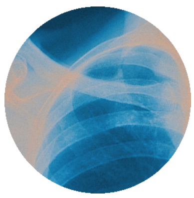
HolmesConsultingGroup

DETAILED SEISMIC ASSESSMENT

STRUCTURAL AND CIVIL ENGINEERS



DETAILED SIESMIC ASSESSMENT FOR WAIKARAKA PARK

PREPARED FOR

AUCKLAND COUNCIL

113123

NOVEMBER 2015

AUCKLAND COUNCIL

DETAILED SEISMIC ASSESSMENT OF WAIKARAKA PARK GRANDSTAND

Prepared For: AUCKLAND COUNCIL

Date: 3 November, 2015
Project No: 113123
Revision No: 2

Prepared By:	Reviewed By:
Michael O'Leary	Chris Mackenzie
DESIGN ENGINEER	BUSINESS DEVELOPMENT/GOVERNANCE

Holmes Consulting Group LP
Auckland Office

CONTENTS

LIST OF TABLES

LIST OF FIGURES

Holmes Consulting Group LP have been commissioned to evaluate the likely seismic performance of the existing Waikaraka Park grandstand. A quantitative assessment has been undertaken to determine the overall extent to which the building complies with the current building standards in relation to its seismic performance.

The grandstand located at Waikaraka Park, Onehunga is located adjacent the club rooms which is accessible via a bridge at level 2 of the grandstand. The grandstand has two suspended levels and is approximately 67.0m long by 10.5m deep and 11.6m high.

Existing buildings are generally assessed against the Design Basis Earthquake (DBE), also called the New Building Standard (NBS), which is the earthquake hazard level for which an equivalent new building constructed on the same site must be designed for under the current building standards. For the grandstand at Waikaraka Park the DBE is based on an earthquake with a 1000 year return period. This corresponds to the current Importance Level 3, (IL3) use of the building as a grandstand.

Two performance limit states are considered for seismic design of new buildings. These are the Ultimate Limit State (ULS), and Collapse Limit State (CLS) which is based on a seismic event much larger than ULS. A new building designed in accordance to the current Building Act has the following minimum performance objectives;

- At ULS, significant damage to the structure occurs, but some margin against either partial or total collapse remains.
- At CLS, substantial damage to the structure occurs, and the building is on the verge of partial or total collapse.

To determine the extent to which the building complies with new building standards a series of three dimensional linear elastic dynamic analyses were carried out using Microstran and ETABS, both of which are commercially available analysis software programmes. Based on a series of analyses at various levels of seismic loading, it was determined that the building has a capacity equivalent to approximately 10-20% DBE at ULS and is considered to be “potentially earthquake prone” under the Building Act classification for the assessment of buildings subject to seismic actions.

In accordance with Building Act 2004 a building is deemed to be earthquake prone if its ultimate capacity (strength) would be exceeded in a “moderate earthquake” and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined as one that would generate loads one-third as strong as those used to design an equivalent new building, or 33%DBE.

For buildings defined as earthquake prone the Territorial Authority may issue notice under Section 124 of the Building Act and restrict occupancy, or require the building owner to strengthen or demolish those elements which are considered earthquake prone.

The overall building strength for the existing Waikaraka Park Grandstand is primarily governed by the ability of the structure to resist the seismic actions from the upper tier. In the east-west direction two walls exist which act to resist the seismic response of the upper tier and also act to resist torsion in the structure. One of the full height walls is supported by a column below the lower tier which has been identified as a key critical structural weakness that limits the structures overall ability to resist seismic loads. However, strengthening of this wall alone may not be sufficient alone to raise the %DBE rating to above 33%. There are also the performance of various other elements, as discussed in the body of this report, that would likely limit the %DBE to below 33%.

The Waikaraka Park grandstand resilience measured as the buildings ability to resist CLS loads based upon realistic strengths as opposed to probably strengths which are used to assess ULS is also expected to be low. New buildings are expected to have a resilience in the order of 1.5.

Four local structure vulnerabilities were identified as part of this assessment.

1. Stair detailing for drift not present
2. Inadequate confinement of columns to accommodate CLS drift
3. The ability of the tiered seating slab to act as a diaphragm
4. The durability and condition of the existing concrete

The first two vulnerabilities noted above exist due to the high level of expected drift. As opposed to strengthening these elements for the level of expected drift it is recommended that the seismic load paths and torsional stiffness of the existing structure are improved to reduce the drift demand on these elements.

There are a several strengthening options for the building owner to consider as discussed within the body of this report. To achieve a %DBE rating between 33-67% the building owner would need to consider a combination of these strengthening schemes. There is also the option for partial or complete demolition of the structure which may be the preferred method, depending on the long-term vision for Waikaraka Park.

Along with strengthening of the structure other element such as the balustrades, gravity support to the stairs and durability need to be addressed. Therefore if a seismic retrofit is the preferred option, strengthening of the balustrades and stair supports along with addressing other defects affecting the durability of the structure would need to be included as part of the work. For further information with regards to the durability and temporary remedial work refer to the previous Holmes Consulting Group LP letter regarding the condition of the structure and immediate public safety issues, dated December 2014.

The seismic evaluation has been restricted to the structural performance of the grandstand at Waikaraka Park. Continued operability of a building after an earthquake is not assured in the absence of structure damage as damage to components, services and content may impair functionality. The seismic resistance of these items has not been assessed.

1. INTRODUCTION

1.1 INTRODUCTION

Following an initial assessment for the condition of the Waikaraka Park Grandstand, Holmes Consulting Group LP were commissioned by Auckland Council to complete a detailed seismic assessment of the primary structure. This detailed seismic assessment includes a linear elastic computer analysis of the Waikaraka Park Grandstand structure, accompanied with hand calculations where appropriate to determine the performance of the primary structure relative to a Design Basis Earthquake (DBE).

1.2 SCOPE OF WORK

The evaluation is restricted to a detailed assessment of the lateral load resisting system and does not explicitly consider the gravity load capacity of the floors nor the performance of non-structural components and contents.

Stages involved in completing this scope of work are:

1. Conduct a qualitative site survey to gather relevant information with regards to geometry of the structure, characteristics of structural members and to identify any alterations made that are not reflected in available existing documentation.

2. Develop a computer model for the building with use of available existing documentation and qualitative site survey information. Models have been developed both in Microstran and ETABS as linear elastic structures using an equivalent static analysis procedures for the review.

3. Evaluate the seismic response of the models in terms of the requirements of NZS1170 (loadings standard), NZS3101 (concrete materials standard for detailing) and NZS3404 (steel materials standard for detailing).

4. Determine the existing seismic performance of the structures as a percentage of the current loading requirements (%DBE).

5. Prepare a detailed seismic analysis report. This report summarises the technical aspects of the assessment and any assumptions made. The report also includes modelling parameters, results and discussion of any vulnerabilities identified.

6. Identify primary structural systems which may require strengthening to improve the seismic performance of the global structure.

1.3 INFORMATION USED FOR THE EVALUATION

The following information was used for the analysis:

- A set of original structural drawings sourced from work completed in 1984 and 1995
- A low-resolution copy of some of the original structural drawings from the 1940's
- A geotechnical report by Soil Engineering Ltd. for a nearby site (dated July 2001).

Where information was not available, assumptions were made based on typical configurations of buildings of this vintage.

Some of the key missing information and assumptions made include the following;

Missing information includes, but is not limited to:

- Actual material strengths for concrete, reinforcing and structural steel.
- Exact detailing of reinforced concrete beams, reinforced concrete columns, reinforced concrete shear walls, reinforced concrete diaphragms and structural steel column connections.
- Diaphragm shear wall reinforcing starters and reinforced concrete frame reinforcing starters.
- Allowable bearing capacity of soil.

Assumptions made include, but are not limited to:

- Material strengths were assumed based on era of construction
- Nominal reinforcing which may be reasonably expected based on era of construction and member sizes have been assumed.
- Based on the geometry and seismic resisting systems it has been assumed that the diaphragm detailing is sufficient to distribute seismic loads to the seismic resisting elements.
- A safe ultimate bearing pressure has been assumed based on existing information for adjacent structures.

1.4 THE BUILDING

Located at Waikaraka Park the grandstand is to the west of the racing circuit. The grandstand has two levels of bleachers and public facilities beneath the lower tier.

Figure 1-1 Waikaraka Park Grandstand (Google Maps)

The structure is approximately 67.0m long by 10.5m deep and 11.6m high measured from the ground floor slab to the height of the concrete/masonry wall forming part of the rear balustrade structure above the top tier. There is a construction/separation joint central to the longitudinal direction which effectively splits the grandstand into two separate structures. The following figure illustrates the location at which the grandstand is separated. The building is therefore treated as two separate structures, the north grandstand and south grandstand. The structure is located on flat ground.

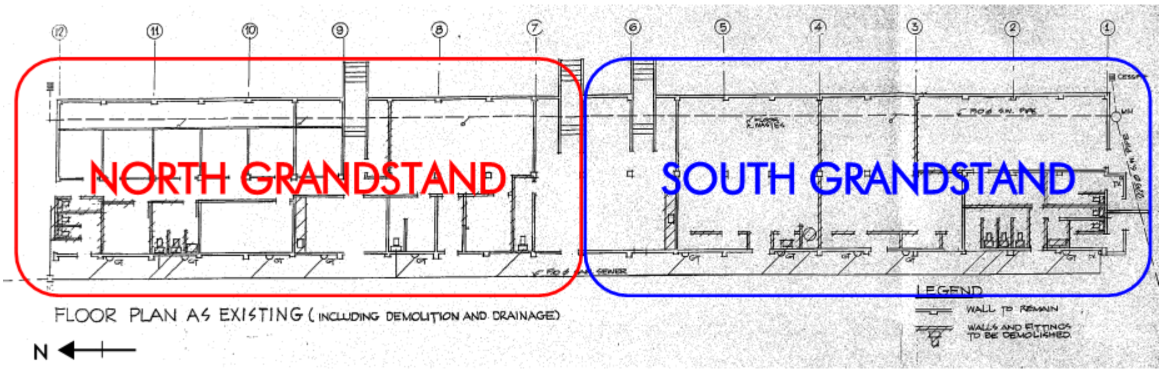


Figure 1-2 Plan of North & South Grandstand structures

1.5 STRUCTURAL DESCRIPTION

1.5.1 Foundation

Foundations consist of reinforced concrete footings bearing on soil/rock below. Geotechnical information is contained in a previous report for a neighbouring site, titled "Geotechnical Investigation for Proposed Onehunga Sports Club at Waikaraka Park, Onehunga" by Soil Engineering Ltd., dated 20th July 2001. Based on this report the site subsoil would likely be considered class D in accordance with Section 3 of NZS1170.5:2004.

The geotechnical investigation has taken samples from locations adjacent the site which relates to the area occupied by the Sports Club building. Generally, the site is underlain by a thin layer of fill (<0.3m) with natural volcanic soils between 0.3-2.6m of varying strength and alluvial material comprising of stiff grey clayey silts to a maximum depth of 4.8m. The depth to the basalt rock surface was highly variable, ranging from less than 1m to over 4m in depth below the ground surface.

1.5.2 Basement Floor Slab

The basement floor appears to be a reinforced concrete slab on grade supported directly by subgrade materials.

1.5.3 Suspended Floors

The suspended floors, or bleachers, appear to be 100mm reinforced cast insitu concrete slabs formed to the tiered profile of the bleachers.

1.5.4 Frames

The primary seismic resisting system in the transverse (East-West) direction is a combination of reinforced concrete walls and reinforced concrete frames. The frames also form part of the primary gravity support structure. The frames have been assessed as elastic due to the limited information with regards to the detailing and based on assumptions given the era of construction.

1.5.5 Structural Walls

The primary seismic resisting system in the longitudinal (North-South) direction is reinforced concrete shear walls. Shear walls also comprise part of the primary seismic resisting system in the transverse (East-West) direction. These reinforced concrete shear walls span between primary concrete columns and are present throughout the height of the building. Shear walls have been assessed as elastic in both shear and flexure in accordance with the appropriate material standards.

1.6 LIMITATIONS

Findings presented as a part of this report are for the sole use of Auckland Council. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Our observations have been visual only and have been restricted to structural aspects only. Intrusive works have not been carried out to ascertain structural details not available from drawings or visual observations.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of non-structural components or building contents (including the stairs) under earthquake loads.

1.7 STATUTORY REQUIREMENTS

In the consideration of existing buildings, the relevant sections of the Building Act 2004 are:

- Section 112: *Alterations to existing buildings.* Essentially, this section of the Act requires that a building will comply no less after the alteration than before. This precludes weakening, or the addition of significant mass without a proportional increase in strength. However, no structure need be stronger than required by code, so a structure with excess reserves of strength may be weakened.

- Section 114: *Change of Use of buildings, etc.* This requires that when a building's use is changed, it shall comply with the structural requirements of the code, "as nearly as is practically possible to the same extent as if it were a new building".

- Section 122: *Meaning of Earthquake Prone Building.* Section 122 of the Building Act 2004 deems a building to be earthquake prone if its ultimate capacity (strength) would be exceeded in a "moderate earthquake" and it would be likely to collapse causing injury or death, or damage to other property. The associated Building Regulations 2005 define a moderate earthquake as one that would generate loads one-third as strong as those used to design an equivalent new building.

- Section 124: *Powers of Territorial Authorities (TA's) in relation to exercise of powers.* This allows the TA's to take into account a number of criteria in considering the extent to which they may enforce their powers under the Building Act. Included are factors such as use, numbers of occupants, heritage value, etc.

- Section 131: *Earthquake Prone Building Policy.* This section of the Building Act requires all Territorial Authorities to adopt a specific policy on dangerous, earthquake prone, and unsanitary buildings.

2. EVALUATION PROCEDURE

2.1 MICROSTRAN ANALYSIS

The seismic evaluation utilises a computer model of the building to simulate the effects of horizontal earthquake forces. This model is developed using information from the original structural drawings of the building and likely properties of materials used at the time of construction. It also incorporates known major modifications to the building since the time of construction which may have a significant effect on building structural response. The building model includes the main structural members and in particular those which make up the primary lateral force resisting system. As a minimum this includes the beams, columns, walls, braces and floor plates of the building.

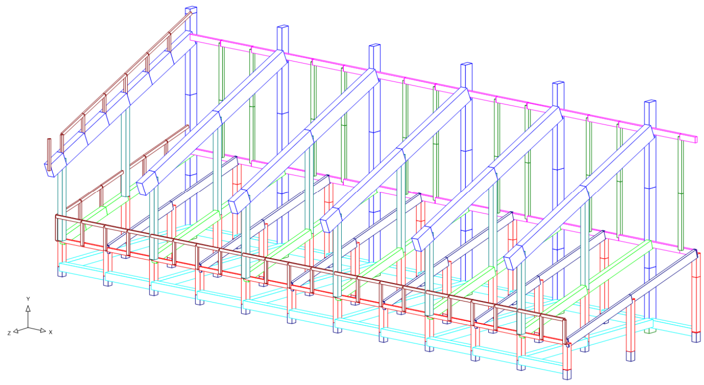


Figure 2-1 Perspective View of Microstran Analytical Model

Microstran is an industry standard structural analysis package for linear elastic equivalent static analysis. The simulated equivalent static loads are scaled to meet the minimum requirements of the chosen building standard. For this project the forces are scaled in accordance with the New Zealand loadings standard, NZS1170.5:2004.

One of the objectives of this assessment is to determine the extent to which the building complies with the current building standard (reported as a percentage of the Design Basis Earthquake - % DBE). As such, various scale factors are iterated through to arrive at the approximate seismic strength relative to the above-mentioned standard.

For each load case investigated, equivalent static forces are applied to individual elements to represent a horizontal acceleration which results from a seismic excitation applied at the base of the building. The amplitude of the acceleration is dictated by the fundamental period of vibration of the building along its two principal axes. Tall flexible structures have long periods, whilst short stiff buildings have short periods. As can be seen in Figure 2-2 following, the longer the period of a structure is, the lower the corresponding horizontal acceleration. For a building period of less than 0.4 seconds the corresponding horizontal ground acceleration in accordance with NZS1170:2002 is 0.51g, compared to a building period of 0.8 seconds where the horizontal acceleration is approximately 0.39g.

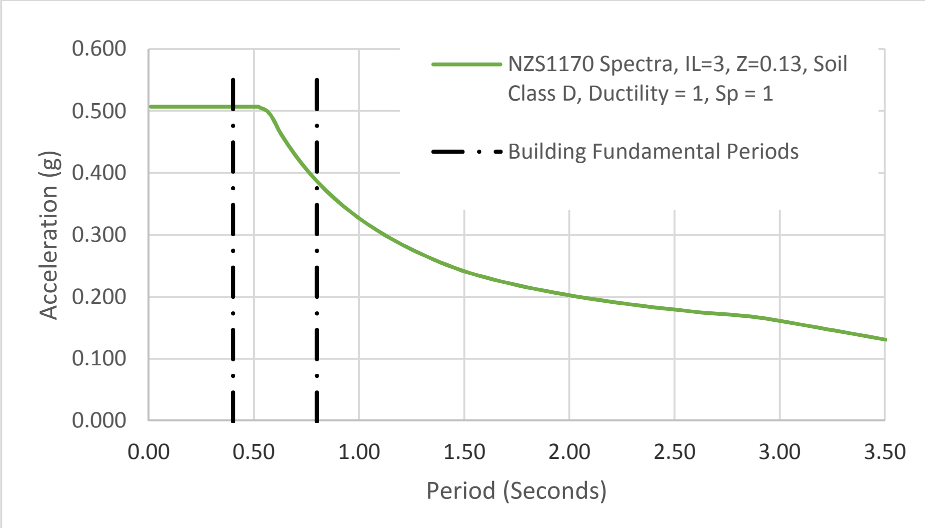


Figure 2-2 Example Seismic Response Spectra

2.2 ETABS ANALYSIS

ETABS is an industry standard structural analysis package for linear elastic dynamic modal response spectrum analysis. The simulated dynamic loads are scaled to meet the minimum requirements of the chosen building standard. For this project the forces are scaled in accordance with the New Zealand loadings standard, NZS1170.5:2004 and the relevant material standards NZS 3101:2006; Concrete Structures Standard and NZS3404:1997 Steel Structures Standard.

Due to the torsional nature of the building ETABS was used in additional to the Microstran analysis to ensure accuracy of results. Structures are generally designed to ensure fundamental modes exists in the two orthogonal directions for the structure. Structures may then be designed for equivalent static forces which relate to the stiffness in either orthogonal direction. In the case of this structure and the idealised Microstran model the fundamental modes are torsional and therefore the stiffness in the two orthogonal directions may be an inaccurate means for determining equivalent static loads for the actual response of the building. The ETABS model also incorporates the wall at Grid 6 which has the effect of increasing the torsional stiffness.

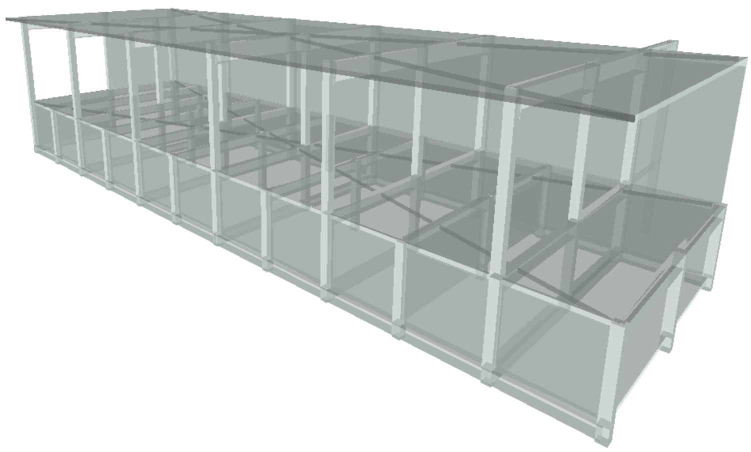


Figure 2-3 Perspective View of ETABS Analytical Model

The ETABS model includes the expected stiffness of the structure and from this the dynamic response resulting from the imposed accelerations can be determined. Since most structures for which this type of analysis is undertaken have a number of periods or modes, the output is made up of a combination of these. Statistical methods are used to combine these results to determine the overall building response.

Typical analysis output will include earthquake induced member forces (in beams, columns, walls and braces), as well as floor-to-floor and overall lateral displacements (drifts). These member forces are subsequently checked against expected strength and detailing requirements, while building drift is a good indicator of potential damage at the given level of load.

2.3 GLOBAL PERFORMANCE CRITERIA

The performance criteria used to evaluate the building in this assessment are a combination of displacement or building drift criteria stipulated by the loadings standard, and building detailing and strength criteria stipulated by the relevant materials standard. In this case the building is predominantly constructed of reinforced concrete and structural steel such that NZS 3101:2006, Concrete Structures Standard and NZS3404:1997, Steel Structures Standard are the relevant material code of reference.

The New Zealand Society for Earthquake Engineering (NZSEE) recommendations suggest that any building with a capacity below 67%DBE be regarded as an '*earthquake risk*' building and that every effort should be made to improve the structural performance of these buildings to 100%DBE where possible, or at least 67%DBE. Table 2-1 taken from the NZSEE recommendations indicate the relative seismic risk of a building based on the assessed %DBE.

%DBE	Relative risk	Description
>100	<1 time	Low risk
80-100	1-2 times	Low risk
67-80	2-5 times	Low risk
33-67	5-10 times	Moderate risk
20-33	10-25 times	High risk
<20	>25 times	High risk

Table 2-1 NZSEE Recommendations Relative Risk

The NZSEE has recommended the following classifications in the assessment of buildings subject to seismic actions

Grade	A+ Excellent	A Good	B Good	C Potential Earthquake Risk	D Potential Earthquake Prone	E Potential Earthquake Prone
%NBS	>100	80-100	67-79	34-66	20-34	<20

Table 2-2 NZSEE Building Classifications

The Auckland Council's guidelines for strengthening of earthquake prone buildings require building owners to strengthen to a minimum of 34% NBS.

2.3.1 Building Drift

Inter-storey drift is the relative horizontal displacement of vertically adjacent floors in a building. Drift is a good indicator of structural performance as damage to both structural and non-structural building components can be related to drift. The maximum inter-storey drift allowed by NZS1170 under ULS earthquake loads is 2.5% of the storey height being considered.

2.3.2 Structural Ductility

Ductility is a measure of a building or its individual components ability to undergo sustained inelastic displacements whilst maintaining sufficient residual strength to carry load. The term inelastic refers to actions beyond the base yield strength of the building or component being considered. The more ductile a building, the more energy it is able to dissipate. Since ductility inherently requires building structural components to be stressed beyond yield there will be some permanent damage associated with this form of energy dissipation.

By considering available building ductility, the magnitude of the seismic forces for which the building is being assessed are able to be reduced to capture the effect of the energy dissipation. Structural ductility is highly dependent on the type of building and the individual member detailing. Highly ductile concrete members for example need to be well confined with closely spaced reinforcing ties in order to maintain their residual strength as they hinge or become damaged.

Member detailing for ductility is a relatively modern concept. As such many older structures have little to no inherent ductility and are therefore considered elastic or nominally ductile and will not be expected to perform as well under higher levels of load. Certain materials also exhibit brittle or non-ductile behaviour such as unreinforced masonry. For analysis of this structure, no ductility has been allowed for due to the age, condition, materials and detailing used/assumed.

2.3.3 Component Strength

The individual members which make up the earthquake load resisting system are assessed for strength and detailing requirements. Component strengths are determined in accordance with the current edition of the applicable materials standard for the component being evaluated.

2.3.4 Material Properties

The properties of the building were based on the dimensions and materials noted in the structural drawings. Historic material strengths have been sourced from original construction drawings where possible, however information is very limited. As such, material strengths have been based on typical construction practises of the era.

3.1 BUILDING CONFIGURATION

The building lateral load system is reinforced concrete shear walls in north-south direction and reinforced concrete frames and concrete shear walls in the east-west direction. The typical flooring consist of cast insitu 100mm thick concrete reinforced slab supported by concrete beams. In order to accurately predict the building response the computer model includes all of the above mentioned structural elements modelled in varying accuracy as stipulated by the limitations of the analysis software.

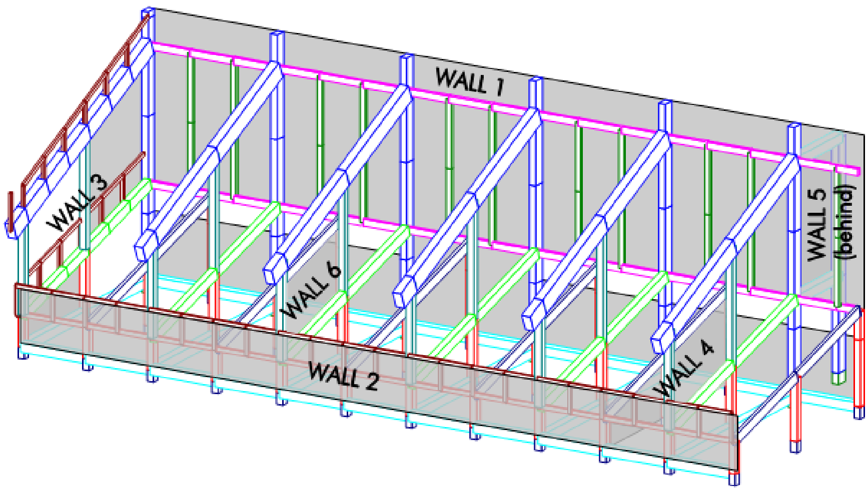


Figure 3-1 Perspective View of Microstran Analytical Model with indication of wall locations

3.2 COMPUTER MODELS

Two programmes were used for the analysis of the structure. Each programme has limitations. The Microstran model more accurately models the frames in terms of stiffness but provides a poor estimate for the overall building stiffness and response. The ETABS model provides a more accurate overall building stiffness and response but is limited in that the height of each floor cannot be modelled with a slope and therefore does not provide the correct flexural demands.

The following information contained in Section 3 refers to the ETABS model.

Whilst ETABS commercial software provides the analysis engine, the linear elastic analysis models are built using a Holmes Consulting Group LP proprietary input excel spreadsheet called DuctileIN. This allows the rapid assembly of the basic building geometry and for the element properties (section sizes, stiffness and material types) to be defined.

3.3 GEOMETRY

The model geometry is described by a series of column numbers to identify the plan location of structural members while elevations are defined to model elements in the vertical plane. Each node represents the geometric location of a beam, column, wall, brace or floor element.

A total of three levels for the building were defined in the vertical direction, as listed in Table 3-1 below. Level elevations are based on dimensions from the original drawings. The basement floor has been included to allow for modelling of foundations to better model the axial stiffness of the supporting shear walls.

Level Identification (Top Down)	Number of Levels	Storey Height (m)	Floor Area (m^2)	Seismic Weight (KPa)	Seismic Mass (t)	Seismic MMI ($t\text{-}m^2$)
ALL LEVELS	3	8.355				
L02	1	5.105	376	7.38	283	29464
L01	2	2.750	342	6.83	238	24402
B01	3	0.500	342	0.00	0	0

Table 3-1 ETABS Elevations and Floor Mass

*the above table refers to the ETABS model in which the slope of the floors cannot be modelled and therefore interstorey heights reflect an average height of the storey.

3.4 SECTION PROPERTIES

Properties were determined from a near complete set of original structural drawings, however information is limited and therefore assumptions were made based on the era of construction.

- The minimum concrete compressive strength was assumed at 25MPa with a probable concrete strength of 30MPa.
- The probable yield strength of reinforcing steel was assumed to be 300MPa.
- Because this analysis is being undertaken to assess the capacity of the existing building probable strengths have been used for analysis as opposed to characteristic strengths which is typical for the design of new buildings.

For the columns and beams used to assemble the model, individual member area, second moment of inertia, section modulus and other geometric properties are explicitly calculated. The specific column, beam and wall elements are defined in Table 3-2, Table 3-3 and Table 3-4 respectively.

		Property	Section Name (ETABS)	Library	Shape Code	Material ID	Major Dimension	Minor Dimension
L01	530x305 COL	1	RECT		RECT	1	0.530	0.305
	305x305 COL	2	RECT		RECT	1	0.305	0.305
	305x305 COL	3	RECT		RECT	1	0.305	0.305
	305x305 COL	4	RECT		RECT	1	0.305	0.305
L02	305x530 COL	5	RECT		RECT	1	0.530	0.305
	300x300 COL	6	RECT		RECT	1	0.300	0.300
	300x300 COL	7	RECT		RECT	1	0.300	0.300
B01	305x305 COL	8	RECT		RECT	1	0.305	0.305
L01	250x320 COL	9	RECT		RECT	1	0.250	0.320

Table 3-2 Column Properties

	Property	Section Name (ETABS)	Library	Shape Code	Material ID	Depth Below	Depth Above	Width
760x460 BEAM	1	RECT		RECT	1	0.760		0.460
460x305 BEAM	2	RECT		RECT	1	0.460		0.305
460x150 BEAM	3	RECT		RECT	1	0.460		0.150
305x150 BEAM	4	RECT		RECT	1	0.305		0.150
200x100 BEAM	5	RECT		RECT	1	0.200		0.100
400x400 FOUND	6	RECT		RECT	1	0.400		0.400

Table 3-3 Beam Properties

Property	Type	Material ID	Wall Thickness
1	MEMB	1	0.090
2	MEMB	1	0.300

Table 3-4 Wall Properties

3.5 MASSES AND WEIGHTS

The seismic weight of the buildings was assembled from element self-weight (explicitly defined beams, columns and walls) plus distributed floor weights. The floor seismic weight for each level is based on the average self-weight of the floor systems, plus an allowance for secondary structural elements including infill masonry walls and secondary steel beams, plus a superimposed dead load of 0.25kPa, plus an average seismic live load over the total floor area. This provides a total floor superimposed seismic weight for each floor. The seismic floor weights for each level are included in Table 3-5 below.

User Loads			EQ Wt
Basement	G	2.50	3.90
	Q	3.00	
	SDL	0.50	
Ground	G	3.17	6.42
	Q	5.00	
	SDL	0.25	
First Floor	G	3.58	6.83
	Q	5.00	
	SDL	0.25	

Table 3-5 Seismic Weight for each Floor

Gravity load case weights are calculated separately and evaluated in combination with the earthquake induced actions in the output spreadsheet DuctileOUT.

3.6 PERFORMANCE OUTPUT

The results of the ETABS analysis are processed by the Holmes Consulting Group LP proprietary post processing Excel spreadsheet, DuctileOUT. The results can then be displayed visually to show demand/capacity ratios which reflect the ability of individual elements to resist actions equivalent to 100%DBE loading.

Table 3-6 illustrates the colour code used in the 3D output to represent the level of utilisation of the capacity of individual elements.

D/C ratio		
>1.10		
1.10-1.00		i.e. capacity is exceeded
1.00-0.95		
0.95-0.9		
0.90-0.8		
0.80-0.7		
0.70-0.5		
0.50-0		i.e. well within expected capacity

Table 3-6 Demand/Capacity Ratio Colour Code on 3D Output

Refer to Figure 5-4 and Figure 5-5 for the 3D outputs from DuctileOUT.

4.1 NEW BUILDING IMPORTANCE LEVELS

The magnitude of seismic design loads in the current loadings standard NZS1170 is a function of the type of structure, as listed in Table 4-1 below. The Waikaraka Park Grandstand has been evaluated as an Importance Level 3 structure, which is appropriate for this type of structure as it may contain crowds of greater than 300 people.

Importance Level	Earthquake Annual Exceedance Probability	Risk of Exceedance in 50 Year Design Life	Risk Factor	Comment	Examples
1 (IL1)	1/100	40%	0.5	Structures representing a low degree of hazard to life and property.	Small structures, farm buildings, fences, masts, walls
2 (IL2)	1/500	10%	1.0	"Normal" structures and structures not in other importance levels.	Hotels, offices, apartments
3 **(IL3)**	**1/1000**	**5%**	**1.3**	**Structures that may contain people in crowds or contents of high value to the community.**	**Schools, emergency medical and other emergency facilities but not essential post-disaster healthcare facilities.**
4 (IL4)	1/2500	2%	1.8	Structures with special post-disaster functions.	Designated civilian emergency facilities, medical emergency facilities with post disaster functions.

Table 4-1 Building Performance Levels

4.2 EXISTING BUILDING PERFORMANCE OBJECTIVES

Existing buildings are generally assessed against the Design Basis Earthquake (DBE), also called the New Building Standard (NBS), which is the earthquake hazard level for which an equivalent new building constructed on this site must be designed for under the current building standards. For this building the %DBE is based on an earthquake with a 1000 year return period (Importance Level IL3) as highlighted in Table 4-1 above. This corresponds to the current use for the building, where crowds of greater than 300 may congregate.

Two performance limit states are considered for seismic design of new buildings. These are the Ultimate Limit State (ULS), and the Collapse Limit State (CLS) which is based on a seismic event much larger than ULS. A new building designed in accordance to the current Building Act has the following minimum performance objectives;

- At ULS, significant damage to the structure occurs, but some margin against either partial or total collapse remains.
- At CLS, substantial damage to the structure occurs, and the building is on the verge of partial or total collapse.

New buildings are generally designed to resist ULS loads and there is no specific requirement to check CLS. New buildings are however required to satisfy stringent detailing provisions which provide the buildings with resilience, giving a sufficiently low probability of collapse under a seismic event in excess of 150% DBE at CLS. The CLS performance of new buildings is therefore implicit in the current building standards.

The earthquakes in Christchurch have emphasized the need to ensure that all buildings have a sufficient margin against partial or total collapse beyond ULS (often termed resilience). The Engineering Advisory Group (EAG) which was formulated after the February 2011 earthquakes has recommended that CLS be specifically checked when reviewing existing buildings, especially when existing buildings do not satisfy the stringent detailing provisions of the current building standards.

Table 4-2 below shows the recommended earthquake hazard levels for both new and existing buildings (as a percentage of the DBE). The earthquake hazard levels at the ULS are in accordance with the New Zealand Society for Earthquake Engineering recommendations (NZSEE, 2006). The earthquake hazard levels at the CLS have been determined by providing a margin of 1.5 above that used for the ULS. This margin has been adopted on the basis that it is at the lower end of the range that would be expected for a new building designed in accordance with AS/NZS1170.

The Building Act also defines a building as earthquake prone (EPB) if it will have its ultimate capacity exceeded in a moderate earthquake (defined as a level of earthquake one-third that required for an equivalent new building); and would be likely to collapse.

Building Performance Level	Earthquake Hazard Level	
	Performance Limit State	
	Ultimate Limit State (ULS)	Collapse Limit State (CLS)
New Building - Minimum Legal Standard	100% DBE	150%+ DBE
Existing Building - Recommended Target	67% DBE	100% DBE
Existing Building – NZSEE Interpretation of EPB	34% DBE	50% DBE
Existing Building - Legal Minimum	-	34% DBE

Table 4-2 Existing Building Performance Objectives

4.3 SEISMIC LOADS

The DBE seismic loads are based on the requirements of NZS1170.5:2004. For modal response spectrum analysis, the standard specifies a minimum base shear to be applied. The base shear coefficient is a function of building period, structure ductility and the site geology, including proximity to known fault lines. The assumed seismic parameters for the Grandstand structure at Waikaraka Park are as listed in Table 4-3.

Design Code :	AS/NZS1170.5:2004
Importance Level:	3
Soil Category :	D
R :	1.3
Z :	0.13
μ :	1.0
S_p :	1.0
D :	>= 50 km
t :	< 0.4 s

Table 4-3 Seismic Parameters

The soil category has been determined on the basis of geotechnical information contained in the Soil Engineering Ltd geotechnical report for a neighbouring site titled "Geotechnical Investigation for Proposed Onehunga Sport Club at Waikaraka Park, Onehunga" dated 2001 and in accordance with Section 3 of NZS1170.5:2004.

5. SEISMIC RESPONSE OF EXISTING BUILDING

5.1 DYNAMIC CHARACTERISTICS

A dynamic analysis using Microstran was undertaken to ensure the building has been modelled accurately. Generally, it is expected that the first three fundamental modes with the greatest mass participation correlate to a mode in either principal axis and a torsional mode. The torsional mode is generally expected to have the lowest mass participation for the lowest three modes, although in highly torsional buildings this may not be the case.

Due to the limitations of Microstran the diaphragms and walls have been idealised as rigid elements. Microstran does not have the capabilities of modelling a plate element. However, this idealisation is considered appropriate for an equivalent static analysis and is expected to have minimal impact on the accuracy of the results. This does however suppress the fundamental modes in the principal axes and therefore only the torsional mode has been computed. The torsional mode from Microstran has a period of 0.79s.

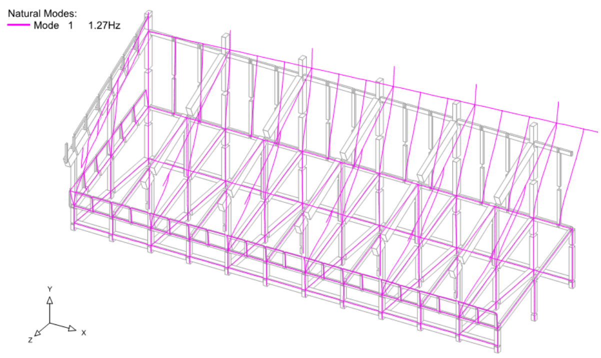


Figure 5-1 Displacement profile for torsional mode from Microstran analysis

It is generally expected that buildings with short shear walls have fundamental periods in the x and z direction <0.4s, however because the structure is highly torsional the first fundamental mode is a torsional mode with a period of 0.79s.

Further analysis was undertaken in ETABS to better determine the likely torsional response for the structure. Six modes of similar mass participation was calculated by the ETABS analysis. These 6 modes correspond to two modes in the orthogonal directions and one torsional mode for each bleacher level. The ETABS model includes the wall supporting the stair at Grid 6, which has the effect of increasing the stiffness of the structure thereby reducing the period of each fundamental mode. The first three modes are illustrated below.

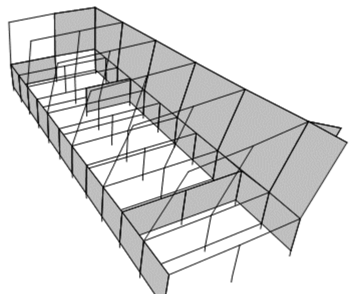
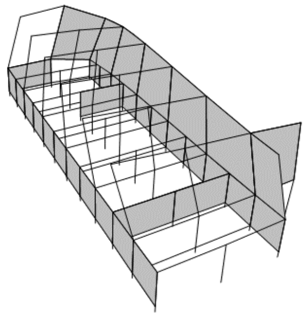
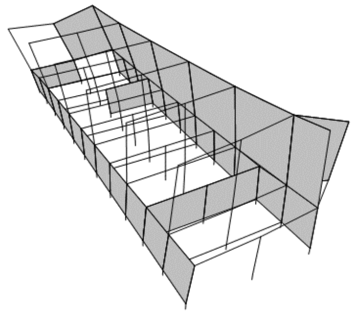
					
Mode 1		Mode 2		Mode 3	
Period	0.20s	Period	0.11s	Period	0.06s
Eff. Mass X	0%	Eff. Mass X	36.4%	Eff. Mass X	27.2%
Eff. Mass Z	39.8%	Eff. Mass Z	4.1%	Eff. Mass Z	9.5%
Eff. Mass R	11.3%	Eff. Mass R	13.4%	Eff. Mass R	33.1%

Table 5-1 First three fundamental modes from ETABS Model 1 Analysis

Because the lateral stiffness between level 1 and level 2 is greater than that assumed in the mircrostran model the period for the torsional mode is lower. However, because the column supporting Wall 5 has been identified as a critical structural weakness and will likely fail at a low level seimsic event, a second ETABS model was created to determine the response of the structure without support of this wall. The following figure illustrates the difference between the two ETABS models.

Model 1: Wall 5 @ G6 supported by Col *Model 2: Wall 5 @ G6 not supported by Col*

Figure 5-2 Difference between the 2 ETABS model

The following table are the results from the second ETABS model:

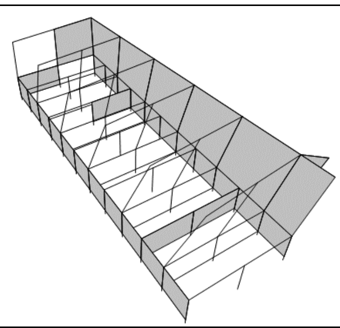
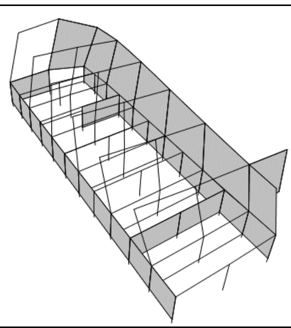
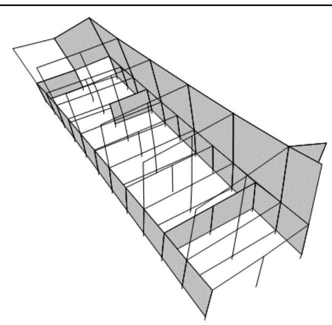
					
Mode 1		Mode 2		Mode 3	
Period	0.69s	Period	0.11s	Period	0.07s
Eff. Mass X	0%	Eff. Mass X	36.2%	Eff. Mass X	27.5%
Eff. Mass Z	32.8%	Eff. Mass Z	7.1%	Eff. Mass Z	10.8%
Eff. Mass R	12.6%	Eff. Mass R	11.3%	Eff. Mass R	31.4%

Table 5-2 First three fundamental modes from ETABS Model 2 Analysis

The results from the 2^{nd} model would tend to suggest that for a seismic event in the Z direction the modal base shear should be scaled to the base shear calculated from an equivalent static analysis with a period of 0.69s. However, upon further investigation it has been deemed more appropriate to derive the wall shear forces based on an equivalent static force for the fundamental periods in the x and z direction of less than 0.4s as per the 1^{st} model. The forces in the frames should also be based upon the 2^{nd} model with a fundamental period in the x and z direction of 0.4s.

The reason for the difference in results between the 1^{st} and 2^{nd} model is the change in seismic load path. The torisonal response from the upper bleachers is resisted by two stiff walls in the 1^{st} model and one stiff wall and a series of flexible frames in the 2^{nd} model. Torsion at the lower bleacher level in both models is resisted by stiff walls around the perimeter of the structure. Therefore the 2^{nd} model has a very low torsional stiffness for the structure supporting the upper bleachers and a very high torsional stiffness for the structure supporting the lower bleachers.

5.2 GLOBAL BUILDING PERFORMANCE

The global building performance and %DBE will be based predominately upon the first ETABS model. However the performance of individual elements, particularly the second level moment resisting frames will be based upon the 2^{nd} ETABS model as the failure of the column supporting Wall 5 is expected and therefore the design forces derived from the 2^{nd} model need to be considered. In the case of the moment resisting frames the 2^{nd} model will result in the most adverse response. The below figure is a representation of the lateral resisting elements for the south grandstand. Beams which form part of the seismic resisting frames as illustrated in green, whilst the yellow lines represent the diaphragm connectivity.

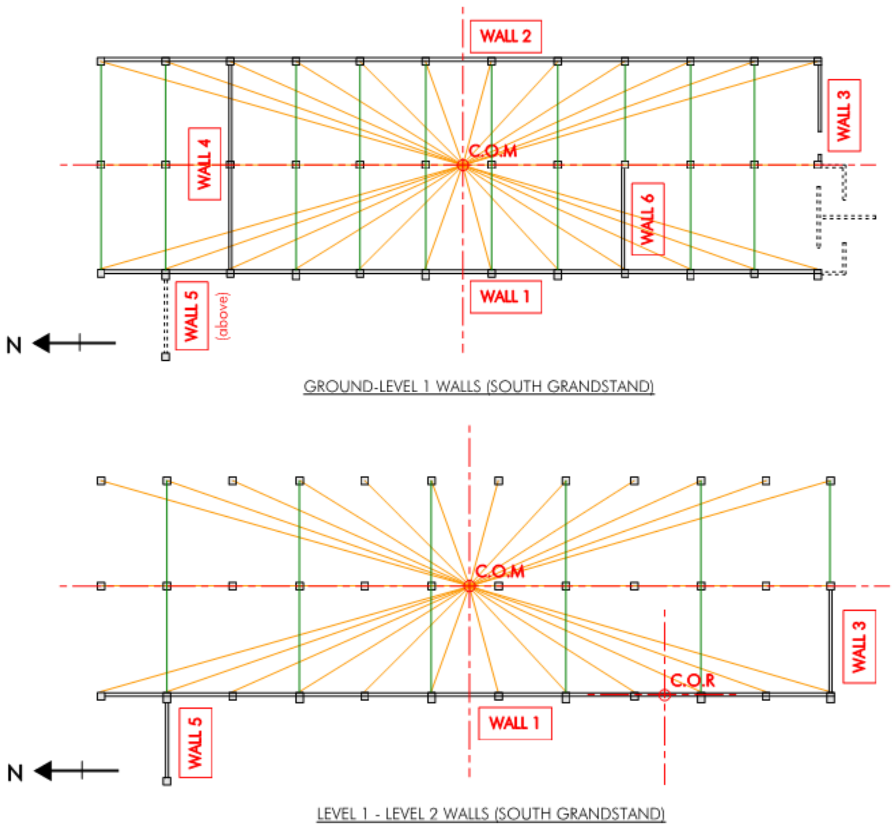


Figure 5-3 Floor Plans of the Wall Layout for the South Grandstand

5.2.1 Building Drift

The following maximum building drifts described henceforth correspond to the 2nd ETABS model. This is considered a more appropriate measure of the actual drift that could occur under a 1/1000 year seismic event.

The lateral load resisting system in the north-south direction consists of reinforced concrete shear walls which is a stiff structural system. In the north-south direction the building displacements were found to be in the order of 50mm at 100% DBE loading, giving the maximum recorded inter-storey drift of 1.2%.

The lateral load resisting system in the east-west direction consists of moment resisting reinforced concrete frames and a stiff concrete shear wall which is required to resist torsion. In this direction the maximum building displacements were found to be in the order of 165mm at 100% DBE loading, giving a maximum recorded inter-storey drift of 3.8%.

The drift is therefore greater than the limit of 2.5% stated in the Loadings Standard NZS1170.5:2004.

5.2.2 Evaluation of Concrete Beams and Columns

The evaluation of the concrete beams and columns comprises two parts, the analysis of the column supporting Wall 5 based upon the response/output of the 1st ETABS model and the analysis of the main seismic resisting frames based upon to response/output of the 2nd ETABS model.

The column supporting Wall 5 has been identified as a critical structural weakness. This column is required to resist the overturning forces of Wall 5. Wall 5 comprises part of the primary seismic load path and accordingly the axial forces in the column supporting Wall 5 are substantial. It is expected that the capacity of this column will be exceeded in a seismic event equivalent to 10-20%DBE loads.

The results from the 2nd ETABS model along with the Microstran model have been used to determine the performance of the frames. The following figure illustrates the demand/capacity ratios for the structural elements at a seismic load equivalent to 30%DBE. This figure shows that the central bays which are required to resist a greater proportion of the torsional forces are exceeded at approximately 10%DBE. This would suggest that upon failure of the column supporting Wall 5 a global failure of the upper level can be expected.

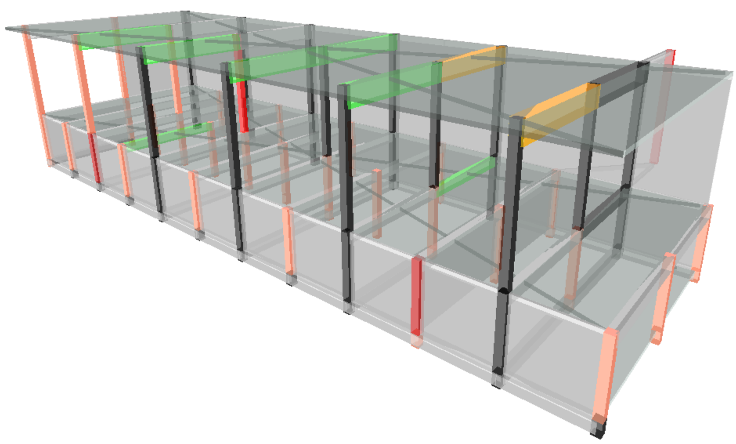


Figure 5-4 Analytical Model displaying Beam and Column Demand/Capacity Ratios

The columns which support the upper level comprise a "100mm Steel Column" and a 300x300 concrete square encasement. The existing structural drawings provide no information on the actual size and thickness of the steel column or any information in regards to the end plate details. Existing structural drawings also show a circular concrete encasement with 6-HD16 longitudinal reinforcing bars, whereas the as built encasement is 300mm square. For the purposes of this exercise the columns have been analysed with an equivalent reinforcing layout as per the circular encasement shown in existing structural and assumptions have been made in regards to the baseplate detail based upon information gained by a site visit.

Based upon our assumptions in regards to the end plate detail, the end plate is insufficient to develop the moment capacity of the column and would require strengthening to ensure better performance of the frames.

5.2.3 Evaluation of Concrete Shear Walls

The concrete shear walls generally perform well under 100%DBE loads. The below model illustrates the demand/capacity ratio for the concrete shear walls under 100%DBE loads based on the 2nd ETABS model output. Based on this analysis the capacity of Wall 3 is exceed at seismic loads equivalent to 75%DBE.

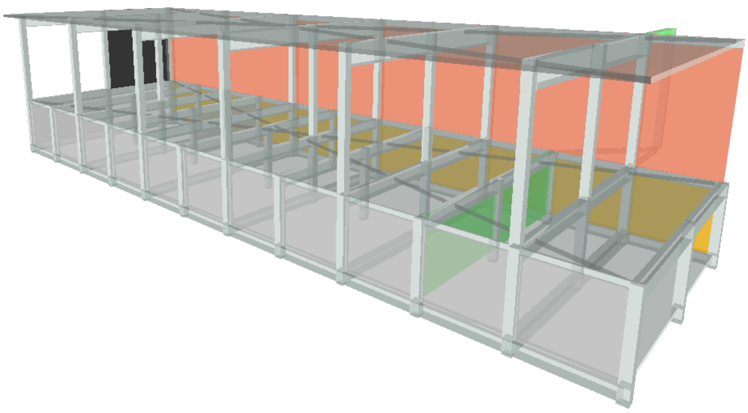


Figure 5-5 Analytical Model displaying Shear Wall Critical Demand/Capacity Ratios

However, the above model does oversimplify the geometry of Wall 1. The actual geometry of Wall 1 is shown in the figure below.

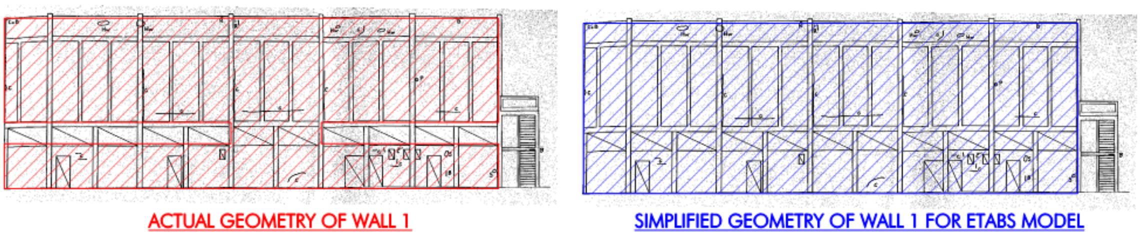


Figure 5-6 Actual Geometry of Wall 1 vs Modelled Geometry

In terms of the shear distribution to the seismic frames and transverse walls the oversimplification of Wall 1 would have little impact on their response and therefore the design forces provided by the ETABS output are considered appropriate. However the actual shear force resolved by Wall 1 below the lower tier is overestimated in the ETABS model. Hand calculation have been undertaken to determine a more realistic design shear force for Wall 1 below the lower tier. Hand calculations suggest that because the stiffness of Wall 2 is much greater than the stiffness for the actual geometry of Wall 1 the diaphragm would act to transfer shear between Wall 1 and Wall 2. Although this type of response is acceptable it is recommended that the stiffness of Wall 1 is increased to avoid this type of response.

Another weakness is the detailing of the diaphragm ties for Wall 5. In the 1st ETABS model Wall 5 is required to resist shear forces from the upper tier and transfer this shear force back into the diaphragm at the lower tier. This creates large tie forces which are currently not allowed for in the detailing of Wall 5, again suggesting that Wall 5 does not have the ability act as a reliable load path beyond low level seismic events.

5.2.4 Evaluation of Floor Diaphragms

Because of the tiered shape of the diaphragm its ability to transfer seismic forces is expected to be poor. Major diaphragm transfer forces occur at level 1 from Wall 1 and Wall 5. This is because the stiffness of both Wall 1 and 5 vary with height and the diaphragm is required to transfer the shear forces from the upper portion of Wall 1 and 5 to more stiff elements at the lower tier level. The below figures illustrate the expected shear transfer at the lower diaphragm from these walls.

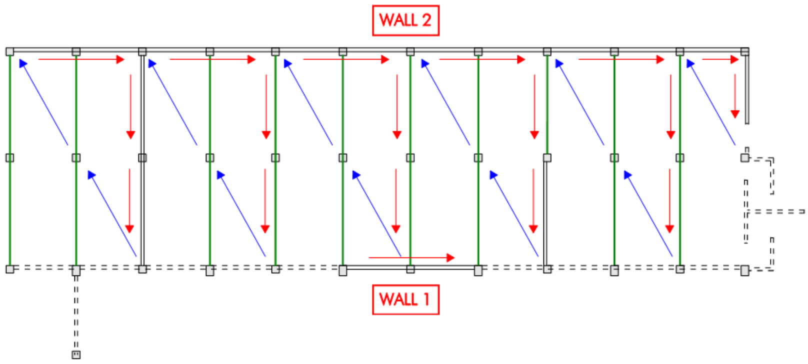


Figure 5-7 Diaphragm Shear transfer from Wall 1 to Wall 2

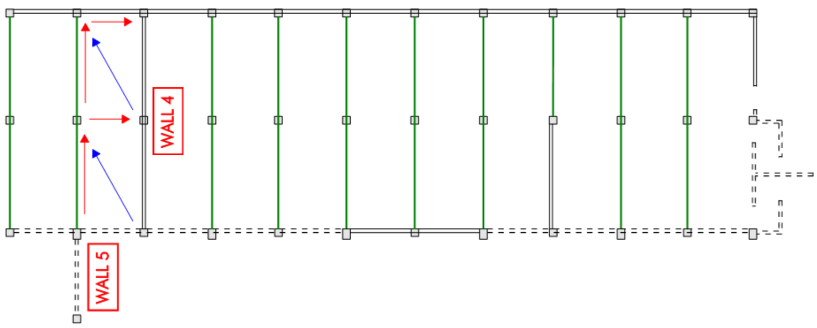


Figure 5-8 Diaphragm Shear transfer from Wall 5 to Wall 4

As opposed to strengthening the diaphragm it is recommended that the stiffness of Wall 1 and 5 are increased at the lower level to prevent shear transfer forces through the diaphragm.

5.2.5 Evaluation of Foundations

The foundation pads form part of the seismic resisting structure and are required to resist the overturning actions from the shear walls. By inspection the critical seismic design actions for the foundations will correspond to the response of Walls 3, 4, 5 and 6.

However, without further information regarding the soil conditions it is difficult to determine the seismic performance of the foundations. In order to gain an indication of the likely allowable ultimate bearing pressure a gravity assessment of the foundations has been undertaken for long-term serviceability and ultimate limit state loads. For the purpose of this assessment, it has been assumed that the allowable ultimate bearing capacity is 300kPa. It is also assumed that the basement slab bears upon the soil directly below and does not contribute to the axial force on the foundation pads.

Based on these assumptions the foundation capacity is exceeded at 75% of the long-term serviceability gravity load and 42% of the ultimate gravity load. These performance ratios relate to the central columns which support both the upper and lower tier. With exception of these central columns the foundation pads generally perform well under gravity loads.

The results of the gravity analysis suggest that the ground conditions are more favourable than predicted. It is possible that the structure has been constructed on well compacted engineered fill, underlain by bedrock.

It is therefore recommended that a geotechnical engineer is engaged to confirm the ground conditions and site seismic soil category. Pending the geotechnical investigation the seismic coefficient may need to be revised, which would potentially have a favourable effect on the expected seismic performance of the structure based on our analysis.

For example, if the results of a geotechnical investigation deem it is more appropriate to use the subsoil classification B the seismic coefficient that needs to be considered for design would be 63% of that which has been currently considered. By crude interpolation if the site subsoil classification was updated to category B the expected seismic performance of the structure would be in the order of 15-30% DBE and would therefore still be considered "Earthquake Prone".

5.3 LOCAL STRUCTURAL VULNERABILITIES

The following local structural vulnerabilities were identified as part of the assessment:

1. Stair detailing for drift not present
2. Inadequate confinement of columns to accommodate CLS drift
3. The ability of the tiered seating slab to act as a diaphragm
4. The durability and condition of the existing concrete

The first two local structural vulnerabilities arise due to the high level of drift expected at ULS and CLS loads. This high level of drift is directly related to the torsional response of the global structure. Therefore, by reducing the drift of the global structure the potential hazard of these vulnerabilities is greatly reduced. To reduce the level of drift for the global structure the torsional stiffness and seismic load paths need to be improved.

The ability of the tiered seating diaphragm to transfer seismic performance is expected to be poor. It is therefore recommended that the potential shear transfer in the diaphragm is reduced by strengthening selected walls to ensure a more even distribution of seismic forces.

If a seismic retrofit is undertaken, other defects affecting the durability of the structure need to be addressed in order to extend the expected lifetime of the structure. For further information with regards to the durability and temporary remedial work refer to the previous Holmes Consulting Group LP letter regarding the condition of the structure and immediate public safety issues, dated December 2014.

5.3.1 Stair detailing for drift not present

There are two sets of existing stairs attached to the structure. The first set of stairs were constructed with the original structure and are attached to the ends of the grandstand whilst the second set of stairs were constructed in 1984 along with the addition of the walls at grids 6 and 7.

The stairs constructed with the original structure are constructed of reinforced insitu concrete. These stairs are directly attached to the end frames which have infill walls. Although there is a potential for these stairs to act as a brace to resist the seismic forces of the global structure the risk of collapse is low. This is because the stairs are attached to a stiff element where the drifts are relatively low. The stairs are however supported by tall slender concrete columns which may require further strengthening for maintaining gravity support and to improve the long-term durability.

The second set of stairs are supported by Wall 5 and the steel stringers which form the stairs act to support Wall 5 out-of-plane. It is not recommended that these stairs are relied upon to brace Wall 5 and further strengthening is recommended. The support of Wall 5 may also be achieved by introducing bracing elements anchored to the main structure which is the preferred method.

5.3.2 Inadequate detailing of gravity columns to accommodate CLS drifts

The columns which form part of the primary gravity frames are inadequately detailed for the expected drift at both ULS and CLS drift. This is however more a function of the structures inability to resist torsion at the upper tier. It is expected that if the seismic load paths and torsional response of the building is remedied that the amount of drift would be significantly lower and the drift demand on these columns would be minimal. So although these columns are considered a local structural vulnerability under the current global seismic load path, with appropriate strengthening the drift demands on these columns could be significantly reduced and strengthening of the columns for drift may not be required.

5.4 BALUSTRADES

Although the primary purpose of this report is to determine the seismic performance of the structure it was also noted that the balustrade at the upper level appeared poorly detailed. Further investigation of the performance for the balustrade to resist loads from crowds is recommended. Based on a site review of the current connections between the balustrades and the primary structure, strengthening is required. It is recommend that strengthening of the balustrade is undertaken as part of any seismic retrofit.

6. POSSIBLE STRENGTHENING OPTIONS

The following are considered possible strengthening options.

6.1 PROVIDE LIMITED ACCESS

An option to reduce the risk from local or global damage to the structure would be to limit access to the building. It is recommended that at a minimum occupancy of the lower tier is prevented due to the significant risk for collapse of the upper tier in a minor seismic event. Short-term occupancy of the upper tier may be considered by the building owner. In deciding whether to allow short term occupancy of the upper tier the building owner must however understand the significant risk. It is recommended that at minimum strengthening of Wall 5 and its supports is undertaken prior to considering whether to allow short-term occupancy of the upper tier.

6.2 STRENGTHEN EXISTING STRUCTURE

The following strengthening schemes are qualitative concepts, based on the results of the seismic assessment for the existing building, no specific analysis of these schemes has been undertaken. Therefore, the information provided with regards to strengthening is only sufficient to provide an initial concept for the type and extent of work that may be required. Further analysis would be needed to determine the required strengthening to meet the desired %DBE. However, prior to further analysis, a more accurate geotechnical assessment of the grandstand site and intrusive materials testing of the existing structure is recommended.

The options listed henceforth are loosely ordered in terms of the cost/benefit.

<u>Strengthen existing structure</u>

i) Infill Wall 5 and strengthen corresponding foundations
ii) Infill Wall 5, construct internal walls adjacent Wall 5 and strengthen corresponding foundations
iii) Strengthen foundations of transverse walls
iv) Strengthen Wall 1 below the lower tier
v) Brace the upper tier to the lower tier transverse walls
vi) Strengthen balustrades, stairs and address durability issues
vii) Introduce additional structural elements to further improve the seismic performance

<u>Partial demolition.</u>

i) Remove the upper tier, including stairs and retain the lower tier structure. Potentially a new upper tier could be designed and constructed.

<u>Complete demolition and rebuild</u>

i) Completely demolish the existing structure and build from new.

Many of the problems that exist in the current structure can be resolved by improving the seismic load paths and increasing the stiffness of the global structure supporting the upper tier through additional walls.

The strengthening options described below typically address a particular element or issue and individually would have little impact on the overall building performance. Therefore, a combination of these strengthening schemes would need to be considered to improve the performance of the structure as a whole. For example, one method of improving the performance of the structure would be to increase the stiffness of Wall 1 below the lower tier, extend Wall 5 to ground level, introduce additional transverse walls and improve the foundations for overturning actions. It is also suggested that out-of-plane support of Wall 5 is provided by braces anchored to the main structure as opposed to relying on the steel stairs to provide this support.

6.2.1 Infill Wall 5 and strengthen foundations

This would be considered the minimum strengthening that would be needed to improve the performance of the structure. As previously mentioned the column supporting Wall 5 is a critical structural weakness and is therefore a principal element that requires strengthening.

This scheme would involve extending Wall 5 to ground level by constructing an insitu reinforced concrete infill wall and improving the foundations which support Wall 5. Improvement of the foundation would likely comprise constructing either a posthole or pile which is anchored in solid rock below. Refer Figure 6-1 below.

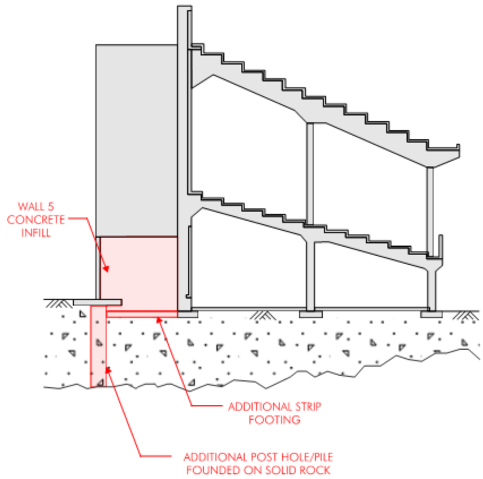


Figure 6-1 Strengthening of Wall 5: Grid 6 and 7

Although strengthening of Wall 5 does address the performance of the column supporting Wall 5 it would have little impact on the overall global performance (%DBE) of the structure. To improve the global performance of the structure beyond addressing the critical structural weakness further strengthening is required.

6.2.2 Infill wall 5 and internal walls adjacent Wall 5 and strengthen foundations

The first stage of improving the global performance whilst also addressing critical structural weaknesses would be improving the lateral load path along Grid 6 and 7. Given the current structural layout of transverse lateral resisting elements, Wall 5 is required to transfer the shear force from the upper tier into the lower tier. This shear force is then transferred through the diaphragm to other transverse lateral resisting elements.

The following strengthening creates a direct load path from the top tier to ground which prevents a shear transfer in the diaphragm. The introduction of new foundations would also resist the overturning actions imposed by Wall 5. Refer Figure 6-2 below.

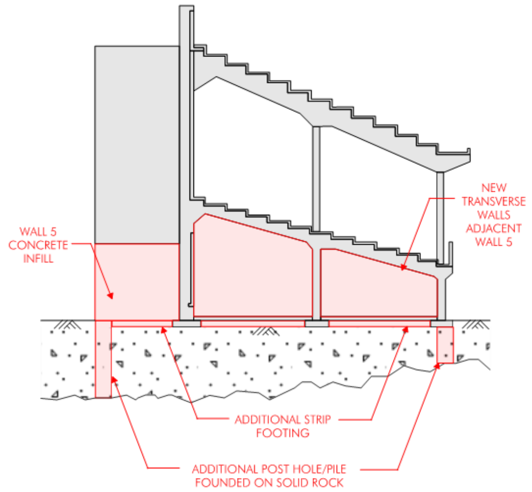


Figure 6-2 Strengthening of Wall 5 and adjacent walls and foundations: Grid 6 and 7

This scheme would require the construction/improvement of reinforced concrete walls along Grid 6 and 7 and construction of new foundations with postholes/piles at each end of the lower tier wall along with an additional strip footing. The postholes/piles would need to be founded on solid rock or be constructed to a depth that's required to resist the actions from overturning.

6.2.3 Strengthen additional transverse walls for overturning

Although the previous two strengthening options address the issues with the transverse walls along Grid 6 and 7, various other transverse walls which provide lateral resistance also lack the required foundations to resist significant overturning actions. Therefore, strengthening for overturning is required. Refer Figure 6-3 below.

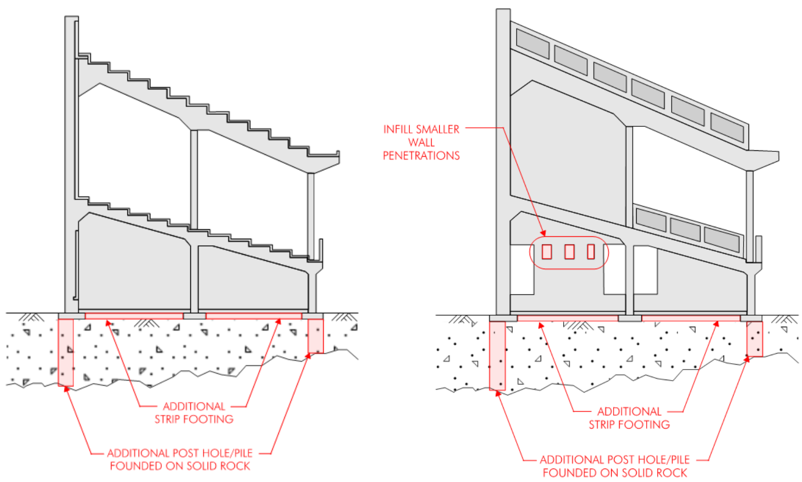


Figure 6-3 Strengthening of transverse walls for Overturning: Grid Varies

This scheme would involve the strengthening the foundations of all transverse walls which act to resist lateral loads by introducing postholes/piles at each end of the wall along with the addition of a strip footing along the length of the wall. Again, the postholes/piles would need to be founded on solid rock or be constructed to a depth that's required to resist the actions from overturning. In some instances it is recommended that penetrations in these transverse walls are infilled for strength. Further investigation is required to determine the extent of infill required.

6.2.4 Strengthen Wall 1 to improve diaphragm performance

The aforementioned strengthening schemes typically address the seismic response in the transverse direction. However strengthening to the longitudinal direction is also required, specifically in regards to reducing the shear transverse in the diaphragm. As opposed to strengthening the diaphragm it is recommended that Wall 1 is stiffened by providing reinforced concrete infills along its length. This would result in a more even shear distribution and reduce the forces through the diaphragm.

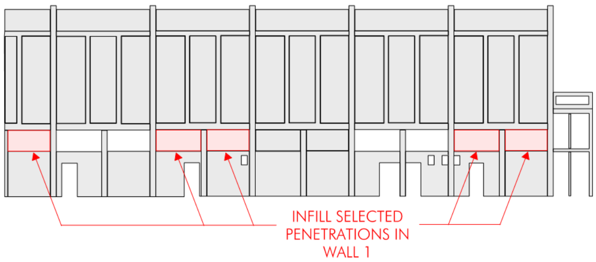


Figure 6-4 Improve the Stiffness of Wall 1 via Reinforced Concrete Infill Walls

This scheme would involve constructing insitu reinforced concrete infill walls in selected penetrations throughout Wall 1 to improve its stiffness. To determine the extent and number of reinforced concrete infills further analysis is required.

6.2.5 Brace upper tier to lower tier

Along with some of the aforementioned strengthening for the response in the transverse direction, additional braces may also be introduced at intermediate grids to improve the overall %DBE for the global structure. This could be in the form of steel braces or reinforced concrete infill walls. Refer Figure 6-5 below.

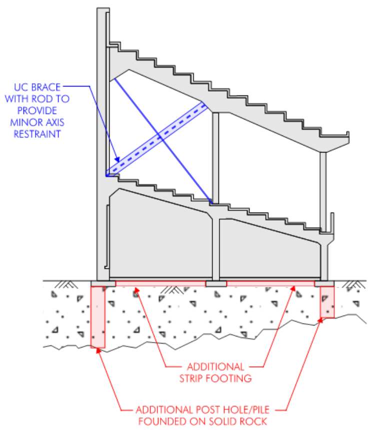


Figure 6-5 Brace Upper Tier to Lower Tier via Steel Braces: Grid Varies

In the previous figure the location of the steel brace is shown to align with a lower tier transverse wall which is needed to create a direct load path from the upper tier to ground. The above figure also depicts a steel brace which would act both in tension and compression and is retrained in the minor axis by a steel rod. The practicability and exact arrangement for the option of installing steel braces requires further analysis and it may be more practical to instead construct reinforced insitu concrete walls.

The concept of bracing the upper tier to the lower tier is to address issues with regards to the torsional response of the upper tier. Although this scheme will improve the performance of the upper tier the performance of the lower tier is not fully addressed.

6.2.6 Strengthen balustrades, stairs supports and address durability issues

As part of any strengthening work the balustrade, stair support and durability issues need to be addressed. Based on our qualitative site review the upper tier steel balustrade detailing appears insufficient for crowd loading. It is recommended that opposed to strengthening the existing balustrade it is instead replaced with a new balustrade.

Although no specific gravity analysis for the stairs was undertaken, from our review of the existing structural drawings it is recommended that the concrete columns which support the weight of the stair are strengthened. This will however, be dependent on the expected drift for the strengthened structure and further analysis is required to determine the extent of work.

As previously mentioned, other defects affecting the durability of the structure need to be addressed in order to extend the expected lifetime of the structure. For further information with regards to the durability and temporary remedial work refer to the previous Holmes Consulting Group LP letter regarding the condition of the structure and immediate public safety issues, dated December 2014.

6.2.7 Introduce additional structural elements to further improve the seismic performance

To further improve the %DBE beyond the strengthening options already mentioned, additional structural elements including reinforced concrete walls, steel braces, diaphragm ties and foundations can be introduced at both the upper tier and lower tier in each direction. The nature of this work would be similar to that already mentioned. The extent of this work will depend on the target %DBE desired by the building owner.

6.3 PARTIAL DEMOLITION

Partial demolition of the structure would involve removing the upper tier and stairs whilst retaining the lower tier structure. By removing the upper tier the structure would no longer have a pronounced torsional response and the overturning actions on the transverse shear walls will be reduced. In considering this option it is recommended that an analysis of the lower tier structure is undertaken to determine the performance (%DBE) of the lower tier alone. In order to improve the performance of the lower tier alone some minor strengthening to the foundations may be required. A new upper tier could be designed and constructed.

6.4 COMPLETE DEMOLITION AND BUILD FROM NEW

Pending the financial viability of strengthening the structure, and considering the long-term vision for Waikaraka Park, the preferred option may be to demolish or demolish and rebuild.

7. CONCLUSIONS

Holmes Consulting Group LP have been commissioned to evaluate the likely seismic performance of the grandstand at Waikaraka Park. To determine the extent to which the building complies with new building standards, a three dimensional linear elastic dynamic analysis was carried out using Microstran and ETABS, commercially available analysis software programmes.

7.1 EXISTING CAPACITY

Based on a series of analyses at various levels of seismic loading, it was determined that the existing building has a capacity equivalent to approximately 10-20% DBE at ULS and would be considered earthquake prone. The overall building strength is governed by the overall layout and capacity of the concrete shear walls supporting the upper tier and several identified critical structural weaknesses.

Due to a lack of information with regards to the soil conditions it is recommended that a geotechnical engineer is engaged to undertake an investigation of the site and confirm an appropriate site subsoil classification.

7.2 ADDRESSING LOCAL STRUCTURAL VULNERABILITIES

The assessment also identified critical structural weaknesses and local structural vulnerabilities for the structure. The following two items are considered critical structural weaknesses;

1. The global torsional stiffness of the structure supporting the upper tier
2. The support of Wall 5.

Both of these items would need to be addressed in any strengthening scheme for the structure.

The following four items were identified as local structural vulnerabilities;

1. Stair detailing for drift not present
2. Inadequate confinement of columns to accommodate CLS drift
3. The ability of the tiered seating slab to act as a diaphragm
4. The durability and condition of the existing concrete

The potential hazard of the first two local structural vulnerabilities as mentioned above, are directly correlated to the level of expected drift. A strengthening scheme that would reduce the expected drift of the structure would greatly improve the performance of these items.

It is also recommended that the potential shear transfer in the diaphragm is reduced by strengthening selected walls to ensure a more even distribution of seismic forces. Further strengthening of the diaphragm may be required pending further detailed analysis. Other defects affecting the durability of the structure should also be addressed in order to extend the expected lifetime of the structure.

For further information with regards to the durability and temporary remedial work refer to the previous Holmes Consulting Group LP letter regarding the condition of the structure and immediate public safety issues, dated December 2014.

7.3 POSSIBLE STRENGTHENING OPTIONS

There are a several strengthening options for the building owner to consider as discussed within the body of this report. To achieve a %DBE rating between 33-67% the building owner would need to consider a combination of these strengthening schemes. There is also the option for partial or complete demolition of the structure which may be the preferred method depending on the long-term vision for Waikaraka Park.

The performance of the structure is predominately governed by its ability to resist torsion at the upper level and the ability of Wall 5 to act as a seismic load path. Therefore, one possible method for improving the torsional performance of the structure, and the performance of Wall 5, is to extend Wall 5 to ground level and improve its foundations.

Along with strengthening the global structure other structural elements such as the balustrades, gravity support to the stairs and durability also need to be addressed.

Further analysis work would be needed to determine the required strengthening to meet the improved %DBE target desired by the building owners. Along with further analysis, a more accurate geotechnical assessment of the grandstand site and intrusive materials testing of the existing structure would be needed to develop an appropriate strengthening scheme.

8. REFERENCES

[1] Standards New Zealand, Structural Design Actions Part 5: Earthquake Actions NZS1170.5:2004, Wellington, New Zealand, 2004.

[2] Standards New Zealand, Concrete Structures Standard Part 1: The Design of Concrete Structures, NZS 3101:Part 1:2006, Wellington, New Zealand, 2006.

[3] Standards New Zealand, Steel Structures Standard Part 1: The Design of Steel Structures, NZS 3404: Part 1:1997, Wellington, New Zealand, 1997.

[4] New Zealand Society for Earthquake Engineering, Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, 2006.

[5] Soil Engineering Ltd, Geotechnical Investigation for Proposed Onehunga Sports Club at Waikaraka Park, Onehunga, 20th July 2001.

[6] Development Consultancy, Waikaraka Park Grandstand Column Encasement, Structural Drawings, January 1996

[7] Babbage Partners Limited, Onehunga Borough Council, Waikaraka Par Grandstand, Structural Drawings, July 1984.

[8] Borough of Onehunga, Reinforced Concrete Grandstand, Waikaraka Reserve, Original Structural Drawings, Date Unknown.