Petone Recreation Ground Grandstand

Detailed Seismic Assessment

For Hutt City Council

Project 10131

July 2023

Sawrey Consulting Engineers
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Detailed Seismic Assessment

Figure 1 – Photograph of the building from the east.

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Executive Summary

Background
This report has been prepared by Sawrey Consulting Engineers Ltd (SCEL) for its client, Hutt City Council, in accordance with a written brief. It presents the findings of the Detailed Seismic Assessment (DSA) carried out for the above property by SCEL, reported according to the limitations of the brief and the report.

Building Description
The grandstand is 43m long by 13m wide, and it is 13.5m high. Beneath the bleachers are two levels of facilities, including a 120m2 hall, a kitchen, changing areas, showers, storage, and toilets. The roof is corrugated iron, the bleachers are reinforced concrete, the L1 floor is timber framed with timber partitions, and the ground floor walls are reinforced concrete, reinforced block, and un-reinforced brick masonry infill. It had been built circa 1939 with some structural improvement circa 1979 and some seismic strengthening circa 2014.

Assessed Earthquake Rating
The results of this DSA indicate the building's earthquake rating to be 20%NBS (IL3) as assessed in accordance with the guideline document *The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments*, dated July 2017. This earthquake rating is for an Importance Level 3 (IL3) structure in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002.

This building has been assessed as being seismic Grade D. The seismic grading scheme, developed by the New Zealand Society for Earthquake Engineering, compares the life risk (due to earthquake damage) of this building to other buildings. Grade D buildings represent a high risk to occupants and neighbours, and it is equivalent to 10 to 25 times the risk expected for a new building.

A building with an earthquake rating of less than 34%NBS fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building with a rating less than 67%NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering. This DSA has found the rating to be less than 34%NBS(IL3) and therefore it may be categorized an earthquake prone building by the Territorial Authority.

Results
Table 1 summarizes the scores for in this assessment for elements that are less than 34%NBS(IL3).

Table 1 - Results of this Assessment

Staad.Pro ref	Member Type	Level	Nearest Column ref	Notes	%NBS(IL3)
1370	Reinf. concrete walls (2)	L3 to Rf.	Col 13	Reinf. concrete walls. Bending. CSW.	20%
1776	Reinf. concrete beam (1)	L1	col 10	Corridor lintel reinforcement in tension. CSW.	20%
493	Raking beam	L1	col 3	Grid C transverse beam at lower end in bending. Rust affected. CSW.	25%
1470 1810	Brick Wall Panels	G	16 – 17 14 - 15	URM out of plane bending. CSW.	25%
241 1455 1469	Reinf. concrete columns (4)	G to L1	15 17 14	Structural system including columns in bending.	30%
145	Block walls (2)	G to L1	col 10 to 11	Black wall in shear.	30%
End Walls	Mullions to glazing	L1 to Rf.	1 – 13 6 - 18	Rotten timber. SSW.	<33%
End Walls	Steel posts. (4)	L1 to Rf.	Above col 1 & col 6	Rusted steel posts both sides of walkway. SSW.	<33%

A severe structural weakness (SSW) is a structural weakness for which rupture would lead to a collapse and for which the probable capacity may not be reliably assessed based on current knowledge; and a critical structural weakness (CSW) is the lowest scoring weakness determined in the DSA.

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1.0 Introduction

Sawrey Consulting Engineers Ltd (SCEL) has been engaged by Hutt City Council to carry out a Detailed Seismic Assessment (DSA) of the Petone Recreation Ground (PRG) Grandstand at 16N Udy Street, Petone. The building is accessible from Kirks Avenue. This DSA is carried out to quantify the seismic risk.

Figure 2 - Grandstand Seating with 1979 Roof Bracing Above

Figure 3 - Exterior North-West Corner

Figure 4 - Upstairs Hall

Figure 5 - Downstairs Corridor

1.1 Previous Seismic Assessments

This DSA supersedes the results of previous seismic assessments as summarised below in Table 2 – Earlier Assessments.

- This DSA results in a lower building rating than of earlier DSA assessments.
- This is because of latest industry knowledge, a more detailed model with a greater number of modelled scenarios, and the results of corrosion testing with an intrusive durability investigation.
- Partial strengthening has been accounted for.
- The Guidelines which incorporate research, knowledge and experience obtained from the Kaikoura earthquake and significant New Zealand earthquakes between 2010 and 2016.
- The DSA result is influenced by inferences made from the results of sampling and testing of the structure and the results of modelling the structure.

We understand that at present this building is not on the Hutt City Council's "Earthquake Prone Building Register". We are aware of the following earlier assessments.

Table 2 - Earlier Assessments

Date	Assessment Type	By
2014	ISA	GHD
2014	DSA	Sawrey
2015	Strengthening	Sawrey
2020	ISA	Sawrey

1.2 Detailed Seismic Assessment (DSA) Regulatory Standards

The performance of a building relative to the current earthquake code requirements is defined as a rating of 'percentage new building standard' (%NBS). Every building is categorized into one of four importance levels. Normal buildings are importance level 2 (IL2). This structural benchmark for "normal buildings" is lower than the equivalent structural benchmark for importance level 3 buildings (IL3); this category includes buildings that may contain people in crowds, such as grandstands.

A building with an earthquake rating of less than 34%NBS fulfils one of the requirements for the Territorial Authority to consider it to be an earthquake prone building (EPB) in terms of the Building Act 2004. A building rating of less than 67%NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering. Ultimately, the Hutt City Council has authority to decide on the building's status, which would then trigger a timeline for the building to be structurally improved.

The seismic assessment guidelines: The Guidelines - The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments, dated July 2017, are an integral part of the EPB methodology produced by the Ministry of Business, Innovation and Employment (MBIE) under section 133AA to 133AY (subpart 6A) of the Building Act. The legal definition of an earthquake prone building is given in Appendix A.

A rating of 33%NBS represents approximately 20 times the risk of a new building, whereas a rating of 66%NBS represents approximately 3 times the life risk of a new building. The aim of the assessment is to evaluate the earthquake strength of the existing building and compare it to the strength required in an equivalent new building.

1.3 Background and Building Category

This DSA is based on the building being Importance Level 3 (IL3) in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002. This is a category that applies to buildings that may contain people in crowds.

1.4 Site Earthquake Hazard Classification

Please refer to Table 3- Site Earthquake Hazard Classification (Greater Wellington GIS Viewer). Refer also to the geotechnical report by TetraTech Coffey Appendix D for further information.

Table 3- Site Earthquake Hazard Classification (Greater Wellington GIS Viewer)

Earthquake Hazard:	Earthquake Hazard	Hazard Classification
PRG Grandstand (address unavailable in the GIS Viewer)	Combined Hazard	High
	Ground Shaking	High
	Liquefaction Potential	High
	Nearest Fault	Wellington Fault (450m North West)
	Slope Failure	Low

2.0 Description

The grandstand is 43m long, 13m wide, and 13.5m high. The structure is made of heavy materials such as reinforced concrete and masonry, with some lightweight elements such as the roof and the level 1 clubrooms. Because it is a grandstand, it is an irregular structure: i.e. it lacks symmetry both vertically and horizontally. The roof is corrugated iron, the bleachers are reinforced concrete, and the level 1 floor is predominately timber flooring with some reinforced concrete slab. The walls are a combination of reinforced concrete, reinforced block, or un-reinforced masonry infill.

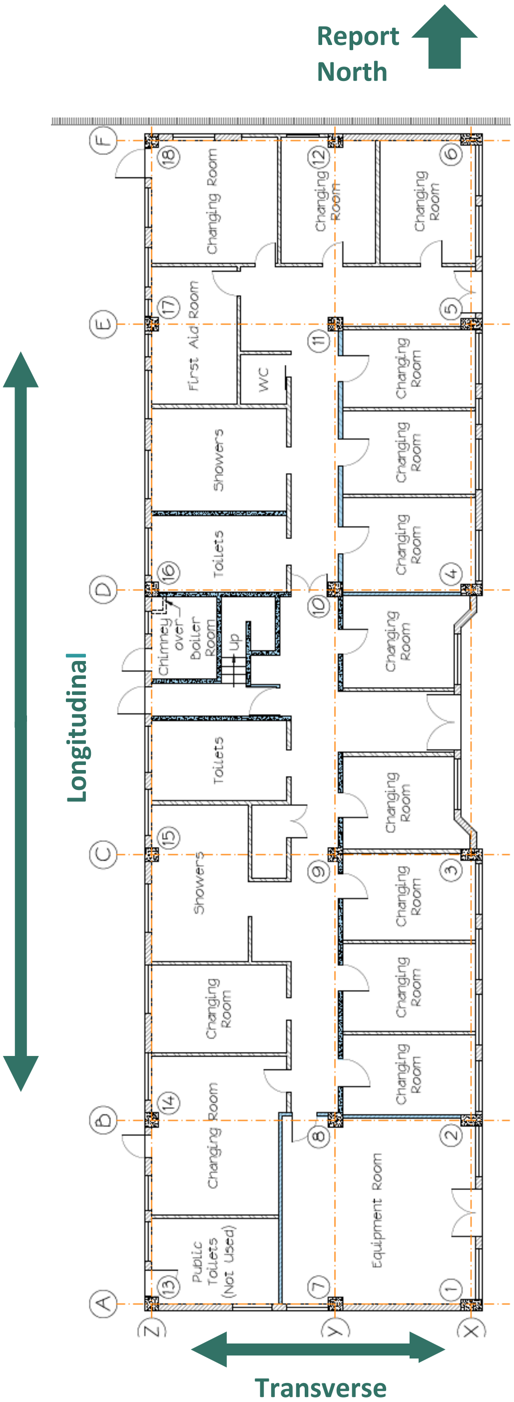

Figure 6 - Ground Floor Plan (2014, SCEL)

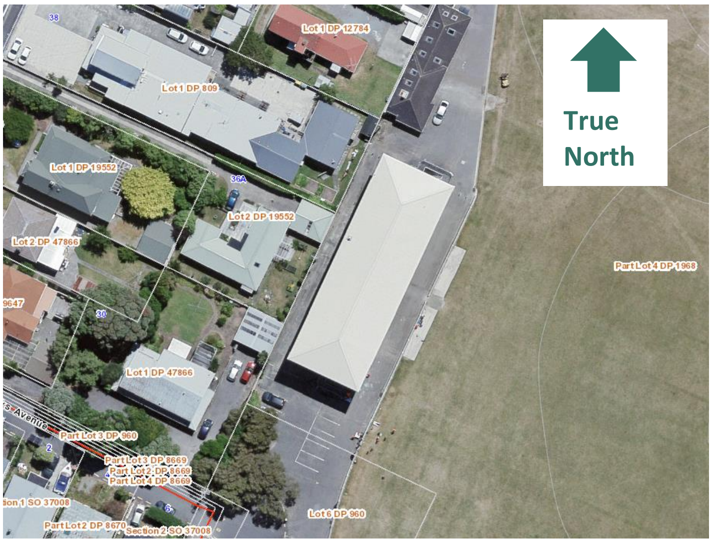

Figure 7 - Arial Photograph (Greater Wellington GIS Viewer)

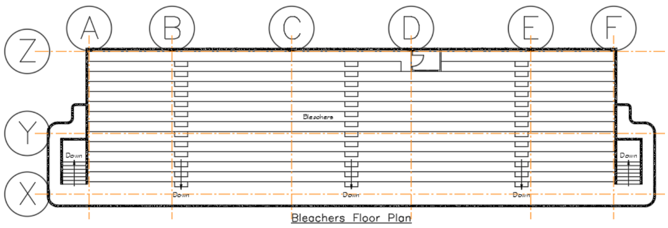

Figure 8 – Bleacher Plan for Reference Gridlines

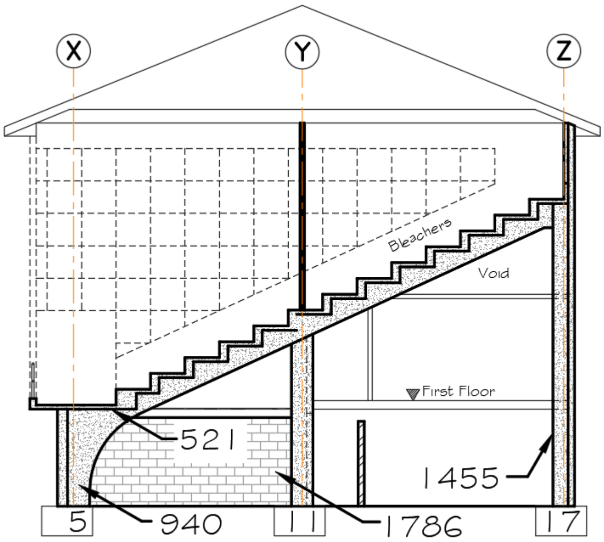

Figure 9 - Building Section – Grid E

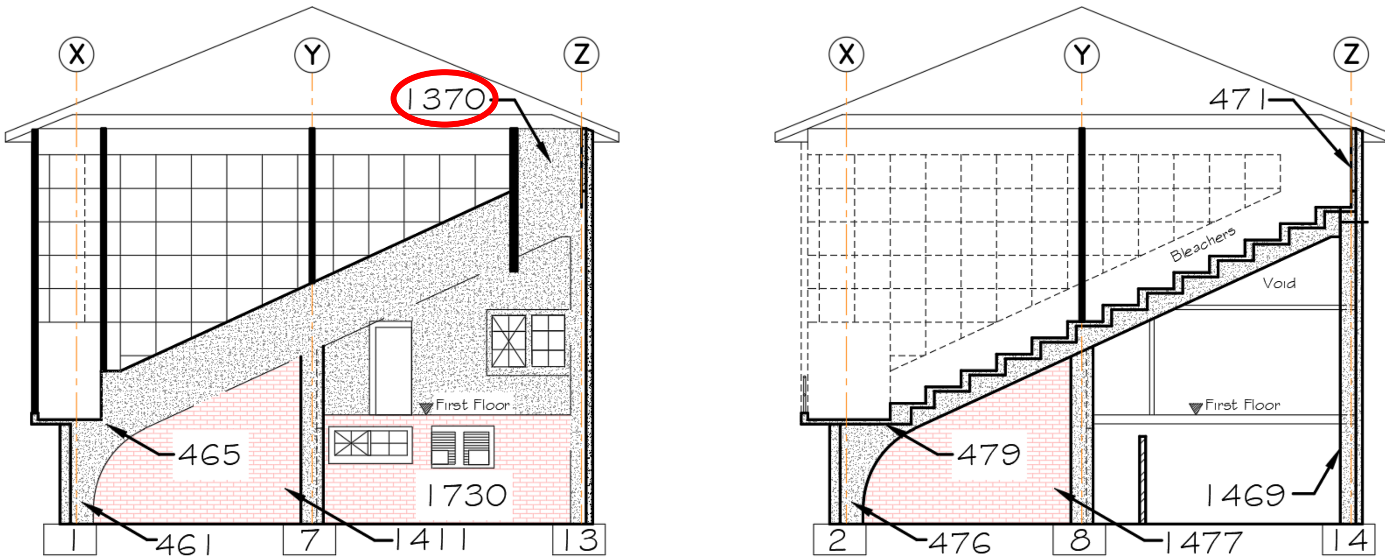

Figure 10 - Left: South End Elevation (Sketch) on Gridline A, Right: Building Section (Sketch) on Gridline B.

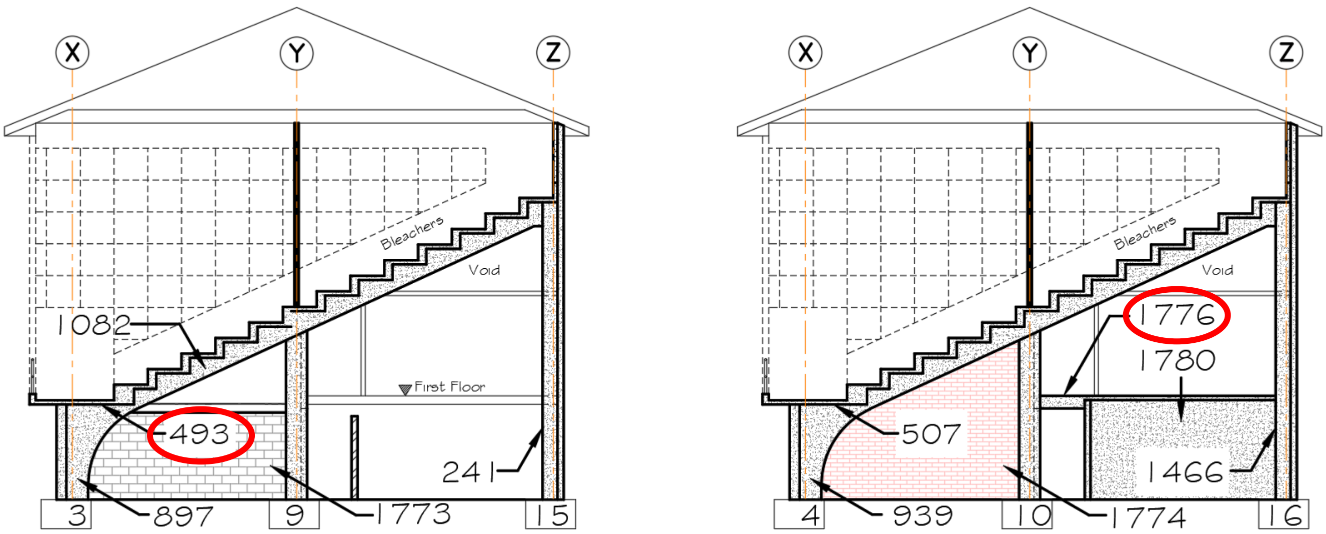

Figure 11 - Left: Building Section (Sketch) on Gridline C, Right: Building Section (Sketch) on Gridline D

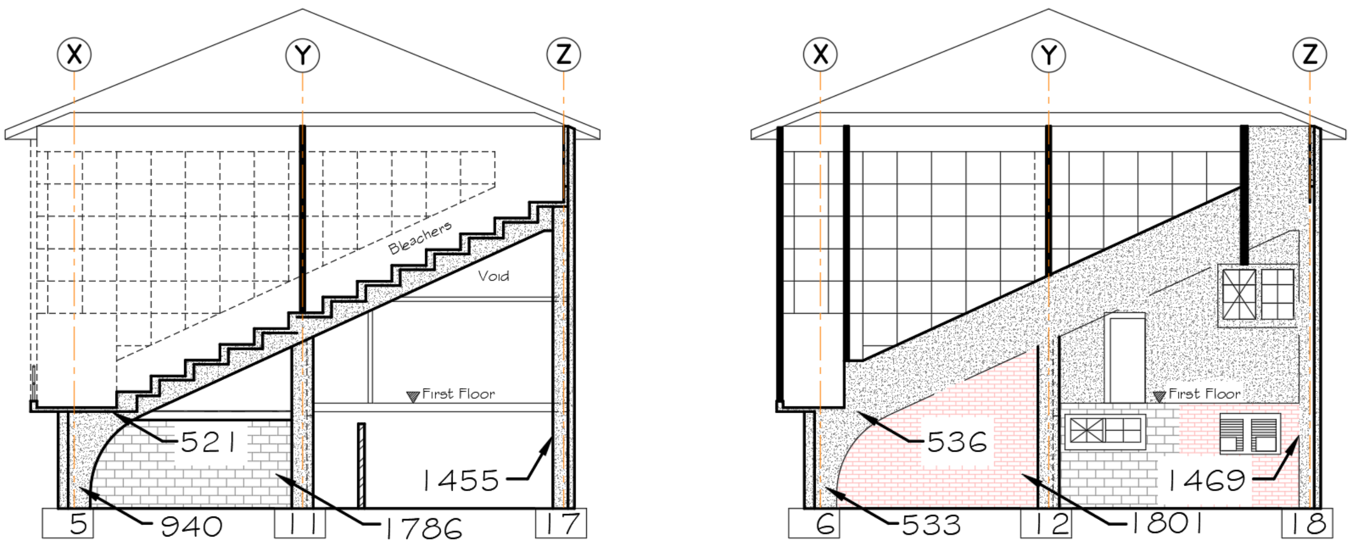

Figure 12 – Left: Building Section (Sketch) on Gridline E, Right: North End Elevation (Sketch) – Grid F

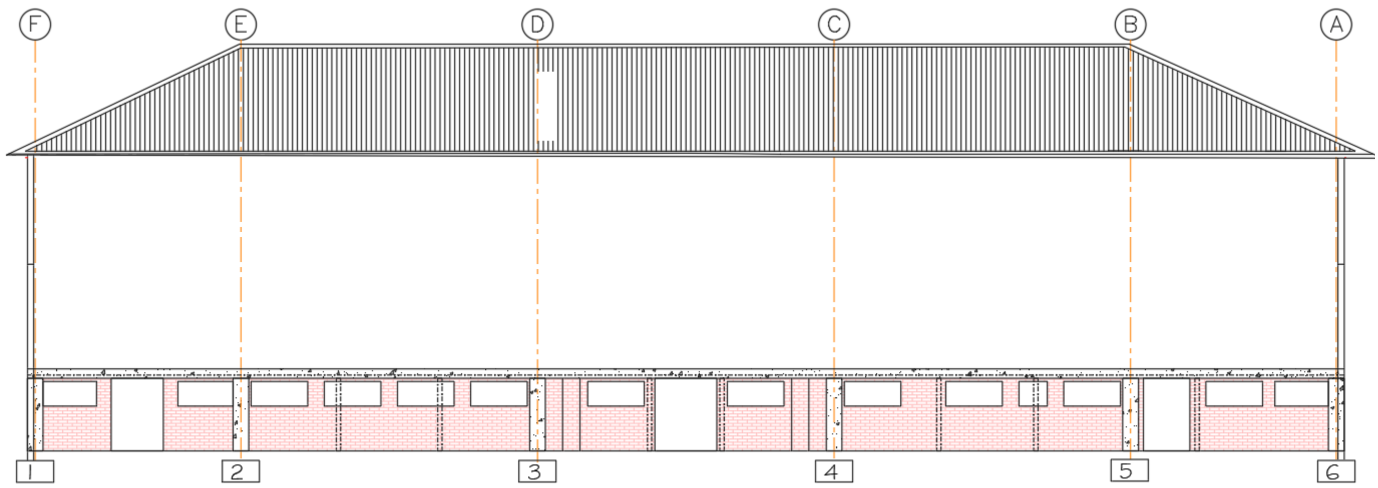

Figure 13 - East Elevation (Sketch) along Gridline X

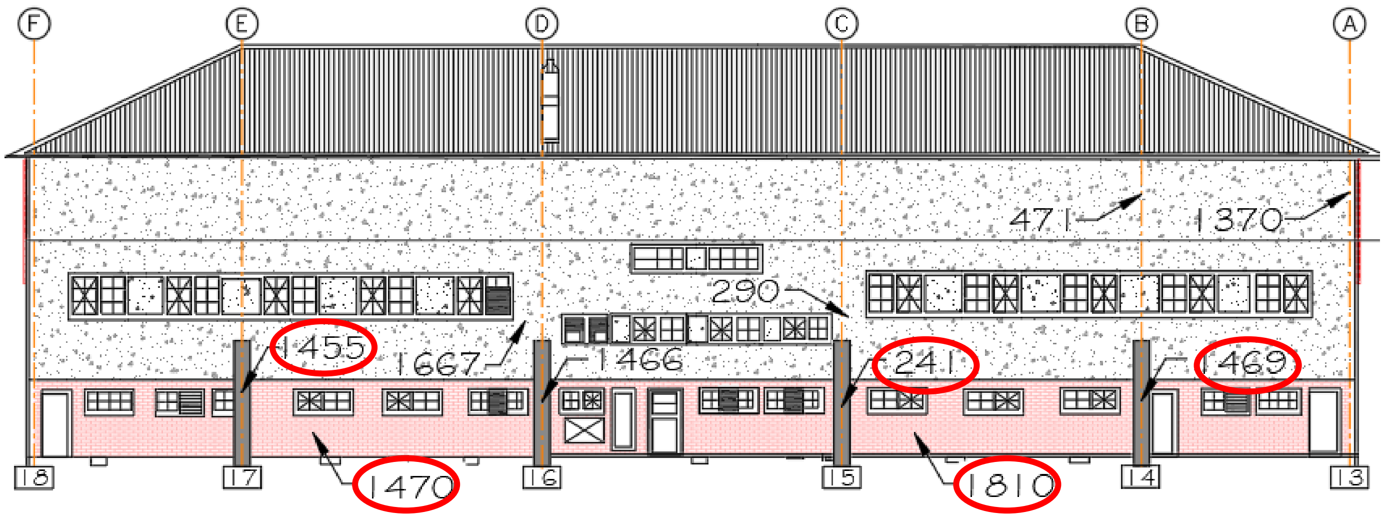

Figure 14 - West Elevation (Sketch) along Gridline Z

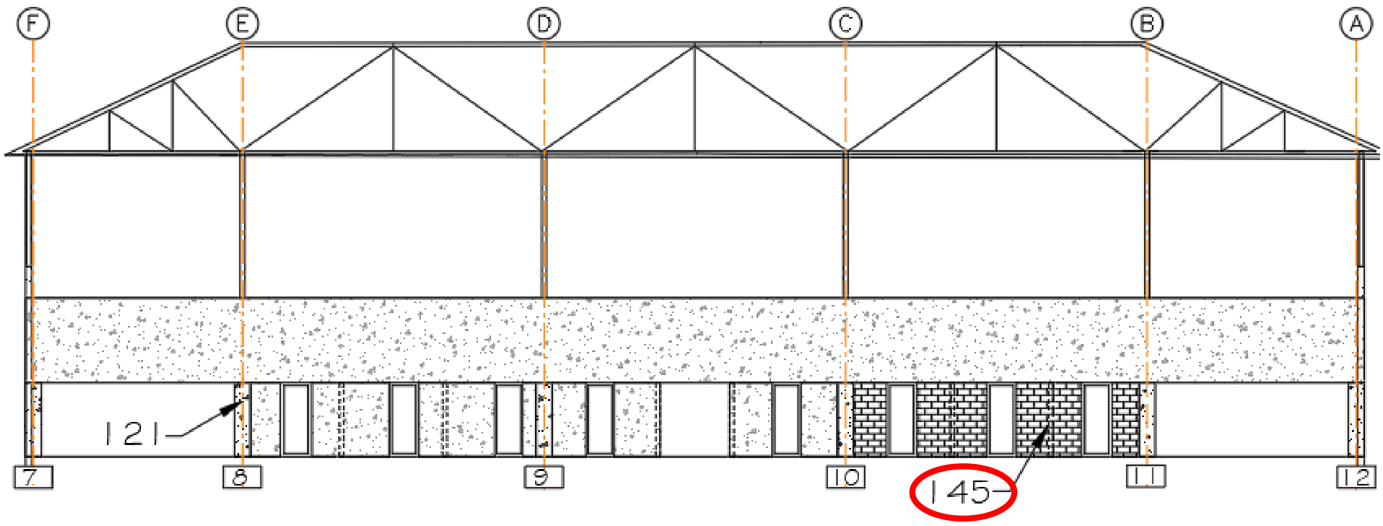

Figure 15 - Building Section (Sketch) along Gridline Y

The grandstand consists of 13 tiers of seating between 2.5 and 7.5 metres above ground level. Over the grandstand is a corrugated iron hip roof. The area under the grandstand has two levels, at ground level there are changing rooms and ablution facilities, and at Level 1 there is a social hall, kitchen, toilets, and offices. The level 1 floor is half the width of the overall building width.

There are external stairs at each end and internal stairs in the middle of the Level 1 floor area.

2.1 History

The grandstand was constructed by Nicholls & Pearce in the late 1939 for the Petone Borough Council. Some structural improvement work had been carried out circa 1979, including roof bracing and the replacement of some brick walls with reinforced concrete masonry. Some seismic strengthening work had been carried out circa 2014, including seismic restraint of the chimney, building up the west wall columns, and strengthening the end walls between bleachers & roof.

2.2 Original 1939 Structure

The seismic structure is five bays long and two bays wide. The gravity structure for the bleachers consists of 14 transverse beams. These raking beams are supported on a wall at each end and a wall/beam in the middle i.e. they have two spans.

Over the grandstand is a corrugated iron hip roof supported on steel angle trusses. These trusses are supported on the rear (west) wall and steel I section columns along the center of the grandstand. The trusses cantilever approximately 7.5m from these columns over the front of the grandstand. Lateral restraint of the roof is provided by the rear wall and two short structural walls at the ends, near the rear corners. The lack of bracing to the ground at the front of the roof makes it a highly torsional sub-structure.

The bleachers on which the seating is located are made of cast in-situ reinforced concrete of about 50mm thickness, reinforced with a narrow mesh in the treads and longitudinal bars with links in the risers.
It is supported by fourteen raking beams which are supported by:

(i) the rear (west) wall,
(ii) a deep beam supported on six columns at the center and
(iii) a beam approximately 1.2m inside the front (east) wall which in turn spans between six faceted columns.

All of the above-described bleacher support is cast in-situ reinforced concrete.

At ground floor level the exterior walls are mainly 230mm brick infill panels between columns and openings. From the level of the top of the ground floor door and window openings to the underside of the bleachers, the exterior walls are 203mm concrete reducing to 152mm concrete above the bleachers.

At ground floor level the internal walls are predominately 190mm blockwork partitions terminating approximately 200mm below the ceiling with small circular hollow steel members bridging the gap. These blockwork partitions may not be original structure. At some locations there are 220 and 152mm thick concrete walls and single skin brick walls full height.

At the level 1 timber floor, the deep beam supporting the centre of the bleachers forms the eastern wall of the level 1 space. Other internal walls are timber framed except for a small number around the interior stairs which are 152mm thick concrete. The upper floor has timber joists spanning between walls and beams below, except for a small area around the interior stair which is concrete.

At each end of the building there is an exterior concrete stair providing access to the bleachers and a cantilever walkway providing access to the upper floor interior area. They are supported by 152mm concrete walls and the end walls to the main structure.

The longitudinal reinforcing is predominately ISTEG twisted round bar, with some plain round bar.

The bleachers are cast in-insitu reinforced concrete supported by fourteen transverse raking beams with supports at each end and one support in the middle. Six of these (one per gridline) beams are supported by columns; the remainder are supported by secondary beams or walls.

2.3 Structure Added

Additional roof bracing covers half of the roof along the top of the west wall (grid Z), and the roof bracing members are 100 N.B. x 4.5 tube. The original roof bracing members are 19mm rods covering the whole roof area.

There is an external PFC and there are built up columns on the rear wall. The chimney has seismic restraint and the reinforced concrete block appears to be added.

2.4 Inspection and Documentation Review

A set of the 1939 construction drawings by John S Swan and W.E Lavelle has been provided by the Hutt City Council together with drawings of additional roof bracing added in 1979 to a design by Spencer Holmes Miller and Jackson. No documentation was found for alterations to the changing rooms and facilities on the ground floor that appear to have been designed and built in the 1970's.

The 2014 drawings, DSA report, DSA calculations, design calculations and specification are available.

The historical structural documents that exist are:

Drawings:
- Original construction Drawings by John S Swan and W.E Lavelle, 1939. (Refer to Appendix E.)
- Structural drawings of additional roof bracing, Spencer Holmes Miller and Jackson, 1979. (Refer to Appendix F).
- No documentation was found for alterations to the changing rooms and facilities on the ground floor that appear to have been designed and built in the 1970's.
- Strengthening Drawings by Sawrey Consulting Engineers Ltd, 2014. Ref 8757 S000o,S101o, S102o, S201o, S202A, S301o, S302o, S303o. S304o, S401o, S402o, S403o, S404o, S405o, S406o. (Refer to Appendix G).
- Balustrade replacement drawings by Sawrey Consulting Engineers, 2018.

Reports.
- Structural DSA 2014 by Sawrey Consulting Engineers Ltd.
- Scoping report by Sawrey Consulting Engineers Ltd, 2018.
- ISA 2020 by Sawrey Consulting Engineers Ltd.
- Geotechnical Desktop Study Petone Recreation Ground Grandstand by Coffey Geotechnics, 1 September 2014.
- Fire Safety Report – Petone Recreation Ground Seismic Upgrade Works by National Consultants Ltd 6, November 2014.

Reports Procured for this DSA
- Petone Recreation Grandstand - Geotechnical Investigation and Assessment Reference: 773-WLGGE317986 by Tetra Tech Coffey Rev-1 21 April 2023. (Refer to Appendix D).

- Independent Concrete and Reinforcing Investigation Report; Project: Kirks Avenue, Petone Rec Grandstand by Concrete Structure Investigations Ltd, 2 June 2023. (Refer to Appendix C).

There have been three site visits by Sawrey Consulting Engineers, one site meeting with the Hutt City Council on the 21st of February 2023, and two site meetings with Concrete Structure Investigations, one in March 2023 and one in May 2023.

There is a significant discrepancy between the original drawings and the 2014 drawings for the walls at the ground floor level. Inspection of the structure has confirmed that the 2014 drawings are a better representation of what has been built. Some of the discrepancies include the thickness of the external walls, the material of external walls, the wall layout of internal walls, and the height of internal walls. The 2014 DSA report notes that "A number of walls were drilled to confirm the material they were constructed from".

The 2014 DSA reported that "Schmidt hammer testing on concrete elements was undertaken which indicated concrete compressive strengths between 16 MPa and 49 MPa. As only 2 of 24 tests indicated strengths less than 25 MPa we have adopted 25 MPa in our assessment". [30MPa has been adopted in the current assessment – see section 3.8.]

The 2014 DSA reported that "punch tests as defined in Table 10.2 of the NZSEE 2006 [1] Document had been carried out on the brick mortar in several places. Results varied from 5 to 10mm penetration indicating stiff mortar".

2.4 Geotechnical Investigation

A geotechnical investigation and assessment report has been provided by Tetra Tech Coffey (NZ) Ltd (21 April 2023) (Appendix D) providing advice on the ground conditions at the site of the grandstand. A previous desktop study was completed by Coffey in September 2014 for the previous DSA.

The current report involves a shallow intrusive investigation consisting of a site walkover and three window sampler boreholes to 3m depth. The report provides information regarding updates to the original desktop study from 2014, estimated site subsoil class, soil spring stiffness, assessment of the liquefaction risk at the site and recommendations on future investigation and geotechnical assessment works.

The assessed ground model for the site using the window sampler boreholes and the wider available data is tabulated below.

Table 4 – Subsoil Description (by Tetra Tech Coffey Limited, 21 April 2023)

Unit	Description	Top Depth (m)	Bottom Depth (m)	Consistency	DCP (blows/100mm)	Cone Resistance qc (Mpa)#
A	Silt/ silty sand	0.1	0.3/0.7	Soft to stiff/ loose to dense	2-6	-
B	Sandy Gravel	0.3/0.7	2.2/2.5	Medium dense to very dense	4-12	-
C	Gravelly Sand	2.2/2.5	3.0+	Dense to very dense	8-18	-
D	Sand and Gravel*	3	6	Generally described as dense	-	12-30+
E	Sand*	6	10	Medium dense to dense	-	8-12
F	Silt/ sand/ organics*	10	>15	Loose to medium dense/ firm to stiff	-	2-10

* Estimated from nearby NZGD data # assessed from the CPT trace available on NZD

The conclusions and recommendations from the report are:

- The ground model comprises variable soils to up to 0.7m depth underlain by medium dense to very dense sandy gravel and dense to very dense gravelly sand to 3m depth. Gravel dominate soils are anticipated to extend to ~6m underlain by sandy soils to ~10m based on the publicly available data.
- Groundwater is at 1.25m depth.
- The site is considered to have a 3m thick non-liquefiable crust and likely up to 6m thick non-liquefiable crust. Therefore, the potential for surface deformation or differential settlement is considered low.
- Bearing capacity is generally considered to be 300kPa, however foundations founded shallower than 0.7m depth at the southern end of the structure should be checked for 200kPa ultimate bearing capacity.
- If any in ground improvements are required as part of the strengthening works or further certainty is required about the ground conditions or liquefaction potential below 3m depth a deeper intrusive investigation is recommended. We (Tetra Tech Coffey) would recommend 2 x CPTs tests to 15m depth with a DPSH if required to penetrate gravelly layers.
- Three test pits carefully dug within the building have also been suggested.

  Refer to Appendix D for the report.

2.5 Limited Durability Investigation

Rust is visible on structural steel members and on reinforcing steel to concrete elements where concrete cover is absent.

Concrete Structure Investigations Limited has undertaken onsite testing and measuring to assist in the estimation of strength loss, of steel and concrete. Samples of the following building elements have been looked at:

- Core testing to concrete samples – chloride testing and compressive strength testing
- Measuring reinforcing and structural steel thicknesses after rust has been removed.
- Measurement of cover concrete thicknesses

Chloride testing has been undertaken by WSP following the reference test methodology cited by NZS3101. Interpretation of the results have been provided by CSI in Section 13.6.7 of the report, and some extracts are listed below.

- Considering the overall testing results it is unlikely that chloride is a contributing factor regarding corrosion.
- It is very unlikely that sea salts are a contributing factor for the deterioration of the structure, although this is based on one sample only.

Some extracts from the Proposed actions of the report are listed below.

- Insufficient cover may have contributed to the development of corrosion.
- Some measurements taken during the investigations had low cover which will have contributed to corrosion.
- [This can be due to] The minimum cover was not met during the construction.
- Recommendation: Scanning to identify the areas of low cover.
- [This can be due to] The original minimum cover has been reduced by the process of carbonation.
- Recommendation: Extracting concrete samples for carbonation testing.
- To understand the extent of corrosion half-cell potential testing is required.

Figure 16 – Corrosion of Steel Column - Base Connection to Concrete - Column 1 Above Bleachers

Figure 17 – Corrosion of Steel Column - Base Connection to Concrete - Column 1 Above Bleachers

Figure 18 - Corrosion of Reinforcing Steel - Concrete Bleacher Beam - Column 3 Above Bleachers

Figure 19 - Corrosion of Reinforcing Steel - Concrete Bleacher Beam - Column 3 Above Bleachers

Part of the scope of work had been "visual inspection to determine up to what height replacement [of the steel columns] may be required above the lintel". In this regard, CSI have commented that "it is not possible to make an informed statement due to the general bad condition of the steel. A detailed assessment by a qualified and certified steel expert is required for such a conclusion".

Further durability investigations may include ultrasonic testing of selected external structural steel members, a cover concrete survey, and half-cell potential testing of reinforcing steel. Refer to the CSI report in Appendix C for further information.

2.6 As-built Structure Investigation

Concrete Structure Investigations Limited have carried out destructive and non-destructive investigation at selected positions under the bleachers. They have investigated:

- The height of brick walls. The height of the brick infill wall at ground floor between columns 2 and 8 and between columns 4 and 10 has been verified, these are full height from ground floor to the underside of the primary raking bleacher beam.
- Reinforcing steel in a lintel. The reinforcing steel in the lintel over the ground floor corridor near column 10 has been verified. This lintel has two longitudinal bars. The spacing of vertical bars is greater than 300mm, and the diameter of longitudinal bar is similar to the diameter of the transverse reinforcing bar.
- Concrete core samples. The concrete compressive strength has been tested using the core sample at one position. Refer to the CSI report in Appendix C for further information. The recorded compressive strength of the sample was 35MPa.

Figure 20 - Reinforcing Steel to Lintel Over the Ground Floor Corridor Adjacent to Col 10.

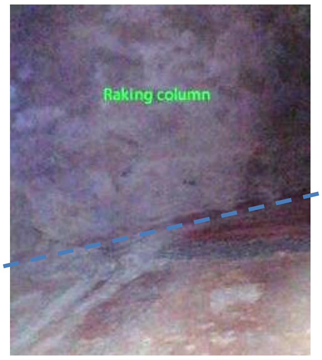

Figure 21 - Height of the Un-reinforced Masonry (URM) Infill Wall - Btn Col 4 & 10 at Ground Floor.

Figure 22 - Concrete Core Sample for Compressive Strength Test

3.0 Basis for Seismic Assessment

Our work has been limited to assessing the ultimate limit state capacity of the building as necessary to estimate the percentage new building standard. The serviceability limit state is not included in the brief for this DSA and has not been assessed. The serviceability state (SLS) is the point at which damage begins to occur, and the ultimate limit state (ULS) is the point at which the building may be severely damaged but still has adequate structural integrity to allow people to escape.

3.1 Regulatory Standard

The building weight has been evaluated in accordance with ASNZS1170.1, and earthquake actions have been evaluated using NZS1170.5.

The structural design standard: NZS1170.5:2004 – Earthquake Actions NZ[2] was cited by the Department of Building and Housing as the approved document (i.e. "current code") for B1 VM1[3], effective from 1 December 2008. Buildings compliant with the "current code" or having the structural equivalence (as assessed with the guidelines) are referred to as having 100%NBS (Percentage New Building Standard).

The seismic loading required by NZS1170.5[2] is dependent on the soil on which the building is founded. Loadings are typically lower for buildings founded on strong rock (soil Class A) and increase for those founded on weaker soils to a maximum for very soft soil (soil Class E).

The seismic assessment guidelines: The Guidelines[1] - *The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments*, dated July 2017, are an integral part of the EPB methodology produced by the Ministry of Business, Innovation and Employment (MBIE) under section 133AA to 133AY (subpart 6A) of the Building Act 2004.

3.2 Load Path for East-West Earthquake Shaking

After viewing drawings and after sample inspections and investigations, the following is a description of inferred load paths for use in modelling.

The east-west seismic actions at roof level are thought to be transferred through roof bracing to reinforced concrete walls at each end. The end reinforced concrete walls extend above bleacher level to the roof level at the rear. The rear wall and its columns cantilever out-of-plane above the top edge of the bleachers. The east-west seismic actions from the bleachers are carried directly to the ground through six reinforced concrete moment frames. One of these frames has an unreinforced masonry (URM) clay brick infill, the two end frames have clay brick infills plus stairs, and one internal wall has URM clay brick infill plus four reinforced concrete shear walls. Two have partial height block infill walls. There is some load sharing through the reinforced concrete bleachers which are thought to act as a stepped diaphragm. The foundations are shallow pads, ground beams and strip footings.

3.3 Load Path for North-South Earthquake Shaking

The north-south seismic actions at roof level are transferred through the roof bracing to the west wall. The primary lateral load resisting structure above L1 floor is reinforced concrete. Below the L1 floor there are three main bracing lines and an internal concrete core. The external east bracing line is 240mm thick clay brick infill, the external west bracing line is 240 mm thick clay brick infill, and the middle bracing line 150mm thick reinforced concrete with some 140mm thick concrete block masonry. The concrete core is a reinforced concrete box with timber framed stairs and a L1 reinforced concrete floor. The reinforced concrete bleachers act as a stepped diaphragm to share loads between the three bracing lines in this direction also. The foundations are shallow pads, ground beams and strip footings.

3.6 Building Structure Assumptions/Model

Four stiffness models have been adopted because a strut and tie model has been used at the positions of some unreinforced masonry walls and some concrete walls. Each strut is orientated from high to low in the direction of the seismic actions to effectuate a compression-only member. The four stiffness models are listed below.

1) North to South plus East to West (high end of the strut is at the north end and at the east side)
2) South to North plus East to West (high end of the strut is at the south end and at the east side)
3) North to South plus West to East (high end of the strut is at the north end and at the west side)
4) South to north plus West to East (high end of the strut is at the south end and at the west side)

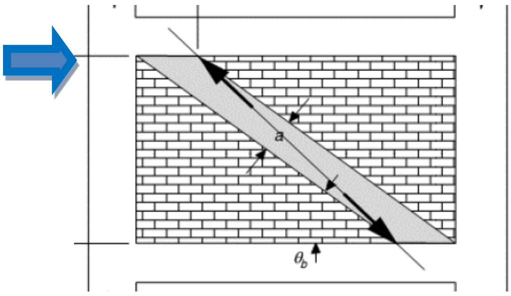

Figure 23 - Equivalent Strut for URM Infill Wall - North to South Shaking

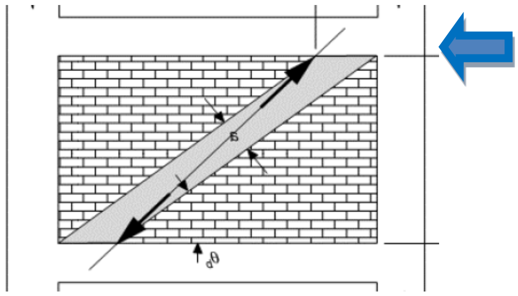

Figure 24 - Equivalent Strut for URM Infill Wall - South to North Shaking

A pseudo push over analysis has been carried out on the west wall to determine whether to model the structure as either an infill wall, a partial infill wall with short columns, or a moment resisting frame.

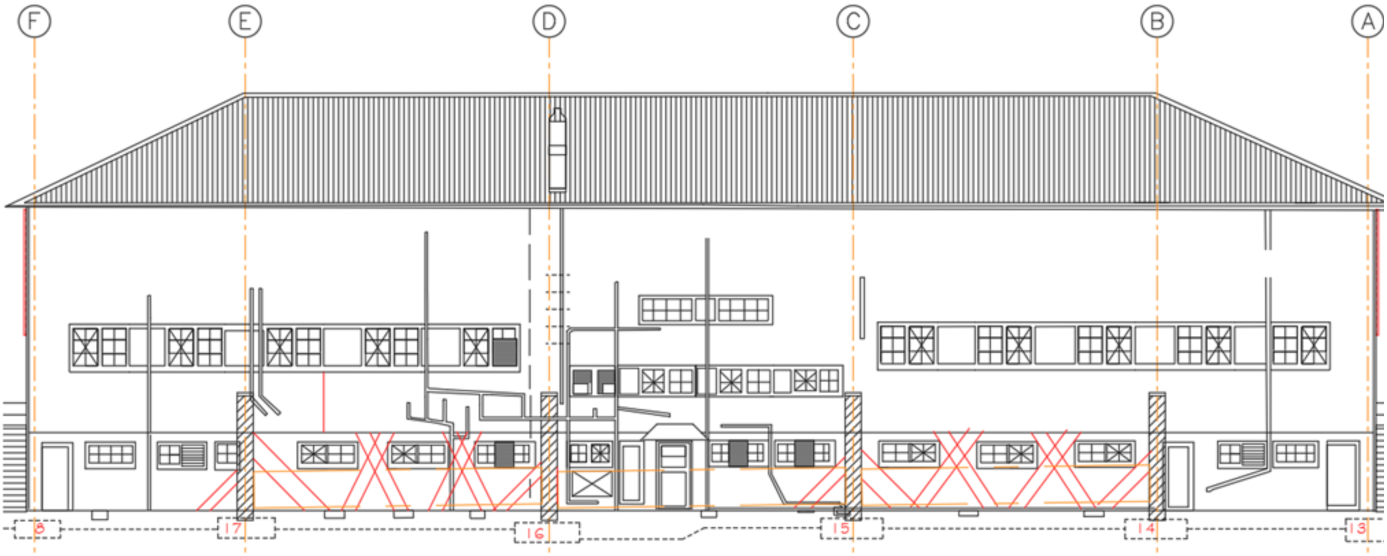

Figure 25 – Diagonal compression only struts (indicated in red) - elevation of the back wall.

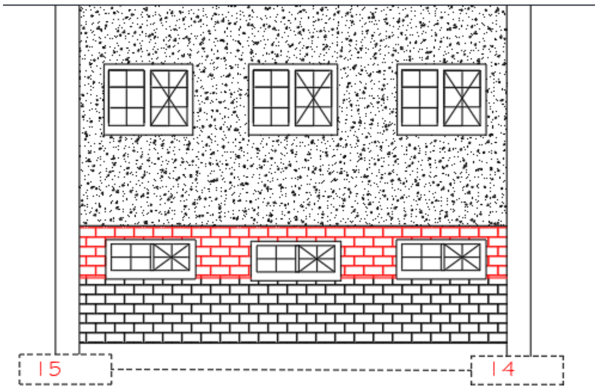

Figure 26 - URM between the windows.

These have been removed from the model because of low toe-crushing strength.

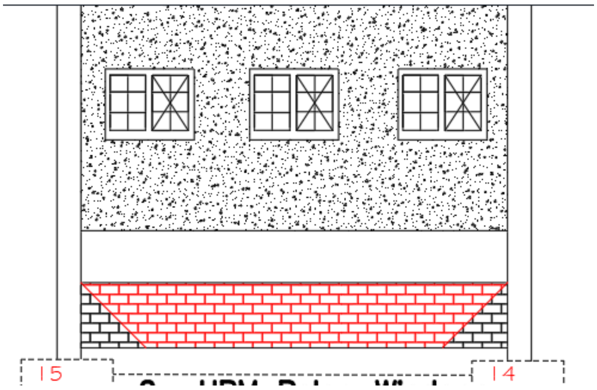

Figure 27 - URM walls below the windows for out of plane actions.

These have been removed from the Staad.Pro model because they have an independent period of vibration.

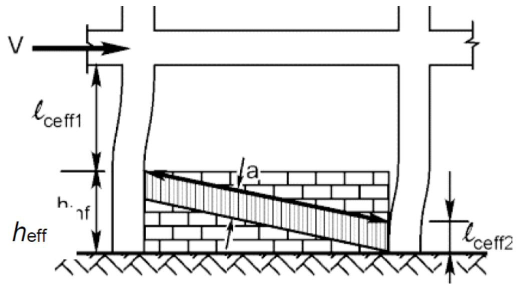

Figure 28 - URM walls below the windows for in-plane actions.

These have been modelled in accordance with figure C7.13 of the guidelines.

Figure 29 - Photograph of the back wall between columns 16 and 17.

The timber joists have been removed from the Staad.Pro model because of low strength connections. Rational Simple Lateral Mechanism Analysis (SLaMA) has been carried out and the end fixings are expected to exhibit higher ductility capacity, higher displacement capacity and significantly lower yield strength than the primary seismic structure. Therefore, the end fixings are expected to yield at low levels of earthquake shaking and contribute little to the primary lateral load resisting system. Two skew nails have been assumed for fixing floor joists to the bearer.

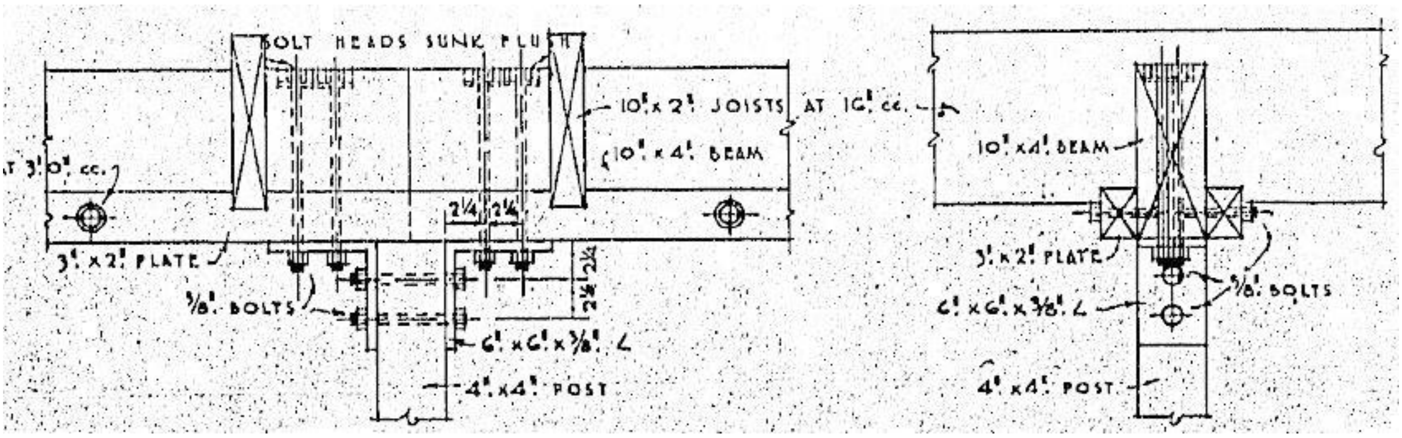

Figure 30 – Construction detail for the seating of Level 1 floor joists.

The reinforced concrete bleachers have been modelled as a series of parallel 'Z' shaped beams. Some concrete walls and all masonry infill walls have been modelled as a strut and tie system. Most concrete walls and concrete block walls are modelled with two diagonal struts or as centreline models. The OOP cantilever action of the rear wall to the roof has not been modelled, whereas the column cantilever bending action has been modelled. Ground beams have been modelled with fixed pin supports at the position of column pads.

3.7 Earthquake Actions Assumptions/Model

A modal response spectrum (MRS) method of analysis has been carried out to account for structural irregularity.

The MRS base shear scale factors have been evaluated in each direction based on the Equivalent Static Method horizontal design action coefficient and the based on the Rayleigh Method of estimating the period of vibration. The input parameters are tabulated below.

An earthquake combination factor of 0.3 has been adopted for floor live load as opposed to 0.6.
A area live load reduction factor of 1.0 was adopted for the grandstand.

Table 5 – Assumptions/ model for horizontal seismic actions

Elastic Site Spectra, C(T)			Modal Response Spectrum Method (MRS)	**X (transverse)**	**Z** (longitudinal)
Hazard Factor, Z	0.4		Fundamental Period of vibration	0.13 sec	0.09 sec
Soil Class (except parts)	D		Period of vibration (shortest)	0.05 sec	0.05 sec
Importance Level	3		Period of vibration (longest)	0.29 sec	0.29 sec
Risk Factor, R	1.3		Statistical Combination	CQC	CQC
Structural Performance Factor, Sp	1.0		No. Modes at 90% Mass	15	25
Near Fault Factor	1.0		Base Shear at 90% Mass	7,670 kN	5,328 kN
APE	1/1000		Scale Factor	1.72	2.48
Equivalent Static Method (ESM)			Mode Cut-off	25	25
Period of Vibration (Rayleigh Method) From previous DSA	0.29 sec		Mass Participation	100%	90%
Horizontal design action coefficient	1.560		Applied Direction (100% Eu + 30% Eu in the orthogonal direction)	1.72 (X) +0.516 (Z)	2.48Z (Z) +0.744 (X)
Base Shear	13,205 kN				
Ductility, μ	1.0				

Table 6 - Assumptions/model for parts, and for vertical seismic actions

Vertical Seismic Actions			Parts, Design Response Coefficient		
Vertical Seismic Actions	4.5		**Height of Structure, hn**	10.0 m	
Period of Vibration, T	0.15		**Classification of Part**	2 & 3	
Ductility, μ	1.0		**Part Risk Factor, Rp**	1	

Table 7 - Assumptions/model for seismic actions on un-reinforced masonry walls.

	Period (Tp)	Height of the part, hj	Design Action Coefficient, Fph
Infill URM – back wall	0.82 sec	1.2 m	2.0
Cantilever URM – back wall	0.96 sec	0 m	NA use charts
Infill URM – transverse frames 120mm thick	1.0 sec	1.8 m	1.743
Infill URM – transverse frame 230mm thick	1.0 sec	1.8 m	1.743

3.8 Material Properties Assumptions/Model

The ISTEG reinforcement system had been developed in Germany and introduced to the UK in the early 1930's. The ISTEG Steel Corporation patented the machine that manufactures it. An article published by the Journal of the American Concrete Institute (17 June 1935, D.B. Steinman, Isteg Steel for Concrete Reinforcement) has been referred to and compared with the requirements of the guidelines. The testing referenced by the article was carried out by the Civil Engineering Testing Laboratories of Columbia University (September 1934 to March 1935).

This DSA has adopted the probable yield stress of 325MPa for Isteg reinforcing steel. The Isteg steel is cold drawn and the manufacturing strain has been calculated as 3.11%. Shrinkage strain of 0.1% has been allowed for and the strain limit of 4% has been adopted as this is the maximum strain allowed for cold drawn mesh (table C5.4 of the guidelines). The test data indicates that the average yield stress of an Isteg bar is 348 MPa and it concludes that this is 1.41 times the average yield stress of a plain round bar. It is noted that the ISTEG bars in the study did not exhibit a defined yield plateau and this is consistent with observations described for cold drawn mesh in note 3 of Table C5.4 of the guidelines. The assumed stress-strain curve is shown below.

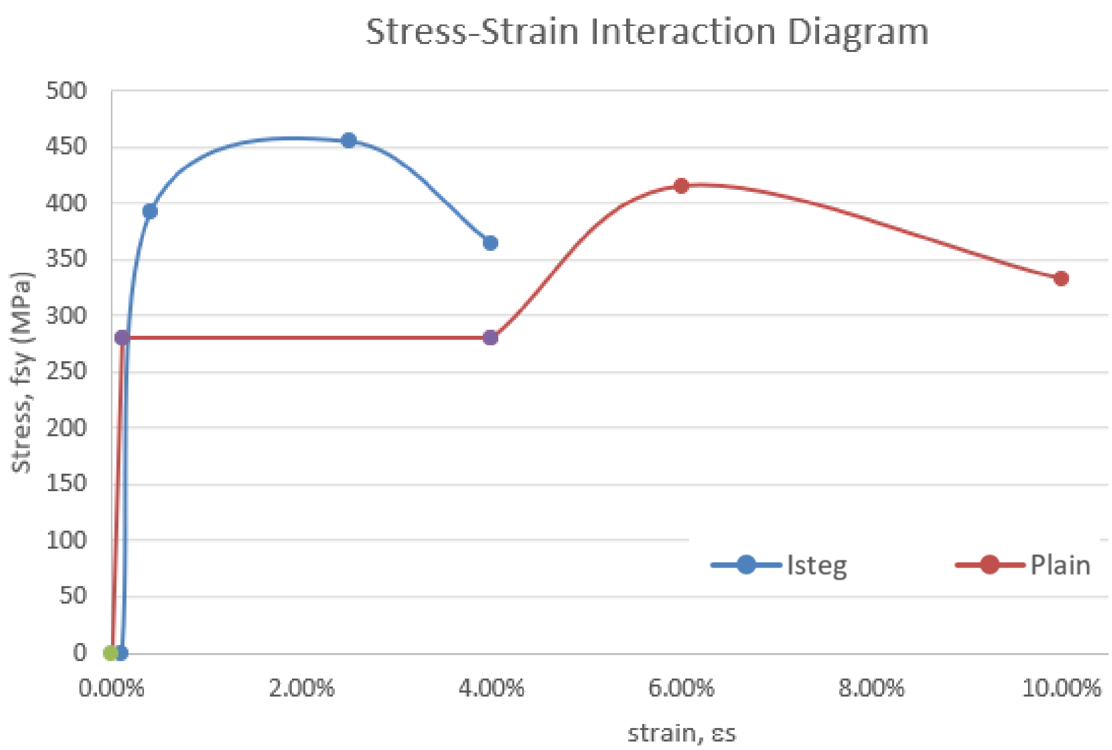

Figure 31 - Assumptions/model for stress/strain interaction of ISTEG reinforcing steel bars (cf plain round bars)

The original concrete strength has been assumed to be 30MPa because this is the lower of the two options 1) a single concrete core compressive strength test specimen and 2) evaluated from the concrete mix design shown on the original drawings. The existing concrete strength has been tested to be 35 MPa by WSP Opus Limited (refer to Appendix C for further information), which is greater than the 30MPa that had been evaluated from section C5.4.2.2 of the guidelines and the concrete mix design shown on the drawings. The mix design is similar to the 17.5/20 MPa mix prescribed in NZS 3104.:2021 table 3.1. A factor of 1.5 has been applied to the design concrete strength as per *the guidelines*.

Cracked section properties have been used for all concrete elements.

Table 8 - Assumptions/model for probable material properties.

Reinforcing Steel			
ISTEG, Yield Stress fy	325 MPa	Engineering Judgement	
1939 round bar, Yield Stress, fy	280 MPa	Table C5.4	
2014 deformed bar, yield stress, fy	324 MPa	Table C5.4	
2014 round bar, yield stress, fy	324 MPa	Table C5.4	
Reduction for corrosion	varies	Engineering Judgement	
Clay Brick Masonry			
Masonry Compressive Strength, f'm	12 MPa	Section C8.7.3	
Elastic Modulus, E	8.4 GPa	Equation C7.1	
Shear Modulus, G	3.4 GPa	Equation C8.5	
Concrete Structure			
Original Concrete, f'c	30 MPa	Engineering Judgement	
Original Structural Steel			
Original Structural Steel, fy	210 MPa	Table C6B.5	

The level of corrosion of reinforcing steel has been assumed; and it is accounted for by reducing the cross sectional area of the reinforcing steel shown in the original plans. Generally, for a particular member, this DSA has assumed that all outer layer reinforcing steel has the same remaining area of steel as the bar with the worst corrosion. The exception is at gridline C above column 3, which has been considered an outlier; and in this case the bar with the worst visible corrosion had 3% remaining steel (see below). Reinforcing steel near the inside face of concrete structure have not been reduced.

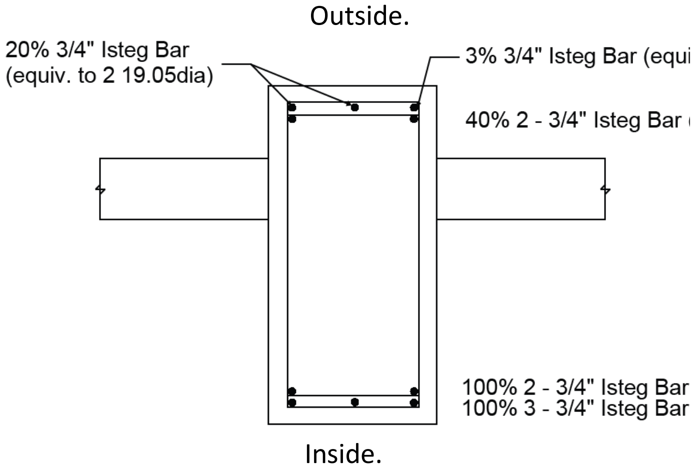

Figure 32 - Assumptions/model for reduced strength of corroded reinforcing steel: bleacher beam, col 3.

Figure 33 - Photograph of corroded reinforcing steel: bleacher beam above col 3.

3.9 End Walls Above the Bleachers Assumptions/Model

The end wall structure is a combination of steel posts, timber mullions, glass panels and a concrete wall. The %NBS score of the timber mullions has not been evaluated because deterioration of the timber is severe and has not been quantified. The glass panel dimensions are assumed to be 0.6m wide by 1.8m high and the panels are considered to be light weight in accordance with Table A4.1 of the guidelines. The timber mullions and the steel posts with glass panels attached are a building parts positioned over egress paths classified as a space class 1 (table A4.2 of the guidelines). Therefore, following this guidance, the timber mullions and steel posts are expected to be a significant life safety risk.

There are six vertical steel posts above the bleachers, three at the north end and three at the south end. The %NBS score has not been evaluated because the level of corrosion. The posts are expected to bend about their strong axes due to out-of-plane seismic actions plus they are in axial compression under both horizontal and vertical seismic actions. The end connections are assumed to be pins. The steel posts are part of the primary seismic structure and have been included in the seismic model. However, the study indicated that the loss of a post would not necessarily lead to an immediate roof collapse. There is an alternative load path: the cantilever steel roof trusses which are fully enclosed. However, vertical displacements of the cantilever roof truss may cause a secondary failure such as out of plane deflections that increase the risk of glass panels falling.

Wind actions are not the topic of a seismic assessment, but out-of-plane wind actions imposed on these end wall elements would also necessitate reliable posts and mullions. Currently, the load path for out of-of-plane actions is considered un-reliable because of high levels of observed corrosion and timber decay: however, the structures have performed adequately in recent weather events. A special wind study has not been carried out.

The guidelines recommend non-destructive testing to gain assurance of the mechanical properties of older steelwork (C6.4.1) particularly when the steel is "of unknown origin". Furthermore, CSI have suggested non-destructive testing for obtaining further information on the extent of corrosion. The guidelines section C6.2.4 building condition (deterioration over time) mentions the reduction in member strength and ductility, and section C6.2.3 observes that Inadequate load paths through connections is the most common cause of local failures in steel buildings.

Generally observations from bleacher level indicate the amount of corrosion in the steel posts is most severe near the bottom of the posts at the connections: i.e. to the lintel and to the concrete base. The steel is described in the Independent Concrete and Reinforcing Investigation Report (2023, CSI) as being in bad condition generally and with severely corroded spots noticeable. It is concluded that in the light of the poor condition of the steel at the connections, the load path should be considered as unreliable, pending further detailed study (or structural improvement).

3.10 Geotechnical Assumptions/Model

Bearing demands for the foundations have been assessed to determine their %NBS under seismic conditions. The bearing capacity provided by Tetra Tech Coffey is 200kPa for foundations founded shallower than 0.7m depth, and 300kPa for deeper foundations, both with a reduction factor of 1.0. The foundation bearing area has been assessed incorporating the column pads, ground beams, and (about 400mm of) internal slab each side of the foundation beams.

Base shear resistance has been assessed, looking at passive resistance against the ground beam sides and sliding on a plane at the underside. The 400mm width of slab along ground beams contributes to the passive bearing and around has been determined by assessing the vertical reaction imposed on the slab during 'sliding with weak floor slab' principle from module 4: earthquake resistant foundation design. The slab was analysed by determining under what bearing capacity is may yield.

Further geotechnical assessment is required within the building platform to determine more representative soils parameters: this is outside the scope of this assessment.

Sliding to this building may occur under seismic ULS loadings. The shallow foundation system for the building appears to be integrated well, with ground beams and the floor slab tying the column pads together. It is therefore expected that the building will slide as a whole, with limited foundation damage. The commentary in "Module 4: Earthquake resistant foundation design" suggests 'it may not be practical to prevent buildings on mat foundations from sliding at the ULS level of shaking (and may not be critical to the safe performance of the building)'. Therefore, some sliding is expected but this may not affect the %NBS rating for the building. If seismic strengthening of the building was to be undertaken then work to the foundations may be required to address the extent of the sliding especially under the SLS level of shaking.

An overturning assessment has been carried out and 49%NBS(IL3) is the result. The assessment conservatively included live loading in the seismic demand and the restoring actions were limited to 0.9 times the building weight only.

4.0 Result of Assessment

A severe structural weakness (SSW) is a structural weakness for which rupture would lead to a collapse and for which the probable capacity may not be reliably assessed based on current knowledge. The critical structural weakness (CSW) is the lowest scoring weakness or weaknesses determined in the DSA whose rupture would cause a collapse. The CSWs are described in Table 9. All other results of the assessment are tabulated in Table 10 to Table 13 below. Calculations are appended. Please refer to Figure 8Figure 9 to 15 for the locations of structural members that are identified in the tables as being found to have a capacity less than 34%(NBS)IL3.

Table 9 – Structural Elements with Earthquake Scores less than 34%NBS(IL3)

Structural Elements and Systems Evaluated to be less than 34%NBS(IL3)			
Structural Element / System	**Strength / Capacity Load Path**	**Modelling of Observed Structure**	**Potential Mode of Failure and Physical Consequence**
Tops of Reinforced Concrete End Walls Ref. Fig 10 (& 12) **Above the bleachers, the wall at each end is mostly glazing with a short length concrete wall near the back. Minimal structural improvement of the wall has been carried out to increase the bending strength but this benefit is reduced due to the presence of significant corrosion of reinforcing steel, as investigated by CSI.**	The concrete wall bending capacity and shear capacity have been evaluated with contribution from reinforcing steel and the external PFC, but axial load contribution is assumed to be zero. Bending capacity governs the strength of this element. Two reduction factors have been assumed; 0.8 for the area of vertical reinforcing steel, and 0.3 for the shear capacity of the post installed anchors.	The tie to the parapet/barrier is assumed to provide no contribution, therefore the critical section is assumed to be at bleacher floor level. The vertical PFC member is attached to the face of the column with post installed epoxy anchors. The concrete in the edge of the shear wall lacks confinement steel and the anchor group's proximity to the edge means its full capacity may not be mobilised without modification.	If the end reinforced concrete wall fails in bending then the out-of-plane load on the back wall will increase, and this could be the final structural system to fail before a roof collapse occurs. **A Critical Structural Weakness, CSW.**
Ground Floor concrete lintel across corridor. Ref. 1776 Fig. 11. **Overhead, across the ground floor corridor is a concrete lintel which, when modelled, is significantly overstressed. The as-built structure has been investigated by CSI.**	The lintel has been evaluated for axial tension. This lintel ties the primary bleacher beam to a group of concrete walls and the concrete L1 floor slab. Two 9.5mm longitudinal bars (one at the top and one at the bottom) have been identified by CSI.	There is a timber floor above the lintel, but it is inferred that the timber floor spans across the corridor from concrete wall to concrete wall as shown on the plans. This beam has been modelled to distribute loadings to the concrete walls that are in line with it.	Failure of this lintel in tension would increase the load on the bleachers as a transfer diaphragm, on the bleacher beam as a frame, and on the URM infill wall. The diaphragm or the URM wall could be the final structural system to fail before collapse occurs. **A Critical Structural Weakness, CSW.**
The rear wall infill masonry walls and associated columns. Ref Fig 14. **Along the rear wall at ground floor level are reinforced concrete columns with fenestrated URM infill walls 230mm thick.**	There are five bays of URM infill walls. A failure sequence has been inferred: (i) The URM between windows would fail* first, (ii) the URM between doors and columns may fail second, (iii) the remaining URM walls may fail out-of-plane * third, and (iv) the reinforced concrete columns may then fail in bending fourth. * Refer to calculations.	Two bays of partial height URM infill walls have been modelled as compression only diagonal struts. Therefore, two out of six columns would be expected to attract most of the load in this model and assumed failure sequence.	The URM rear wall (out-of-plane) could be the final structural element to fail before collapse, but there are window frames and other URM walls that may hold the walls in-place. Therefore, the reinforced concrete columns could be the final structural member to fail before collapse occurs. **A Critical Structural Weakness, CSW.**

Structural Elements and Systems Evaluated to be less than 34%NBS(IL3)			
Structural Element / System	**Strength / Capacity Load Path**	**Modelling of Observed Structure**	**Potential Mode of Failure and Physical Consequence**
<u>One of the middle transverse reinforced concrete bleacher beams – the lower end of the raking beam. Ref. 493 Fig 11.</u> **This structural weakness has been identified because on the outside, at bleacher floor level near the front of the grandstand, the diameter of reinforcing steel rods has been significantly reduced by corrosion.**	The bending capacity and the shear capacity has been evaluated and bending strength governs. There are five longitudinal 'ISTEG' reinforcing steel bars near the outside face. Therefore, these are assumed to be subject to the highest level of corrosion.	The bending capacity has been evaluated with contribution from these five bars, but the contribution from axial load has been assumed to be negligible. One bar has been tested by CSI and it has 3% remaining steel, two bars are assumed to have 20% remaining steel and two bars are assumed to have 40% remaining steel.	There are six transverse moment frames and the internal four attract the most load. If this moment frame fails then the load on the bleachers as a transfer diaphragm would increase and the load attracted by the moment frames to either side would increase. The transfer diaphragm or the URM infill wall on the adjacent frame could be the final structural element to fail before collapse occurs. **A Critical Structural Weakness, CSW.**
<u>Ground Floor Block Wall Along Corridor Ref. Fig 15.</u> **At ground floor there is a block wall along the corridor.** **The wall is assumed to have all cells concrete filled, reinforced with horizontal steel 9.5mm in diameter at 400mm centres, and with vertical starter bars cast in-situ.**	The wall shear strength has been evaluated with contribution from the masonry and horizontal reinforcing steel, but zero axial load contribution because the load is assumed to go through the deep beam above, to the columns and to the column pads.	Along this bracing line, there are two long block walls and two short block walls. There are two long concrete walls and 5 short concrete walls. These have been modelled in staad.pro as columns fixed to the beams, top and bottom.	Longitudinally, the two long block walls and the two long concrete walls attract most of the north-south seismic actions along the bracing line. Failure of four long walls would increase the load on the bleachers as a transfer diaphragm and the back longitudinal URM infill wall, and either of these could be the final structural system to fail before collapse occurs.

Table 10 - Results for primary lateral force resisting structure.

	Staad.Pro	Member Type	Height	Column Number	Notes	ϕNn NBS(IL3)	ϕVn NBS(IL3)	ϕMn NBS(IL3)
Previously	583	truss	roof	col 13 to 8	bottom chord	40%		
Identified	591	truss	roof	col 13 to 8	top chord	45%		
Elements	887	brace	roof	col 13 to 8	pipe	50%		
(2014)	121	wall	G to L1	col 8 to 9	concrete		60%	
	145	wall	G to L1	col 10 to 11	block		30%	
	1370	End wall	L3 to roof	Col 13	improved		40%	20%
	471	column	L3 to roof	col 14	0		100%	70%
	1412	stairs	G to L1	col 6 to 12	0	100%		
	1776	beam	L1	col 10	lintel	20%		
Back	290	wall	L1 to L2	15	strut & tie		60%	
Wall	1667	wall	L1 to L2	16	strut & tie		50%	
	1469	column	G to L1	14	improved		60%	30%
	241	column	G to L1	15	improved		55%	30%
	1466	column	G to L1	16	improved		60%	34%
	1455	column	G to L1	17	improved		55%	30%
	1470	wall	G to L1	16 to 17	URM IP URM OOP		40%	 25%
	1810	wall	G to L1	14 to 15	URM IP URM OOP		34%	 25%
Transverse	461	Column	G	col 1	Grid A		100%	100%
Bleacher	465	beam	L1	col 1	Grid A		100%	95%
Beams	476	Column	G	col 2	Grid B		60%	90%
	479	beam	L1	col 2	Grid B		100%	100%
	897	Column	G	col 3	Grid C		35%	60%
	493	beam	L1	col 3	Grid C		45%	25%
	939	Column	G	col 4	Grid D		60%	95%
	507	beam	L1	col 4	Grid D		100%	80%
	940	Column	G	col 5	Grid E		60%	55%
	521	beam	L1	col 5	Grid E		70%	100%
	533	Column	G	col 6	Grid F		75%	100%
	536	beam	L1	col 6	Grid F		100%	100%
Transverse	1411	wall	G to L2	col 1 to 7	240 brick		100%	
URM Infill	1477	wall	G to L2	col 2 to 8	120 brick		45%	
walls	1773	wall	G to L2	col 3 to 9	140 block		55%	
	1774	wall	G to L2	col 4 to 10	120 brick		60%	
	1786	wall	G to L2	col 5 to 11	140 block		80%	
	1801	wall	G to L2	col 6 to 12	240 brick		100%	

Table 11 - Results for vertical seismic actions.

Staad.Pro	Member	Height	Column	Notes	Actions	ϕVn
Reference	**Type**		**Number**			**NBS(IL3)**
1470	wall	G to L1	16 to 17	URM**	OOP	40%
1810	wall	G to L1	14 to 15	URM**	OOP	40%
674	truss	roof	col 9	bot chord	Eu vertical	100%
668	truss	roof	col 9	top chord	Eu vertical	60%

Table 12 - Results for building parts.

Staad.Pro	Member	Height	Column	Notes	Actions	ϕVn
Reference	**Type**		**Number**			**NBS(IL3)**
end wall	mullions	L1 to roof	col 6 to 18	timber	OOP	< 33%
end wall	2 posts	L1 to roof	col 6	steel	axial	< 33%*
end wall	Mullions	L1 to roof	col 1 to 13	timber	OOP	< 33%
end wall	2 posts	L1 to roof	col 1	steel	axial	< 33%*

Table 13 - Results for vertical bearing pressures of the foundations.

Staad.Pro	Member	Height	Column	Notes	Rx	Ry	Rz
Reference	type		number		**NBS(IL3)**	**NBS(IL3)**	**NBS(IL3)**
1	foundation	Ground	col 14	pad		100%	
2	foundation	Ground	15	pad		100%	
3	foundation	Ground	16	pad		100%	
4	foundation	Ground	17	pad		100%	
8	foundation	Ground	8	pad		100%	
9	foundation	Ground	col 13	pad		90%	
11	foundation	Ground	1	pad		100%	
12	foundation	Ground	18	pad		100%	
13	foundation	Ground	6	pad		100%	
15	foundation	Ground	2	pad		100%	
16	foundation	Ground	3	pad		100%	
17	foundation	Ground	4	pad		100%	
18	foundation	Ground	5	pad		100%	

5.0 Discussion

5.1 Risk Assessment

This building has been assessed as seismic Grade D.

Table 14- The New Zealand Society for Earthquake Engineering Grading Scheme

Percentage of New Building Standard (%NBS)	Alpha Rating	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to medium risk
35-66	C	5-10 times greater	Medium risk
20-34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

The seismic grading scheme in Table 14 above, developed by the New Zealand Society for Earthquake Engineering, compares the life risk (due to earthquake damage) of this building to other buildings. Grade D buildings represent a High risk to occupants, which is estimated to be equivalent to 10-25 times the risk expected for a new building.

5.2 Seismic Assessment: A Continuum of Information Gathering, Modelling and Assessment.

The grandstand structure is complex relative to most other low rise building structures both in its form and because of its level of decay. Levels of seismic assessment can vary, as can the reliability of conclusions. This depends on the level of assessment, and the complexity of the building structure and its situation.

The limited on-site investigation by CSI for this DSA provided a snapshot of levels of material decay in a few selected locations. Inferences have been made from the results. This type of testing with is inferences would not be required if the building was known to be in good condition. Further testing may be undertaken in the future if a greater knowledge is required.

The building is an irregular structure, vertically and horizontally. This means that its structure's earthquake response is relatively unpredictable. A complex dynamic analysis has been carried out on the building model but the level of reliability of the results should be considered lower than for the results of a simple analysis of a regular building. Engineering judgement is required (i) to estimate current material properties in areas of the building that have not been tested, (ii) to identify the assumptions for the creation of the structural model, (iii) to decide on the approach to its analysis, and (iv) to interpret the results. Therefore, while the results of this DSA are reported as an "earthquake score", the inexact nature of adopting a model and making assumptions should be considered: the actual %NBS is point on a probability distribution.

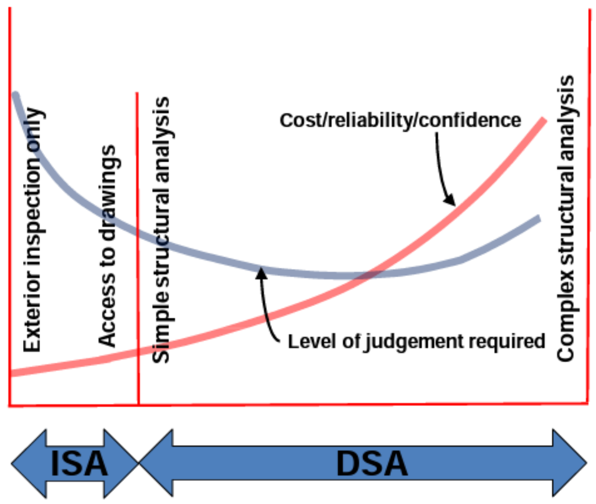

Figure 34 - The continuum of seismic assessment.

The continuum of seismic assessment curve illustrates how the level of cost, reliability and confidence may increase relative to the level of the engineering judgement required. There is a vertical line to mark the transition from Initial Seismic Assessment to a Detailed Seismic Assessment. The curve illustrates that the continuum for the ISA ranges between exterior inspection only up to access to drawings. Additionally, the curve illustrates that the continuum for a DSA, ranges between a simple structural analysis and a complex structural analysis.

Refer to the following publications for further discussion: Guidance for Territorial Authorities and Property Owners on Initial Seismic Assessments (Ministry of Business, Innovation & Employment, November 2014), and The Seismic Assessment of Existing Buildings, Part A Assessment Objectives and Principles (July 2017, Ministry of Business, Innovation and Employment).

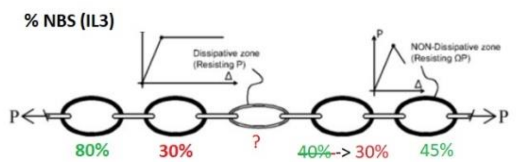

Figure 35 - The weakest link in the chain analogy.

One analogy for a building model used in a seismic assessment is a chain where the earthquake is pulling on the chain and each link has the potential to cause the chain to break. Numerous assumptions have been made and these may affect the weakest link (i.e. the modelling of the weakest part of the building). Thus the level of reliability of a complex assessment may be affected by the assumptions that are made.

5.3 Non-Structural Elements

Ceilings, partitions etc are non-structural elements and have not been specifically assessed. During an earthquake, the occupant safety can be put at risk due to non-structural items falling. These items should be adequately seismically restrained where possible, to NZS4219:2009 – The Seismic Performance of Engineering Systems in Buildings. An assessment of non-structural items is recommended for future action.

5.4 Interim Securing

A seismic assessment rating [%NBS] is used solely for building assessments. Structural improvement may be proposed as a percentage of the ultimate limit state earthquake action Eu, from the NZ earthquake loading standard NZS1170.5 (reference 2). A minimum allowable structural improvement would be 34% of Eu.

An interim securing option includes reduction of the loading and changing the Building Importance Level while improving three of the four critical structural weaknesses and the two severe structural weaknesses. As stated earlier, this building has an “Importance Level” of IL3 because more than 300 people can congregate in one space. The NZ Building Code requires a 30% increase of the design earthquake loading to be imposed on the model, compared with the design earthquake loading that is imposed on a similar IL2 building in which a crowd of fewer than 300 persons would be expected to congregate.

A course of action that council could consider is to close off parts of the viewing area so the capacity is less than 300 persons. Not only will this change the importance level of the building, but also the imposed weight to closed off areas will be reduced, as will the associated inertial seismic load.

A change from IL3 to IL2 would automatically mean 20%NBS(IL3) structural members become (at least) 26%NBS(IL2). With the removal of live loading from the grandstand, the latter figure may improve marginally. The building would be less than 34%NBS(IL2) but interim securing would likely be less onerous to reach a selected lower bound seismic ultimate capacity.

Staad.Pro	Member	Height	Column	Notes	ϕNn	ϕVn	ϕMn
Ref.	Type		Number		NBS IL3/IL2	NBS IL3/IL2	NBS IL3/IL2
145	Walls (2)	G to L1	col 10 to 11	Corridor Block		30%/39%	
1370	Walls (2)	L3 to roof	Col 13	End Walls		40%/52%	20%/26%
1776	Beam (1)	L1	col 10	Corridor Lintel	20%/26%		
1470 & 1810	Brick Wall Panels (2)	G	16 – 17 & 14 - 15	URM OOP			25%/32%
241, 1455, 1469	Columns (4)	G to L1	15, 17,14	Rear wall		55%	30%/39%
493	Raking Beam* (1)	L1	col 3	Grid C Transverse beam		45%	25%/33%*
end walls	Mullions	L1 to roof	col6 to 18	timber OOP			< 33%
end walls	Posts (4)	L1 to roof		Steel posts both sides of walkway.			< 33%

*This figure is expected to become greater than 34%NBS(IL2) in the absence of the grandstand crowd loading.

5.5 Long Term Structural Improvement Options

Long term structural improvement options include the following.

(i) Undertake the above recommendations in 5.4, with a strategy in place for the long-term protection of the materials and restricting uses so the importance level would become IL2 permanently. The future level of earthquake resistance should be selected carefully because of the range of building work involved. The building uses would be permanently restricted to changing room and clubroom facilities. The grandstand roof and crowd live loading would be removed and a new lightweight roof would be placed over the bleachers.

(ii) Retrofit the grandstand, maintaining the IL3 importance level, with a strategy in place for the long-term protection of the materials. The level of earthquake resistance should be carefully selected because that would affect how many parts of the building would require structural improvement. The work involved in retrofitting to Grade B or Grade A (i.e.67% & 80% of earthquake ultimate loading) would be significant.

Both the strength and the durability of the building structure would be addressed in the above options.

This report is looking solely at the existing building structure. The assessment of alternative opportunities to identify and meet the current and future objectives of the facility are not included in the brief. The earthquake rating of the building needs to be addressed, irrespective of its future usability. Other options for making the building safe may be identified when the Hutt City Council considers and decides upon its strategy for the building.

6.0 Conclusion

All conclusions are to be read in context of the limitations, inferences and assumptions referred to herein.

- The assessment result indicates an earthquake rating of 20%NBS(IL3) and a seismic grade of D.

- Critical Structural Weaknesses were identified as the end walls above the bleachers, an internal transverse lintel, the rear wall structure and a raking beam to a main transverse frame.

- Earthquake scores for other structural elements and systems have been provided and Severe Structural Weaknesses identified.

- Rust to reinforcing steel and structural steel, and timber decay has contributed to the results of the assessment.

- A broad strategy for interim securing for a restricted use has been identified.

- Further work is required to identify practicable long-term securing options to enable continued use as a grandstand and/or changing facilities and clubrooms.

7.0 Recommended Next Steps

We recommend;

1 The status of the building should be reconsidered in the light of this assessment.

2 Short-term and long-term options for the building should be identified and carefully considered by the Hutt City Council, in the formation of an overall strategy.

3 An assessment of non-structural elements should be done.

4 The short-term work should include:

   a. The structural improvement of the end walls above the bleachers; including removal or replacement of the glazing and timber framing, the urgent strengthening of the four steel posts at the building ends (two at each end), structural improvement of an internal transverse lintel and the rear wall structure.

   b. Maintenance at positions of spalled concrete, crumbling concrete, and exposed reinforcing steel at external concrete walls, beams and columns.

5 While remediation for continued use as a grandstand remains a possible future outcome, the following should be done:

   a. Consideration of geotechnical aspects in conjunction with strengthening scenarios is required. Further on-site investigations may be required for a structural improvement design.

   b. Further durability investigations may include ultrasonic testing of selected external structural steel members, a cover concrete survey, and half-cell potential testing of reinforcing steel. Refer to the CSI report in Appendix C for further information.

8.0 Applicability

This report contains the professional opinion of Sawrey Consulting Engineers Ltd as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill normally exercised by professional engineers providing similar services in similar circumstances. No other express or implied warranty is made as to the professional advice contained in this report.

We have prepared this report in accordance with the brief as provided and our terms of engagement. The information contained in this report has been prepared by Sawrey Consulting Engineers Limited at the request of its client, the Hutt City Council, and is exclusively for its use and reliance. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Sawrey Consulting Engineers Limited. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

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References

[1] "The Seismic Assessment of Existing Building – Technical Guidelines for Engineering Assessments – July 2017" – The New Zealand Society for Earthquake Engineering, Structural Engineering Society of New Zealand, New Zealand Geotechnical Society Inc, Ministry of Business Innovation and Employment & NZ Earthquake Commission.
This document is referred to as "the guidelines" in the this DSA report.

[2] "NZS 1170.5: 2004 Structural Design Actions – Part 5: Earthquake Actions - New Zealand" – Standards New Zealand – Ministry of Business Innovation and Employment

[3] "New Zealand Building Code Verification Method B1/ VM1" - New Zealand Government

[4] "NZS4230:2004 Design of Reinforced Concrete Masonry" – Standards New Zealand – Ministry of Business Innovation and Employment

[5] "NZS1170.5 Supp1:2004 Structural Design Actions Part 5: Earthquake Actions - New Zealand Commentary" – Standards New Zealand – Ministry of Business Innovation and Employment

[6] "NZS3101.1:2006 Concrete Structures Standard"– Standards New Zealand – Incorporating Amendment No. 3 (2017)

[7] "NZS3404:Part 1:1997 Steel Structures Standard" – Standards New Zealand – Ministry of Business Innovation and Employment - Incorporating Amendment No. 2 (2007)

[8] "Guidance for Territorial Authorities and Property Owners on Initial Seismic Assessments"– Ministry of Business, Innovation & Empowerment, Structural Engineering

[9] "ISTEG Steel for Concrete Reinforcement"– American Concrete Institute (17 June 1935)