beam Connection — Page No: 10 of

By: D.J.S. — Date: 3-6-04

Mezzanine Connections

460UB75

Connect 700WB115

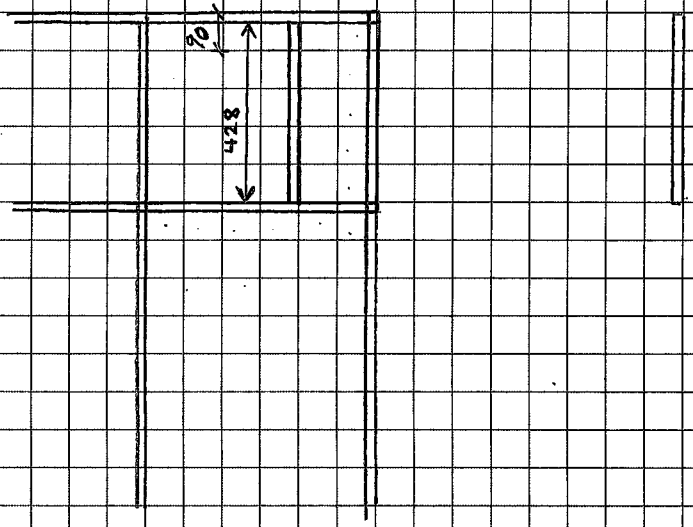
$V^* = 304$ kN + axial load from EQ loads.

$N^* = \frac{1}{3} \times 1349$

$N^* = 450$kN

S30182

→ changed 4/6/04. to 350UC137 — no change to connect Plate. DJS.



Bolts 4@90 ge =

or 5@70 ge =

Bolts

$F^* = \sqrt{N^{*2} + V^{*2}}$

$= \sqrt{450^2 + 304^2}$

$= 543$kN

4M24 8.8 x/s = 4 × 186

$\phi V_{sf\,n\,x/s} = 744$kN

543kN ≤ 744kN ✓

4M24 8.8 x/s.

$V_b = 3.2 \times 10 \times 24 \times 440$

$= 338$kN

$V_p = 45 \times 10 \times 440$.

$= 198$kN

$\phi V_p = 178$kN.

4 × 178 ≥ 543kN.

712 ≥ 543kN ✓

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Calculation Sheet

Job name: Queensgate mall — Job No: 2711265/1009

Subject: Zone 3 LVL2 Perimeter Connection — Page No: 11 of

By: DJOS — Date: 3-6-04

Shear on steel

$V^* \leq \phi V_w$

$300kN \leq 0.9 \times 0.6 \times 300 \times 12 \times (428 - 4 \times 36)$

$300kN \leq 630kN$ ✓

Axial

$N^* \leq \phi N_t$

$450kN \leq 0.9 \times 300 \times 12 (428 - 4 \times 26)$

$\leq 1050kN$ ✓

Combined action

$$\frac{N^*}{\phi N_t} + \left(\frac{V^*}{\phi V_w}\right)^2 \leq 1$$

$$\frac{450}{1050} + \left(\frac{300}{630}\right)^2 \leq 1$$

$0.417 \leq 1$ ✓

check shear transfer:

$\phi F_{shear} = 0.9[0.6 \times 300 \times 10 \times 2 \times 37 + 10 \times 440 \times 3 \times (90 - 26)]$

$= 860kN$ ✓

N^*

Weld use full 6m fillet around shoe, both sides.

$\phi V_{weld} = 0.835 \times 2 \times (428 + 2 \times 90)$

$= 1015kN$

$N^* + V^* = 855kN$

$855kN \leq 1015kN$ ✓

6mm fillet both sides all round shoe as shown

Calculation Sheet

Job name: Diaphragm wall
Job No: 271281/119
Subject: Zone F ever flow box Connection
Page No: 12 of
By: DWJS
Date: 3.6.04

Web check:

$\phi V_{uw} = 0.9 \times 0.6 \times 300 \times 10 \times (3 \times 90 + 2 \times 45 - 4 \times 26)$

$= 414.7\,kN$

$\phi N_t = 0.9 \times 300 \times 10 \times (360 - 4 \times 26)$

$= 691\,kN$

$\left(\frac{304}{414}\right)^2 + \left(\frac{450}{691}\right)^2 \leq 1$

$0.96 \leq 1$ ok so provide double plate. 10 — wt 8 — welds

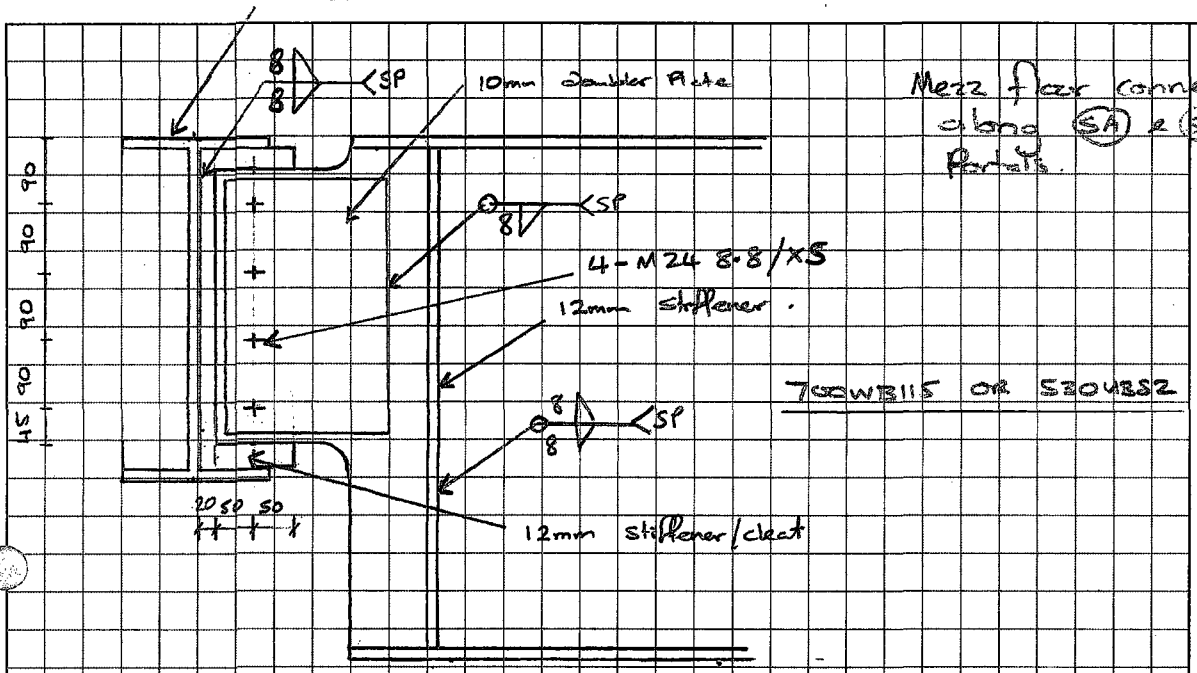
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Calculation Sheet

Job name: _____ Job No: 2711265/109

Subject: _____ Page No: 13 of _____

By: _____ Date: 3-6-04



Mezz floor connections along (5A) & (5B) to be 7WCWB115 OR 530UB82

Detail same both sides and on ends add 12mm stiffener to outer of 45.

CT 10
3/6/04

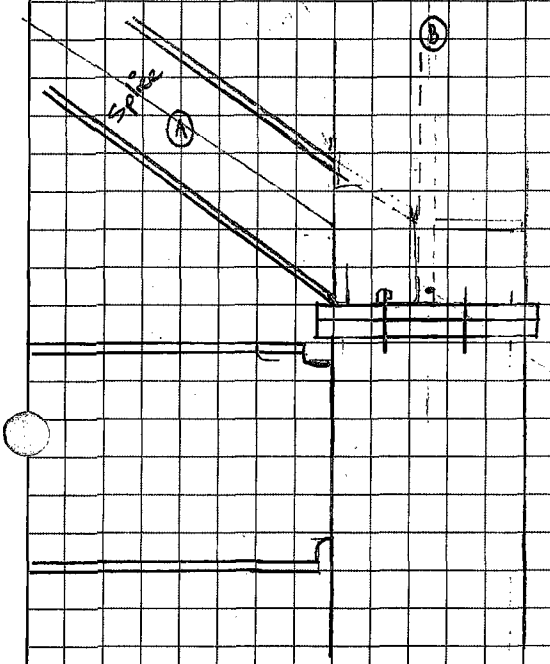
Calculation Sheet

Job name: Queensgate mall — Job No: 2711265/109

Subject: Zone J LVL2 Connections. — Page No: 14 of

By: DJDS. — Date: 4-6-04

Mezzanine Portals. Brace connections.



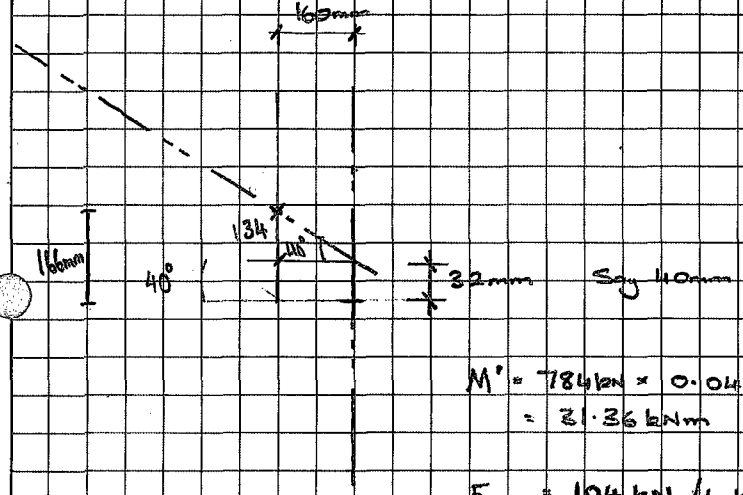
(A) $N^* = 1024\,kN$ T or C

(B) $N^* = 744\,kN$ C
$V^* = 100\,kN$

Loads to column from (A)

$N^* = 658\,kN$ T or C
$V^* = 784\,kN$

Check angles to determine possible moment load from brace.



$M^* = 784\,kN \times 0.04$
$= 31.36\,kNm$

$F_{bolt} = 104\,kN$ /bolt.

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left(\frac{884}{4\times 291}\right)^2 + \left(\frac{104}{373}\right)^2 \leq 1.0$$

$0.65 \leq 1.0$ ✓

$V_b = 3.2 \times 20 \times 30 \times 440$.
$= 844\,kN$.

$V_p = 50 \times 20 \times 440$. $\quad \phi V_p = 396 \geq \frac{884}{4}$ ✓

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Calculation Sheet

Job name: Queensgate — Job No: 2711265/109

Subject: Zone 1 LVL 2 Connections — Page No: 15 of

By: — Date: 4-6-04

weld check for ML2 brace

Flange butt welds:

$\phi N_w = 0.8 \times 0.6 \times 14 \times 410 \times 254\,mm$

$= 700kN$

$2 \times \phi N_w = 1400kN$

$N^* = 1024$ kN ✓

butt weld flanges + 6mm fillet to web both sides.

+ plates 12mm

Stiffeners 8mm

Rivets.

Splice for brace. use friction bolts!

$\mu = 0.35$

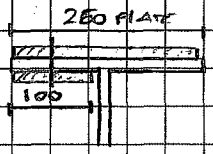
$V^* \leq \phi V_f$ $V^* = 1024kN$.

Bolts... Double shear

M16 $\phi V_f = 51.5kN$

$\frac{1024}{2 \times 51.5} = 10$ bolts. put 5 per flange.

Plates.



$1024kN \leq 0.9 \times 300 \times 2 \times (250 + 2 \times 100 - 4 \times 32) \times 12$

$1024kN \leq 2086kN$ ✓

12mm plates

$N^* \leq \phi N_s$.

$\leq \phi N_c = \phi \alpha_c N_s$

$N_s = kr A_o f_y$

$= 1 \times 100 \times 12 \times 300$

$= 360kN$

$l_e = 0.7 \times 150$

$= 105mm$

$r = \sqrt{\frac{12^2}{12}}$

$= 3.46$

$\lambda_n = 30.3$

$\alpha_b = 1$

$\alpha_c = 0.888$

$N^* \leq 0.9 \times 0.888 \times 360$

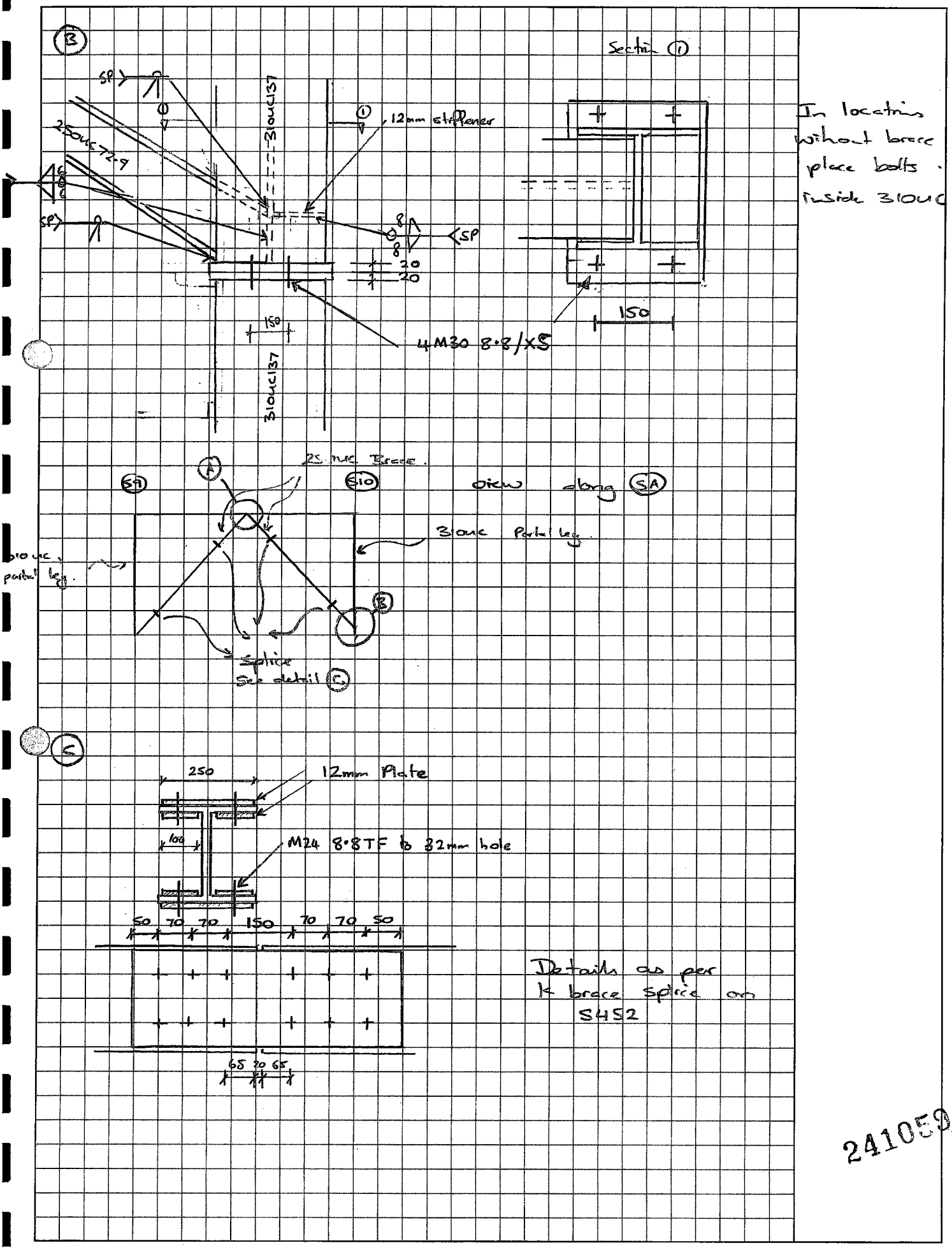
$\leq 287.7kN$.

$N^* = \frac{1024}{8} = 128kN$

$128kN \leq 287kN$ ✓

Calculation Sheet

Job name: Queensgate
Job No: 2711265/109
Subject: Zone 3 LVL7 Connection
Page No: 16 of
By: [illegible]
Date: 4-6-04



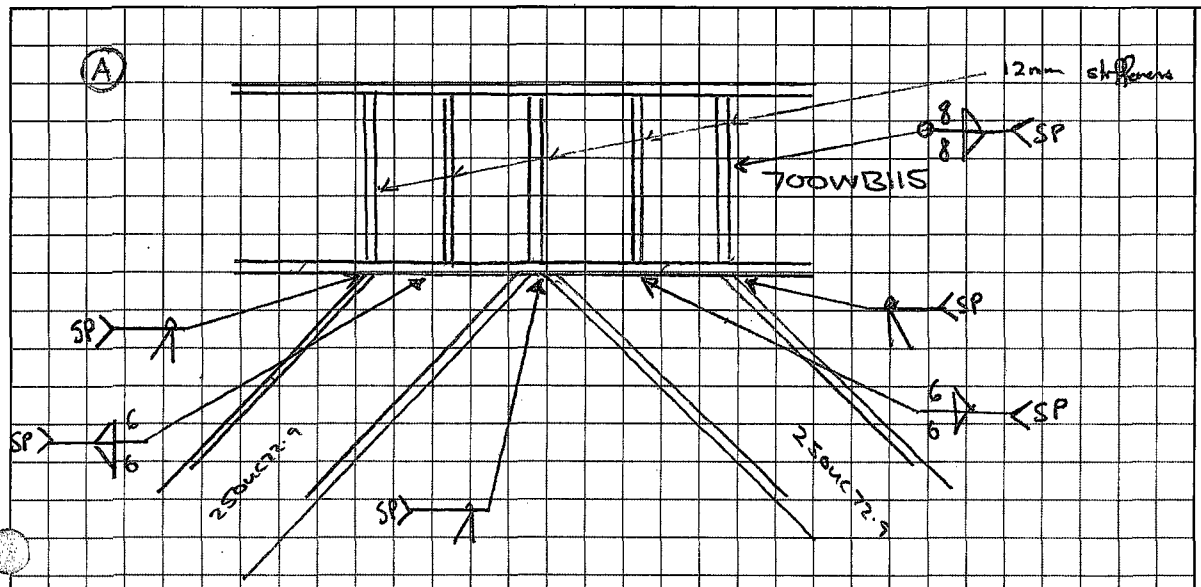
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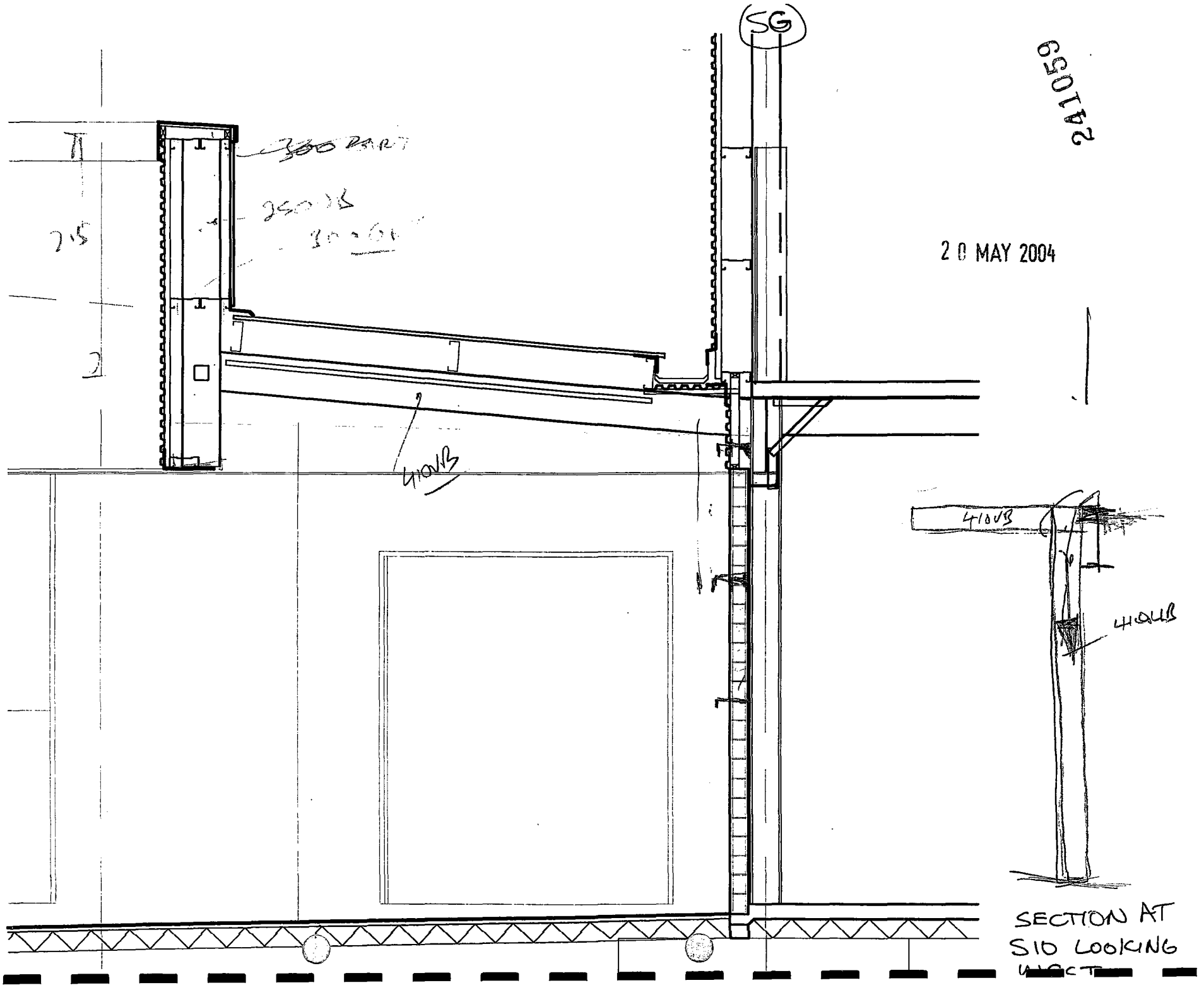
Calculation Sheet

Job name: Queensgate **Job No:** 2711265/109

Subject: Zone J LVL2 Connections **Page No:** 17 of

By: DWS. **Date:** 4-6-04





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20 MAY 2004

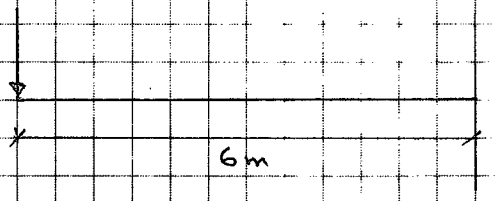
SECTION AT S10 LOOKING

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Loading

1) Roofing = 0.25 KPa, including decking, purlins and lighting services

2) Live load = 0.25 KPa

$W_u = 1.2G + 1.6Q = 0.7$ KPa

Panel width 10.5m.

$W_u = 0.7 \times 10.5 = 7.35$ KN/m

Point load = $0.25 \times 3.5 \times 10.5 = 9.1875$

$P_u = 1.2 \times 9.1875 = 11.025$ KN

$M_u = 11.025 \times 6 + 7.35 \times \frac{6^2}{2} = 198.45$ KN-m.

Use 410 UB 53.7

$M_{ux} = 206$ KN-m. with one fly brace at 3m.

Wind load —

Ultimate upward wind = -1.35 KPa

$0.9G + W_{u\,up} = 0.9 \times .25 - 1.35 = -1.125$ KPa

$W_u = -1.125 \times 10.5 = -11.8125$ KN/m

$M_u = -11.8125 \times \frac{6^2}{2} = -212.62$ KN-m

For this case the bottom flange will be in tension and top in compression.
The Top flang will be restrained by purlins @ 2.0m c/c
∴ L_b for this case is 2.0m.

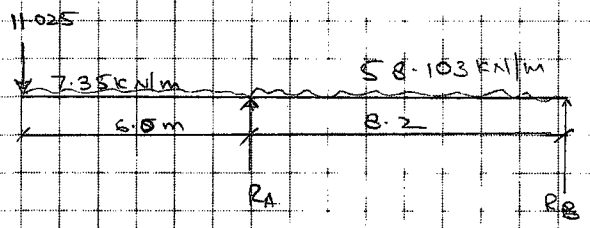
$M_{ux} = 250$ KN-m $L_b = 2.0$m ok.

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Loading:

A) Cantilever Portion

Roofing = 0.25 including decking + lighting

Live Load = 0.25 kPa

$W_u = 1.2G + 1.6Q = 0.7$ kPa

Panel width = 10.5 m.

$W_u = 0.7 \times 10.5 = 7.35$ KN/m.

Point load from Perpet wall

$P_u = 1.2 \times (0.25 \times 3.5) \times 10.5 = 11.025$ KN.

B) Deck Beam.

Dead Load

i) Slab = 3.17 kPa

ii) SDL = 0.5 kPa

$G = 3.67$ kPa

Live Load =

$Q = 14.54$ kPa

$W_u = 1.2G + 1.6Q = 27.66$ kPa

Panel width = 2.1 m

$W_u = 58.103$ KN/m

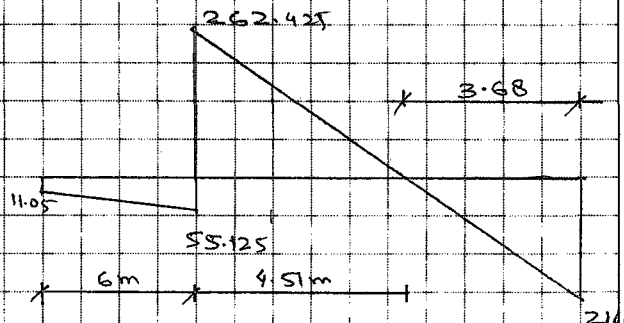
$\Sigma M @ B = 0 \quad (\curvearrowright -ve)$

$8.5 R_A = 11.025 \times 14.2 + 7.35 \times 6 \times (3 + 8.2) + 58.103 \times 8.2 \times \frac{8.2}{2}$

$R_A = 317.55$ KN

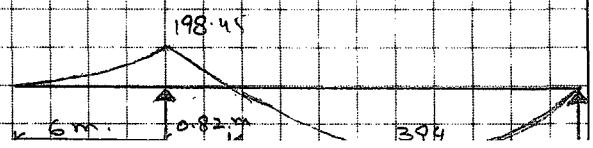
$R_B = 11.025 + 7.35 \times 6 + 58.103 \times 8.2 - 317.55$

$R_B = 214.02$ KN



-ve $M_u = 198.45$ KN-m

+ve $M_u = 393.79$ KN-m



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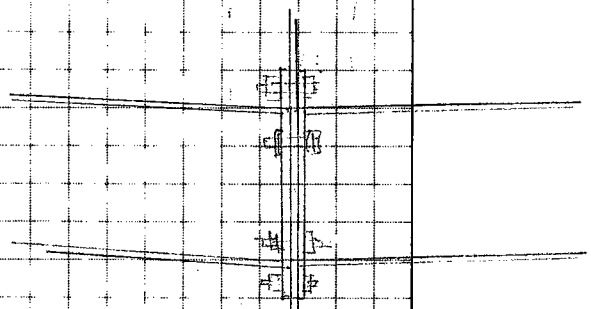
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Connection



$M_u = 198 \simeq 206$ kN-m

$V_u = \simeq 265$ kN-m

Assuming the Bottom Flange of the Beam is in compression and compressive force is acting at ½ of flange.

Lever arm from Top Flange to Bolt is 410 mm

$l_a = 410$ mm

$T = \frac{M}{l_a} = 502.4$

Using 4 Bolts at top, the tension per bolts = 125.61 kN

Total Shear $V_u = 265$ kN/m

Shear Per bolts $= \frac{265}{8} = 33.125$ kN

The axial capacity of 20 mm G8.8 Bolts is 163 kN
The Shear capacity of 20mm G8.8 Bolt is 92.6 kN.

Bolts Subject to combined Shear & axial tension

$$\left(\frac{V_f}{\phi V_f}\right)^2 + \left(\frac{N_{tf}}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left(\frac{33.125}{92.6}\right)^2 + \left(\frac{125.61}{163.0}\right)^2 \leq 1.0$$

$0.7218 \leq 1.0$ Okay.

The connection is similar to

MEP- G1 50/25 connection for

410 UB 53.

$M_{up} = 216$ kN o/c.y

$V_{up} = 325$ kN

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265027

SUBJECT: Zone J, Canopy Cantilever Beam (0.9G + Wu wind Load Case) — PAGE NO: 1 OF 3

BY: Jamil Khan — DATE: 11/6/04

DESIGN DATA

Enter trial section size here 410 UB 53.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	213	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	2.000	m
Length between Y axis restraints	L_y =	2.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	1060000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	173000	mm^3
Moment of inertia about X axis (table)	I_x =	188000000	mm^4
Moment of inertia about Y axis (table)	I_y =	10300000	mm^4
Torsion constant (table)	J =	234000	mm^4
Warping constant (table)	I_w =	3.94E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	6890	mm^2
Radius of gyration about X axis (table)	r_x =	165	mm
Radius of gyration about Y axis (table)	r_y =	38.6	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

241059

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	213 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	2.000 m
Length between Y axis restraints	L_y =	2.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \phi M_{sx}$ and
$M_x^* \le \phi M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \phi M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \phi M_{sx}$
$\le \phi f_y Z_{ex}$

Strength reduction factor	ϕ =	0.9
Yield stress of flanges	f_y =	300 MPa
Effective section modulus about X axis (table)	Z_{ex} =	1060000 mm^3
	ϕM_{sx} =	**286 kNm**

M_x^*		ϕM_{sx}		
213 kNm	≤	**286 kNm**	∴ **O.K.**	**410 UB 53.7**

$M_y^* \le \phi M_{sy}$
$\le \phi f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	173000 mm^3
	ϕM_{sy} =	**47 kNm**

M_y^*		ϕM_{sy}		
0 kNm	≤	**47 kNm**	∴ **O.K.**	**410 UB 53.7**

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265027
SUBJECT: Zone J, Canopy Cantilever Beam (0.9G + Wu wind Load Case)
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$
$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00
Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs
$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}
$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	E =	200 GPa
Moment of inertia about Y axis (table)	I_y =	10300000 mm^4
Shear modulus of elasticity	G =	80 GPa
Torsion constant (table)	J =	234000 mm^4
Warping constant (table)	I_w =	3.94E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00
	L_{ex} =	2.000 m
	M_{oa} =	1040 kNm
	αs =	0.8718186
	$\varnothing M_{bx}$ =	**250 kNm**

$\varnothing M_{bx}$		$\varnothing M_{sx}$			
250 kNm	≤	**286 kNm**	∴	**Ø Mbx =**	**250 kNm is critical**

M_x^*		Ø Mbx =		
213 kNm	≤	**250 kNm**	**∴ O.K.**	**410 UB 53.7**

Section size	410 UB 53.7	O.K.

Use
410 UB 53.7

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DESIGN DATA

Enter trial section size here 410 UB 53.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	198.45 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	3.000 m
Length between Y axis restraints	L_y =	3.000 m
Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Young's modulus	E =	200 GPa
Shear modulus of elasticity	G =	80 GPa
Effective section modulus about X axis (table)	Z_{ex} =	1060000 mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	173000 mm^3
Moment of inertia about X axis (table)	I_x =	188000000 mm^4
Moment of inertia about Y axis (table)	I_y =	10300000 mm^4
Torsion constant (table)	J =	234000 mm^4
Warping constant (table)	I_w =	3.94E+11 mm^6
Moment modification factor	αm =	1.00
Ratio of smaller to larger bending moment	βm =	0
Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	0 kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250 MPa
Gross area of cross section (use nett if approp.) (table)	A =	6890 mm^2
Radius of gyration about X axis (table)	r_x =	165 mm
Radius of gyration about Y axis (table)	r_y =	38.6 mm
Effective length factor about X axis	k_{ex} =	1
Effective length factor about Y axis	k_{ey} =	1
Form factor (by calculation)	k_f =	0.954
Section constant, α_b from table 6.3.3(1)	α_b =	0

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0 kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85
Nett area of cross section (by calculation)	A_n =	12500 mm^2
Tensile strength	f_u =	410 MPa

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265027
SUBJECT: Zone J, Canopy Cantilever Beam (1.2G+1.6Q Load Case)
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 11/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	198.45 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	3.000 m
Length between Y axis restraints	L_y =	3.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Effective section modulus about X axis (table)	Z_{ex} =	1060000 mm^3
	$\varnothing M_{sx}$ =	**286 kNm**

M_x^*		$\varnothing M_{sx}$		
198 kNm	**≤**	**286 kNm**	**∴ O.K.**	**410 UB 53.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	173000 mm^3
	$\varnothing M_{sy}$ =	**47 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	**≤**	**47 kNm**	**∴ O.K.**	**410 UB 53.7**

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Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment | JOB NO: 2711265027

SUBJECT: Zone J, Canopy Cantilever Beam (1.2G+1.6Q Load Case) | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor	$\alpha m =$	1.00
Ratio of smaller to larger bending moment	$\beta m =$	0

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	10300000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	234000 mm^4
Warping constant (table)	$I_w =$	3.94E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$	3.000 m
$M_{oa} =$	487 kNm
$\alpha s =$	0.7187861
$\varnothing M_{bx} =$	**206 kNm**

$\varnothing M_{bx}$		$\varnothing M_{sx}$			
206 kNm	≤	**286 kNm**	∴	**Ø Mbx =**	**206 kNm is critical**

M_x^*		Ø Mbx =		
198 kNm	≤	**206 kNm**	**∴ O.K.**	**410 UB 53.7**

Section size	**410 UB 53.7**	**O.K.**

Use

410 UB 53.7

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Cantilever beam at GL (S11)

i) Roofing = 0.25 kPa

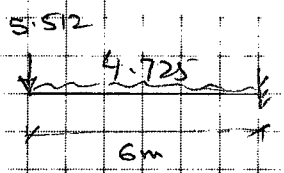
ii) LL = 0.25 kPa

iii) Parapet = 0.25 × 3.5 = 0.875 kN/m

$$w_u = (1.2 \times .25 + 1.6 \times 0.25) \times \frac{10.5}{2} + 0.875 \times 1.2$$

$$w_u = 4.725$$

$$P_u = 0.875 \times 1.2 \times \frac{10.5}{2} = 5.512$$



$$M_u = 5.512 \times 6 + 4.725 \times \frac{6^2}{2} = 118.12 \text{ kN-m}$$

Use 410 UB 59.7

For $L_b = 6m$ $M_{us} = 129$ kN-m.

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265027

SUBJECT: Zone J, Canopy Cantilever Beam @ (S11) (1.2G+1.6Q Load Case) — PAGE NO: 1 OF 3

BY: Jamil Khan — DATE: 15/6/04

DESIGN DATA

Enter trial section size here 410 UB 59.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	118	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	6.000	m
Length between Y axis restraints	L_y =	6.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	1200000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	203000	mm^3
Moment of inertia about X axis (table)	I_x =	216000000	mm^4
Moment of inertia about Y axis (table)	I_y =	12100000	mm^4
Torsion constant (table)	J =	337000	mm^4
Warping constant (table)	I_w =	4.67E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	7640	mm^2
Radius of gyration about X axis (table)	r_x =	168	mm
Radius of gyration about Y axis (table)	r_y =	39.7	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	$M_x^* =$	118 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	6.000 m
Length between Y axis restraints	$L_y =$	6.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing =$	0.9
Yield stress of flanges	$f_y =$	300 MPa
Effective section modulus about X axis (table)	$Z_{ex} =$	1200000 mm^3
	$\varnothing M_{sx} =$	**324 kNm**

M_x^*		$\varnothing M_{sx}$		
118 kNm	**≤**	**324 kNm**	**∴ O.K.**	**410 UB 59.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	$Z_{ey} =$	203000 mm^3
	$\varnothing M_{sy} =$	**55 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	**≤**	**55 kNm**	**∴ O.K.**	**410 UB 59.7**

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5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$
$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00
Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs
$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}
$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	12100000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	337000 mm^4
Warping constant (table)	$I_w =$	4.67E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 6.000 m
$M_{oa} =$ 187 kNm
$\alpha s =$ 0.398072

$\varnothing M_{bx} =$ 129 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
129 kNm	≤	**324 kNm**	∴	**Ø Mbx = 129 kNm is critical**

M_x^*		Ø Mbx =		
118 kNm	≤	**129 kNm**	**∴ O.K.**	**410 UB 59.7**

			Use
Section size	**410 UB 59.7**	**O.K.**	**410 UB 59.7**

Design
Of
Roof Purline

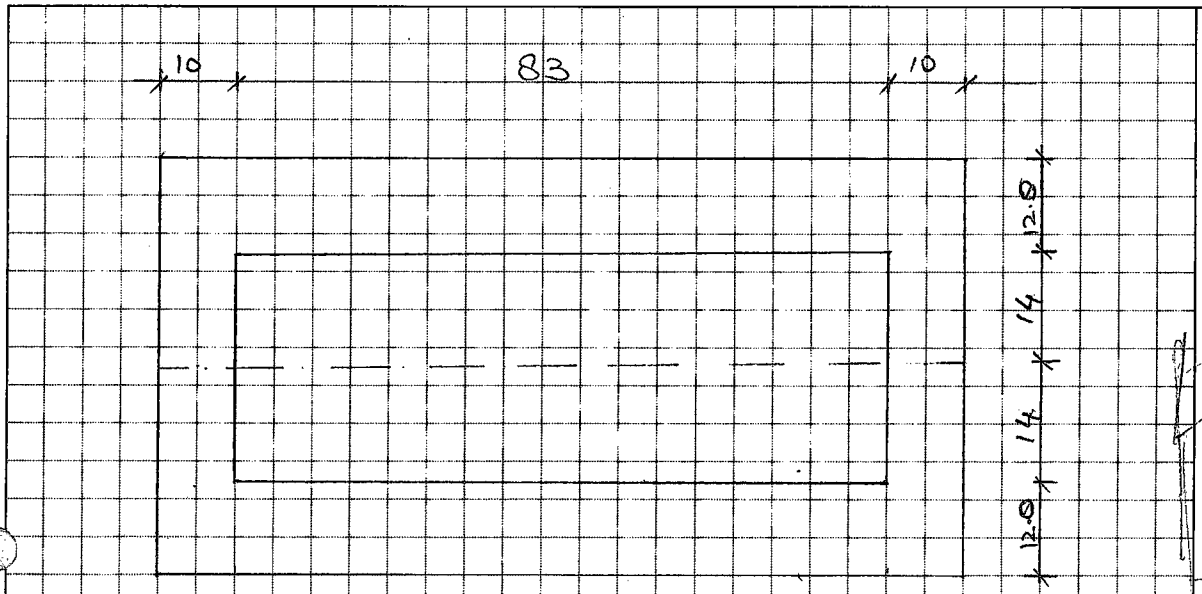
241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Dead load of roofing including purlin wt = 0.15 KPa

Imposed Live Load = 0.25 KPa

Outward wind load.

$P_e = C_{pe} k_a k_l k_p q_z$

$q_z = 1.0$ KPa.

$k_a = 1.0$

$k_l = 1.5$ for area = 10.4×10.4

$k_l = 2.0$ for area = $\frac{10.4}{2} \times \frac{10.4}{2}$

our purlin span is 10.5m

$\therefore k_l = 1.5$ m

$C_{pe} = -0.9$

$P_e = 1.35$ KPa

Ultimate $P_e = 1.35$ KPa

Service $P_e = 0.81$ KPa

0.2b = 10.4
0.2d = 20.6
ht = 14.5.
∴ a = 10.4m

241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Loads on Purlins

1) Gravity Load

DL = 0.15 kPa including Purlins self wt

LL = 0.25 kPa

2) Wind Load For edge zone

Outward Ultimate Load = −1.35 kPa

Outward Service Load = −0.81 kPa

3) Wind Load for central zone

Outward Ultimate Load = −0.50 kPa

Outward Service Load = −0.30 kPa

Critical Design Load Combination (Ultimate)

For Edge Zone

$W_u = 1.2D + 1.6L = 0.58$ kPa ↓

$W_u = D + W_{wind} = 0.15 - 1.35 = -1.2$ kPa ↑

For central zone

$W_u = 1.2D + 1.6L = 0.58$ kPa ↓

$W_u = D + W_{wind} = 0.15 - 0.5 = -0.35$ kPa ↑

Critical Design Load Combination for Service

For Edge Zone

$W_s = D + L = 0.15 + 0.25 = 0.40$ kPa ↓

$W_s = D + Wind = 0.15 - 0.81 = -0.66$ kPa ↑

For central zone

$W_s = D + L = 0.40$ kPa ↑

$W_s = D + W_{ind} = 0.15 - 0.3 = -0.15$ kPa ↑

Calculation Sheet

Job name: Job No:

Subject: Page No: of

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Design of Purlines

For Edge zone

Limit State	Load Combination	Load on Purlins (KN/m) 2m s/p	1.5m s/p	1.25m s/p
Ultimate	1.2D + 1.6L	1.16 ↓	0.87 ↓	+ 0.725
	D + Wind	−2.40 ↑	−1.80 ↑	− 1.5
Service	D + L	0.8 ↓	+0.6 ↓	+ 0.5
	D + Wind	−1.32 ↑	−0.99 ↑	− 0.825

[2.0m s/p

For 10.5m s/p. The Purlins DHS 300/18-2B lapped end spans can carry an Ultimate load of 2.50 KN/m with 3 Braces and Service load of 2.40 KN/m for $\frac{L}{8}$ = 150 compliances

∴ Use DHS 300/18-2B @ 2 m c/c

For Central Zone

Limit State	Load Combination	Load on Purlins (KN/m) 2m s/p
Ultimate	1.2DL + 1.6L	1.16 ↓
	D + Wind	−0.7 ↑
Service	D + L	0.8 ↓
	D + Wind	− 0.3 ↑

Load carry capacity of DHS 300/15-1B is 1.41 KN/m and WB capacity is 2.0 KN/m.

241059

Purlines over Mezzanine Floor Area

The design purlins over the mezzanine floor area are governed by ultimate wind outward load, without additional dead load of ceiling & services. The additional ceiling and services dead load over mezzanine will be beneficial for wind and deal load combination. For ultimate gravity load combination the current design is able to support the additional ceiling & service load. For further explanation please refer to the following calculations.

Dead Load

1) Roof + Purlins	0.15 KPa
2) Ceiling + Services	0.50 KPa (Only for Mezzanine roof)
Total DL	0.65 KPa

Live Load 0.25 KPa

Wind Load

1) Outward Ultimate	-1.35 KPa
2) Outward Service	-0.81 KPa

Ultimate combinations

1.2G + 1.6Q	= 1.18 KPa	for 2m spacing	2.36 KN/m
G + Wind Ult.	= -0.70 KPa	for 2m spacing	-1.40 KN/m

Service Combinations

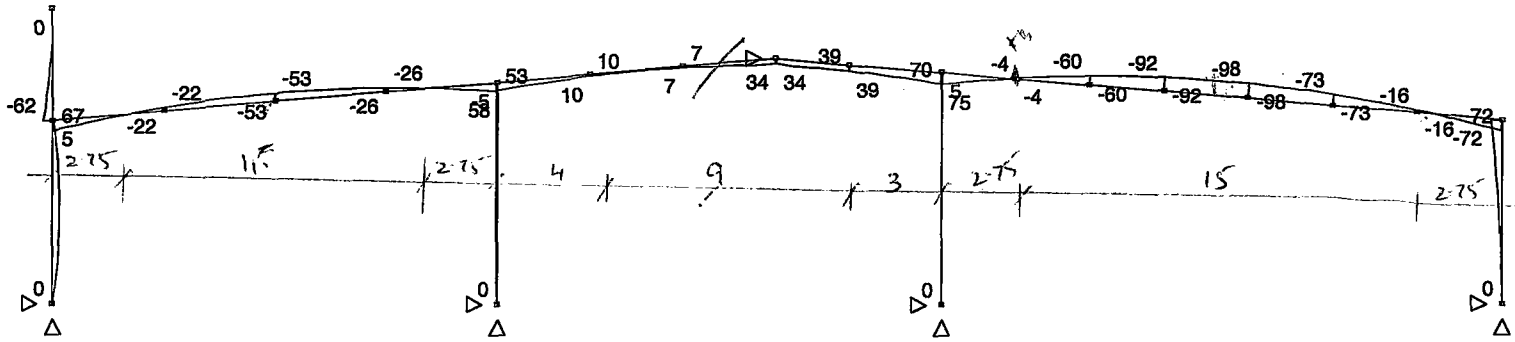
G + Q	= 0.90 KPa	for 2m spacing	1.80 KN/m
G + Wind Serv	= -0.16 KPa	for 2m spacing	-0.32 KN/m

For roof over Mezzanine we are proposing DHS 300/18-2B Lapped Purlines
For 10.5m end span the purlines can carry 2.50 KN/m ultimate load and 2.40 KN/m service load for $L/\delta = 150$

Design
Of
Roof Rafters

241059

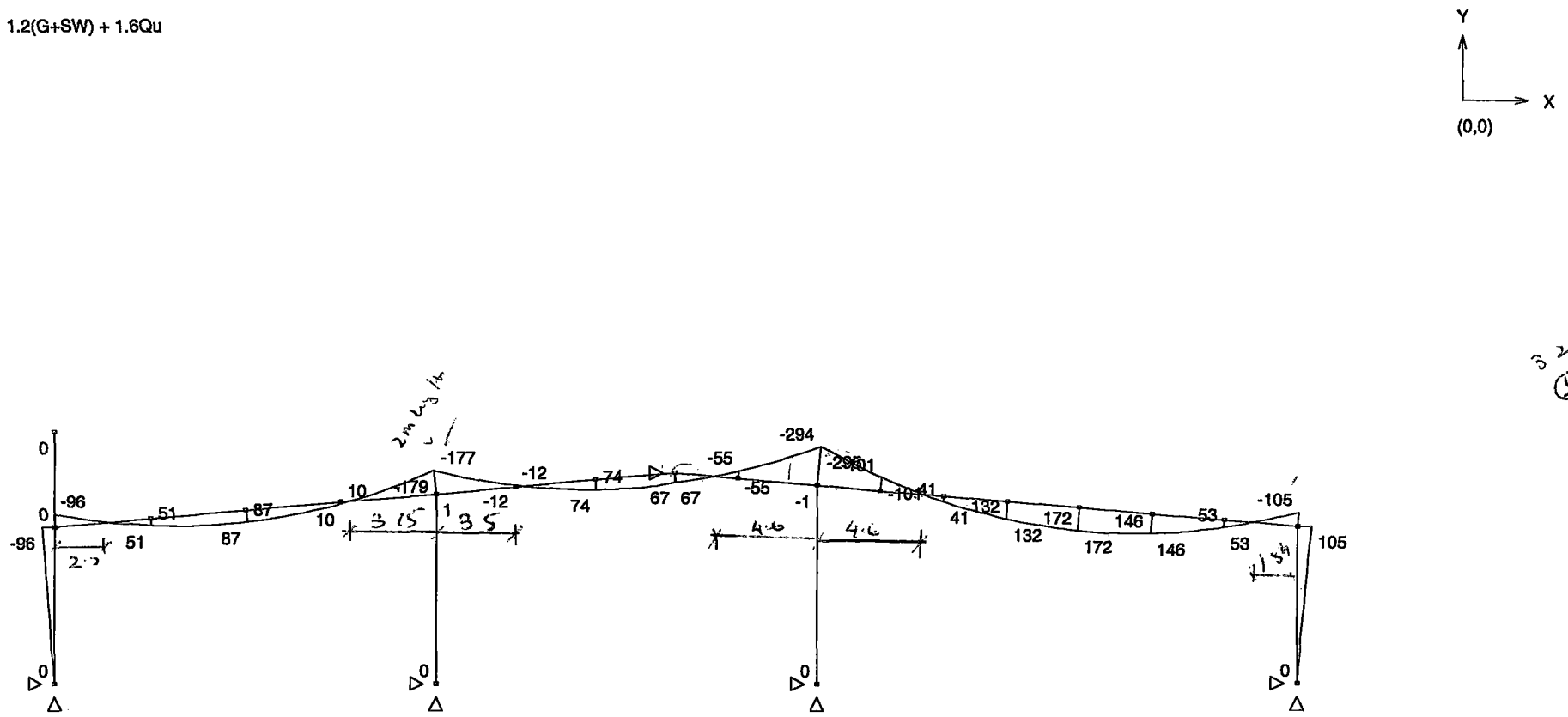
Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

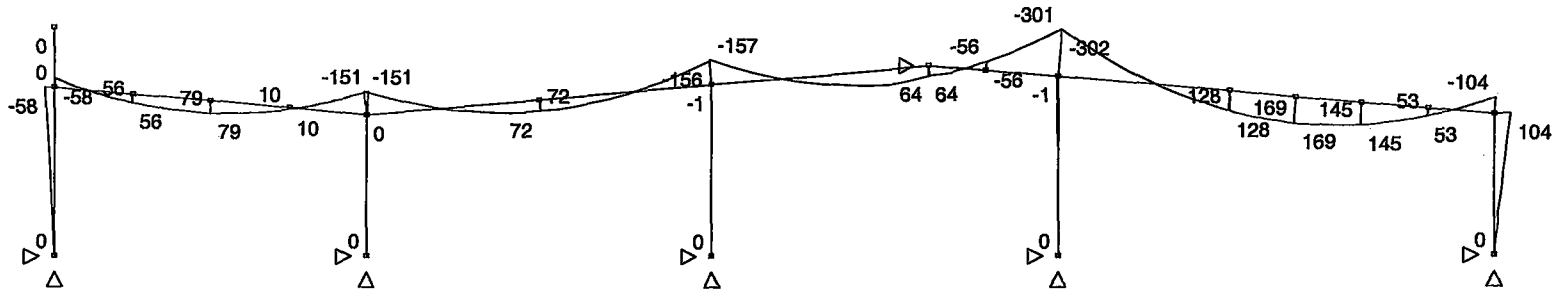
Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S9 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, M... kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Ax... one, Torsion: None

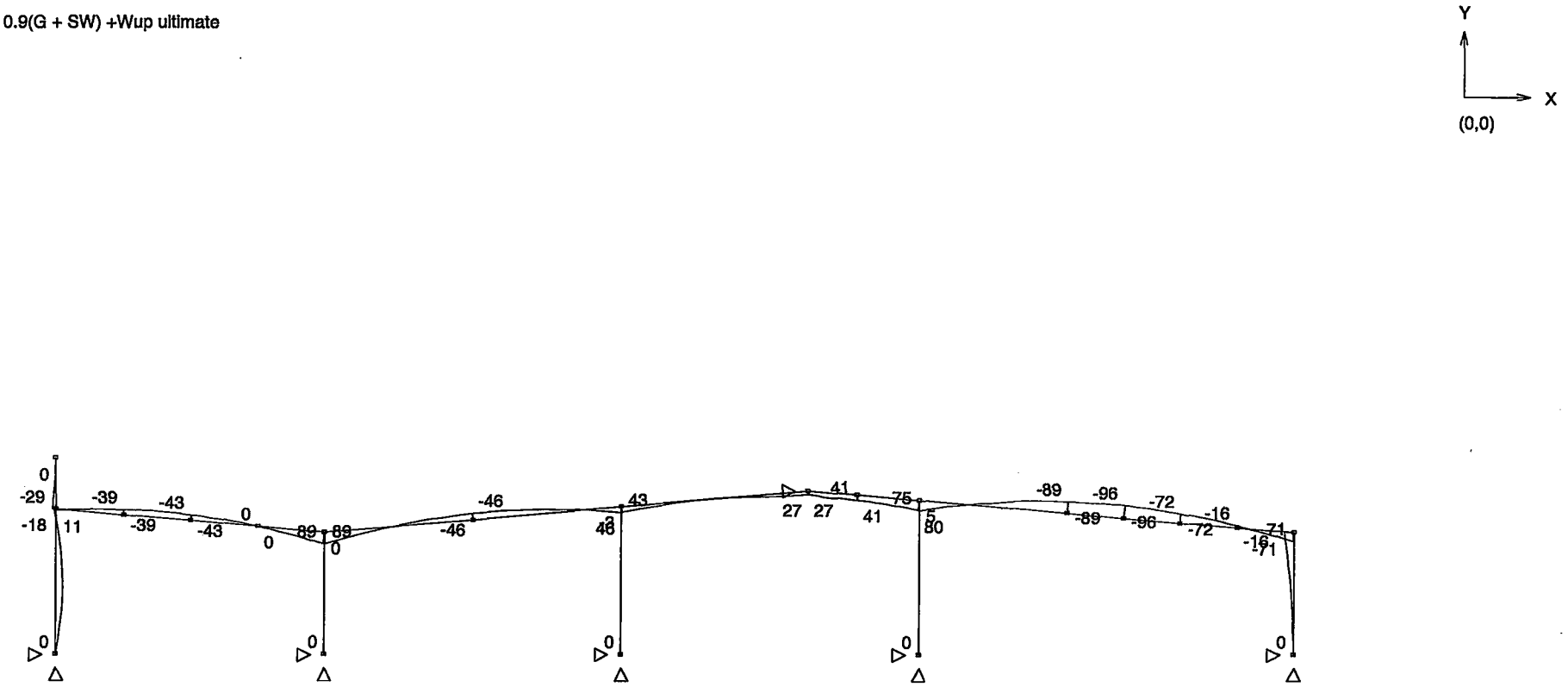
Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S5 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, M[illegible] kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:360, Load: None, Disp: None, Moment: 49, Shear: None, Ax[illegible]lone, Torsion: None

Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S5 for Warehouse Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, M[illegible]: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:360, Load: None, Disp: None, Moment: 49, Shear: None, A[illegible]None, Torsion: None

DESIGN DATA

Enter trial section size here 360 UB 50.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	39	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	9.000	m
Length between Y axis restraints	L_y =	9.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	897000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	168000	mm^3
Moment of inertia about X axis (table)	I_x =	142000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9600000	mm^4
Torsion constant (table)	J =	241000	mm^4
Warping constant (table)	I_w =	2.84E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	6470	mm^2
Radius of gyration about X axis (table)	r_x =	148	mm
Radius of gyration about Y axis (table)	r_y =	38.5	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

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5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	$M_x^* =$	39 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	9.000 m
Length between Y axis restraints	$L_y =$	9.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing =$	0.9
Yield stress of flanges	$f_y =$	250 MPa
Effective section modulus about X axis (table)	$Z_{ex} =$	897000 mm^3
	$\varnothing M_{sx} =$	**202 kNm**

M_x^*		$\varnothing M_{sx}$		
39 kNm	≤	**202 kNm**	∴ **O.K.**	**360 UB 50.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	$Z_{ey} =$	168000 mm^3
	$\varnothing M_{sy} =$	**38 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	≤	**38 kNm**	∴ **O.K.**	**360 UB 50.7**

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone Z | JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SC) to (SE) Wind Case Centre Portion | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	9600000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	241000 mm^4
Warping constant (table)	$I_w =$	2.84E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 9.000 m

$M_{oa} =$ 78 kNm

$\alpha s =$ 0.2896242

$\varnothing M_{bx} =$ 58 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$			
58 kNm	**≤**	**202 kNm**	**∴**	**Ø Mbx =**	**58 kNm is critical**

M_x^*		Ø Mbx =		
39 kNm	**≤**	**58 kNm**	**∴ O.K.**	**360 UB 50.7**

Section size	**360 UB 50.7**	**O.K.**

Use

360 UB 50.7

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Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone Z — JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SE) to (SG) Wind Case Centre Portion — PAGE NO: 1 OF 3

BY: Jamil Khan — DATE: 11/6/04

DESIGN DATA

Enter trial section size here 360 UB 50.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	53	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	6.000	m
Length between Y axis restraints	L_y =	6.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	897000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	168000	mm^3
Moment of inertia about X axis (table)	I_x =	142000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9600000	mm^4
Torsion constant (table)	J =	241000	mm^4
Warping constant (table)	I_w =	2.84E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	6470	mm^2
Radius of gyration about X axis (table)	r_x =	148	mm
Radius of gyration about Y axis (table)	r_y =	38.5	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

Beca Carter Hollings & Ferner **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone Z — JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SE) to (SG) Wind Case Centre Portion — PAGE NO: 2 OF 3

BY: Jamil Khan — DATE: 11/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	53 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	6.000 m
Length between Y axis restraints	L_y =	6.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	250 MPa
Effective section modulus about X axis (table)	Z_{ex} =	897000 mm^3
	$\varnothing M_{sx}$ =	**202 kNm**

M_x^*		$\varnothing M_{sx}$		
53 kNm	≤	**202 kNm**	∴ **O.K.**	**360 UB 50.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	168000 mm^3
	$\varnothing M_{sy}$ =	**38 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	≤	**38 kNm**	∴ **O.K.**	**360 UB 50.7**

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Beca Carter Hollings & Ferner **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone 2 JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SE) to (SG) Wind Case Centre Portion PAGE NO: 2 OF 3

BY: Jamil Khan DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	E =	200 GPa
Moment of inertia about Y axis (table)	I_y =	9600000 mm^4
Shear modulus of elasticity	G =	80 GPa
Torsion constant (table)	J =	241000 mm^4
Warping constant (table)	I_w =	2.84E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

L_{ex} = 6.000 m

M_{oa} = 135 kNm

αs = 0.4440296

$\varnothing M_{bx}$ = 90 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
90 kNm	≤	**202 kNm**	∴	**Ø Mbx = 90 kNm is critical**

M_x^*		Ø Mbx =		
53 kNm	≤	**90 kNm**	**∴ O.K.**	**360 UB 50.7**

Section size	360 UB 50.7	O.K.

Use

360 UB 50.7

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone 2 — JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SE) to (SG) Gravity Case End Portion — PAGE NO: 1 OF 3

BY: Jamil Khan — DATE: 11/6/04

DESIGN DATA

Enter trial section size here 360 UB 50.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	179	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	2.000	m
Length between Y axis restraints	L_y =	2.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	897000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	168000	mm^3
Moment of inertia about X axis (table)	I_x =	142000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9600000	mm^4
Torsion constant (table)	J =	241000	mm^4
Warping constant (table)	I_w =	2.84E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	6470	mm^2
Radius of gyration about X axis (table)	r_x =	148	mm
Radius of gyration about Y axis (table)	r_y =	38.5	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

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5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	$M_x^* =$	179 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	2.000 m
Length between Y axis restraints	$L_y =$	2.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing =$	0.9
Yield stress of flanges	$f_y =$	250 MPa
Effective section modulus about X axis (table)	$Z_{ex} =$	897000 mm^3
	$\varnothing M_{sx} =$	**202 kNm**

M_x^*		$\varnothing M_{sx}$		
179 kNm	≤	**202 kNm**	**∴ O.K.**	**360 UB 50.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	$Z_{ey} =$	168000 mm^3
	$\varnothing M_{sy} =$	**38 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	≤	**38 kNm**	**∴ O.K.**	**360 UB 50.7**

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone 2 | JOB NO: 2711265
SUBJECT: Rafter @ GL(S2 & S3) from (SE) to (SG) Gravity Case End Portion | PAGE NO: 2 OF 3
BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$
$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00
Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs
$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}
$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	9600000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	241000 mm^4
Warping constant (table)	$I_w =$	2.84E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 2.000 m
$M_{oa} =$ 868 kNm
$\alpha s =$ 0.8957526

$\varnothing M_{bx} =$ 181 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
181 kNm	**≤**	**202 kNm**	**∴**	**$\varnothing$ Mbx = 181 kNm is critical**

M_x^*		$\varnothing$ Mbx =		
179 kNm	**≤**	**181 kNm**	**∴ O.K.**	**360 UB 50.7**

			Use
Section size	**360 UB 50.7**	**O.K.**	**360 UB 50.7**

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Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone J | JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Gravity Case End Portion | PAGE NO: 1 OF 3

BY: Jamil Khan | DATE: 11/6/04

DESIGN DATA

Enter trial section size here 460 UB 67.1
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	295	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	2.000	m
Length between Y axis restraints	L_y =	2.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	1480000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	230000	mm^3
Moment of inertia about X axis (table)	I_x =	296000000	mm^4
Moment of inertia about Y axis (table)	I_y =	14500000	mm^4
Torsion constant (table)	J =	378000	mm^4
Warping constant (table)	I_w =	7.08E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	8580	mm^2
Radius of gyration about X axis (table)	r_x =	186	mm
Radius of gyration about Y axis (table)	r_y =	41.2	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

Beca Carter Hollings & Ferner **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone J — JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Gravity Case End Portion — PAGE NO: 2 OF 3

BY: Jamil Khan — DATE: 11/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	295 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	2.000 m
Length between Y axis restraints	L_y =	2.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	250 MPa
Effective section modulus about X axis (table)	Z_{ex} =	1480000 mm^3
	$\varnothing M_{sx}$ =	**333 kNm**

M_x^*		$\varnothing M_{sx}$		
295 kNm	≤	**333 kNm**	**∴ O.K.**	**460 UB 67.1**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	230000 mm^3
	$\varnothing M_{sy}$ =	**52 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	≤	**52 kNm**	**∴ O.K.**	**460 UB 67.1**

241059

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone J | JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Gravity Case End Portion | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 2

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	E =	200 GPa
Moment of inertia about Y axis (table)	I_y =	14500000 mm^4
Shear modulus of elasticity	G =	80 GPa
Torsion constant (table)	J =	378000 mm^4
Warping constant (table)	I_w =	7.08E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

L_{ex} = 2.000 m

M_{oa} = 1647 kNm

αs = 0.9131121

$\varnothing M_{bx}$ = 304 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
304 kNm	≤	**333 kNm**	∴	**Ø Mbx = 304 kNm is critical**

M_x^*		Ø Mbx =		
295 kNm	≤	**304 kNm**	**∴ O.K.**	**460 UB 67.1**

Section size	460 UB 67.1	O.K.

Use

460 UB 67.1

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone J
JOB NO: 2711265
SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Wind Case Centre Portion
PAGE NO: 1 OF 3
BY: Jamil Khan
DATE: 11/6/04

DESIGN DATA

Enter trial section size here **460 UB 67.1**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	98	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	8.000	m
Length between Y axis restraints	L_y =	8.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	1480000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	230000	mm^3
Moment of inertia about X axis (table)	I_x =	296000000	mm^4
Moment of inertia about Y axis (table)	I_y =	14500000	mm^4
Torsion constant (table)	J =	378000	mm^4
Warping constant (table)	I_w =	7.08E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	8580	mm^2
Radius of gyration about X axis (table)	r_x =	186	mm
Radius of gyration about Y axis (table)	r_y =	41.2	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

241059

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone J | JOB NO: 2711265

SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Wind Case Centre Portion | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 2

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	14500000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	378000 mm^4
Warping constant (table)	$I_w =$	7.08E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 8.000 m

$M_{oa} =$ 152 kNm

$\alpha s =$ 0.3328837

$\varnothing M_{bx} =$ **111 kNm**

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
111 kNm	≤	**333 kNm**	∴	**Ø Mbx = 111 kNm is critical**

M_x^*		Ø Mbx =		
98 kNm	≤	**111 kNm**	**∴ O.K.**	**460 UB 67.1**

Section size	460 UB 67.1	O.K.

Use

460 UB 67.1

Beca Carter Hollings & Ferner **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone J
JOB NO: 2711265
SUBJECT: Rafter @ GL(S2 & S3) from (SA) to (SC) Wind Case Centre Portion
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 11/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	98 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	8.000 m
Length between Y axis restraints	L_y =	8.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	250 MPa
Effective section modulus about X axis (table)	Z_{ex} =	1480000 mm^3
	$\varnothing M_{sx}$ =	**333 kNm**

M_x^*		$\varnothing M_{sx}$		
98 kNm	**≤**	**333 kNm**	**∴ O.K.**	**460 UB 67.1**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	230000 mm^3
	$\varnothing M_{sy}$ =	**52 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	**≤**	**52 kNm**	**∴ O.K.**	**460 UB 67.1**

241059

Calculation Sheet

Job name: Job No:

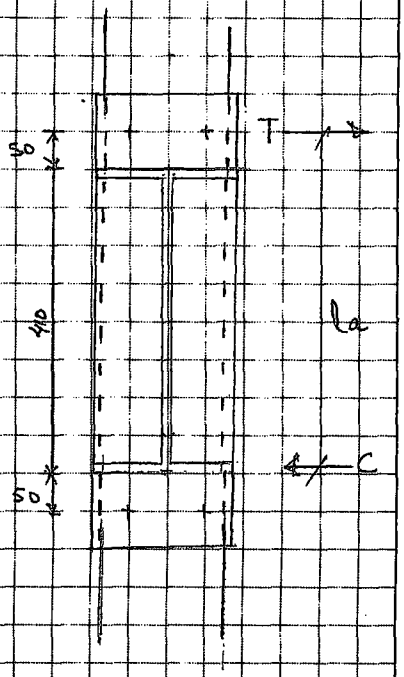
Subject: Page No: of

By: Date:

Connection Design.

Connection between column and Rafter at GL (SA)

M_u = 116 kN-m



Assuming that Bottom flang of Rafter is in compression and the compression force is acting at c/l of flan

Lever arm from Top Bolts to Bottom flang centre

l_a = 50 + 400 = 450 mm = 0.45m

M_u = 116 kN-m

$$T = \frac{M_u}{l_a} = \frac{116}{0.45}$$

T = 257.77 kN.

Tension for single bolt is $\frac{257.77}{2}$ = 128.88 kN

Total Shear force on connection = 70 kN

Shear per single bolt is = $\frac{70}{4}$ = 17.5 kN.

Use M20 Bolts of grade 8.8

Axial tension capacity of M20 Bolt is = ϕN_{tf} = 163 kN

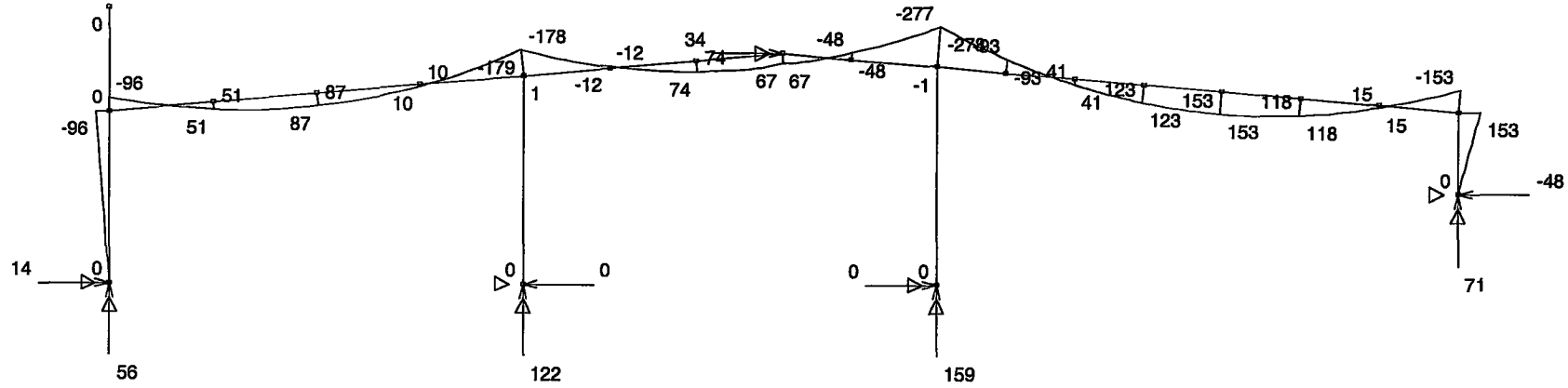
Shear capacity of M20 Bolt is = ϕV_{fn} = 92.6 kN.

Bolts subject to combine Shear & tension

$$\left(\frac{V_f}{\phi V_f}\right)^2 + \left(\frac{N_{tf}}{\phi N_{tf}}\right)^2 \leq 1.0 \Rightarrow \left[\left(\frac{17.5}{92.6}\right)^2 + \left(\frac{128.88}{163}\right)^2 = 0.66\right] \leq 1.0$$

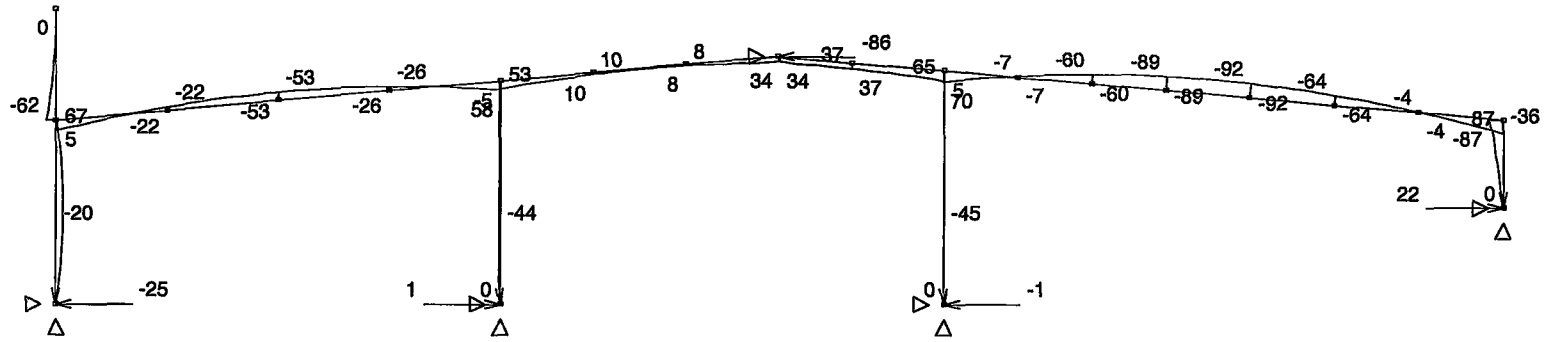
Okay. Use 4 M20 Bolts (2 top & 2 Bottom) with 20 mm Thick end plate to 460UB67.

Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu



No general restraint

Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter @ GL S10 for Zone J Roof Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone 2 — JOB NO: 2711265

SUBJECT: Rafter @ GL(S5 - S8) from (SG) to (SK) Gravity Case End Portion — PAGE NO: 1 OF 3

BY: Jamil Khan — DATE: 11/6/04

DESIGN DATA

Enter trial section size here 360 UB 50.7
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	151	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	3.000	m
Length between Y axis restraints	L_y =	3.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	250	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	897000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	168000	mm^3
Moment of inertia about X axis (table)	I_x =	142000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9600000	mm^4
Torsion constant (table)	J =	241000	mm^4
Warping constant (table)	I_w =	2.84E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	250	MPa
Gross area of cross section (use nett if approp.) (table)	A =	6470	mm^2
Radius of gyration about X axis (table)	r_x =	148	mm
Radius of gyration about Y axis (table)	r_y =	38.5	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

241059

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone 2 | JOB NO: 2711265

SUBJECT: Rafter @ GL(S5 - S8) from (SG) to (SK) Gravity Case End Portion | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	151 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	3.000 m
Length between Y axis restraints	L_y =	3.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	250 MPa
Effective section modulus about X axis (table)	Z_{ex} =	897000 mm^3
	$\varnothing M_{sx}$ =	**202 kNm**

M_x^*		$\varnothing M_{sx}$		
151 kNm	≤	**202 kNm**	**∴ O.K.**	**360 UB 50.7**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	168000 mm^3
	$\varnothing M_{sy}$ =	**38 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	≤	**38 kNm**	**∴ O.K.**	**360 UB 50.7**

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment, Zone 2 | JOB NO: 2711265

SUBJECT: Rafter @ GL(S5 - S8) from (SG) to (SK) Gravity Case End Portion | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 11/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	E =	200 GPa
Moment of inertia about Y axis (table)	I_y =	9600000 mm^4
Shear modulus of elasticity	G =	80 GPa
Torsion constant (table)	J =	241000 mm^4
Warping constant (table)	I_w =	2.84E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

L_{ex} = 3.000 m

M_{oa} = 414 kNm

αs = 0.7638908

$\varnothing M_{bx}$ = 154 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
154 kNm	≤	202 kNm	∴	Ø Mbx = 154 kNm is critical

M_x^*		Ø Mbx =		
151 kNm	≤	154 kNm	∴ O.K.	360 UB 50.7

Section size	360 UB 50.7	O.K.

Use: **360 UB 50.7**

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment Zone J
JOB NO: 2711265
SUBJECT: Level 2, External Col @ GL (SA) form (S9-S10) for 1.2G+1.6Q Comb.
PAGE NO: OF
BY: Jamil Khan
DATE: 11/6/04

DESIGN DATA

Enter trial section size here 310 UB 46.2
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	155	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	4.000	m
Length between Y axis restraints	L_y =	4.000	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	729000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	163000	mm^3
Moment of inertia about X axis (table)	I_x =	100000000	mm^4
Moment of inertia about Y axis (table)	I_y =	9010000	mm^4
Torsion constant (table)	J =	233000	mm^4
Warping constant (table)	I_w =	1.97E+11	mm^6
Moment modification factor	αm =	1.75	
Ratio of smaller to larger bending moment	βm =	2	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

There should be another parameter for buckling length, as the spreadsheet use Lx as buckling length. Th buckling length can be reduced by by introdcuing Fly braces. In absence of Bukling lenth paramenter, we use Lx = Ly = buckling length

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	71	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	5930	mm^2
Radius of gyration about X axis (table)	r_x =	130	mm
Radius of gyration about Y axis (table)	r_y =	39	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	5930	mm^2
Tensile strength	f_u =	410	MPa

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment Zone J | JOB NO: 2711265

SUBJECT: Level 2, External Col @ GL (SA) form (S9-S10) for 1.2G+1.6Q Comb. | PAGE NO: OF

BY: Jamil Khan | DATE: 11/6/04

8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1.4 Have you checked if axial force is significant?

8.2 DESIGN ACTIONS

Factored bending moment about major X - axis	$M_x^* =$	155 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	4.000 m
Length between Y axis restraints	$L_y =$	4.000 m

8.3 SECTION CAPACITY

8.3.2 Uniaxial Bending About Major Principal X-Axis

$M_x^* \le \varnothing M_{rx}$ 196.83

All sections
$\le \varnothing M_{sx}(1-N^*/\varnothing N_s)$ $\varnothing M_{rx} =$ 188 kNm

M_x^*		$\varnothing M_{rx}$	
155 kNm	≤	188 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sx}(1-N^*/\varnothing N_s) \le \varnothing Msx$ $\varnothing M_{rx} =$ 221 kNm

$\varnothing M_{rx}$		$\varnothing M_{sx}$			
221 kNm	≥	197 kNm	∴	Ø Msx =	197 kNm is critical

M_x^*		Ø Msx =	
155 kNm	≤	197 kNm	∴ O.K.

8.3.3 Uniaxial Bending About Minor Principal Y-Axis

$M_y^* \le \varnothing M_{ry}$

All sections
$\le \varnothing M_{sy}(1-N^*/\varnothing N_s)$ $\varnothing M_{ry} =$ 42 kNm

M_y^*		$\varnothing M_{ry}$	
0 kNm	≤	42 kNm	∴ O.K.

I sections to Cl. 8.1.5
$\le \varnothing 1.19M_{sy}(1-(N^*/\varnothing N_s)^2) \le \varnothing Msy$ $\varnothing M_{ry} =$ 52 kNm

$\varnothing M_{ry}$		$\varnothing M_{sy}$			
52 kNm	≥	44 kNm	∴	Ø Msy =	44 kNm is critical

M_y^*		Ø Msy =	
0 kNm	≤	44 kNm	∴ O.K.

241059

SHS or RHS sections to Cl. 8.1.5
$\le \varnothing 1.18M_{sy}(1-N^*/\varnothing N_s) \le \varnothing Msy$ $\varnothing M_{ry} =$ 50 kNm

$\varnothing M_{ry}$ $\varnothing M_{sy}$

Beca Carter Hollings & Ferner **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment Zone J JOB NO: 2711265

SUBJECT: Level 2, External Col @ GL (SA) form (S9-S10) for 1.2G+1.6Q Comb. PAGE NO: OF

BY: Jamil Khan DATE: 11/6/04

50 kNm ≥ 44 kNm ∴ Ø Msy = 44 kNm is critical

M_y^*	Ø Msy =	
0 kNm	≤ 44 kNm	∴ O.K.

8.3.4 Biaxial Bending

Enter $ØM_{rx}$ of applicable section 188 kNm kNm (ie. all, I, or hollow section)
Enter $ØM_{ry}$ of applicable section 42 kNm kNm (ie. all, I, or hollow section)

All sections
$N^*/ØN_s + M_x^*/ØM_{sx} + M_y^*/ØM_{sy} \leq 1.0$ = 0.834

O.K.

I or SHS or RHS sections to Cl. 8.1.5
$(M_x^*/ØM_{rx})^\gamma + (M_y^*/ØM_{ry})^\gamma \leq 1.0$ = 0.758

where $\gamma = 1.4 + (N^*/ØN_s) \leq 2.0$ = 1.446

O.K.

8.4 MEMBER CAPACITY

8.4.2 IN PLANE CAPACITY

8.4.2.2 Compression Members

$M^* \leq ØM_i$

All sections
$\leq ØM_s(1-N^*/ØN_c) \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}(1-N^*/ØN_{cx}) \leq ØM_{rx}$ $ØM_{ix}$ = 178 kNm

$ØM_{ix}$	$ØM_{rx}$		
178 kNm	≤ 188 kNm	∴	Ø Mix1 = 178 kNm is critical

M_x^*	Ø Mix1 =	
155 kNm	≤ 178 kNm	∴ O.K.

- Bending about minor principal y-axis

$\leq ØM_{sy}(1-N^*/ØN_{cy}) \leq ØM_{ry}$ $ØM_{iy}$ = 40 kNm

$ØM_{iy}$	$ØM_{ry}$		
40 kNm	≤ 42 kNm	∴	Ø Miy1 = 40 kNm is critical

M_y^*	Ø Miy1 =	
0 kNm	≤ 40 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5
$\leq ØM_s\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_c) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_c))}\} \leq ØM_{rx}$ or $ØM_{ry}$

- Bending about major principal x-axis

$\leq ØM_{sx}\{[1-((1+\beta_m)/2)^3](1-N^*/ØN_{cx}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/ØN_{cx}))}\} \leq ØM_{rx}$
$ØM_{ix}$ = 320 kNm

Beca Carter Hollings & Ferner | **CALCULATION SHEET**

JOB NAME: Queesgate Mall Redevelopment Zone J | JOB NO: 2711265

SUBJECT: Level 2, External Col @ GL (SA) form (S9-S10) for 1.2G+1.6Q Comb. | PAGE NO: OF

BY: Jamil Khan | DATE: 11/6/04

$\varnothing M_{ix}$		$\varnothing M_{rx}$			
320 kNm	≥	188 kNm	∴	Ø Mrx =	188 kNm is critical

M_x^*		Ø Mrx =	
155 kNm	≤	188 kNm	∴ O.K.

- Bending about minor principal y-axis

$$\leq \varnothing M_{sy}\{[1-((1+\beta_m)/2)^3](1-N^*/\varnothing N_{cy}) + 1.18((1+\beta_m)/2)^3 \sqrt{(1-(N^*/\varnothing N_{cy}))}\} \leq \varnothing M_{ry}$$

$\varnothing M_{iy}$ = 72 kNm

$\varnothing M_{iy}$		$\varnothing M_{ry}$			
72 kNm	≥	42 kNm	∴	Ø Mry =	42 kNm is critical

M_y^*		Ø Mry =	
0 kNm	≤	42 kNm	∴ O.K.

Enter $\varnothing M_{ix}$ of applicable section 188 kNm (ie. all, I, or hollow section)
Enter $\varnothing M_{iy}$ of applicable section 42 kNm (ie. all, I, or hollow section)

8.4.2.3 Tension Members

A member subject to a tensile force and B.M. shall satisfy 8.3 SECTION CAPACITY

8.4.4 OUT OF PLANE CAPACITY

8.4.4.1 Compression Members

$M_x^* \leq \varnothing M_{ox}$

All sections

$\leq \varnothing M_{bx}(1-N^*/\varnothing N_{cy})$ $\varnothing M_{ox}$ = 186 kNm

M_x^*		$\varnothing M_{ox}$	
155 kNm	≤	186 kNm	∴ O.K.

I or SHS or RHS sections to Cl. 8.1.5

$\leq \varnothing\alpha_{bc}M_{bxo}\sqrt{[(1-N^*/\varnothing N_{cy})(1-(N^*/\varnothing N_{oz})]} \leq \varnothing M_{rx}$ $\varnothing M_{ox}$ = 142 kNm

where $1/\alpha_{bc} = 1-\beta_m/2 + (1+\beta_m/2)^3(0.4 - 0.23\ N^*/\varnothing N_{cy})$ = 0.775

α_{bc} = 1.291

$M_{bxo} = M_{bx} / \alpha_m \times \alpha_m = 1.0$ = 131 kNm

$N_{oz} = GJ + (\Pi^2 E I_w/L_z^2) / (I_x + I_y)/A$ = 2336 kN

L_z = Dist between partial or full retraints which prevent twist about centriod = 4.000 m

$\varnothing M_{ox}^*$		$\varnothing M_{rx}$			
142 kNm	≤	188 kNm	∴	Ø Mox2 =	142 kNm is critical

M_x^*		Ø Mox2 =	
155 kNm	≥	142 kNm	∴ N.G. - Reselect trial section size

241059

8.4.4.2 Tension Members

$M_x^* \leq \varnothing M_{ox}$

All sections

$\leq \varnothing M_{bx}(1+N^*/\varnothing N_t) \leq M_{rx}$ $\varnothing M_{ox} =$ 216 kNm

$\varnothing M_{ox}$		$\varnothing M_{rx}$		
216 kNm	**≥**	**188 kNm**	**∴**	**Ø Mrx = 188 kNm is critical**

M_x^*		Ø Mrx =	
155 kNm	**≤**	**188 kNm**	**∴ O.K.**

8.4.5 BIAXIAL BENDING CAPACITY

8.4.5.1 Compression Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{cx} = \varnothing M_{ix}$

For partial or no restraint $\varnothing M_{cx}$ = **Lesser of** $\varnothing M_{ix}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{cx}$ for applicable restraint, $\varnothing M_{cx}$ = 188 kNm

$(M_x^*/\varnothing M_{cx})^{1.4} + (M_y^*/\varnothing M_{iy})^{1.4} \leq 1.0$ = 0.765

O.K.

8.4.5.2 Tension Members

For full lateral restraint to Cl. 8.1.2.3 $\varnothing M_{tx} = \varnothing M_{rx}$

For partial or no restraint $\varnothing M_{tx}$ = **Lesser of** $\varnothing M_{rx}$ **or** $\varnothing M_{ox}$

Enter $\varnothing M_{tx}$ for applicable restraint, $\varnothing M_{tx}$ = 188 kNm

$(M_x^*/\varnothing M_{tx})^{1.4} + (M_y^*/\varnothing M_{ry})^{1.4} \leq 1.0$ = 0.765

O.K.

Calculation Sheet

Job name: Job No:

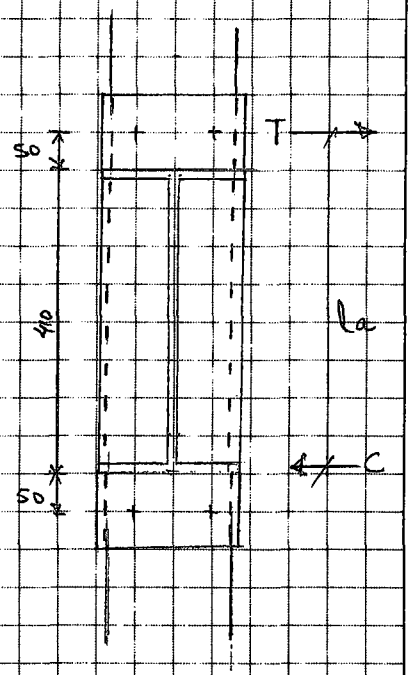
Subject: Page No: of

By: Date:

Connection Design.

Connection between column and Rafter at GL (SA) at Mezzanine Location at GL SA/S8, SA/S9, SA/S10

M_u = 153 KN-m



Assuming that Bottom flang of Rafter is in compression and the compression force is acting at c/l of flan.

Lever arm from Top Bolts to Bottom flang centre.

l_a = 50 + 400 = 450 mm = 0.45 m

M_u = 153 KN-m

$$T = \frac{M_u}{l_a} = \frac{153}{0.45}$$

T = 340. KN.

Tension for single bolt is $\frac{340}{2}$ = 170 KN

Total Shear force on connection = 70 KN

Shear per single bolt is = $\frac{71}{4}$ = 17.75 KN.

Use M20 Bolts of grade 8.8

Axial tension capacity of M24 Bolt is = ϕN_{tf} = 234 KN

Shear capacity of M24 Bolt is = ϕV_f = 133.0 KN.

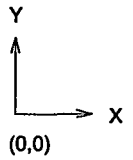
Bolts subject to combine Shear & tension

$$\left(\frac{V_f}{\phi V_f}\right)^2 + \left(\frac{N_{tf}}{\phi N_{tf}}\right)^2 \leq 1.0 \Rightarrow \left[\left(\frac{17.75}{133}\right)^2 + \left(\frac{170}{234}\right)^2 = 0.56\right] \leq 1.0$$

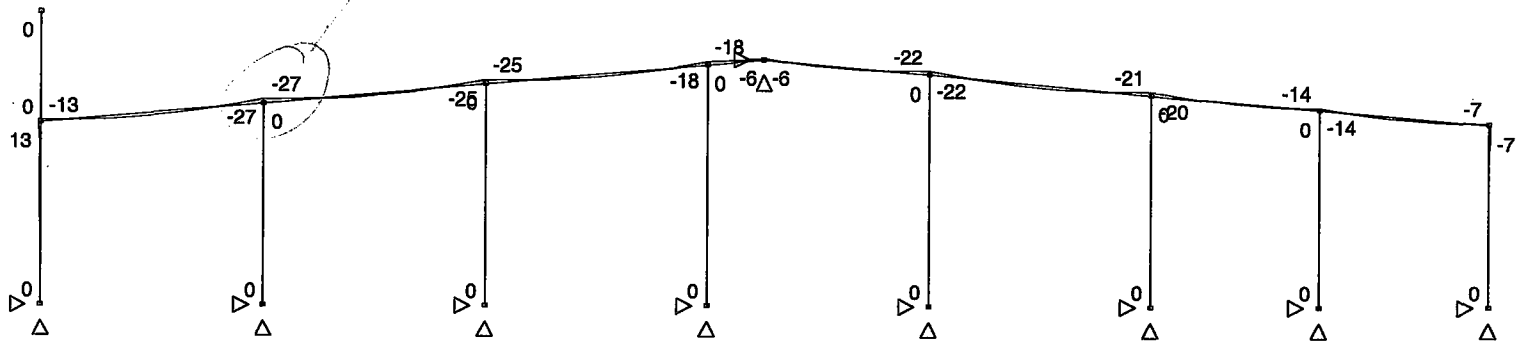
Okay. Use 4 M24 Bolts (2 top & 2 Bottom) with 20 mm Thick end plate to 460 UB 67.

241059

Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu



This is support Moment and Both top & Bottom flange are restraint fully
∴ 250 UB 31

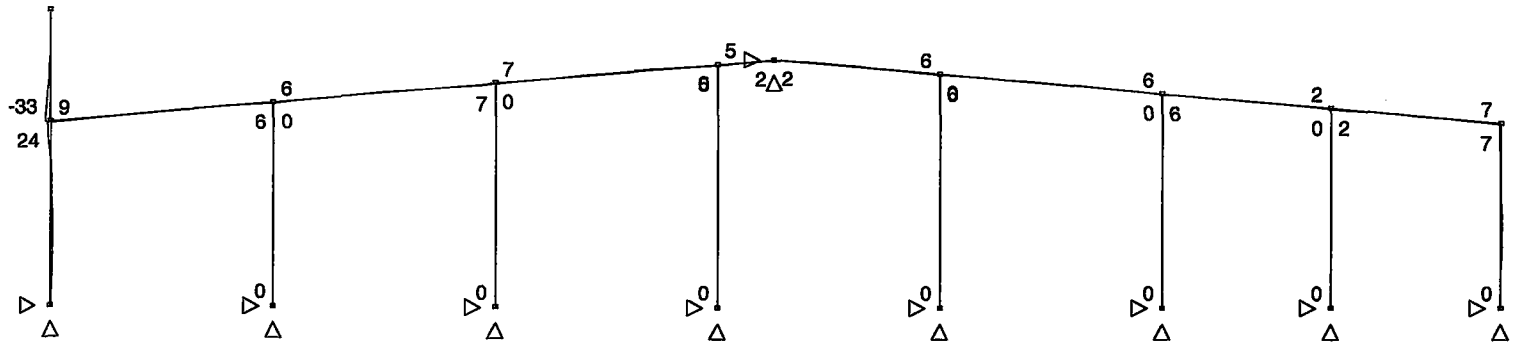


Use 250 UB 31

Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

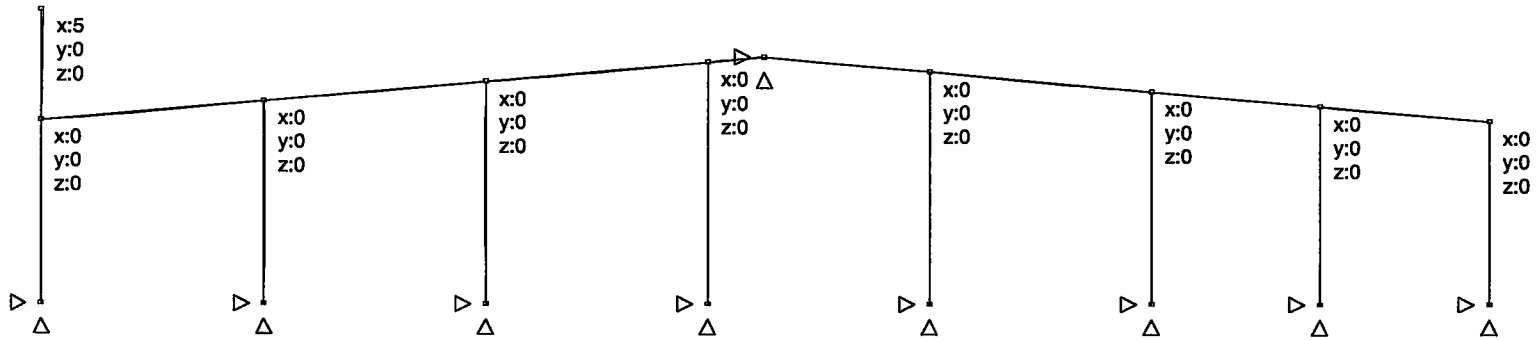
Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, M[illegible]: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, A[illegible]None, Torsion: None

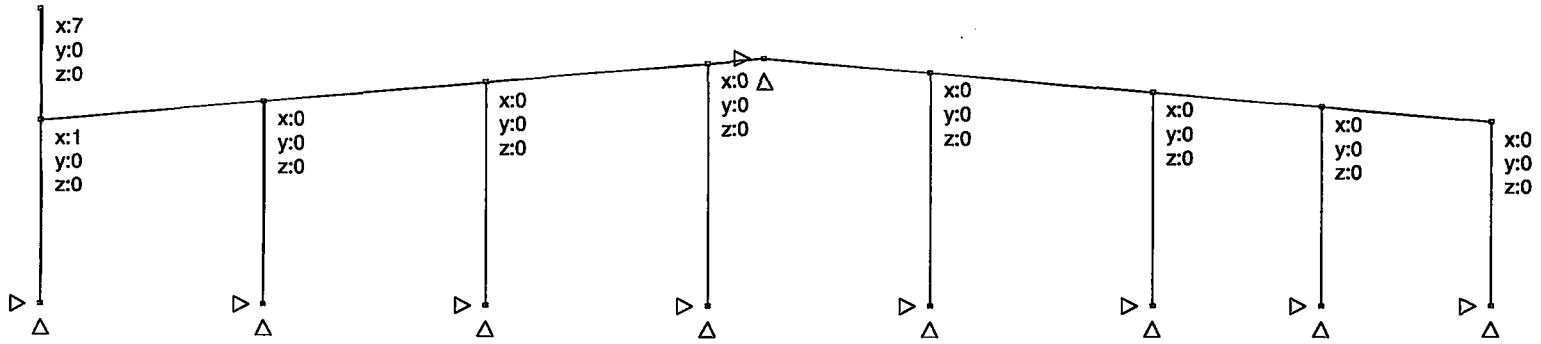
Load cases:
■ 14 ——— (SW) G + SW +Q



Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

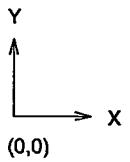
Load cases:
■ 13 ——— (SW) G + SW + Wup service

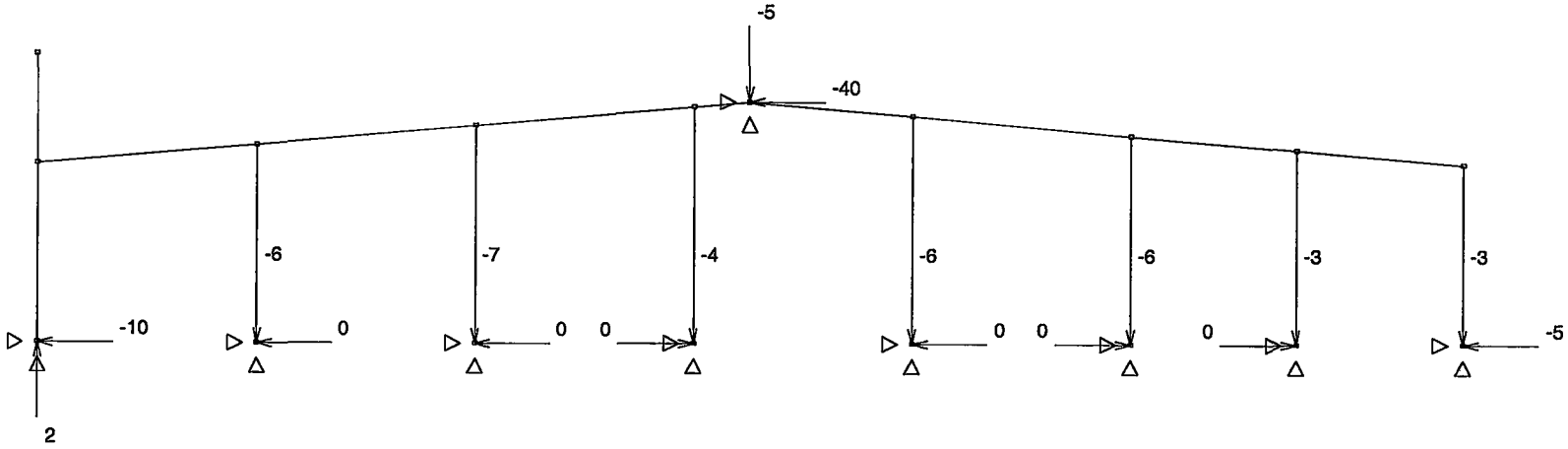


Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, M[illegible]: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: 10, Moment: None, Shear: None, A[illegible]None, Torsion: None

Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate

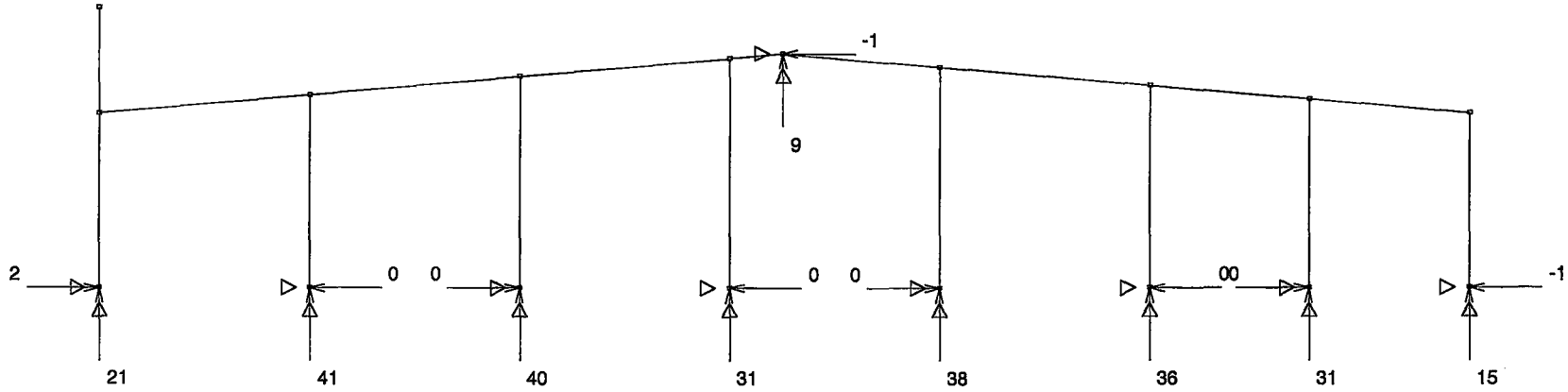




Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

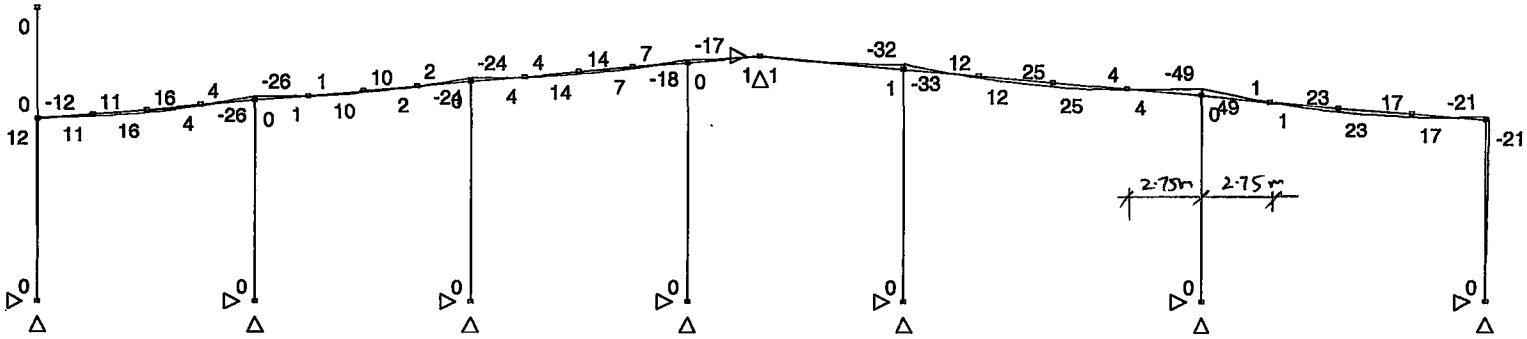
Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu

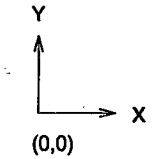


Potal Fram @ GL (S11)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S11Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu





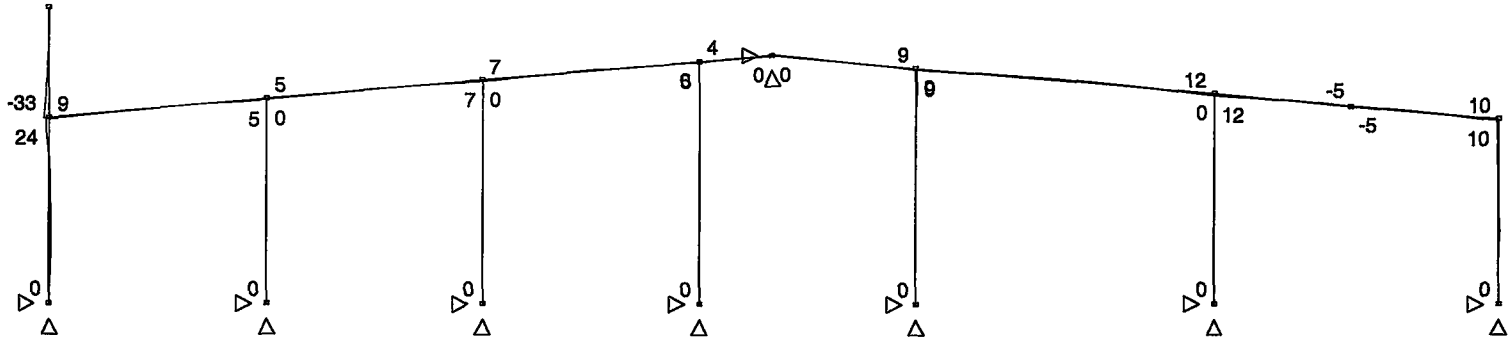
For Le = 3m

M_{NR} for 250UB31 = 64.6 kN-m

Portal Frame @ GL S1
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

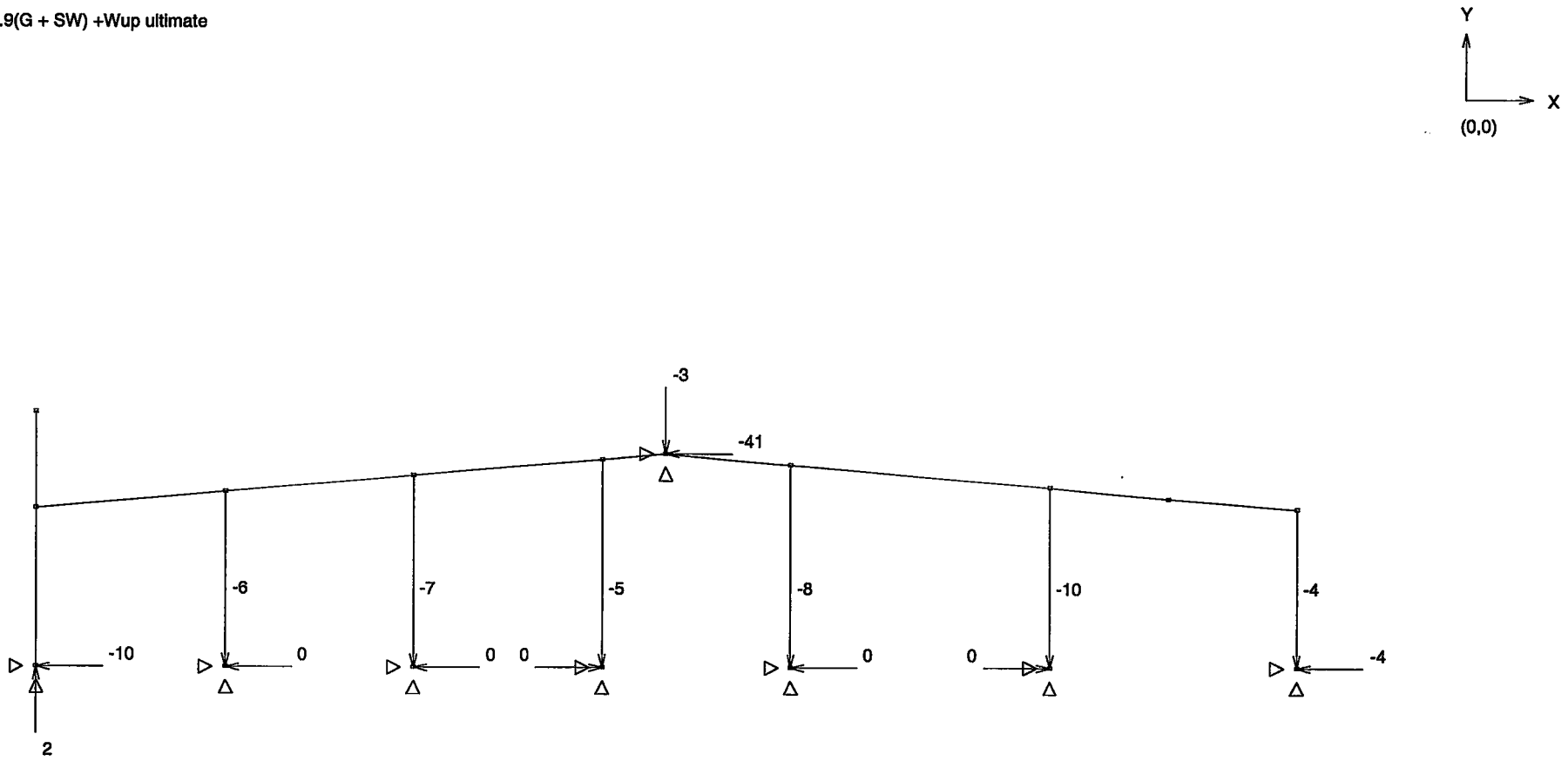
Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



Portal Frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: 49, Shear: None, Axial: None, Torsion: None

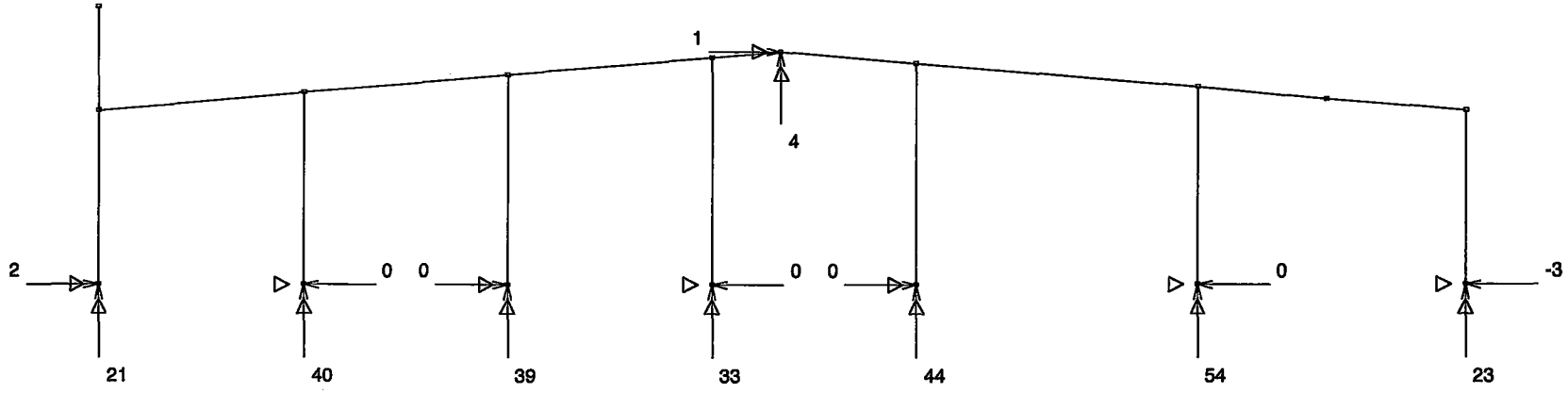
Load cases:
■ 12 ——— (SW) 0.9(G + SW) +Wup ultimate



Portal Frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

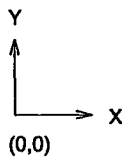
Load cases:
■ 11 ——— (SW) 1.2(G+SW) + 1.6Qu

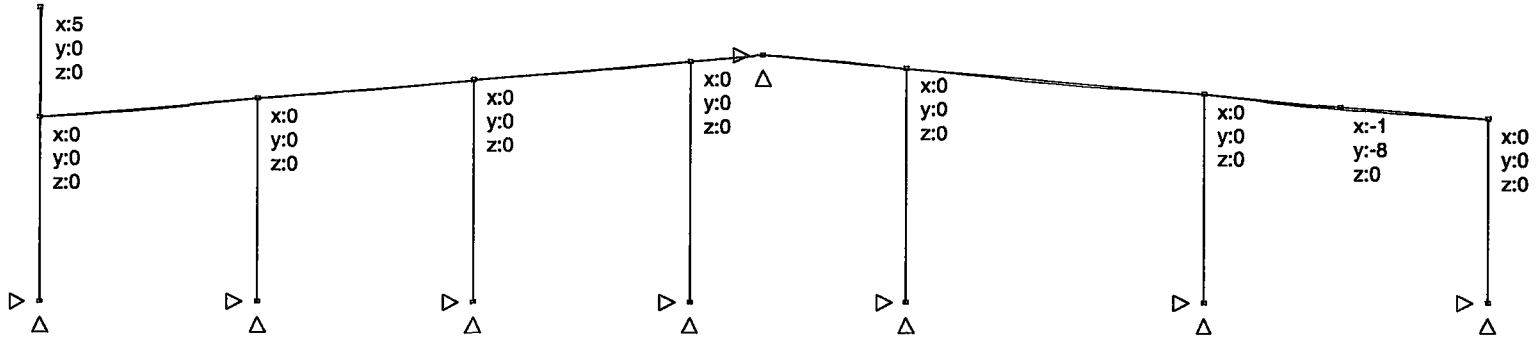


Portal Frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mass: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 14 ——— (SW) G + SW +Q

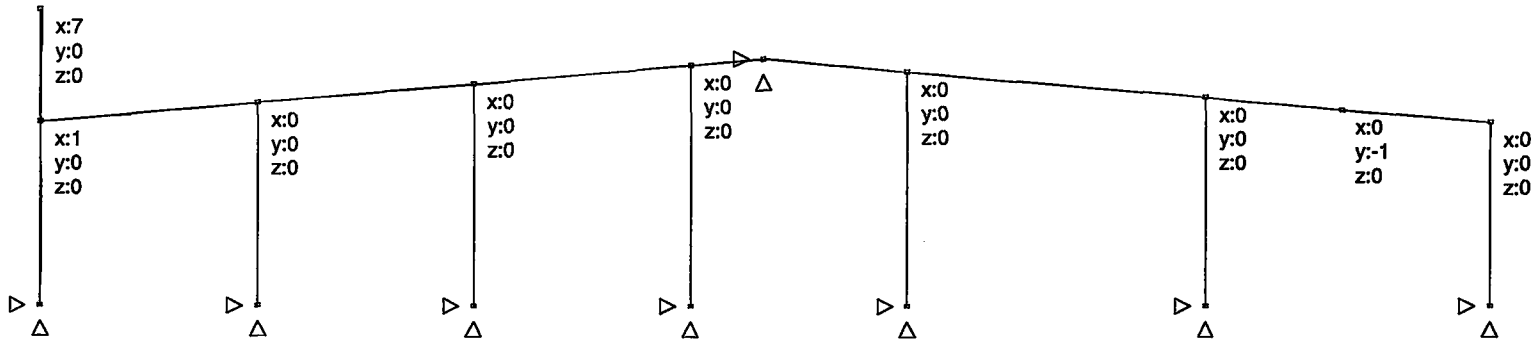




Portal Frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

Load cases:
■ 13 ——— (SW) G + SW + Wup service



Portal Frame @ GL (S1)
No general restraint

Queensgate - Roof Rafter for Warehouse - RBW
Job: C:\...\Queensgate Zone J - Rafter Check for Warehouse Roof @ GL S1 Rev 01
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:280, Load: None, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Supp-A Moment = 38 KN-m. & V = 47

$l_e = 250 + 50 = 300$ mm

$l_a = 0.3$ m

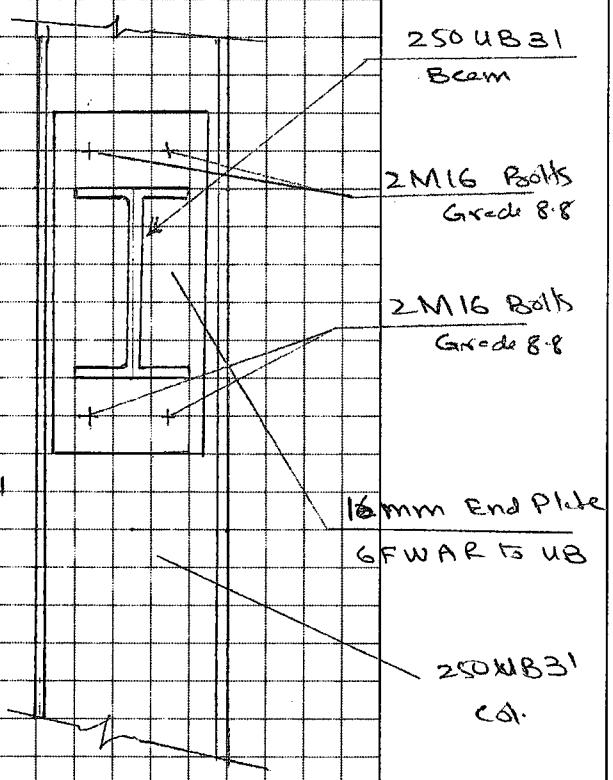
$F = \frac{M_u}{l_a} = 126.67$ KN

Direct tension on single bolt = 63 KN.

Shear Force from Beam = $48/2 = 24$ KN

If we are using 4 Bolts

∴ Shear for single Bolts is 6 KN.



Required.

Direct tension on single bolt = 63 KN

Shear for single Bolts = 6 KN

∴ 2 M16 at Top and 2 M16 Grade 8.8 Bolts should be used with 12 mm Thick end plate

Calculation Sheet

Job name: Queensgate MoU Zone J, **Job No:** 2711265026

Subject: Steel Beam For Platform Roof **Page No:** of

By: JK **Date:** 14 June 2004

Equipment load = 1800 kg
Plat Form load = 0.25 kPa

Plat Form Dimension = 3m width

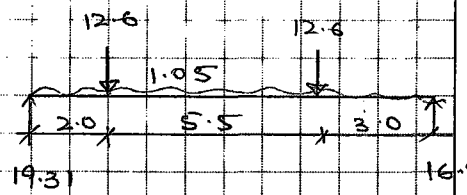
We are providing 2 Beams
∴ Load for each beam

From Equipment = 9 kN.
For Plat Form
DL = 0.25 x 1.5 = 0.375 kN/m
LL = 0.25 x 1.5 = 0.375 kN/m

$P_u = 1.4 \times 9 = 12.6$ kN (Consider Plant load as Variable Dead Load)

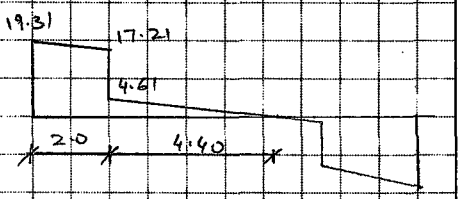
$W_u = 1.2G + 1.6Q = 1.05$ kN/m

Looking from Roof layout the most critical conditions are for Span (S9) – (S10)



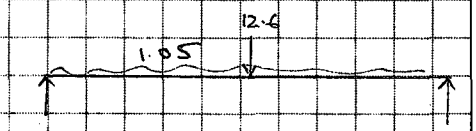
Case (1)

$M_u = \frac{(19.31 + 17.21)}{2} \times 2 + \frac{1}{2}(4.40 \times 4.61)$



$M_u = 46.662$ kN-m.

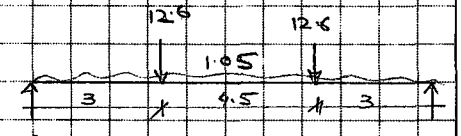
Another Possible Case



Case (2)

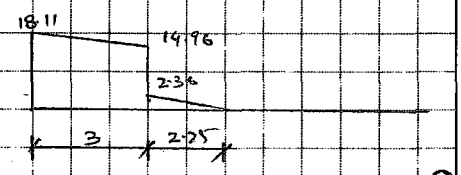
$M_u = 47.54$ kN-m.

Another Possible Case



Case (3)

$M_u = \frac{(18.11 + 14.96)}{2} \times 3 + \frac{1}{2} \times 2.36 \times 2.25$



$M_u = 52.26$ kN-m.

241059

Use 310 UB 40 with 180 PFC To brace the flond.

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone Z
JOB NO: 2711265
SUBJECT: Steel beams for Roof Plant Equipments
PAGE NO: 1 OF 3
BY: Jamil Khan
DATE: 14/6/04

DESIGN DATA

Enter trial section size here **310 UB 40.4**
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	52.26	kNm
Factored bending moment about minor Y - axis	M_y^* =	0	kNm
Length between X axis restraints	L_x =	5.500	m
Length between Y axis restraints	L_y =	5.500	m
Strength reduction factor	$\varnothing$ =	0.9	
Yield stress of flanges	f_y =	300	MPa
Young's modulus	E =	200	GPa
Shear modulus of elasticity	G =	80	GPa
Effective section modulus about X axis (table)	Z_{ex} =	633000	mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	139000	mm^3
Moment of inertia about X axis (table)	I_x =	86400000	mm^4
Moment of inertia about Y axis (table)	I_y =	7650000	mm^4
Torsion constant (table)	J =	157000	mm^4
Warping constant (table)	I_w =	1.65E+11	mm^6
Moment modification factor	αm =	1.00	
Ratio of smaller to larger bending moment	βm =	0	
Twist restraint factor	K_t =	1.00	
Load height factor	K_l =	1.00	
Rotation restraint factor	K_r =	1.00	

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0	kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300	MPa
Gross area of cross section (use nett if approp.) (table)	A =	5210	mm^2
Radius of gyration about X axis (table)	r_x =	129	mm
Radius of gyration about Y axis (table)	r_y =	38.3	mm
Effective length factor about X axis	k_{ex} =	1	
Effective length factor about Y axis	k_{ey} =	1	
Form factor (by calculation)	k_f =	0.954	
Section constant, α_b from table 6.3.3(1)	α_b =	0	

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0	kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85	
Nett area of cross section (by calculation)	A_n =	12500	mm^2
Tensile strength	f_u =	410	MPa

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone Z
JOB NO: 2711265
SUBJECT: Steel beams for Roof Plant Equipments
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 14/6/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	52.26 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	5.500 m
Length between Y axis restraints	L_y =	5.500 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Effective section modulus about X axis (table)	Z_{ex} =	633000 mm^3
	$\varnothing M_{sx}$ =	**171 kNm**

M_x^*		$\varnothing M_{sx}$		
52 kNm	**≤**	**171 kNm**	**∴ O.K.**	**310 UB 40.4**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	139000 mm^3
	$\varnothing M_{sy}$ =	**38 kNm**

M_y^*		$\varnothing M_{sy}$		
0 kNm	**≤**	**38 kNm**	**∴ O.K.**	**310 UB 40.4**

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment, Zone Z
JOB NO: 2711265
SUBJECT: Steel beams for Roof Plant Equipments
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 14/6/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$
$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00
Ratio of smaller to larger bending moment $\beta m =$ 0

Slenderness reduction factor, αs
$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}
$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	7650000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	157000 mm^4
Warping constant (table)	$I_w =$	1.65E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 5.500 m
$M_{oa} =$ 108 kNm
$\alpha s =$ 0.4254083

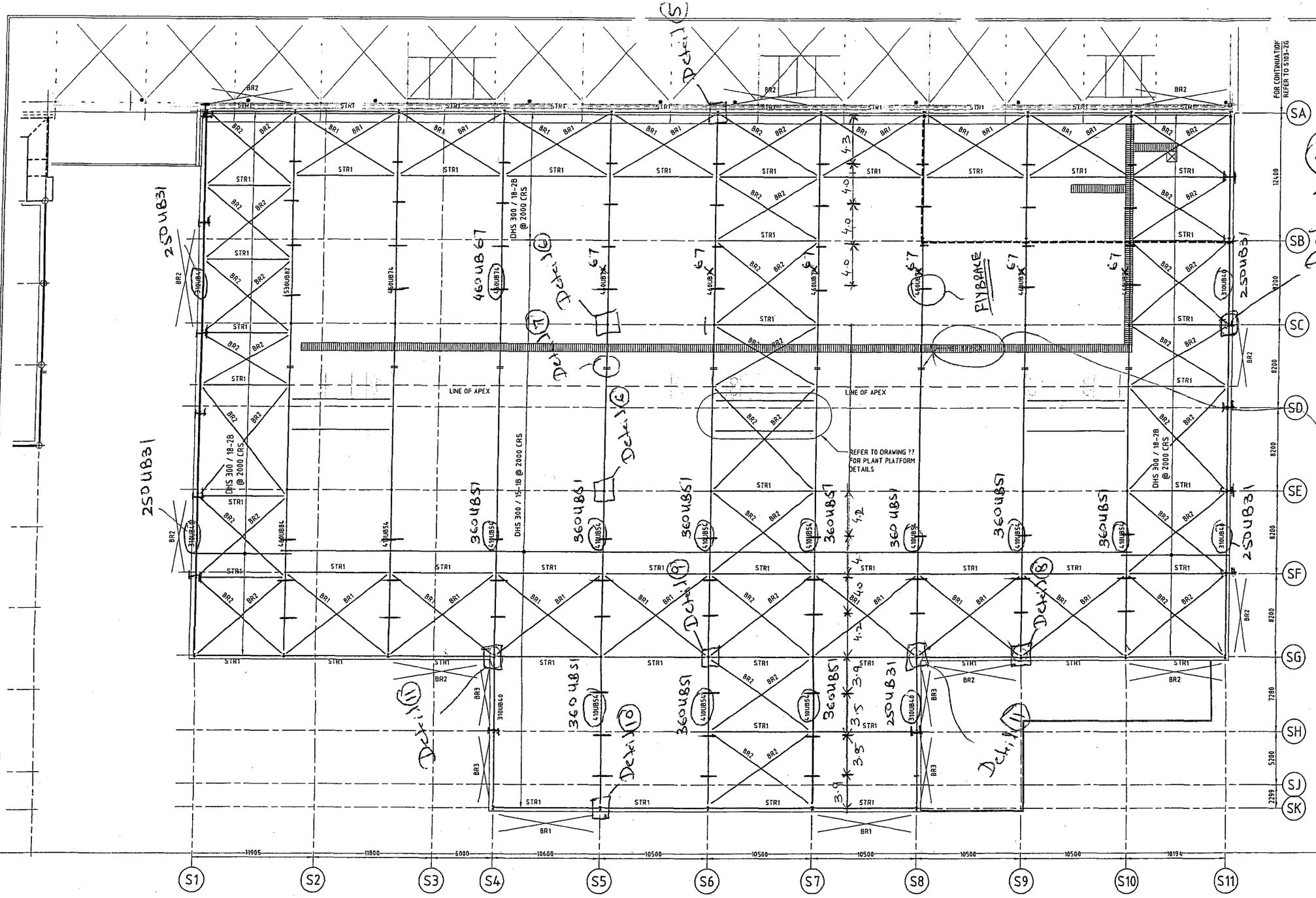
$\varnothing M_{bx} =$ 73 kNm

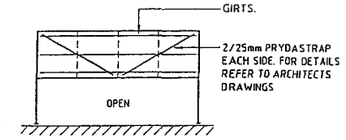
$\varnothing M_{bx}$		$\varnothing M_{sx}$		
73 kNm	**≤**	**171 kNm**	**∴**	**$\varnothing$ Mbx = 73 kNm is critical**

M_x^*		$\varnothing$ Mbx =		
52 kNm	**≤**	**73 kNm**	**∴ O.K.**	**310 UB 40.4**

Section size	**310 UB 40.4**	**O.K.**

Use
310 UB 40.4





DETAIL OCCURS ON GRID SA

NOTE

1. FOR ROOF BRACING & PURLIN CLEAT DETAILS SEE S351

MEMBER	SIZE
STR1	200x6 SHS
BR1	RH24
BR2	125x6 EA
BR3	100x6 EA

2. ALL PURLINS TO BE LAPPED 10% OF SPAN AT SUPPORTS.
3. WALL GIRTS TO BE DHS 300/18-2B@1800crs WITH CHANNEL BEAM (NO RODS). BOTTOM TWO SPANS TO BE AT MAX 900crs
4. REFER TO ARCHITECTS DRAWINGS FOR EXTENT OF PERIMETER PARAPETS
5. FLYBRACES
   -ROOF BEAMS (NUMERIC GRIDS).-ALLOW 12 FLYBRACES PER RAFTER
   -COLUMNS.-ALLOW 2 FLYBRACES PER EXTERNAL WALL COLUMN.

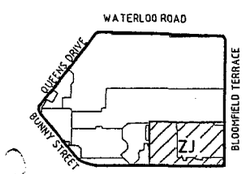
Connection between 460UB67 & 360UB51

NOT FOR CONSTRUCTION

WORKING COPY

241075

PRELIMINARY



NOTE
ALL DIMENSIONS TO BE CHECKED ON SITE. WRITTEN DIMENSIONS ONLY TO BE USED. REFER TO ALL DETAIL & STRUCTURAL MECHANICAL & SERVICES DWGS BEFORE COMMENCING WORK. REFER ANY DISCREPANCIES TO ARCHITECTS.

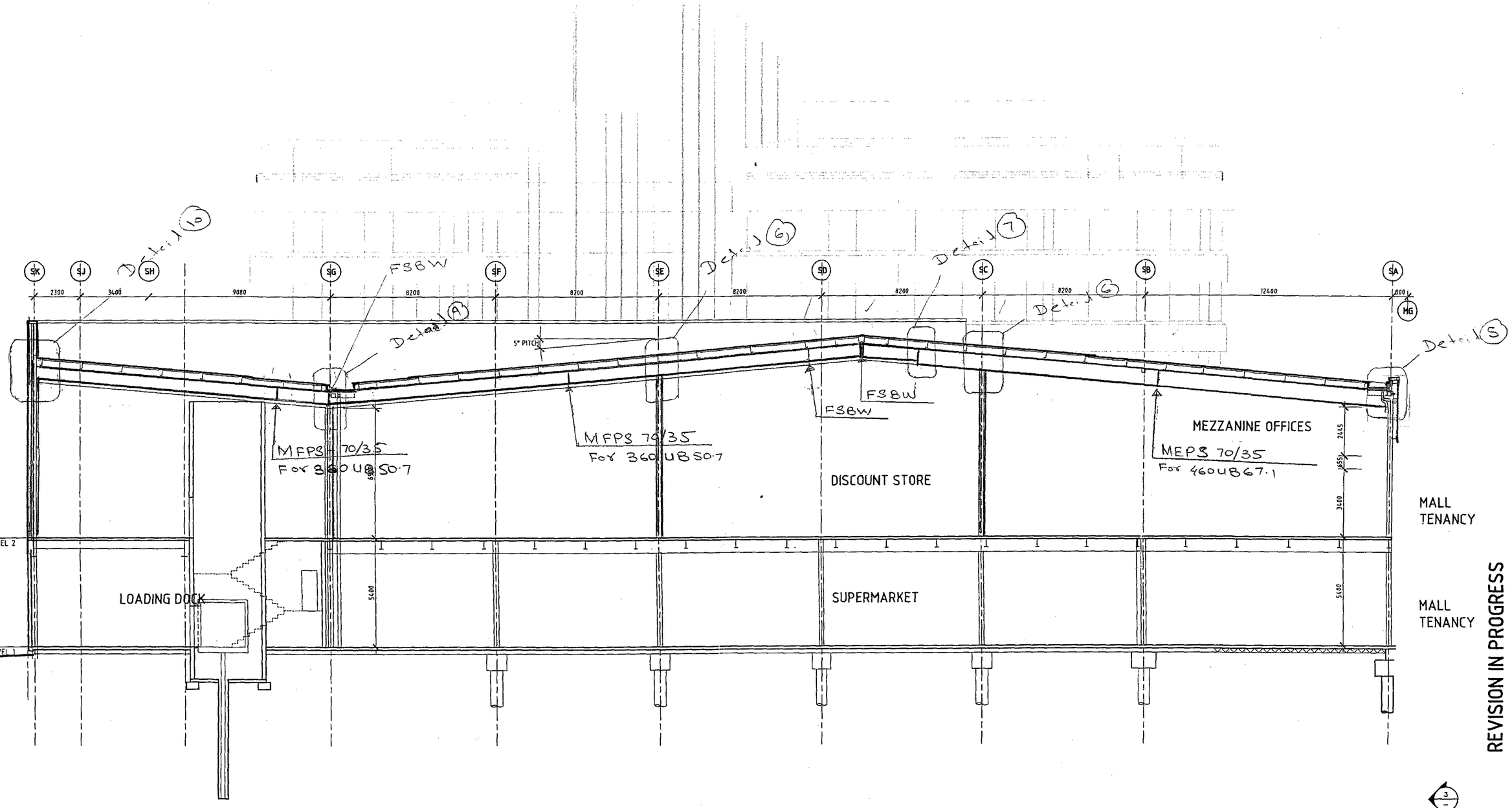
Rev.	Date	By	Description
1	29.3.04	HJB	BILL OF QUANTITIES

Westfield (New Zealand) Limited
L3 cnr REMUERA RD & NUFFIELD ST, New Market, AUCKLAND, New Zealand
Telephone : (09) 978 5050
Fax Num. : (09) 978 5070
Project Architect Mark Oakley Checked

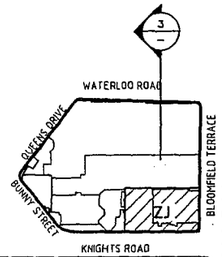
Beca Carter Hollings & Ferner Ltd
Tel 04-473 7551 Fax 04-473 7811
Drawn by M FYFE

Other Consultants
Architects: Tetrad Design Group Limited Tel 04-385 9187 Fax 04-385 1838
Services Engineer: Montgomery Watson NZ Ltd Tel 04-381 6700 Fax 04-381 6738

Project	QUEENSGATE REDEVELOPMENT
Title	ZONE J ROOF PLAN
Scale	1:200
Project/Job Number	2711265
Computer Ref.	S103-ZJ
Drawing Number	S103-ZJ
Library Ref.	M:/QG/CD
Schema	35R
Date Plotted	29/03/04
Revision	1



241059



NOTE

VARIATION NOTE: ATTENTION THE CONTRACTOR

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241059

REVISION IN PROGRESS

PRELIMINARY

Rev.	Date	By	Description	Rev.	Date	By	Description	Rev.	Date	By	Description

Westfield (New Zealand) Limited
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New Market, AUCKLAND, New Zealand
Telephone : (09) 978 5050
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Project Architect Mark Oakley

Checked

Tetrad Design Group Limited
Architects and Interior Designers
5 Napier Street, PO Box 6340, Te Aro, Wellington, New Zealand
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Other Consultants
Structural Engineer
Beca Carter Hollings & Ferner Ltd
Tel 04-473 7551 Fax 04-473 7911
Services Engineer
Montgomery Watson NZ Ltd
Tel 04-381 6700 Fax 04-381 6730

Drawn by

Project: QUEENSGATE REDEVELOPMENT	Scale: A1 1:100, A3 1:200	Project/Job Number: 07004	Computer Ref.: ZJ-SEC02
Title: ZONE J SECTION 5.	Library Ref.: J:/QG/CD	Scheme: 35R	Date Plotted: 18/03/04
			Drawing Number: A305-ZJ
			Revision: 1*

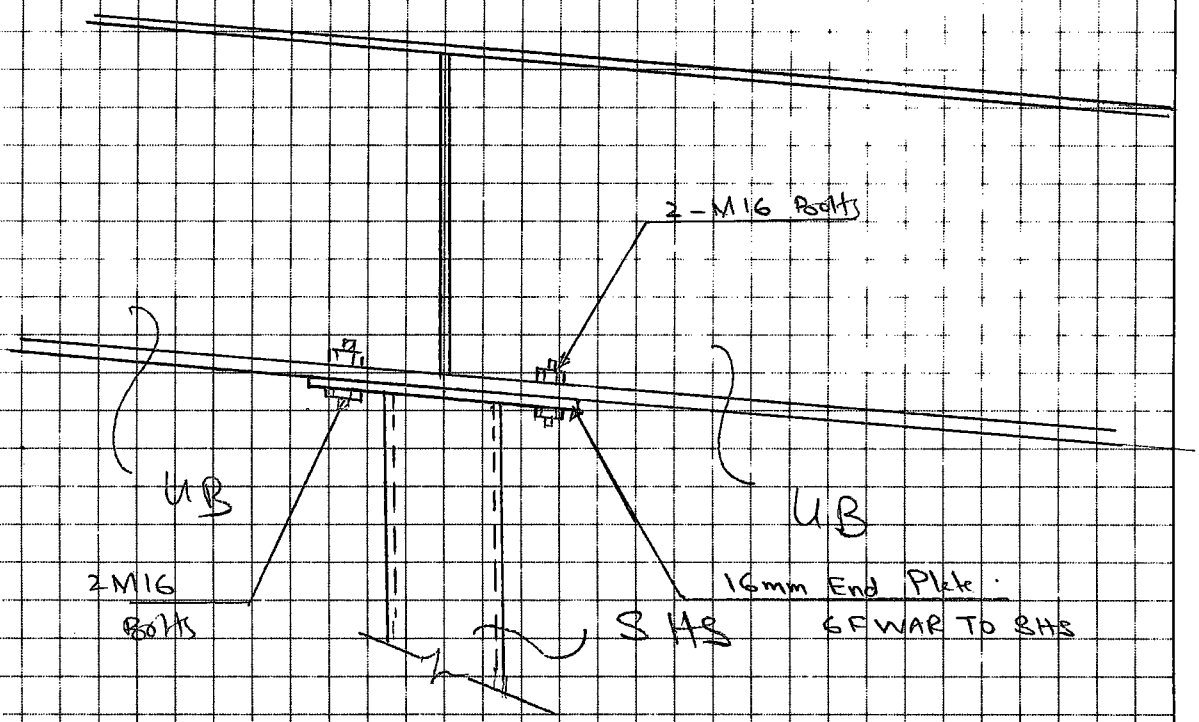
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Calculation Sheet

Job name: **Job No:**

Subject: **Page No:** of

By: **Date:**



Detail (6) Connection between UB Rafter and SHS Column.

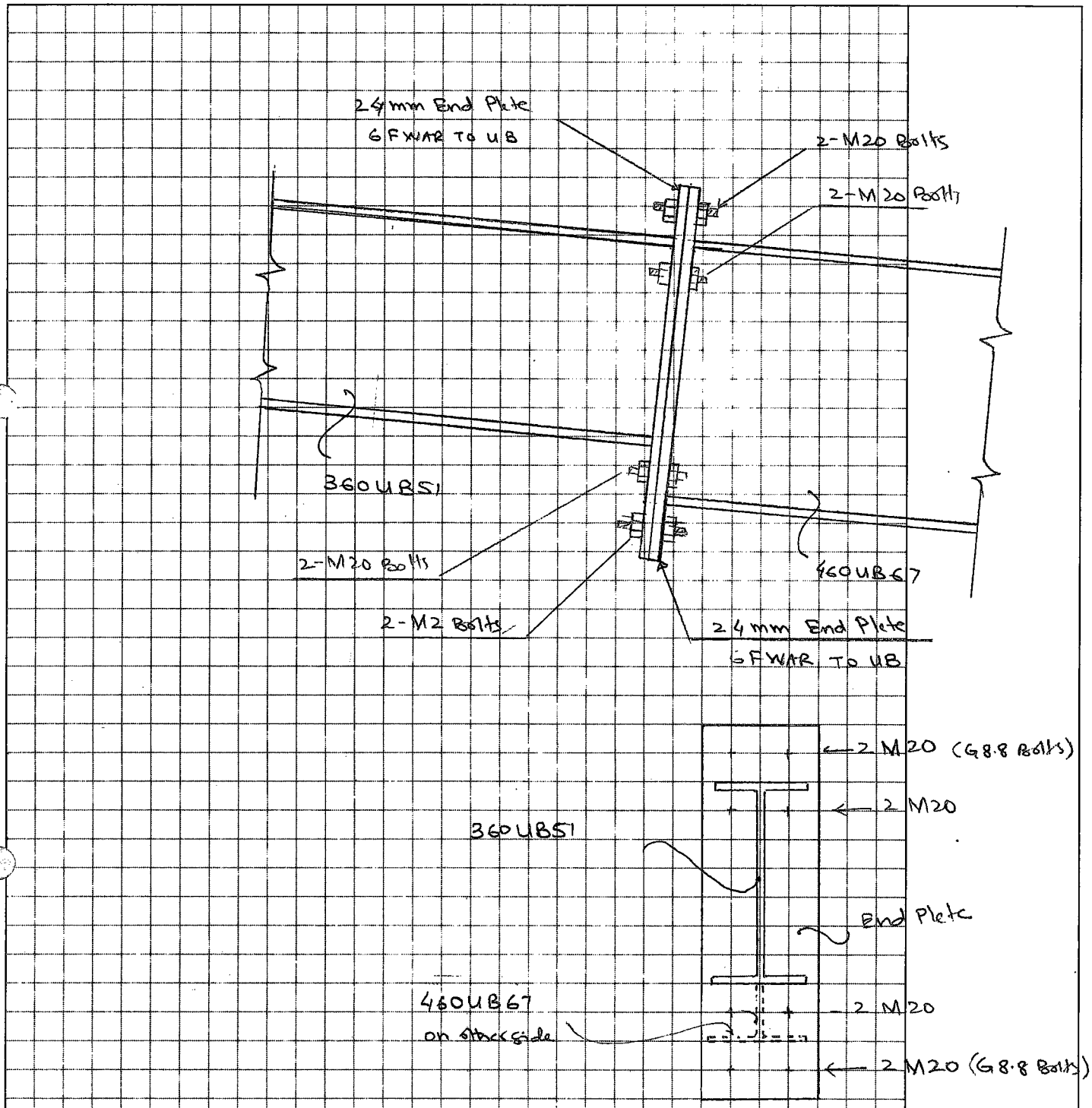
241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



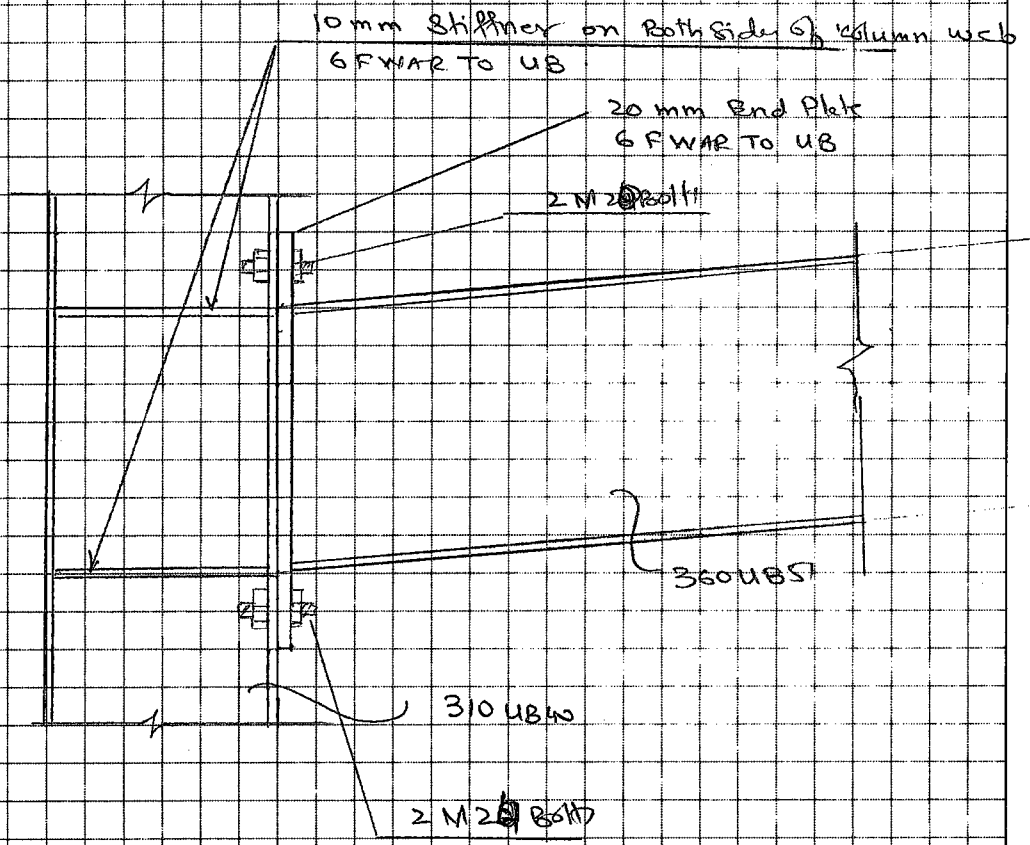
Detail (7) Connection Between 360UB51 and 460UB67 Refer

Calculation Sheet

Job name: **Job No:**

Subject: **Page No:** of

By: **Date:**



Detail (8) Connection Between End column & Rafter @ GL. (8G).

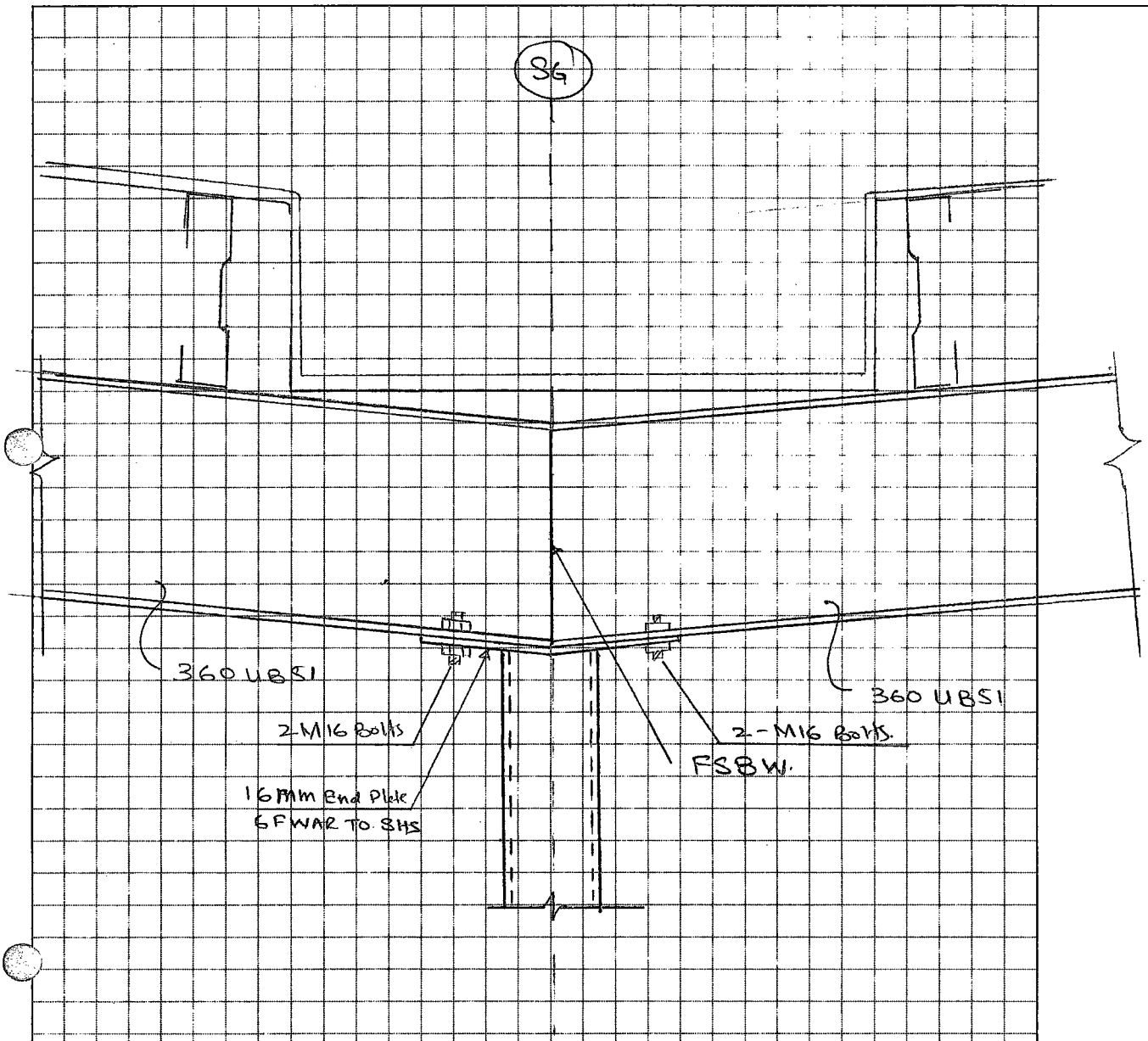
241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Detail (9) Connection between Rafter and Column

@ S5/S6, S6/S6, S7/S6

Calculation Sheet

Job name:

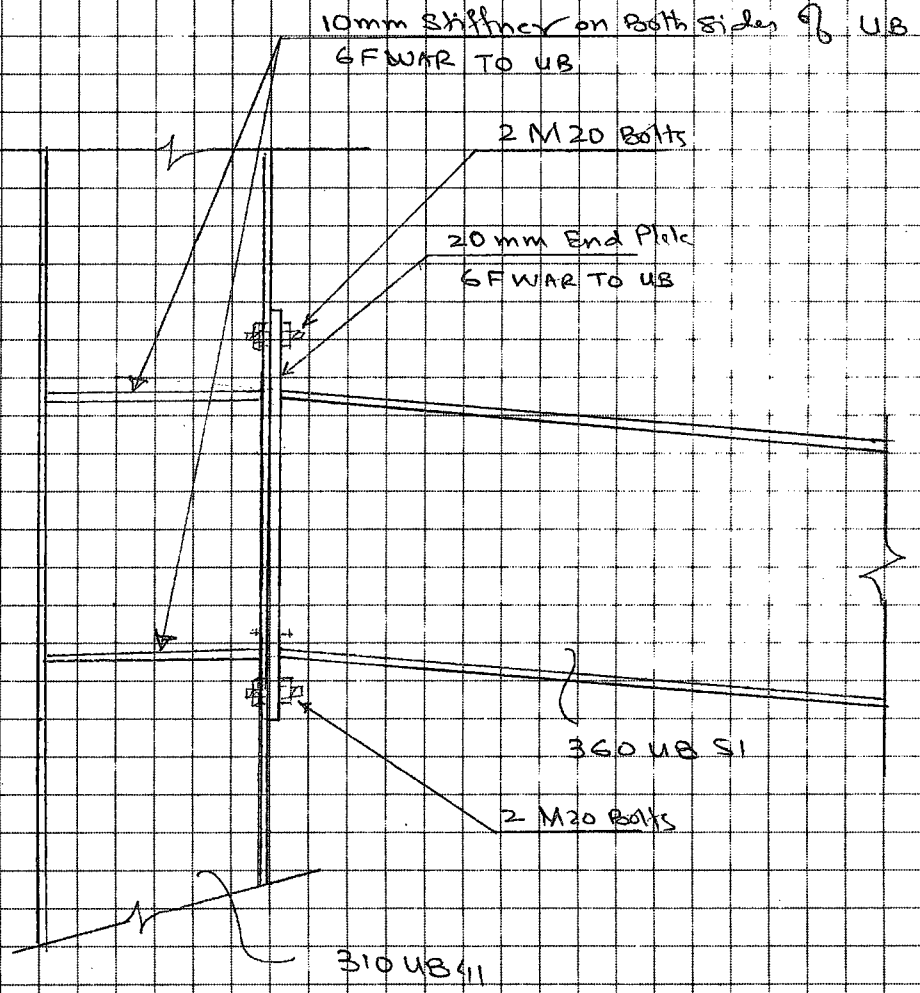
Subject:

By:

Job No:

Page No: of

Date:



Detail (10) Connection Between Column and Rafter

@ S5/SK, S6/SK, S7/SK.

Hamdt 021 443577

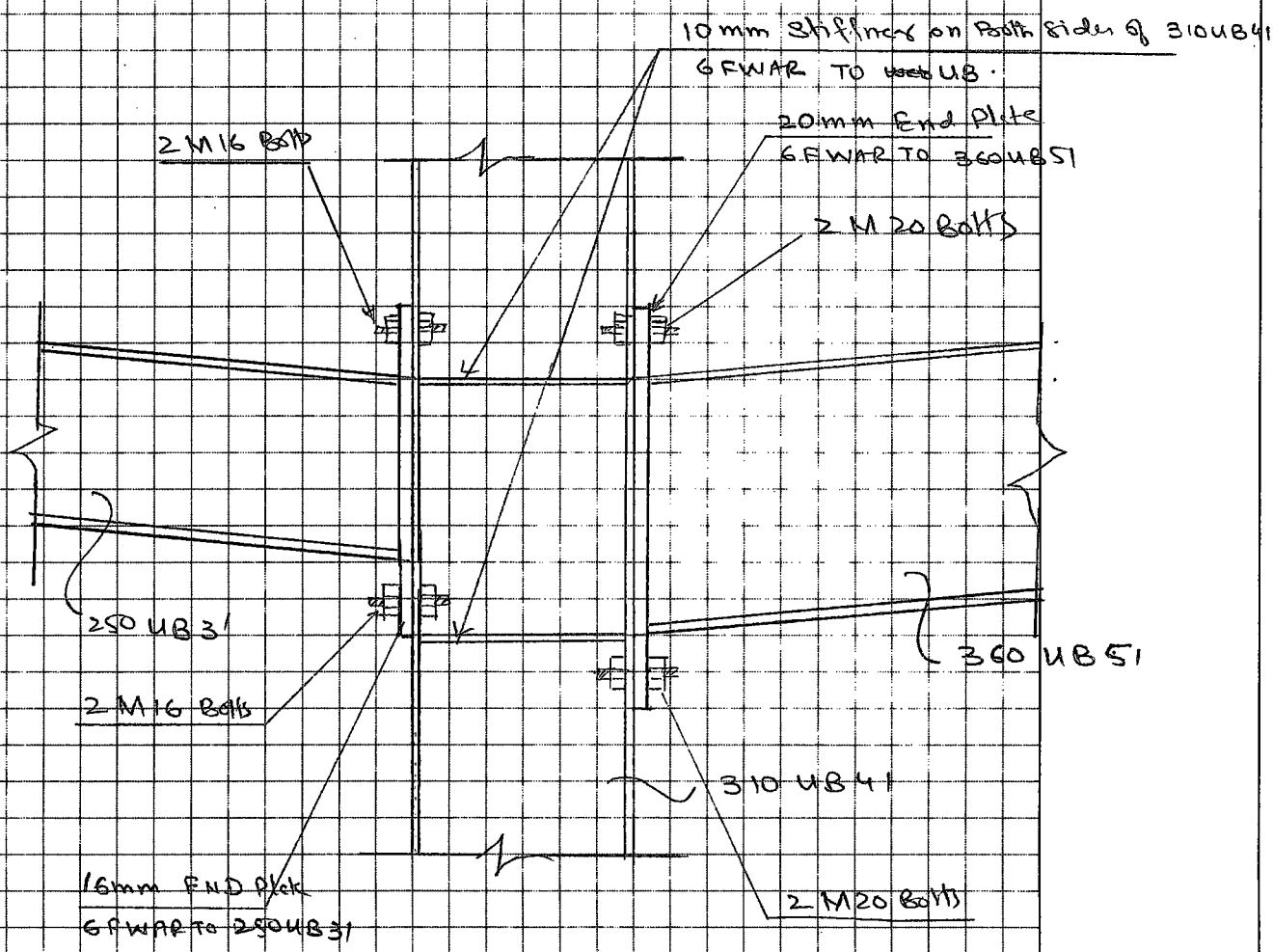
241053

Calculation Sheet

Job name: **Job No:**

Subject: **Page No:** of

By: **Date:**



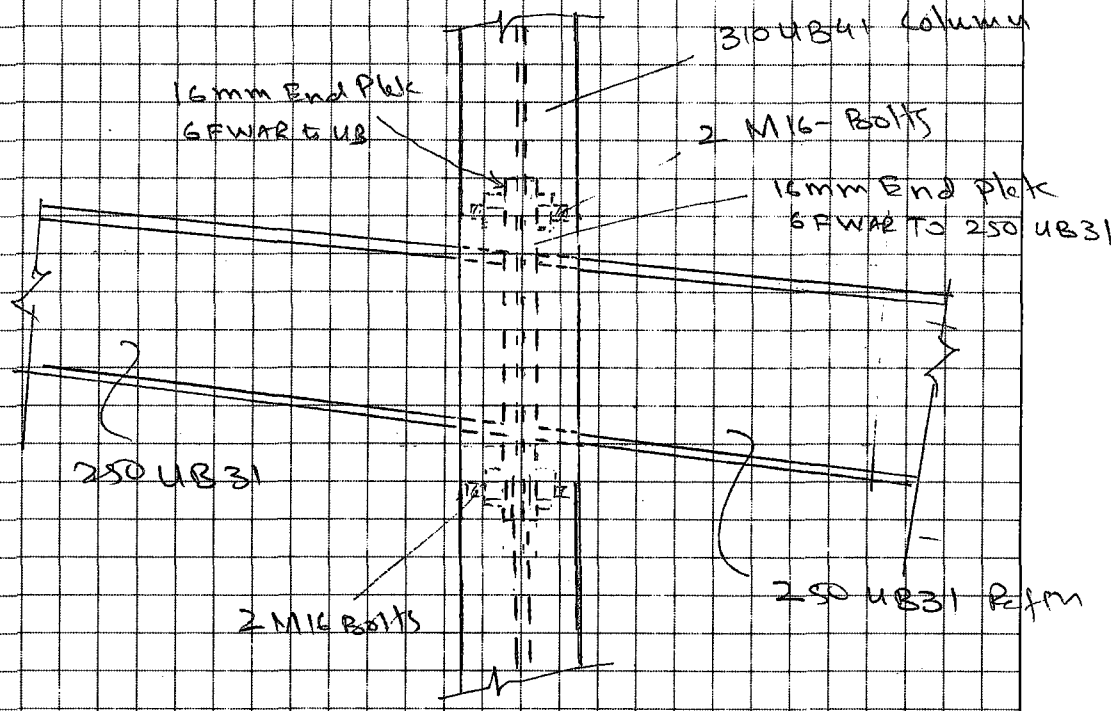
Detail (11) Connection between Rafters and columns at S4/S6 and S8/S6.

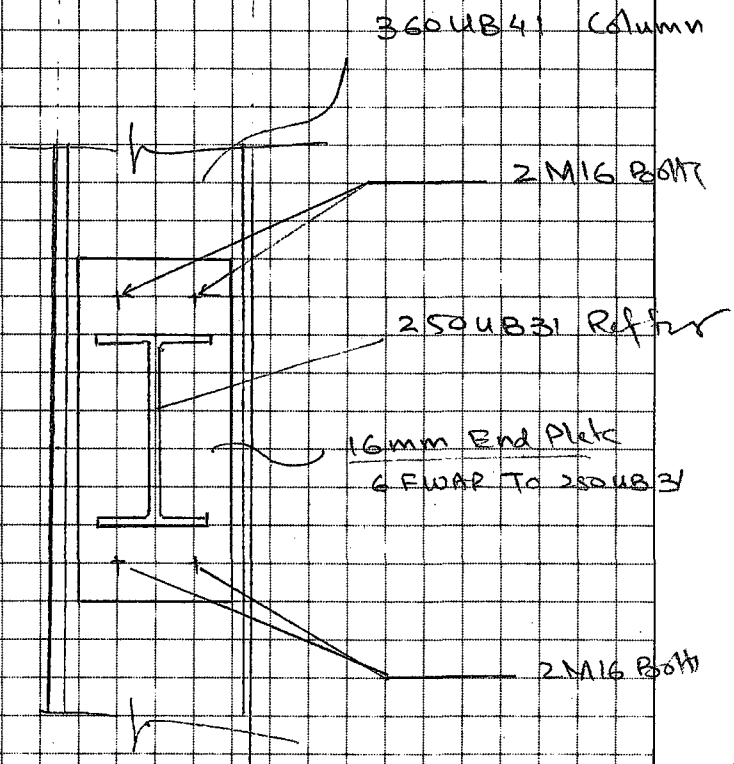
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:





241059

Detail (12) Connection between column and Refm at GL (S11), (S1), S4/SH and S8/SH

Design
Of
Roof Bracing

241059

Calculation Sheet

Job name: Queensgate Redevelopment
Subject: Parts & Parts for 23 Roof.
By: DOS.

Job No: 2711265/109
Page No: 1 of 1
Date: 10-5-04.

P.I ∴ $R_p = 1.10$

$F_{ph} = C_{pL} W_p R_p$ $\quad W_p = 215\,kN$.

$C_{fi} = C_{fn}$ @ roof level

$C_{fn} = \dfrac{C_h(T_1, \mu_0)}{C_h(T_1, \mu)} \times \dfrac{F_n}{W_n}$ $\quad \mu_0 = \dfrac{\mu}{q_0}$

$= \dfrac{6}{1.56 \times 1.2}$

$= 3$

$= \dfrac{C_h(0.1, 3)}{C_h(0.1, 6)} \times \dfrac{7493\,kN}{48,976\,kN}$

$= \dfrac{0.37}{0.19} \times \dfrac{7493}{48,976}$

$= 0.298$

$C_{fi} = 0.298$

$C_{fi} = 0.298$

$C_{pi} = C_h(T_{pi}, \mu_p) \cdot C_{fi}/0.4$

$= C_h(0.45, 1) \dfrac{0.298}{0.4}$

$= 0.8 \times \dfrac{0.298}{0.4}$

$= 0.596$

$C_{pi} = 0.596$

$F_{pL} = C_{pL} W_p R_p$

$= 0.596 \times 2195\,kN \times 1.1$

$F_{pL} = 1439\,kN$.

241059

Calculation Sheet

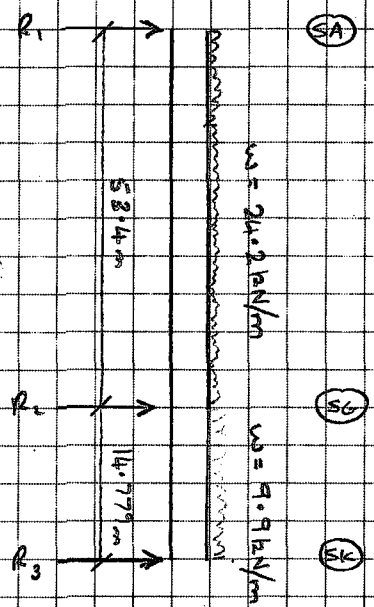
Job name: Queensgate Mall Redevelopment
Subject: Roof Bracing Design Zone 3J
By: Dan Saunders

Job No: 2711265/109
Page No: 1 of
Date: 12-5-04

← loading.

F_{ph} = 1439kN. udl = 23.6 kN/m.

check wind w = 8.3kN/m. Low R_1 & 3 bands.



$R_1 = \frac{wL}{2}$.

$= \frac{24.2\ kN/m \times 53.4m}{2}$

$= 646\ kN$

$R_3 = \frac{wL}{2}$

$= \frac{9.9 \times 14.779}{2}$.

$= 73.2kN$.

$R_2 = R_1 + R_3$.

$= 719\ kN$

R_1 = 646kN
R_2 = 719kN
R_3 = 73.2kN

Roof brace shear = $\frac{646kN}{3bays}$.

$= 215\ kN$

brace shear = 215kN

Tension & compression chords.

$\frac{wL^2}{8 \times 3} \times \frac{1}{5pairs\ of\ brace}$.

$= \frac{24.2 \times 53.4^2}{8 \times 3} \times \frac{1}{10.5m}$

$= 274\ kN$

T & C = 274 kN

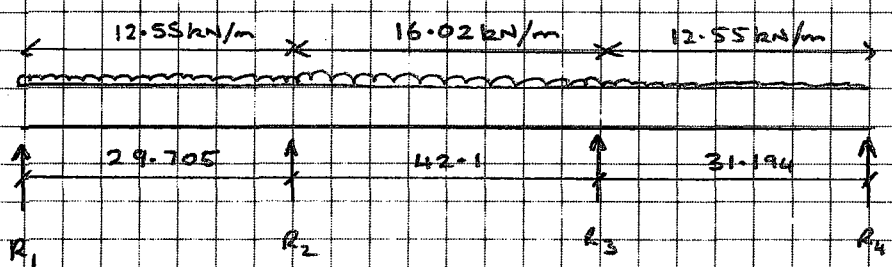
Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Roof bracing Design Zone 23
By: Dean Saunders.

Job No: 27/265/109
Page No: 2 of
Date: 12-5-04

↑ loading.

F_{ph} = 1439kN gowns over wind as per page 1



$R_1 = \frac{wl}{2}$

$= \frac{12.55 \times 29.705}{2}$

$= 186$kN

$R_4 = \frac{wl}{2}$

$= \frac{12.55 \times 31.194}{2}$

$= 196$kN.

$R_2 = R_1 + \frac{wl}{2}$

$= 186 + \frac{16.02 \times 42.1}{2}$

$= 523$kN

$R_3 = R_4 + \frac{wl}{2}$

$= 196 + \frac{16.02 \times 42.1}{2}$

$= 533$kN

R_1 = 186kN
R_2 = 523kN
R_3 = 533kN
R_4 = 196kN

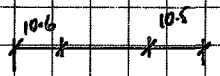
Calculation Sheet

Job name: Queensgate mall Redevelopment
Subject: Roof bracing Design Zone 3
By: Dan Sanderson.

Job No: 2711265/104
Page No: 2 of
Date: 12-5-04

Roof brace shear = $\frac{337}{2 \text{ bays}}$

= 169 kN

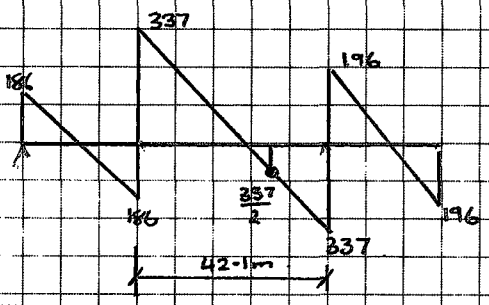


Roof shear = 169kN.

Chord force.

= $\frac{wL^2}{8 \times 2} \times \frac{1}{6.2}$

= 286 kN.



Tee = 286kN.

Chords - Slotted end plate connection to 200×6 SHS.

$N_t^* = 286 kN$
$N_c^* = 286 kN$

$\phi N_t = 0.9 \times 4530 \times 350$
$= 1427 kN$

$0.3 \phi N_t = 0.3 \times 1427 kN$
$= 428 kN$

Check Compression first.

$N_c^* \leq \phi N_s$

$\leq \phi N_c$.

$N_s = k_f A_g f_y$.

$= 1 \times 4530 \times 350$.
$= 1585.5 kN$.

$k_f = \frac{A_e}{A_g}$ $A_g = 4530 mm^2$

$\lambda_e = 31.3 \times \sqrt{\frac{350}{250}}$

$= 37$

$\lambda_{ey} = 40$

$b_e = b$.
$k_f = 1$

$N_c = \alpha_c N_s$ $\lambda_n = \frac{10540}{786} \times \sqrt{1} \times \sqrt{\frac{350}{250}}$

$= 158$

$\alpha_b = 0.5$.
$\alpha_c = 0.288$.

$N_c = 0.288 \times 1585.5 kN$
$N_c = 456 kN$

Calculation Sheet

Job name:

Subject: roof bracing 23

By: DWS

Job No: 2711265

Page No: 4 of

Date: 13-5-04

$N_c^* \le \phi N_c$

$\le 0.9 \times 456$

$286\text{kN} \le 410\text{kN}$ ✓

$\phi V_b = \phi 3.2 \times d_f \times t_p \times f_{up}$ M24 t_p=12 below.

$= 0.9 \times 3.2 \times 24 \times 12 \times 400$

$= 365\text{kN}$

$\phi V_b = \phi \alpha_{b2} \cdot d_f \cdot t_p \cdot f_{up}$

$= 0.9 \times 59 \times 12 \times 400$

$= 280$

$N^* = n_b \phi V_b$

$286 \le n_b \times 280$

$n_b \ge 2$ M24 bolts for compression.

Back to tension

must use $N_t^* = 428\text{kN}$

$N_t = \phi N_t$

$= 0.9 \times 0.85 \times 1 \times (4530 - 2 \times 6 \times 6) + 430$

$= 1466\text{kN}$.

$\phi N_{ts} = 0.9 \times 0.85 \left(k_{t1} A_{si} + 2 k_{t2} A_{so}\right) f_{up}$

$= 0.9 \times 0.85 \cdot \left(0.5 \times 200 \times t_p + 2 \times 1 \times 10 \times t_p\right) 400$.

$428\text{kN} \le 0.9 \times 0.85 \times 400 \times 12 t_p$.

$t_p \ge 10.6\text{ mm}$ use $t_p = 12\text{mm}$ t_p=12mm

Bolts

$N^* = \phi n_b V_f$.

$456\text{kN} \le n_b \phi V_f$

$n_b \ge 5$ for M20

$n_b \ge 4$ for M24

241050

Calculation Sheet

Job name:

Subject: Roof bracing ZJ

By: DWS

Job No: 2711265/109

Page No: 5 of

Date: 13-5-04

Weld

$N^* \leq 4 \times \phi V_w L_s$

$428kN \leq 4 \times 0.835 \times L_s$

$L_s \geq 128\ mm$. make $L_s = 150mm$

$L_s = 150$ mm.

Web bearing & tension

$\phi V_b = 365kN$

$\phi V_b = \phi a_c t_w f_{up}$

$= 0.9 \times 35 \times 12 \times 440$

$= 166kN$

$N^* \leq \phi n_b V_b$

$428kN \leq 4 \times 166kN$

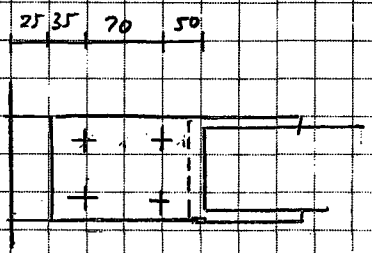
$428kN \leq 665kN$.

Block shear

$\phi N_{tb} = \phi (0.6 f_{up} A_{vg} + f_{up} A_{tg})$ or $\phi (0.6 f_{up} A_{vs} + f_{up} A_{tg})$

$= 0.9(0.6 \times 300 \times 2520 + 440 \cdot 528)$ or $0.9(0.6 \times 440 \cdot 1584 + 300 \cdot 840)$

$= 617kN$ or $603kN$



$428kN \leq 617kN$ ✓

Cleat Compression

$N_c = \phi N_c = \dfrac{\phi P_c}{1 + \left(\dfrac{P_c}{n_r \cdot P_y}\right)}$

$P_{cy} = f_{ye} t_c b_c$

$= 300 \times 220 \times 12$

$= 792kN$

$P_{ec} = \dfrac{4\pi^2 E I_c}{L_c^2}$

$= 7720kN$

$\lambda \leq \sqrt{2}$

$P_c = P_y \left(1 - \dfrac{P_y}{4 P_{ec}}\right)$

$= 772kN$

Calculation Sheet

Job name:

Subject: Roof bracing P3

By: DWS

Job No: 2711265/109

Page No: 6 of

Date: 13-5-04

$\alpha = \frac{140}{165}$ $\beta = 1$ $\gamma = \frac{155}{165}$

$= 0.848$ $= 0.939$

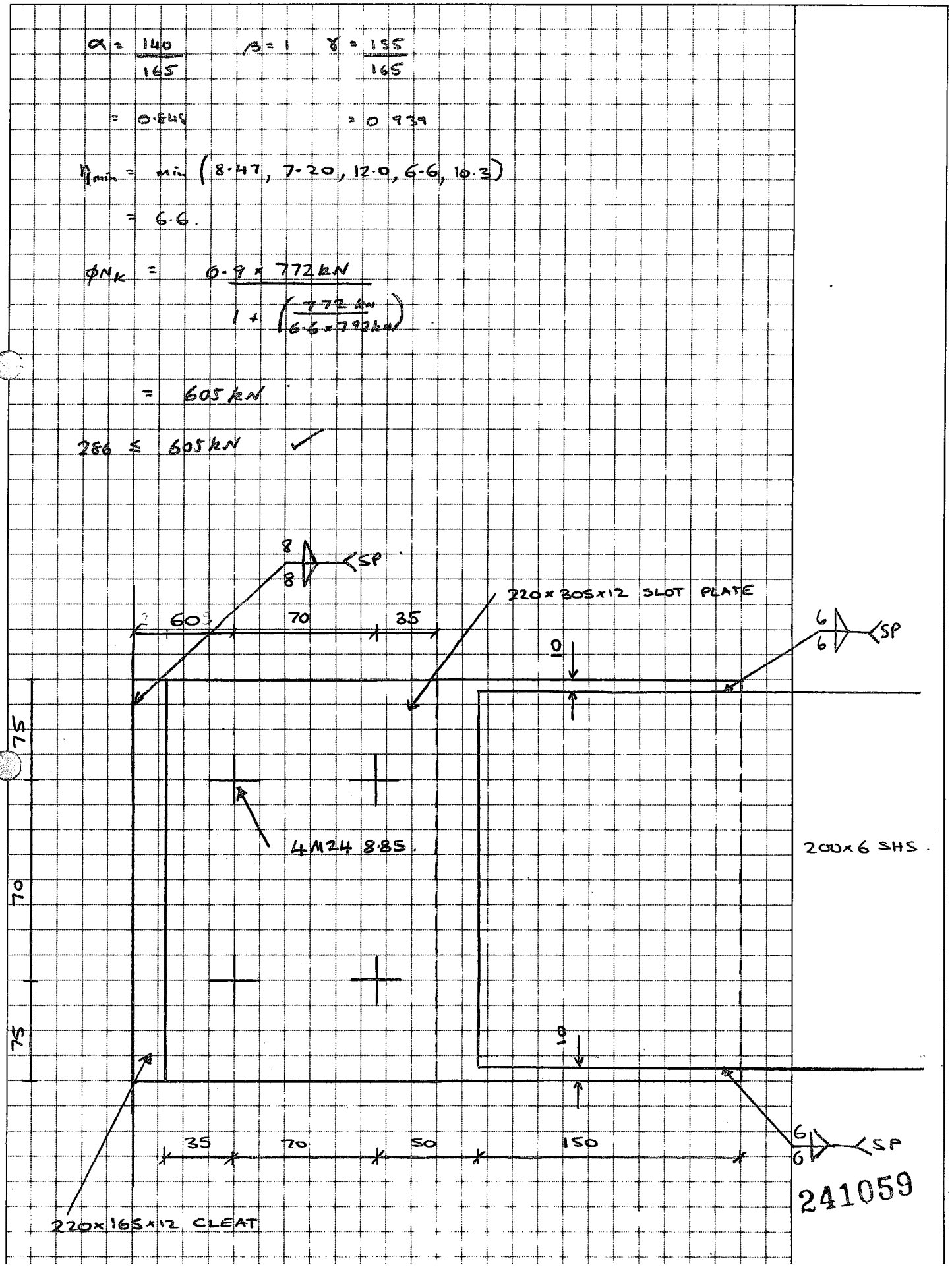
$\eta_{min} = \min(8.47, 7.20, 12.0, 6.6, 16.3)$

$= 6.6.$

$\phi N_k = \frac{0.9 \times 772\,kN}{1 + \left(\frac{772\,kN}{6.6 \times 772\,kN}\right)}$

$= 605\,kN$

$286 \leq 605\,kN$ ✓



Calculation Sheet

Job name:
Subject: Roof bracing 2 3
By: DWS

Job No: 2711265/109
Page No: 7 of
Date: 13-5-04

Diagonal Bracing:

← $V^* = 215\,kN$.

worst angle = 44°

$N^* = \frac{215\,kN}{\cos 44°}$

$= 300\,kN$

check

$N^* = 276\,kN$

can use 3M20

$\phi N_t = \phi A_g f_y$ or $\phi \times 0.85 k_t A_n f_u$

$= 0.9 \times A_g \times 300$ or $0.9 \times 0.85 \times 0.85 \times (A_g - df \times t_p) \times 440$.

$300\,kN \le \frac{270 A_g}{1000}$ or $300\,kN \le 0.9 \times 0.85^2 \times 440 \times (A_g - df \times t_p)$

$300{,}000 \le 286.11 (A_g - df t_p)$

$A_g \ge 1111\,mm^2$

Try 100×100×8 $A_g = 1500$. with 3 × M24

$300kN \le 286.11 (1500 - 26 \times 7.8)$

$300kN \le 371kN$ ✓

BR2 use 100×100×8

$N^* \le \phi V_b \times n_b$

$\le 0.9 \times 3.2 \times 7.8 \times 24 \times 440 \times 3$

$\le 237 \times 3$

$300kN \le 711kN$ ✓

$N^* \le \phi V_{pn} n_b$

$\le 0.9 \times 35 \times 7.6 \times 440 \times 3$

$300kN \le 324\,kN$. ✓

Calculation Sheet

Job name:

Subject: Roof bracing PT

By: DWS

Job No: 2711265/104

Page No: 8 of

Date: 13-5-04.

↑ $V^* = 169\ kN$

worst angle = 59.7° others 52°

$N^* = \frac{267\ kN}{\cos 59.7°}$ $N^* = 274\ kN$

$= 335 kN$

This is support by BR2 from other braces

Can design for lower shear for BR1

$V^* = \frac{196\ kN}{2\ bays}$

$= 98 kN$

$N^* = \frac{98\ kN}{\cos 59.7°}$

$= 194 kN$

$N^* \leq \phi N_t$

$194{,}000 \leq 0.9 \times A_g \times 500$

$A_g \geq 431\ mm^2$

use RH28 $A_g = 616 mm^2$ $A_{TS} = 492 mm^2$ ✓

RH28

241059

Calculation Sheet

Job name:
Subject: Roof bracing 75
By: DWS

Job No: 2911265/104
Page No: 9 of
Date: 13-5-04

cleats

$N^* = 300kN$

$\phi N_t \leq \phi A_g f_y$

$300,000 \leq 0.9 \times 300 \times A_g$

$A_g \geq 1111$

use 120 × 12 plate $A_g = 1440mm^2$

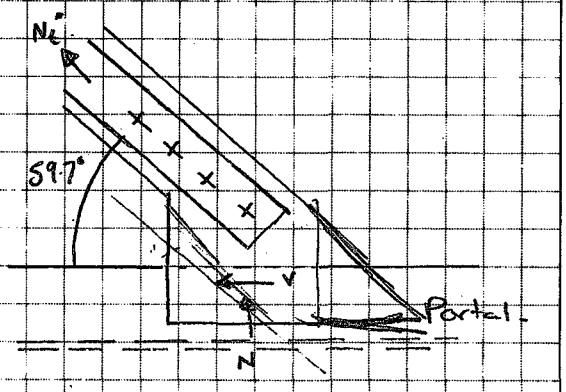
$\phi N_t \leq 0.9 \times 0.85 \times 1 \times (1440 - 26 \times 12) \times 440$

$\leq 380kN$

$300 \leq 380kN$ ✓

bar to plate 120×12EA

weld.



$N = 259 kN$
$V = 151 kN$

$259kN + 151kN \leq \phi v_w \times 2 \times L_s$

$410kN \leq 0.8 \times 1111 \times 2 \times L_s$

$L_s \geq 231mm.$

8 FW SP.
$L_s = 250mm$

Calculation Sheet

Job name:

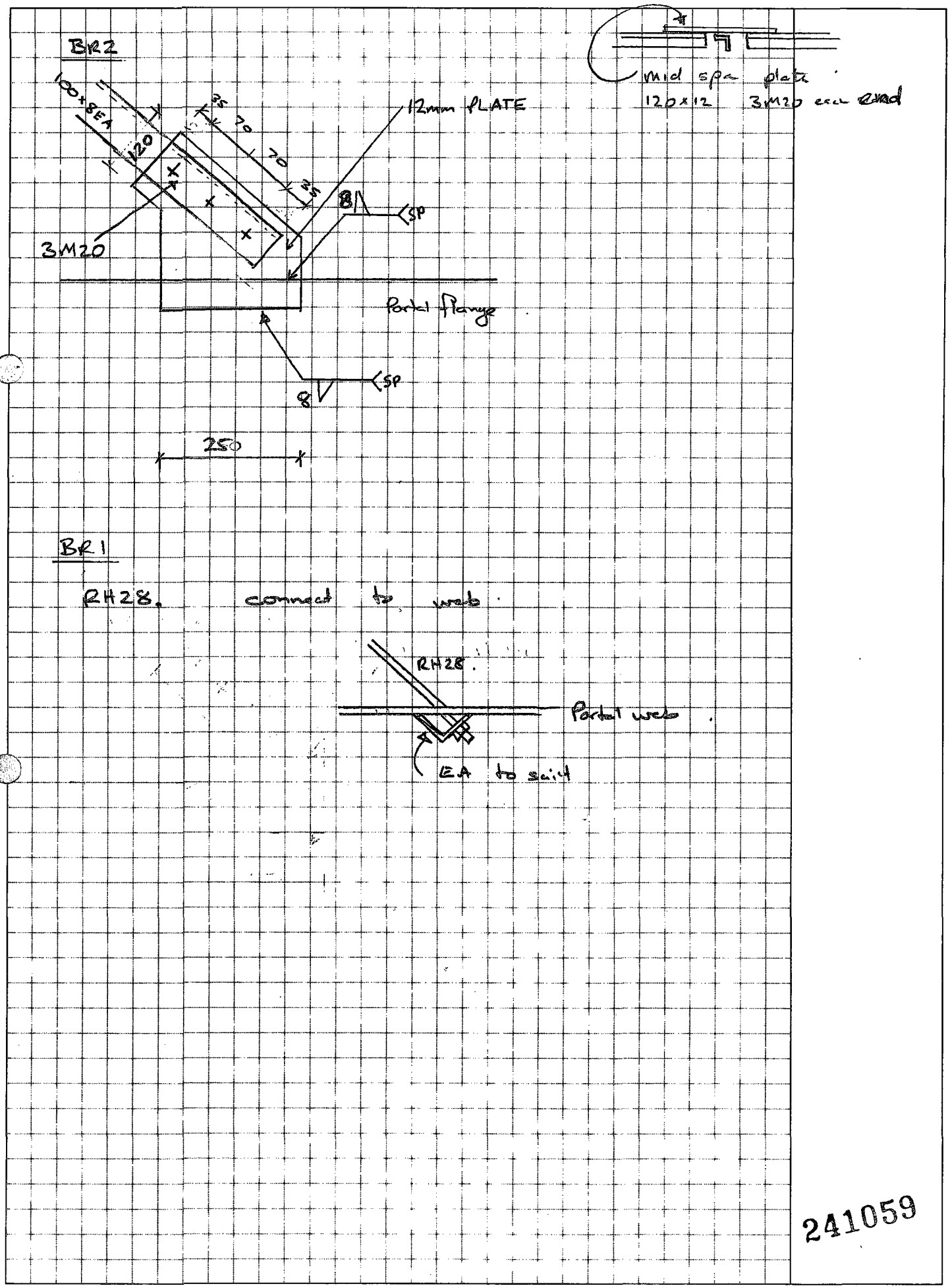
Subject: Roof bracing 7 5

By: DWS

Job No: 2711265/109

Page No: 10 of

Date: 13-5-04



241059

Calculation Sheet

Job name:

Subject: Roof bracing 7 J

By: DWS

Job No: 2711265/109

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Date: 14-5-04

Check Compression members.

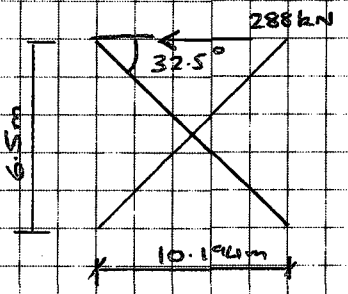
$\phi N_c = 400\,kN$ for ST21 200×6 SHS

Max $N^* = \frac{946\,kN}{3} + 73.2\,kN$

$= 288\,kN$ ✓ ok.

wall bracing:

← load.



$N^* = \frac{288}{\cos 32.5°}$

$= 341\,kN$ ⇒ 3 M24 bolts in shear.

$N^* \le \phi N_t$

$341\,kN \le 0.9 \times 300 \times A_g$

$A_g \ge 1263\,mm^2$

use 125×12 $A_g = 1500\,mm^2$.

$\le \phi N_t$

$\le 0.9 \times 0.85 \times 1 \times (1500 - 26 \times 12) \times 440$.

$341\,kN \le 400\,kN$ ✓

$N^* \le \phi V_b$.

$\le 0.9 \times 3.2 \times d_f \times t_p \times f_{up} \times n_b$ & $\le \phi a_e t_p f_{up} \times n_b$

$\le 0.9 \times 3.2 \times 26 \times 12 \times 440 \times 3$ & $\le 0.9 \times 35 \times 12 \times 440 \times 3$

$341\,kN \le 1185\,kN$ & $499\,kN$. ✓

Calculation Sheet

Job name:

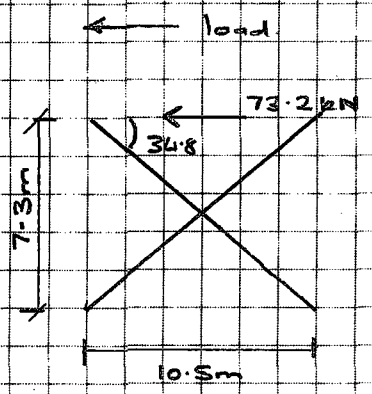
Subject: Roof bracing 7j

By: DWS

Job No: 2711265/104

Page No: 12 of

Date: 14-5-04



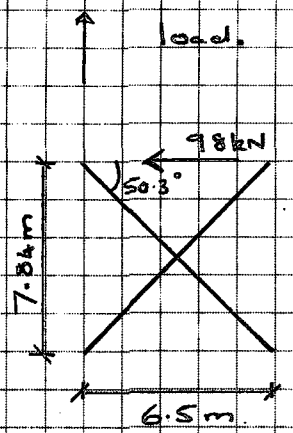
$N^* = \frac{73.2}{\cos 34.8^\circ}$

$= 89\,kN$

$89\,kN \leq 0.9 \times 500 \times A_g$

$A_g \geq 198\,mm^2$ use RH20 min $A_{sf} = 245\,mm^2$ with hexcel

$89\,kN \leq 110\,kN$ ✓



$N^* = \frac{98\,kN}{\cos 50.3^\circ}$

$= 153.4\,kN$

$153.4\,kN \leq 0.9 \times 500 \times A_g$

$A_g \geq 341\,mm^2$ use RH24 min $A_{sf} = 353\,mm^2$ with hexcel.

$153.4\,kN \leq 158\,kN$ ✓

241059

Calculation Sheet

Job name:

Subject: Roof bracing. 2J

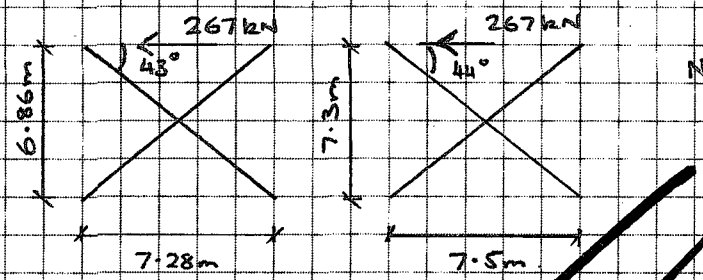
By: DWS

Job No: 2711265/109

Page No: 13 of

Date: 14-5-04

load.



$N^* = \frac{267\text{kN}}{\cos 44°}$

$= 373\text{kN}$. 3M24 in shear.

~~S/S.~~

$373\text{kN} \leq 0.9 \times 300 \times A_g$

$A_g \geq 1380\text{mm}^2$

use 125×12 $A_g = 1500\text{mm}^2$

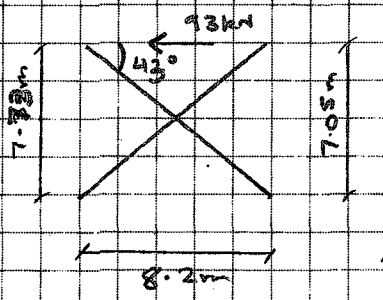
$\leq \phi N_t$

$\geq 0.9 \times 0.85 \times 1 (1500 - 2 \times 6 \times 12) \times 440$

$373\text{kN} \leq 400\text{kN}$ ✓

125×12 PLATE WITH 3M24.

load.



$N^* = \frac{93\text{kN}}{\cos 42°}$

$= 128\text{kN}$.

use 150×90×12 L 2H 28 $\phi N_t = 221\text{kN}$ ✓

Calculation Sheet

Job name:

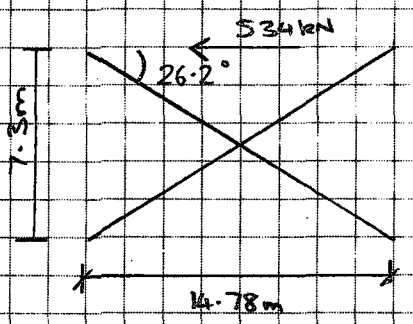
Subject: Roof bracing 2J

By: DWS

Job No: 2711261/109

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Date: 14-5-04



$$N^* = \frac{534\,kN}{\cos 26.2^\circ}$$

$$= 596\,kN.$$

$596\,kN \leq 0.9 \times 300 \times 150 \times t_p$

$t_p \geq 14.7\,mm$

Use 150×16

$N^* \leq \phi N_t$

$596\,kN \leq 0.9 \times 0.85 \times 1 \times (150\times16 - 26\times16) \times 440$

$596\,kN \leq 668\,kN$

150×16 Plate with 5-M24.

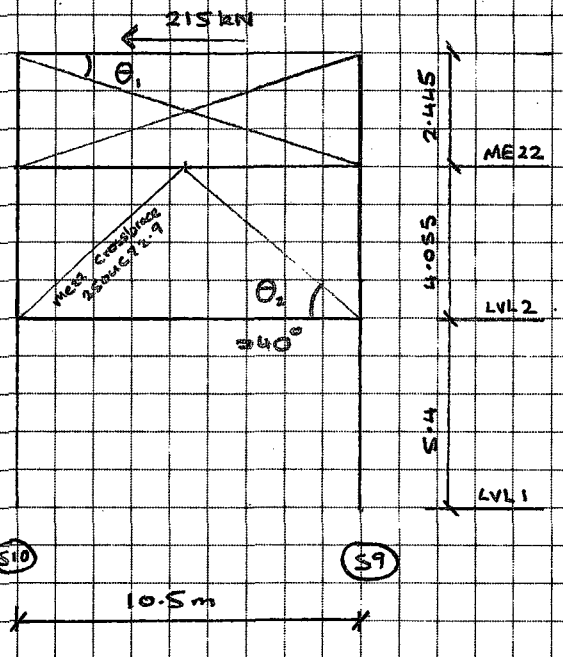
241059

Calculation Sheet

Job name: Queenstown wall Redevelopment
Subject: wall bracing revision.
By: DWS

Job No: 2711265/109
Page No: of
Date: 25-5-04

Shift wall bracing on SA, S10 - S11 to S9-S10 to align with mezzanine cross bracing.



$\theta_1 = \tan^{-1}\frac{2.445}{10.5}$

$= 13.1^\circ$

$N^*_{brace} = 215 \times \frac{1}{\cos\theta_1}$

$= 221\,kN$

Additional load to 250UC72.9

$= \frac{221}{2} \times \frac{1}{\cos\theta_2}$.

$= 144\,kN$

$N^* = 880\,kN + 144\,kN$
$\approx 1024\,kN$

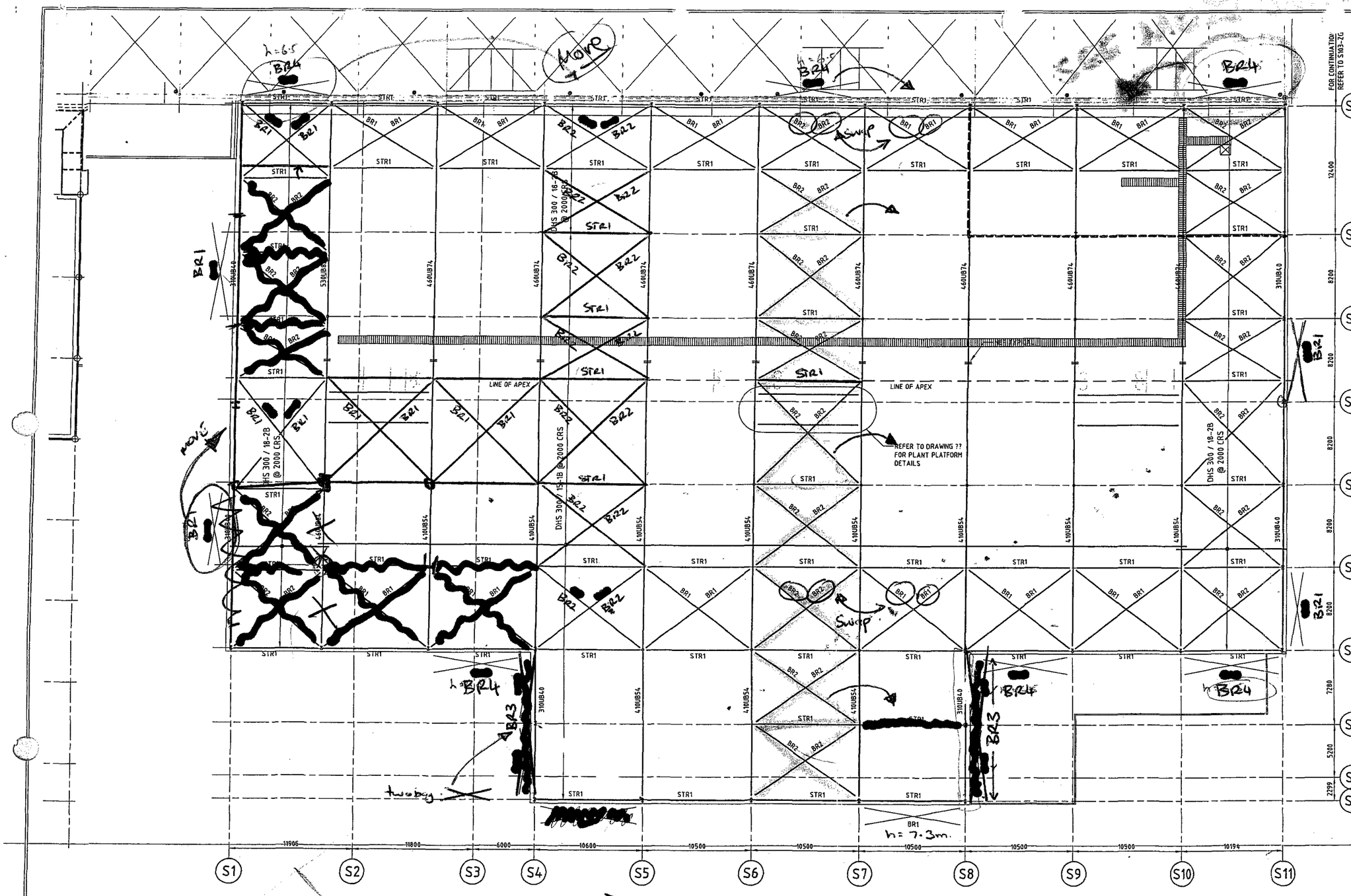
$L_e = 6.63\,m$

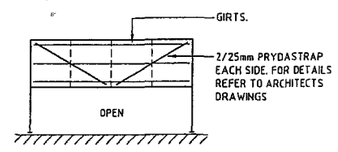
$\phi N_c = 1163\,kN$

$N^* \le \phi N_c$ ✓

Wall brace

$N^*_t = 221\,kN$. use BR14 detail 125x12 PLATE with 3/M24 each end.





DETAIL OCCURS ON GRID SA

NOTE

1. FOR ROOF BRACING & PURLIN CLEAT DETAILS SEE S351

MEMBER	SIZE
STR1	200x6 SHS

2. ALL PURLINS TO BE LAPPED 10% OF SPAN AT SUPPORTS.
3. WALL GIRTS TO BE DHS 300/18-2B@1800crs WITH CHANNEL BEAM (NO RODS). BOTTOM TWO SPANS TO BE AT MAX 900crs
4. REFER TO ARCHITECTS DRAWINGS FOR EXTENT OF PERIMETER PARAPETS
5. FLYBRACES
   -ROOF BEAMS (NUMERIC GRIDS).-ALLOW 12 FLYBRACES PER RAFTER
   -COLUMNS.-ALLOW 2 FLYBRACES PER EXTERNAL WALL COLUMN.

BR2 100x8EA with 3M20 each end

BR1 RH28

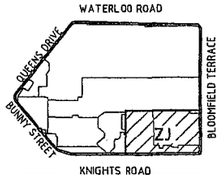
BR4 125x12 PLATE with 3M24 each end.

BR3 150x16 PLATE with 5M24 each end.

NOT FOR CONSTRUCTION

WORKING COPY

PRELIMINARY



NOTE
ALL DIMENSIONS TO BE CHECKED ON SITE. WRITTEN DIMENSIONS ONLY TO BE USED. REFER TO ALL DETAIL DWGS, STRUCTURAL MECHANICAL & SERVICE DWGS BEFORE COMMENCING WORK. REFER ANY DISCREPANCIES TO ARCHITECTS.

274 kW additional load Toric to Portal frame for roof bracing.

Rev.	Date	By	Description
1	29.3.04	HJB	BILL OF QUANTITIES

Westfield (New Zealand) Limited
L3 cnr REMUERA RD & NUFFIELD ST, New Market, AUCKLAND, New Zealand
Telephone : (09) 978 5050
Fax Num. : (09) 978 5070
Project Architect Mark OakleyChecked

Beca Carter Hollings & Ferner Ltd
Tel 04-473 7851 Fax 04-473 7811
Drawn by M FYFE

Other Consultants
Architects
Tatrod Design Group Limited
Tel 04-385 8187 Fax 04-385 1834
Services Engineer
Montgomery Watson NZ Ltd
Tel 04-381 8700 Fax 04-381 8728

Project	QUEENSGATE REDEVELOPMENT
Title	ZONE J ROOF PLAN
Scale	1:200
Project/Job Number	2711265
Computer File	S103-ZJ
Drawing Number	S103-ZJ
Library Ref.	M:/QG/CD
Scheme	35R
Date Plotted	29/03/04
Revision	1

241059

Design
Of
External Wall Girt

241059

Calculation Sheet

Job name: Queensgate Redevelopment — Job No:
Subject: Warehouse — Page No: of
By: Jamil — Date:

Horizontal Purlins.

For Ideal wind loads Purlins span column to column.
Purlins spacing is 2m c/c

Design wind Load Pr.

ULS = 1.0 kPa
SLS = 0.6 kPa

Purlin span = 8.2 m.

$W_S = 2 \times 0.6 = 1.2$ kN/m (1.44)
$W_U = 2 \times 1.0 = 2.0$ kN/m (2.4)

SLS Deflection
For simply supported condition

$$\delta = \frac{5(W_S)L^3}{384EI}$$

$$\delta = \frac{5 \times (1.2 \times 8.2) \times 8200^3 \times 1000}{384 \times 205 \times 10^3 I}$$

$$\delta = \frac{3.44605 \times 10^8}{I}$$

For DHS 200/18 δ = 78.33 mm $\frac{L}{\delta}$ = 104.6

For DHS 250/18 (6.81 kg/m) δ = 42.076 mm $\frac{L}{\delta}$ = 195

For DHS 300/18 δ = 24.62 mm $\frac{L}{\delta}$ = 333.06

For DHS 300/15 (6.70 kg/m) δ = 29.59 mm $\frac{L}{\delta}$ = 277.13 1.44

Use DHS 250/18 Purlin with at least one Bracking
Edge one in both 2B
Roof 1B

Reaction From Purline to vertical col.

$R_U = \frac{W_U l}{2} = 8.2$ kN.

$R_S = \frac{W_S l}{2} = 4.92$ kN.

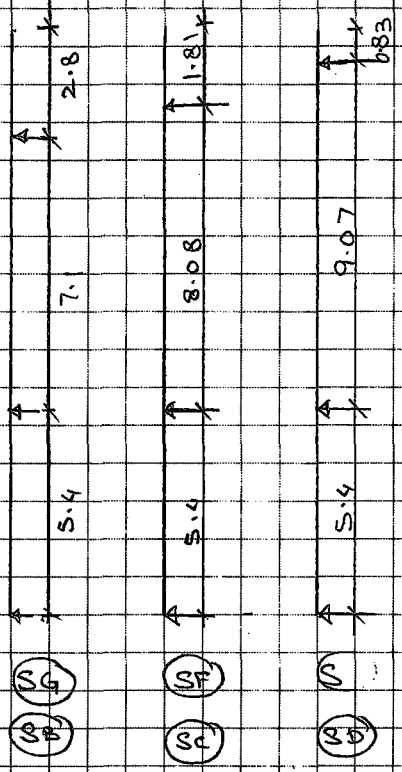
241059

Calculation Sheet

Job name: **Job No:**

Subject: **Page No:** of

By: **Date:**



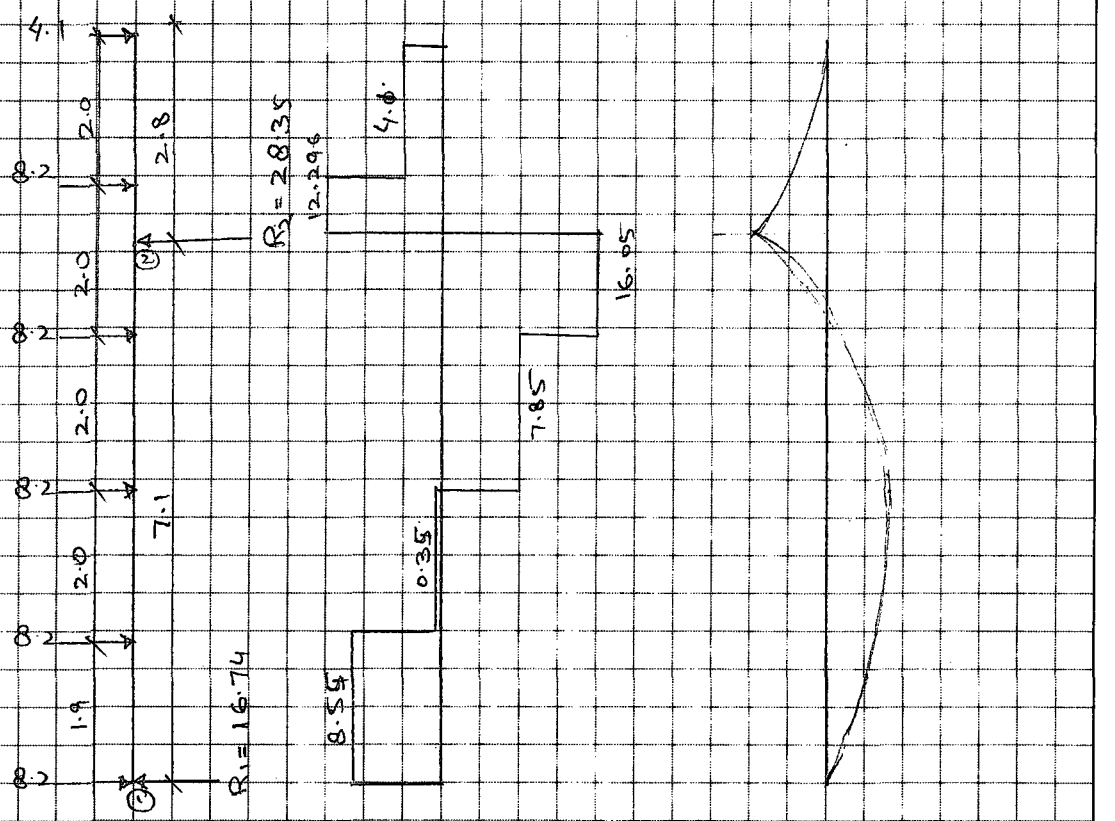
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Vertical Members @ GL. (B6) & (B8)



$\Sigma M @ (1) = 0$ (↻)

$R_2 \times 7.1 - 8.2 \times (1.9 + 3.9 + 5.9 + 7.9) + 4.1 \times 9.9$

$R_2 = 28.35$ kN

$R_1 = 8.2 \times 5 + 4.1 - 28.35 = 16.74$ kN

+ve $M_u = 8.55 \times 1.9 + .35 \times 2 = 16.945$ kN-m

-ve $M_u = 4.1 \times 2 + 12.296 \times 0.8 = -18.032$ kN-m

241059

241[illegible]

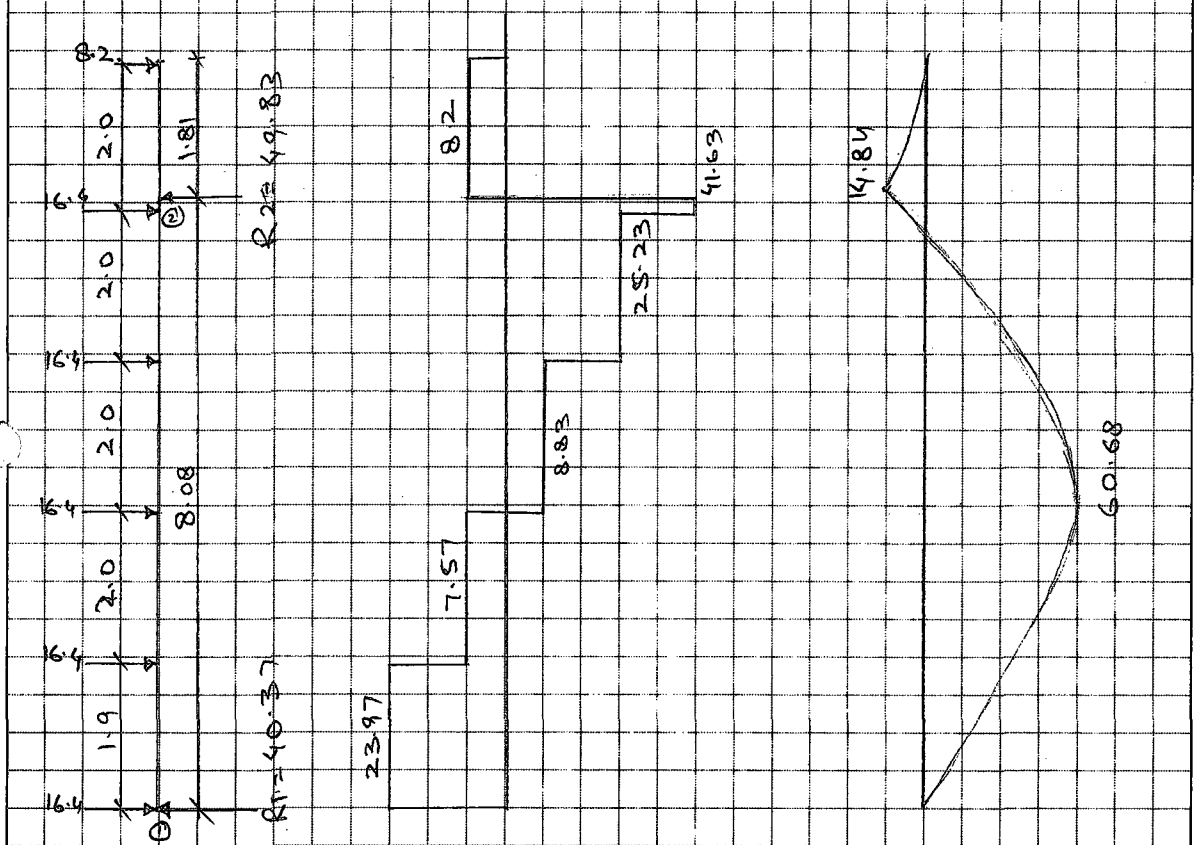
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Vertical Member @ GL (SF) & (SC)



M in @ ① ↻ +ve

$R_2(8.08) = 16.4 \times (1.9 + 3.9 + 5.9 + 7.9) + 8.2 \times 9.9$

$= 16.4 \times 19.6 + 8.2 \times 9.9$

$= 321.44 + 81.18$

$R_2 = \frac{402.62}{8.08} = 49.83$ KN.

$R_1 = 16.4 \times 5 + 8.2 - R_2$

$R_1 = 82 + 8.2 - 49.83$

$R_1 = 90.2 - 49.83$

$R_1 = 40.37$ KN.

+ve $M_u = 23.97 \times 1.9 + 7.57 \times 2 = 60.68$

-ve $M_u = 8.2 \times 1.81 = 14.84$

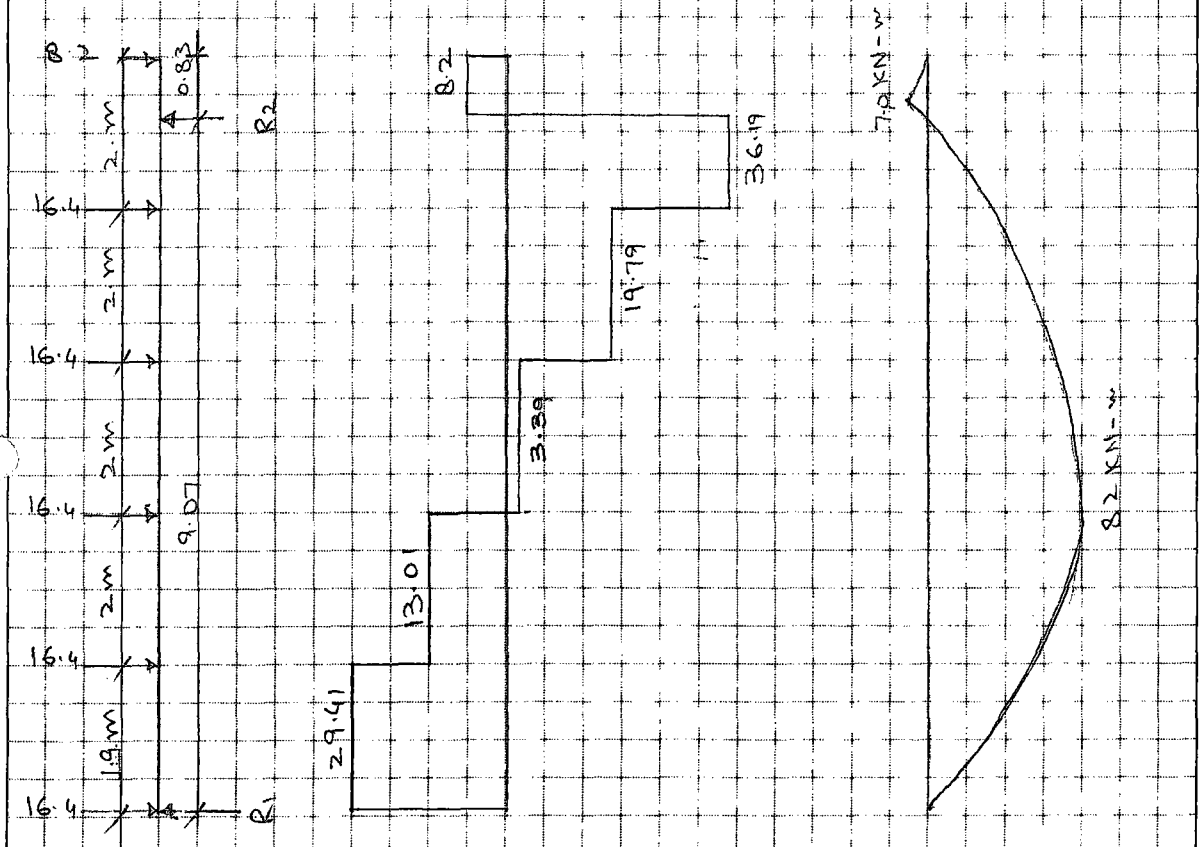
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Vertical Member @ GL (SE) & (SD)



$M_{@①} = 0 \quad \curvearrowleft = \curvearrowright$

$R_2 \times 9.07 = 16.4 \times (1.9 + 3.9 + 5.9 + 7.9) + 8.2 \times 8.9$

$9.07\, R_2 = 16.4 \times 19.6 + 8.2 \times 9.9$

$R_2 = \frac{402.62}{9.07} = 44.39 \text{ kN}$

$R_1 = 16.4 \times 5 + 8.2 - R_2$

$R_1 = 90.2 - 44.39 =$

$R_1 = 45.81 \text{ kN}$

$+ve\ M_u = 29.41 \times 1.9 + 13.01 \times 2 = 81.899 \approx 82 \text{ kN-m}$

$-ve\ M_u = 8.2 \times .83 = 6.80 \text{ kN-m}$

241059
241053

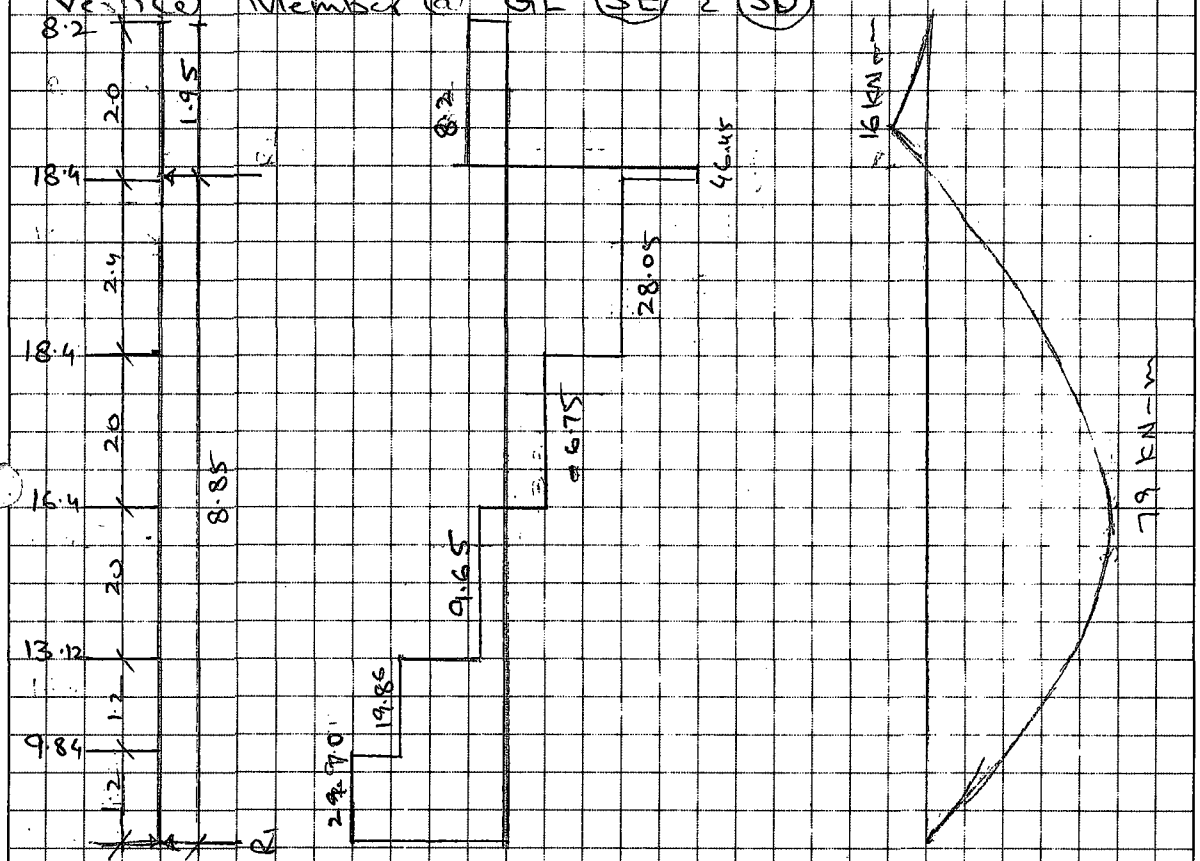
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Vertical Member @ GL (SE) & (SD)



Revised due to change in position of Purlins

$M_M @ ① = 0 \quad (+ve)$

$R_2 \times 8.85 = 9.84 \times 1.2 + 13.12 \times 2.4 + 16.4 \times 4.4 + 18.4 \times (6.4 + 8.8) + 8.2 \times 10.8$

$8.85\, R_2 = 483.696$

$R_2 = 54.65$

$R_1 = 29.70$

+ve $M_u = 1.2 \times 29.7 + 19.86 \times 1.2 + 9.65 \times 2 = 78.77$ kN-m

-ve $M_u = 8.2 \times 1.95 = 16.00$ kN-m

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Vertical Members

Estimation of Vertical load on column @ GL (8E)/(8D)

Dead loads from Roof

i) Roof Sheeting with insulation = 0.25 kPa
ii) Services = 0.20 kPa
iii) Purlins = 6.8/2 = 3.4 kg/m² = 0.03 kPa

0.483 kPa ≈ 0.5 kPa

STR1	200×68 HS - 5.5m	× 29.9	=	165
BR2	125×6 EA - 14 m	× 14.9	=	210
	310UB40 - 8.2m	× 40	=	328
				708 kg

= 6.89 kN ≈ 7 kN

46.28

∴ Load on Vertical Member

i) From Roof = $0.5 \times 8.2 \times \frac{10.2}{2}$ = 20.91 kN

ii) From Structural Beam = 7.0 kN

iii) Side Wall Cladding = 0.25 × 8.2 × 9.9 = 20.29

≈ P_D = 48.205 kN

P_D = 48.205

P_L = 0.25 × 8.2 × 9.9 = 20.29 kN

$P_U = 1.4 P_D + 1.7 P_L$ =

P_U = 101.98 kN

Since the Purlin spacing is 2.0m c/c the effective length of vertical col. can be considered as 2m.

250 UB 31.4 M_{ue} = 78.8 kN-m @ L_e = 2m

250 UB 37.4 M_{ue} = 98.1 kN-m @ L_e = 2m > 8 + use 250UB37.4

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Warehouse Building
PAGE NO: OF
BY: Jamil Khan
DATE: 27/4/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	82 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	2.000 m
Length between Y axis restraints	L_y =	9.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Effective section modulus about X axis (table)	Z_{ex} =	395000 mm^3
	$\varnothing M_{sx}$ =	**107 kNm**

M_x^*		$\varnothing M_{sx}$	
82 kNm	≤	**107 kNm**	**∴ O.K.**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	91400 mm^3
	$\varnothing M_{sy}$ =	**25 kNm**

M_y^*		$\varnothing M_{sy}$	
0 kNm	≤	**25 kNm**	**∴ O.K.**

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment — JOB NO: 2711265

SUBJECT: Warehouse Building — PAGE NO: OF

BY: Jamil Khan — DATE: 27/4/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor — αm = 1.00

Ratio of smaller to larger bending moment — βm = 2

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	E =	200 GPa
Moment of inertia about Y axis (table)	I_y =	4470000 mm^4
Shear modulus of elasticity	G =	80 GPa
Torsion constant (table)	J =	89300 mm^4
Warping constant (table)	I_w =	6.59E+10 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

L_{ex} = 2.000 m

M_{oa} = 296 kNm

αs = 0.8261245

$\varnothing M_{bx}$ = 88 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
88 kNm	**≤**	**107 kNm**	**∴**	**Ø Mbx = 88 kNm is critical**

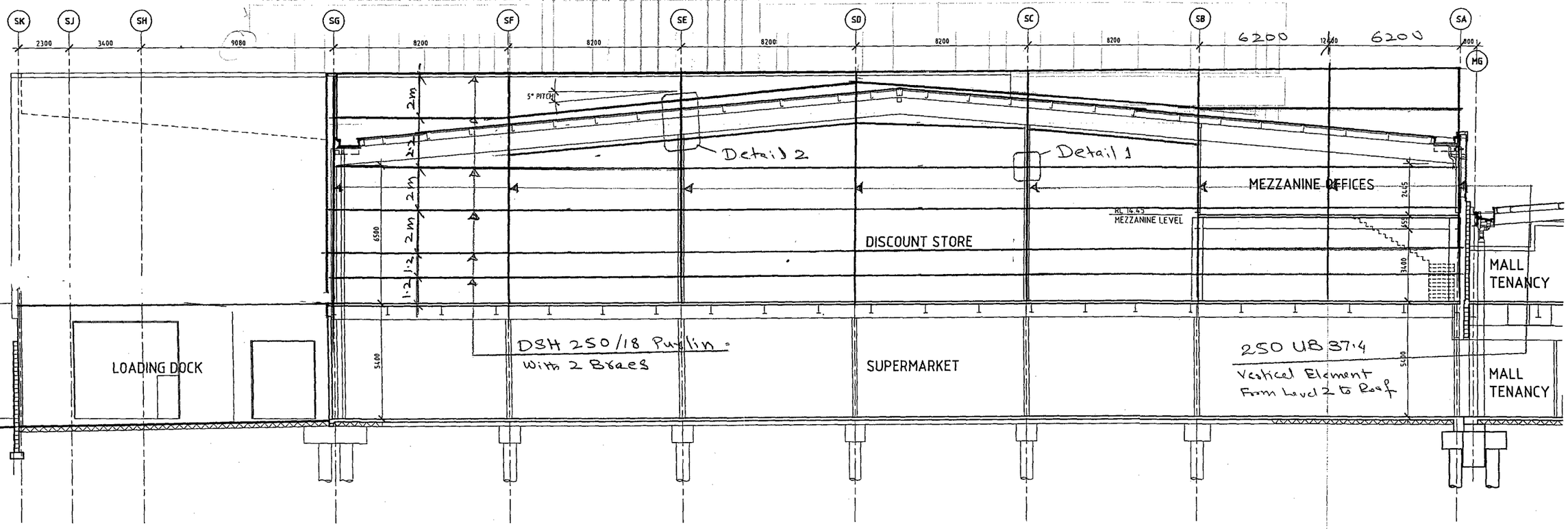
M_x^*		Ø Mbx =	
82 kNm	**≤**	**88 kNm**	**∴ O.K.**

Section size	250 UB 31.4	O.K.

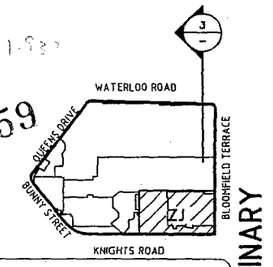
Use

250 UB 31.4

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Frame along Grid (S11) and (S1)



241059

PRELIMINARY

NOTE

VARIATION NOTE: ATTENTION: THE CONTRACTOR

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ALL DIMENSIONS TO BE CHECKED ON SITE. WRITTEN DIMENSIONS ONLY TO BE USED. REFER TO ALL DETAIL DWGS, STRUCTURAL MECHANICAL & SERVICE DWGS BEFORE COMMENCING WORK. REFER ANY DISCREPANCIES TO ARCHITECTS.

a	Rev.	Date	By	Description	Rev.	Date	By	Description

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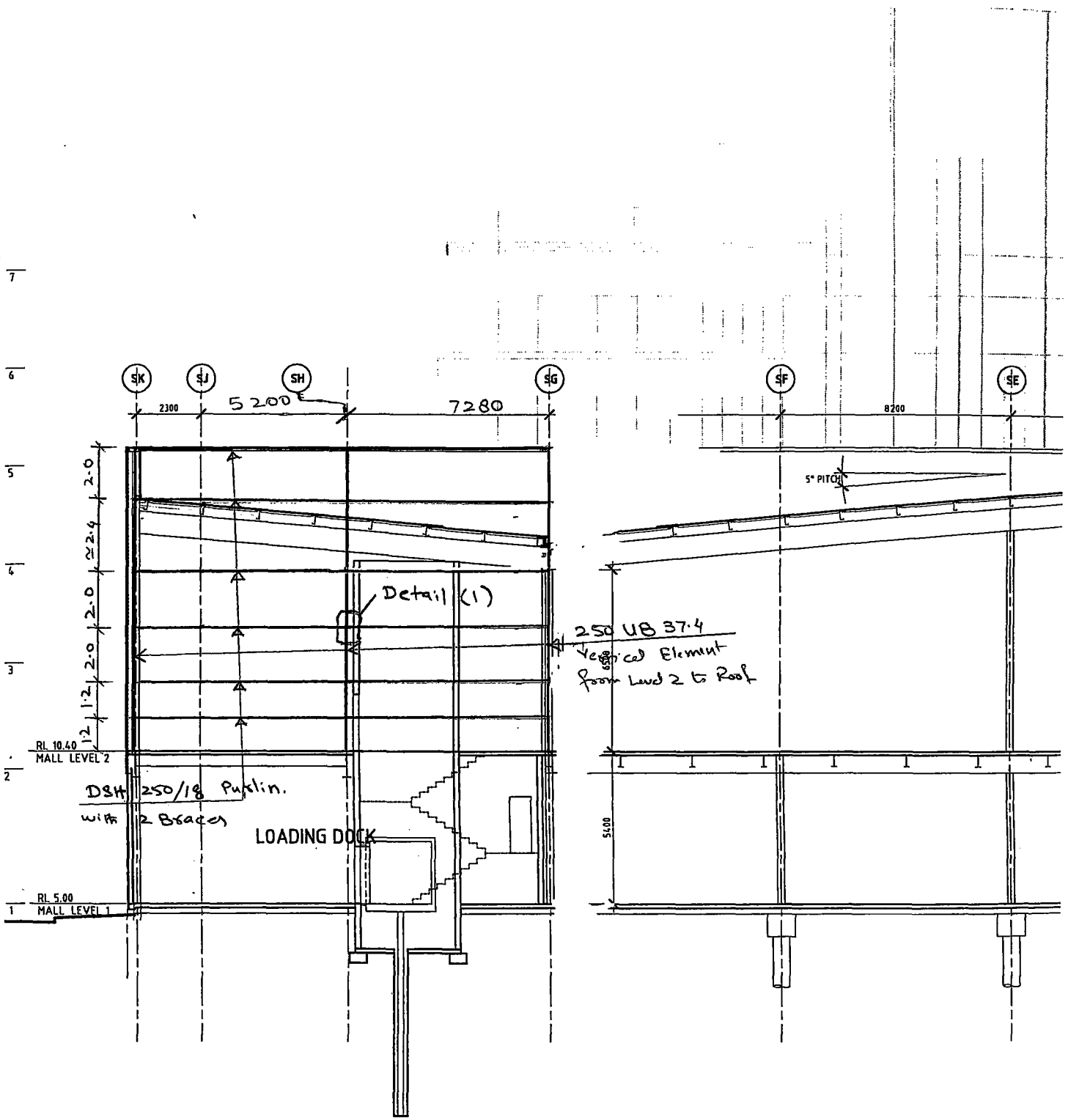
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Glass Carter Holdings & Farmer Ltd
Tel 04-473 7581 Fax 04-473 7911
Services Engineer
Montgomery Watson NZ Ltd
Tel 04-381 6700 Fax 04-381 6720

Checked

Drawn by PR

Project: QUEENSGATE REDEVELOPMENT	Scale: A1 1:100, A3 1:200	Project/Job Number: 04007	Computer Ref.: ZJ-SEC03	Drawing Number: A303-ZJ
Title: ZONE J PART SECTION 3	Library Ref.: J:/QG/CD	Scheme: 35R	Date Plotted: 31/03/04	Revision: 1

300\A303-ZJ.dwg - 31/03/04 - 4



Frame along Grid (S8). and Grid (S4)

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AD FILE No. J:\QQ\CD\A300\A305-ZJ.dwg - 18/03/04 - 5:15

Rev.	Date	By	Description	Rev.	Date	By	Description	Rev.	Date	By	Description

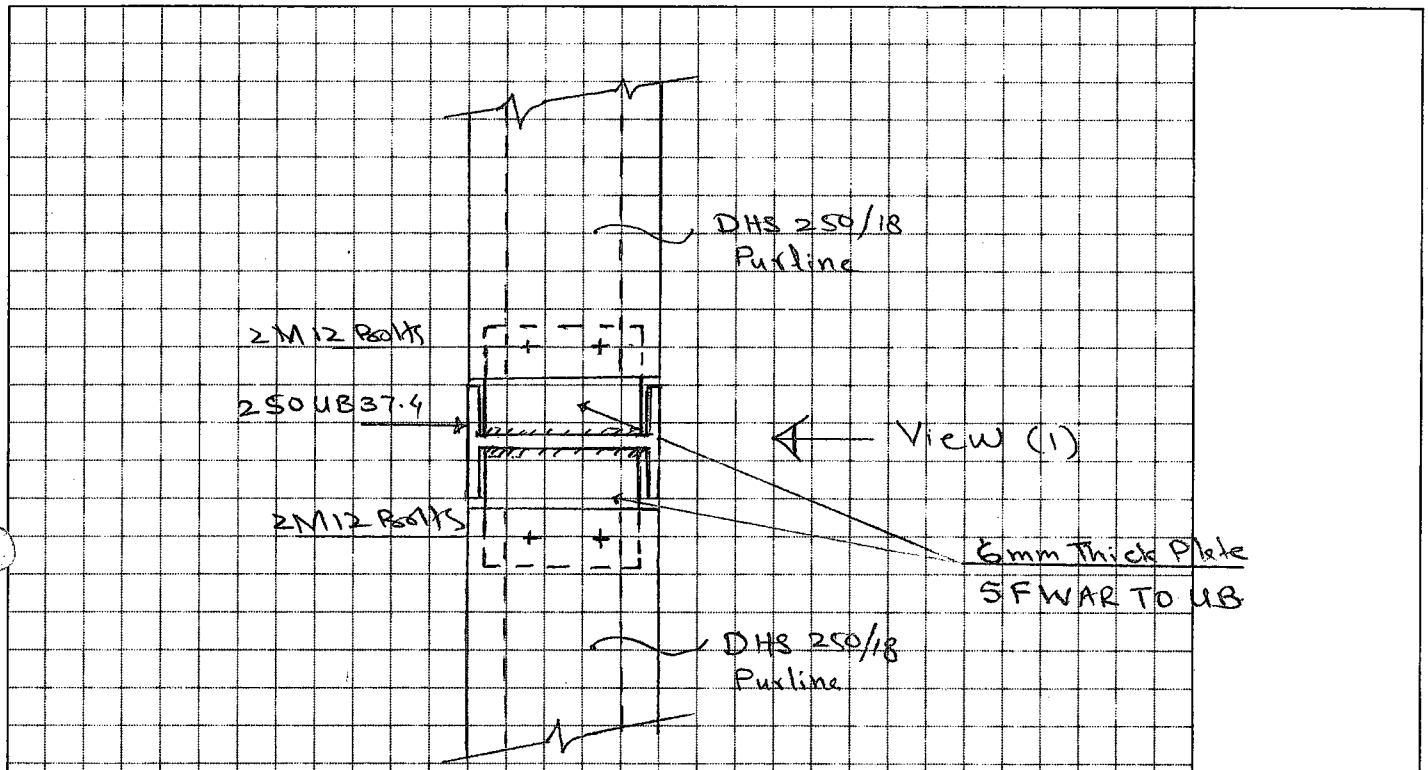
Westfield

Calculation Sheet

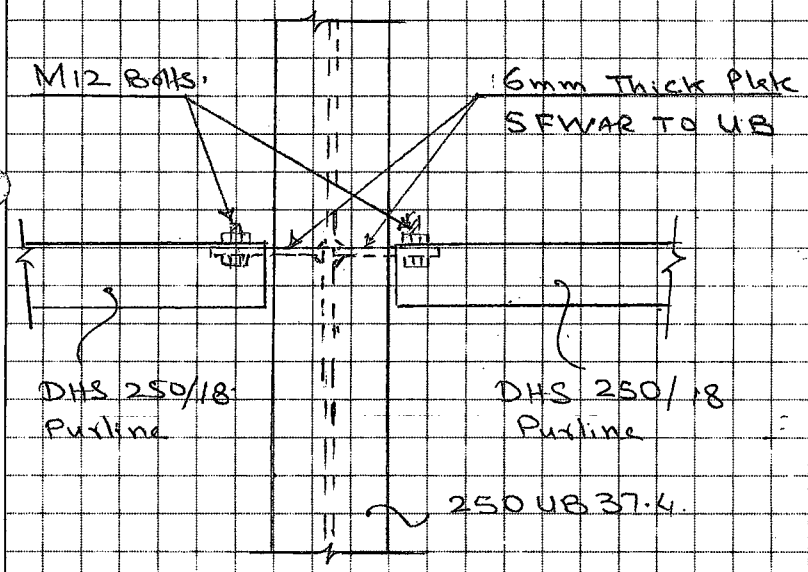
Job name: Job No:

Subject: Page No: of

By: Date:



Plan View of connection
between vertical column & Horizontal Purlins.



Detail (1)

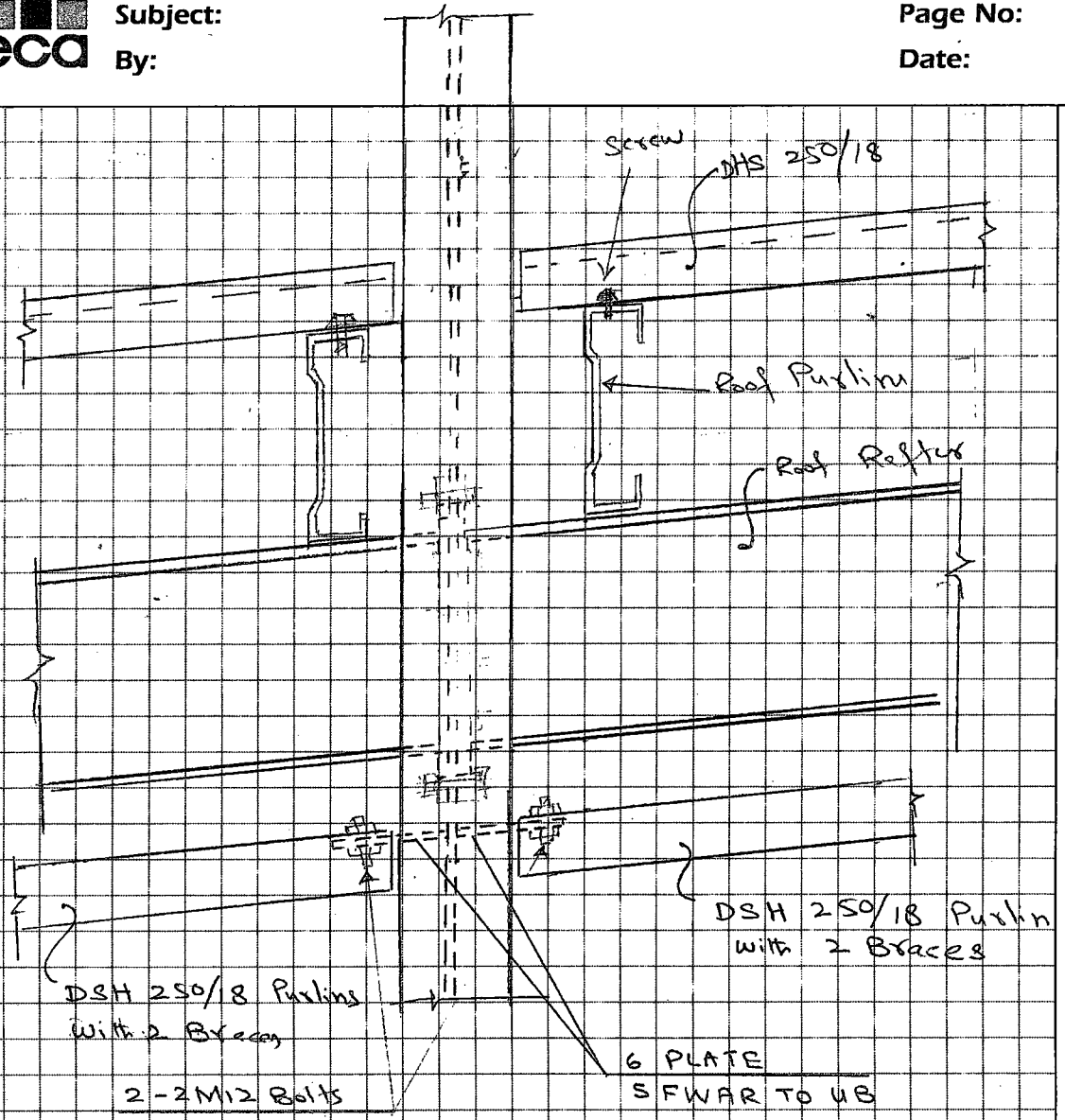
View (1)

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Detail 2

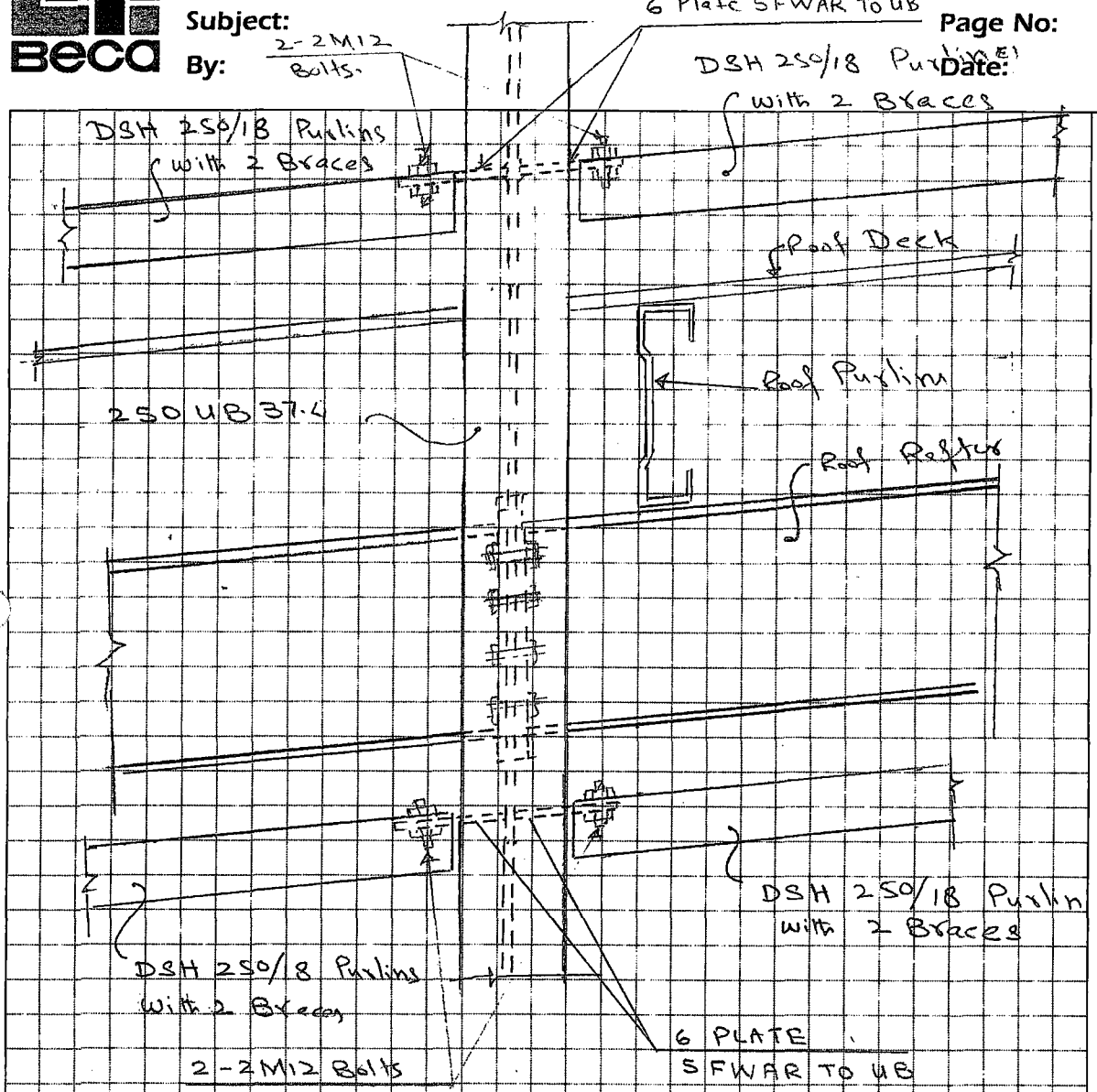
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Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



Detail 2

Calculation Sheet

Job name: Queensgate Redevelopment — Job No:

Subject: Warehouse, External Wall @ GL (5G) From (3B) to (3C) — Page No: of

By: Jamil Khan — Date:

Horizontal Purlins

Design wind load Pz — ULS = 1.0 kPa, SLS = 0.6 kPa

For a Span of 10.5 m

$W_s = 2.0 \times .6 = 1.2$ kN/m

$W_u = 2.0 \times 1 = 2.0$ kN/m

SLS Deflection

For simply supported conditions

$$\delta = \frac{5 W_s L \cdot L^3}{384 E I}$$

$$\delta = \frac{5 \times (1.2 \times 1000) \times 10.5 \times 10500^3}{384 \times 205 \times 10^3 \, I}$$

$$\delta = \frac{9.264 \times 10^8}{I}$$

				Weight
For DSH 250/18	δ = 113.12 mm	$\frac{L}{\delta}$ = 92.82		6.81 kg/m
DSH 300/15	δ = 79.55 mm	$\frac{L}{\delta}$ = 132		6.70 kg/m
DSH 300/18	δ = 66.20 mm	$\frac{L}{\delta}$ = 158.61		8.06 kg/m
DSH 350/18	δ = 45.40 mm	$\frac{L}{\delta}$ = 231.27		8.89 kg/m
DSH 400/20	δ = 29.29 mm	$\frac{L}{\delta}$ = 358		10.81 kg/m

Reaction From Purlins = $2 \times \frac{10.5}{2} = 10.5$

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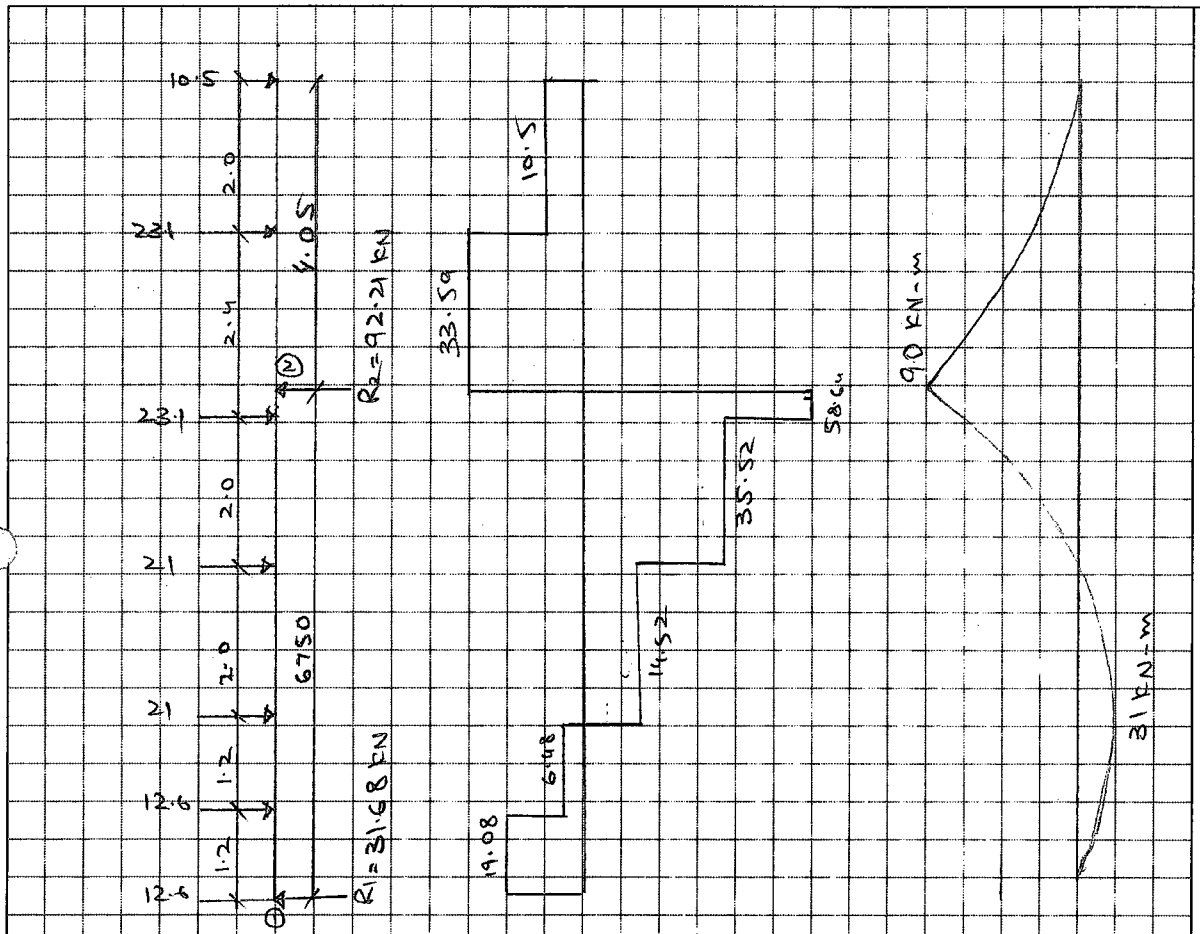
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Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:



$\Sigma M @ ① = 0 \quad (+\circlearrowleft)$

$R_2(6.75) = 12.6 \times 1.2 + 21(2.4 + 4.4) + 23.1 \times (6.4 + 8.8) + 10.5 \times 10.8$

$= 15.12 + 142.8 + 351.12 + 113.4$

$R_2 = 92.21 \text{ kN}$

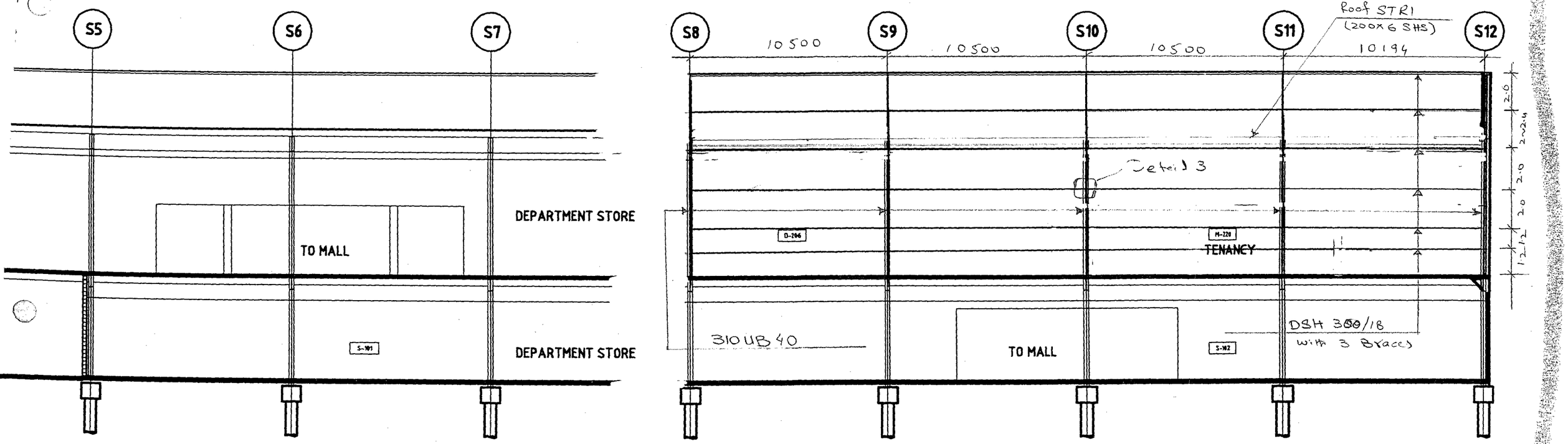
$R_1 = 12.6 + 12.6 + 21 + 21 + 23.1 + 23.1 + 10.5 - 92.21$

$R_1 = 31.68 \text{ kN}$

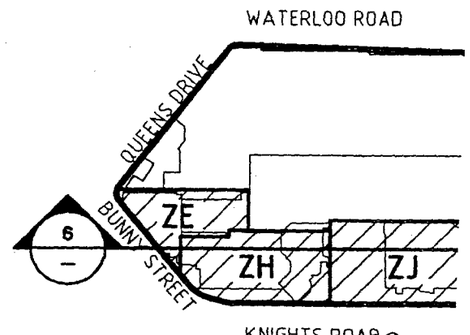
$+ve \; M_u = 19.08 \times 1.2 + 6.48 \times 1.2 = 30.672 \text{ kN-m}$

$-ve \; M_u = 33.59 \times 2.05 + 10.5 \times 2 = 89.85 \text{ kN-m}$

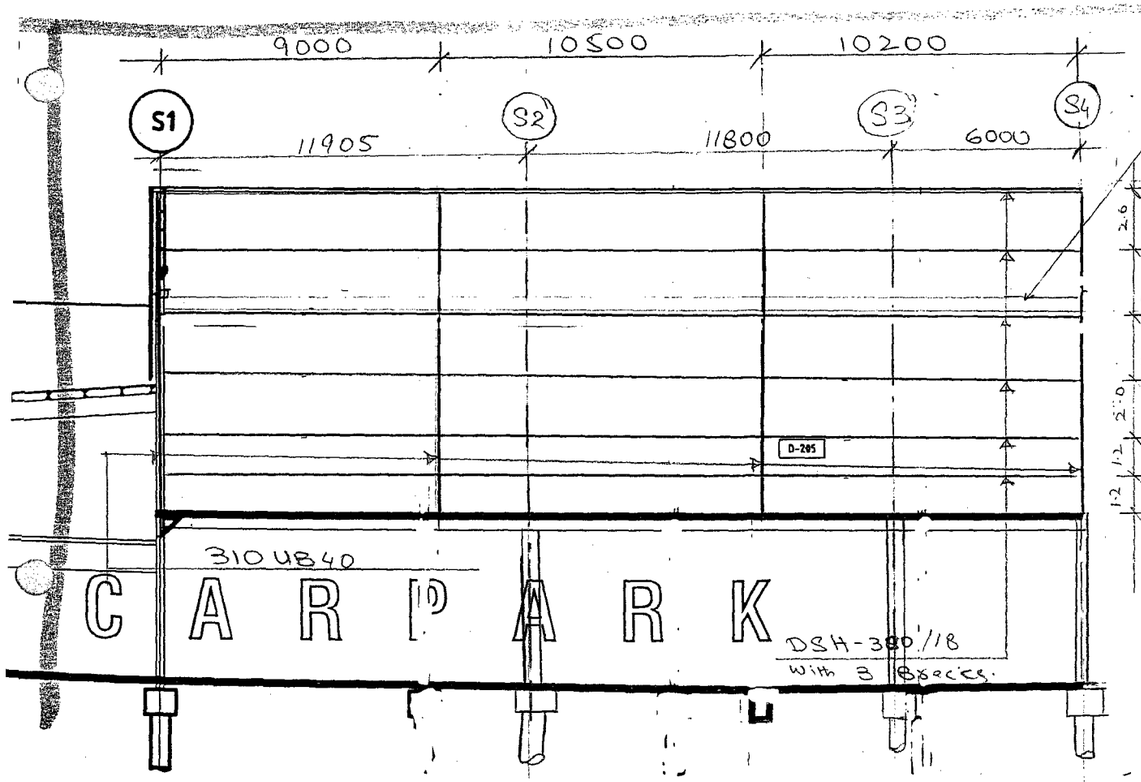
For Section 250 UB 37.4 $M_{us} = 98.1 \text{ kN-m}$ @ $L_c = 2 \text{ m}$

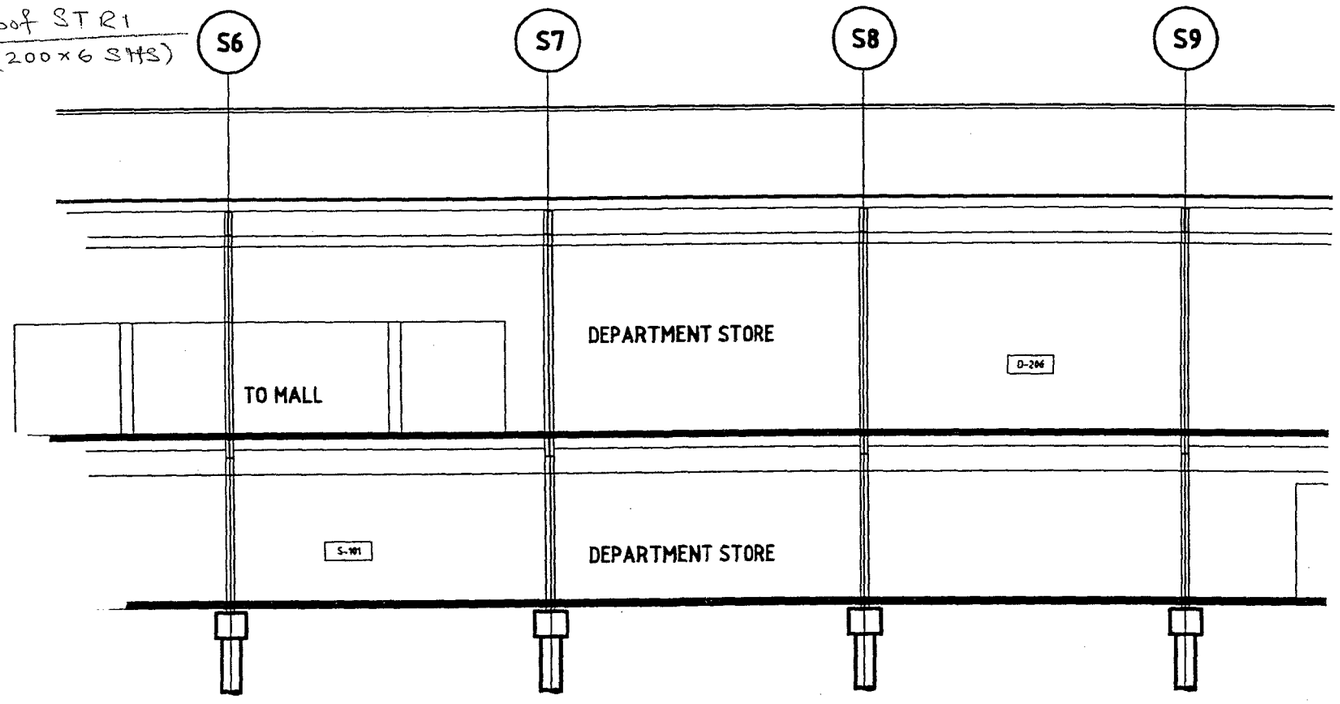


Along Grid line (SG)
From Grid (S8) to (S12)



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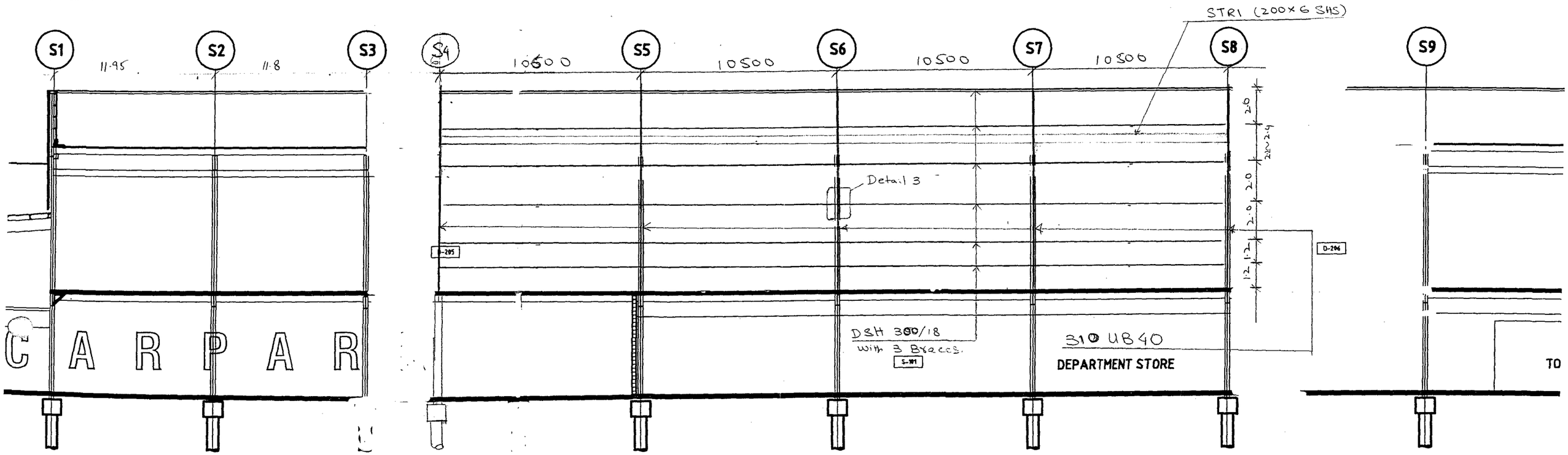




Along Grid line (SG)

From Grid (S1) to (S4)

18-19



Along Grid line (S4)

From Grid (S4) to (S8)

By	Description

Westfield

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Project QUEENSGATE REDEVELOPMENT

Title ZONE E, H, J
SECTION 6

Design
Of
Staircases

241059

STAIRCASE SLAB DESIGN *(Version 1.00)*

Developed by Jamil ur Rahman Khan

Project Title: Queensgate Shopping Centre
Designation: Stair Case for Zone J Building

Project Ref. No. 21-286
Design By: Jamil
Date of Design: 05/28/04

MATERIAL	Concrete grade	f'_c :	40 N/mm2	
	Reinf't grade	f_y :	500 N/mm2	
MARKING	Staircase No.	:	Span 1	Span 2
	Level	:	Typical Flight	Typical Flight
INPUT DATA	Span	Ln, m :	5.200	5.000
	ss oc bc	:	ss	ss
	Waist Slab Thickness	h, mm :	200	200
	Riser	R, mm :	180	150
	Thread	T, mm :	275	250
	Length of Waist Slab	m :	3.20	3.00
	Clear Concrete Cover	mm :	40	40
	Tension Bar diameter	mm :	16	13
	Compression Bar diameter	mm :	0	0
	Effective depth	d, mm :	152	153.5
LOADING	Waist	kN/m^2 :	5.74	5.60
	Steps	kN/m^2 :	2.16	1.80
	Finishes/Services	kN/m^2 :	0.50	0.50
	Screed	kN/m^2 :	0.00	0.00
	Live Load	kN/m^2 :	5.00	5.00
	W=1.2G+1.6Q	kN/m^2 :	18.08	17.48
DESIGN	Moment coeff't	:	0.125	0.125
	Moment	kN-m :	61.10	54.62
	$R_U = M_U / (\phi\, bd^2)$	:	3.11	2.73
	$m = f_y / (0.85\, f'_c)$	:	14.71	14.71
	$\rho = 1/m \{1 - (1 - (2\, m\, R_U) / f_y)^{1/2}\}$		0.0065	0.0057
	ρ_{min} = Max (0.7/ f_y, 0.0014)		0.0014	0.0014
	$As_{Required}$ = Max (ρ, ρ_{min}) bd	mm^2:	994	874
	ρ_w = As / (b d)	:	0.0076	0.0058
	Max spacing	mm :	202	152
REINF'T	Tension Reinforcement	:	T 16-	T 13-
	Provided Spacing	mm :	175	150
	Compression Reinforcement	:	0	0
	Provided Spacing	mm :	300	300
DEFLECTION	Span/Depth ratio	:	25	25
	Flight / Span Mod. Factor		1.15	1.15
	Tension Mod. Factor	:	0.98	0.94
	Compression Mod. Factor		1.00	1.00
	Modified L/h	:	28.16	26.94
	Actual L/h	:	26.00	25.00
	Deflection	:	O.K.	O.K.
	Notation	:	ss = both ends simply supported	
		:	oc = one end continuous	
		:	bc = both ends continuous	

Disclaimer:-

Considerable time & effort have gone into the development of this spreadsheet. The spreadsheet has been thoroughly tested and used. In using the spreadsheet, however, the user accepts and understands, that no warranty is expressed or implied by the developer on the accuracy or the reliability of the spreadsheet. The user must explicitly understand the assumption of the spreadsheet and must independently verify the results.

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241053

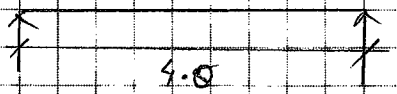
Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Landing Beam



Load on Beam

1) Dead Load

i) From Landing $= \left(5.74 + \frac{2.18 \times 3.2}{5.2} + 0.5\right) \times \frac{5.2}{2} = 19.71$

ii) From 125 Block wall $= 3.2 \times 3 = 9.6$ kN/m

$W_D = 29.31$ kN/m

2) Live Load

$W_L = 5 \times 2.6 = 13$ kN/m

$W_u = 1.2G + 1.6Q = 56.00$ kN/m

$M_u = \frac{W_u l^2}{8} = 111.95$

Use 310 UB 46.2

$R_u = 56 \times 2 = 112$ kN

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Zone J, Stair case @ GL SG/S4, Typical Lending Beam
PAGE NO: 1 OF 3
BY: Jamil Khan
DATE: 28/5/04

DESIGN DATA

Enter trial section size here 310 UB 46.2
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	112 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	4.000 m
Length between Y axis restraints	L_y =	4.000 m
Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Young's modulus	E =	200 GPa
Shear modulus of elasticity	G =	80 GPa
Effective section modulus about X axis (table)	Z_{ex} =	729000 mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	163000 mm^3
Moment of inertia about X axis (table)	I_x =	100000000 mm^4
Moment of inertia about Y axis (table)	I_y =	9010000 mm^4
Torsion constant (table)	J =	233000 mm^4
Warping constant (table)	I_w =	1.97E+11 mm^6
Moment modification factor	αm =	1.00
Ratio of smaller to larger bending moment	βm =	2
Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N* =	0 kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300 MPa
Gross area of cross section (use nett if approp.) (table)	A =	5930 mm^2
Radius of gyration about X axis (table)	r_x =	130 mm
Radius of gyration about Y axis (table)	r_y =	39 mm
Effective length factor about X axis	k_{ex} =	1
Effective length factor about Y axis	k_{ey} =	1
Form factor (by calculation)	k_f =	0.954
Section constant, α_b from table 6.3.3(1)	α_b =	2

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0 kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85
Nett area of cross section (by calculation)	A_n =	12500 mm^2
Tensile strength	f_u =	410 MPa

241059

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Zone J, Stair case @ GL SG/S4, Typical Lending Beam
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 28/5/04

5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	$M_x^* =$	112 kNm
Factored bending moment about minor Y - axis	$M_y^* =$	0 kNm
Length between X axis restraints	$L_x =$	4.000 m
Length between Y axis restraints	$L_y =$	4.000 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \phi M_{sx}$ and
$M_x^* \le \phi M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \phi M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \phi M_{sx}$
$\le \phi f_y Z_{ex}$

Strength reduction factor	$\phi =$	0.9
Yield stress of flanges	$f_y =$	300 MPa
Effective section modulus about X axis (table)	$Z_{ex} =$	729000 mm^3
	$\phi M_{sx} =$	**197 kNm**

M_x^*		ϕM_{sx}	
112 kNm	**≤**	**197 kNm**	**∴ O.K.**

$M_y^* \le \phi M_{sy}$
$\le \phi f_y Z_{ey}$

Effective section modulus about Y axis (table)	$Z_{ey} =$	163000 mm^3
	$\phi M_{sy} =$	**44 kNm**

M_y^*		ϕM_{sy}	
0 kNm	**≤**	**44 kNm**	**∴ O.K.**

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Zone J, Stair case @ GL SG/S4, Typical Lending Beam
PAGE NO: 2 OF 3
BY: Jamil Khan
DATE: 28/5/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$
$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00
Ratio of smaller to larger bending moment $\beta m =$ 2

Slenderness reduction factor, αs
$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}
$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
Moment of inertia about Y axis (table)	$I_y =$	9010000 mm^4
Shear modulus of elasticity	$G =$	80 GPa
Torsion constant (table)	$J =$	233000 mm^4
Warping constant (table)	$I_w =$	1.97E+11 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 4.000 m
$M_{oa} =$ 218 kNm
$\alpha s =$ 0.5994641

$\varnothing M_{bx} =$ 118 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$			
118 kNm	≤	197 kNm	∴	Ø Mbx =	118 kNm is critical

M_x^*		Ø Mbx =	
112 kNm	≤	118 kNm	∴ O.K.

Section size	310 UB 46.2	O.K.

Use
310 UB 46.2

241059

Beca Carter Hollings & Ferner

CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
JOB NO: 2711265
SUBJECT: Zone J, Lift @ GL SG/S5, Typical Lift Wall Beam
PAGE NO: 1 OF 3
BY: Jamil Khan
DATE: 28/5/04

DESIGN DATA

Enter trial section size here 250 UB 31.4
("Copy value" from "Section sizes" worksheet)

5 MEMBERS SUBJECT TO BENDING AND SHEAR
8 MEMBERS SUBJECT TO COMBINED ACTIONS

Factored bending moment about major X - axis	M_x^* =	55 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	3.750 m
Length between Y axis restraints	L_y =	3.750 m
Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Young's modulus	E =	200 GPa
Shear modulus of elasticity	G =	80 GPa
Effective section modulus about X axis (table)	Z_{ex} =	395000 mm^3
Effective section modulus about Y axis (table)	Z_{ey} =	91400 mm^3
Moment of inertia about X axis (table)	I_x =	44500000 mm^4
Moment of inertia about Y axis (table)	I_y =	4470000 mm^4
Torsion constant (table)	J =	89300 mm^4
Warping constant (table)	I_w =	6.59E+10 mm^6
Moment modification factor	αm =	1.00
Ratio of smaller to larger bending moment	βm =	2
Twist restraint factor	K_t =	1.00
Load height factor	K_l =	1.00
Rotation restraint factor	K_r =	1.00

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

Factored axial design load	N^* =	0 kN
Minimum value of yield stress for any element (or weighted average)	f_y =	300 MPa
Gross area of cross section (use nett if approp.) (table)	A =	4010 mm^2
Radius of gyration about X axis (table)	r_x =	105 mm
Radius of gyration about Y axis (table)	r_y =	33.4 mm
Effective length factor about X axis	k_{ex} =	1
Effective length factor about Y axis	k_{ey} =	1
Form factor (by calculation)	k_f =	0.954
Section constant, α_b from table 6.3.3(1)	α_b =	2

7 MEMBERS SUBJECT TO AXIAL TENSION

Factored axial design load	N_t^* =	0 kN
Correction factor for distribution of forces (by calculation)	k_{te} =	0.85
Nett area of cross section (by calculation)	A_n =	12500 mm^2
Tensile strength	f_u =	410 MPa

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CALCULATION SHEET

JOB NAME: Queesgate Mall Redevelopment
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5 MEMBERS SUBJECT TO BENDING AND SHEAR

Factored bending moment about major X - axis	M_x^* =	55 kNm
Factored bending moment about minor Y - axis	M_y^* =	0 kNm
Length between X axis restraints	L_x =	3.750 m
Length between Y axis restraints	L_y =	3.750 m

5.1 DESIGN FOR BENDING MOMENT

5.1.1 Bending About Section Major Principal X-axis

$M_x^* \le \varnothing M_{sx}$ and
$M_x^* \le \varnothing M_{bx}$

5.1.2 Bending About Section Minor Principal Y-axis

$M_y^* \le \varnothing M_{sy}$

5.2 NOMINAL SECTION MOM. CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

$M_x^* \le \varnothing M_{sx}$
$\le \varnothing f_y Z_{ex}$

Strength reduction factor	$\varnothing$ =	0.9
Yield stress of flanges	f_y =	300 MPa
Effective section modulus about X axis (table)	Z_{ex} =	395000 mm^3
	$\varnothing M_{sx}$ =	**107 kNm**

M_x^*		$\varnothing M_{sx}$	
55 kNm	≤	**107 kNm**	**∴ O.K.**

$M_y^* \le \varnothing M_{sy}$
$\le \varnothing f_y Z_{ey}$

Effective section modulus about Y axis (table)	Z_{ey} =	91400 mm^3
	$\varnothing M_{sy}$ =	**25 kNm**

M_y^*		$\varnothing M_{sy}$	
0 kNm	≤	**25 kNm**	**∴ O.K.**

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JOB NAME: Queesgate Mall Redevelopment | JOB NO: 2711265

SUBJECT: Zone J, Lift @ GL SG/S5, Typical Lift Wall Beam | PAGE NO: 2 OF 3

BY: Jamil Khan | DATE: 28/5/04

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 Segments of constant cross section

$M_x^* \le \varnothing M_{bx}$

$\le \varnothing \alpha_m \alpha_s M_{sx} \le \varnothing M_{sx}$

Moment modification factor $\alpha m =$ 1.00

Ratio of smaller to larger bending moment $\beta m =$ 2

Slenderness reduction factor, αs

$\alpha s = 0.6\{\sqrt{[(M_{sx}/M_{oa})^2+3]}-(M_{sx}/M_{oa})\}$

Reference buckling moment, M_{oa}

$M_{oa} = M_o = \sqrt{\{(\Pi^2 E I_y/L_e^2)[GJ+(\Pi^2 E I_w/L_e^2)]\}}$

Young's modulus	$E =$	200 GPa
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Torsion constant (table)	$J =$	89300 mm^4
Warping constant (table)	$I_w =$	6.59E+10 mm^6

Calculate effective length about X axis, $L_{ex} = K_t K_l K_r L_x$

Twist restraint factor	$K_t =$	1.00
Load height factor	$K_l =$	1.00
Rotation restraint factor	$K_r =$	1.00

$L_{ex} =$ 3.750 m

$M_{oa} =$ 101 kNm

$\alpha s =$ 0.552297

$\varnothing M_{bx} =$ 59 kNm

$\varnothing M_{bx}$		$\varnothing M_{sx}$		
59 kNm	**≤**	**107 kNm**	**∴**	**$\varnothing$ Mbx = 59 kNm is critical**

M_x^*		$\varnothing$ Mbx =	
55 kNm	**≤**	**59 kNm**	**∴ O.K.**

Section size	**250 UB 31.4**	**O.K.**

Use 250 UB 31.4

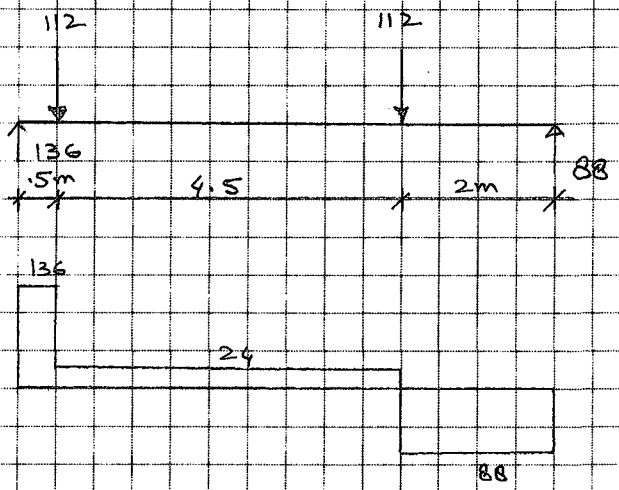
Calculation Sheet

Job name: Queensgate Zone J. Job No: 2711265/027

Subject: Page No: of

By: JK Date:

Ground Beam for Staircase 5. GB3.



$M_u = 88 \times 2 = 176$ KN-m

Additional Moment due to Pile Ecc of 50mm

Pile ultimate Capacity of PC1D = $0.65 \times 1350 = 877.5 \simeq 878$ KN

$M_{u_{add}} = 0.050 \times 878 = 43.9$

$M_{u_{Total}} = 43.9 + 176 = 219.9 \simeq 220$ KN-m

$b = 400$ $h = 500$ $d = 500 - 75 - 10 - 20/2 = 405$

$R_u = \frac{M_u}{\phi b d^2} = \frac{220 \times 10^6}{0.85 \times 400 \times 405^2} = 3.94$ 3.49

$m = \frac{f_y}{0.85 f_c'} = 23.53$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2Rum}{f_y}}\right) = 0.0087885$ 0.00767

$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y} = 0.0025$

13 19

$A_{s_{req}} = \rho b d = 1423.77$ mm²

use 4 DH 25 Bott

For Top

$M_u = 43.9$ KN-m

$R_u = 0.787$ $\rho = 0.00160 < \rho_{min}$

$A_{s_{req}} = \rho_{min} b d = 405$

use 4 DH 16 at Top

241059

Calculation Sheet

Job name: Job No:

Subject: Page No: of

By: Date:

Ground beams for Lift Shaft

Load on C61

1) From Stair Case

i) From 1st Flight = P_u = 60 KN

ii) From 2nd landing = P_u = 112 KN

2) From Lift Maintence Room

Slab = $(1.2\times.13\times24 + 1.6\times7.5)\times\frac{3.75}{2}\times\frac{3.5}{2}$ = 51.66 KN

3) From Wall below Lift Maintence Room

$1.2\times.2\times20\times\left(\frac{3.75}{2}+\frac{3.5}{2}\right)\times1.5$ = 2.61 KN

4) From wall above Lift Maintence Room

$= 1.2\times1\times\left(\frac{3.75}{2}+\frac{3.5}{2}\right)\times4$ = 17.4 KN

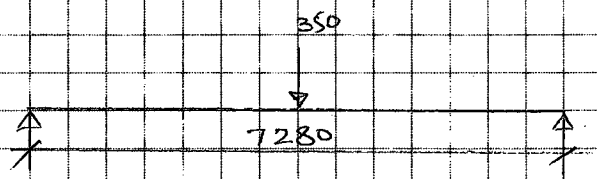
5) From Level 2 Slab

$(1.2\times.13\times24 + 1.6\times6.75)\times\left(\frac{3.75}{2}\times\frac{3.5}{2}\right)$ = 51.66 KN

6) Wall above Level 2

$1.2\times1\times\left(\frac{3.75}{2}+\frac{3.5}{2}\right)\times6.5$ = 28.275 KN

P_u = 347.095 ≃ 350 KN



$M_u = \frac{350\times7.28}{4}$ = 637 KN-m Due to Point load

$M_{u,dl}$ = 0.65×1600×0.05 = 52 KN-m

M_u = 689 KN-m

b = 400 h = 800 d = 800 − 75 − 10 − 32/2 = 699

$R_u = \frac{M_u}{\phi bd^2}$ = 4.1475 m = 23.53

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2Rum}{fy}}\right)$ = 0.009316 ρ_{min} = 0.0025

$A_s = \rho bd$ = 2604.77 mm²

Use 4 D H32 Bott

4 D H20 Top

2 R12-125 Stirrups