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Denton Park Lighting Towers
BU 0770-001 EQ2

Detailed Engineering Evaluation

Qualitative Report

Version FINAL

442 Main South Road, Hornby

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

Denton Park Lighting Towers
BU 0770-001 EQ2

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

442 Main South Road, Hornby

Christchurch City Council

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Reviewed By
CPEng

Date
11th June 2012

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Denton Park Lighting Towers

BU 0770-001 EQ2

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

Version FINAL

442 Main South Road, Hornby

Background

This is a summary of the Qualitative report for above the building structure, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 15th February 2012.

Key Damage Observed

Key damage observed includes:-

- Rusting of tension bolts.
- Rusting of base flange.
- Rusting of securing nuts.

Critical Structural Weaknesses

There were no potential critical structural weaknesses identified in the structure.

Indicative Building Strength

An analysis was carried out to determine if the structures are wind or seismically governed. This was assessed by comparing the loading of a 5 year return period wind event and a 500 year return period seismic event. The results showed that the structures are wind governed with the 5 year wind loading being 10 times greater than the 500 year seismic loading. This would indicate that the structures experience the loading associated with a 500 year return period earthquake on a regular basis and are not at risk of failure as a result of such an event.

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the light towers has been assessed to be in the order of 73% NBS and post-earthquake capacity also in the order of 73% NBS. As a result the structures are neither earthquake prone nor an earthquake risk.

Recommendations

- The building has not been assessed as being Earthquake Prone. As a result, the lighting towers can remain in use, as per CCC's policy.
- CCC are not required to undertake a detailed seismic assessment of the structures.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Denton Oval Lighting Towers.

This report is a Qualitative Assessment of the building structure, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. Construction drawings were made available, and these have been considered in our evaluation of the building. The building description below is based on a review of the drawings and our visual inspections.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity 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 by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Table 1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

Denton Oval and its associated lighting towers are located at 442 Main South Road, Hornby, Christchurch. The site consists of a large cycling track with a spectator stand and club facilities to the south of the track, four flood lighting towers (2 north and 2 south of the track) and an array of smaller lighting towers around the perimeter of the track. The facility was constructed prior to the 1974 Commonwealth Games with a subsequent addition made to the building around 1990.

The flood light towers are constructed of a single galvanised steel circular hollow section (CHS's) with a flange at the bottom for securing them to the foundations. The towers are constructed of approximately 2.5m long sections with welded connections. As the pole extends vertically it tapers.

The mono-poles are supported by piled foundations with a concrete pile cap. The exact pile design is unknown as no drawings have been made available.

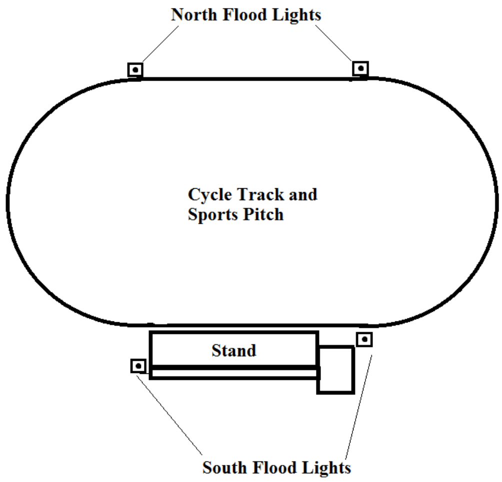

Figure 2 Plan Sketch Showing Site Layout

The dimensions of each of the mono-poles are approximately 0.8m diameter at the base and approximately 22m tall. The approximate area of each of the pile caps is 2.5m^2.

The nearest buildings to any of the light towers are the spectator stands and the club facilities. There are no waterways in the area.

No plans were made available for these structures.

4.2 Gravity Load Resisting System

The gravity loads of the structure are transferred directly down through the steel CHS mono-pole to the pile cap and through to the piled foundation.

4.3 Lateral Load Resisting System

Lateral loads are resisted in both directions by the cantilever actions of the mono-pole and piled foundation.

5. Assessment

An inspection of the structure was undertaken on the 15th of February 2012. Only the exterior of the poles were inspected. The piled foundations were not able to be viewed.

The inspection consisted of observing the structure to determine the structural systems and likely behaviour of the mono-poles during an earthquake. The site was assessed for damage, including examination of the ground conditions, checking for damage in areas where damage would be expected for the type of structure and noting general damage observed throughout the site in both structural and non-structural elements.

The IEP assessment spread-sheet developed by CERA for the purpose of carrying out the DEE inspections of buildings is not considered to be directly applicable to these light tower structures. For the purpose of analysis of the structures a comparison of the current wind and seismic loading was undertaken. The reasoning behind this is that if the wind loading for a shorter return period on the structure is greater than the seismic loading for a longer return period event, then the structure is dominated by wind loading and the seismic capacity of the structure will be sufficient.

The %NBS score determined for this building has been based on the IEP procedure described by the NZSEE and based on the information obtained from visual observation of the structure.

6. Damage Assessment

6.1 Surrounding Buildings

The closest buildings to the lighting towers are the spectator stand and club facility. Minor cracking was noted in the concrete masonry of the spectator stand structure and the changing room structure. Though not covered by this assessment, cracking was noted to several of the smaller concrete light poles surrounding the cycling track (See Photograph 11). Metal extensions at the top of these poles have suffered damage as a result of contact with encroaching trees (See Photograph 12).

6.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during the inspection of the lighting towers.

Rusting of the tension bolts and securing nuts at the base of all four towers was noted. This is shown in Photographs 1, 3, 6, 8 and 9.

6.3 Ground Damage

There was no evidence of ground damage on the property or surrounding neighbours land.

7. Critical Structural Weakness

No critical structural weaknesses have been identified in the structures.

8. Geotechnical Consideration

8.1 Introduction & Site Description

This report outlines the ground conditions, as indicated from sources quoted within the document. This is a desktop report and no site visit has been undertaken by Geotechnical personnel.

This report is only specific to the grandstand and amenities at Denton Oval, 422 Main South Road, Hornby. The park is located between the Main South Railway line to the north and Main South Road (SH1) to the south. It is bound to the east by commercial properties and west by residential properties. The property is owned and maintained by the Christchurch City Council.

The site is situated within a recreational reserve, within the suburb of Hornby in western Christchurch. It is relatively flat at approximately 30m above mean sea level. It is approximately 2.5km west of the Heathcote River, and 15km west of the coast (Pegasus Bay) at New Brighton.

8.2 Published Information on Ground Conditions

8.2.1 Published Geology

The geological map of the area indicates that the site is underlain by Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial gravel, sand, and silt of historic river flood channels.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that eight boreholes are located within a 200m radius of the site. Of these boreholes, six of them had a lithographic log of which the four most relevant are summarised below. The site geology described in these logs shows the area is predominantly sandy gravels with varying amounts of silt and clay.

Table 2 ECan Bore Log Summary Table

Bore Name	Depth (m bgl)	Log Summary
M35/1865	0 – 1	Hardfill
(110m SE of site)	1 – 21	Fine to coarse GRAVEL and SAND
	21 – 49	Medium dense to dense GRAVEL, with some sand and clay
	49 – 52	Dense GRAVEL, with sand and clay
	52 – 79	Fine to medium GRAVEL, with traces of clay
	79 – 86	Sandy medium GRAVEL
	86 – 88	PEAT
	88 – 94	Dense GRAVEL, and stiff CLAY
	94 – 102	Dense Sandy GRAVEL, with some yellow clay

Bore Name	Depth (m bgl)	Log Summary
M35/3546	0 – 0.3	Filling Material
	0.3 – 3.9	SILT
	3.9 – 40.5	Sandy GRAVEL, with some clay
	40.5 – 49.5	CLAY, with some gravel and peat
	49.5 – 52	Dense GRAVEL, with some clay
	52 – 95.8	Layers of CLAY, SAND and GRAVEL
M35/7739	0 – 6	Gravelly SAND
	6 – 23.5	Sandy GRAVEL, with traces of silt and clay
	23.5 – 29.5	Sandy GRAVEL
M35/7743	0 – 1	Clayey GRAVEL
	1 – 9	Sandy GRAVEL, with some clay and silt
	9 – 10.8	Sandy GRAVEL
	10.8 – 12.5	Slightly clayey, fine SAND
	12.5 – 20.7	Clayey GRAVEL and sandy GRAVEL

It should be noted that the purpose of the boreholes the well logs are associated with, were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

8.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the site.

8.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

The site is in the “not applicable” technical category. Not applicable means that non-residential properties in urban areas, properties in rural areas or beyond the extent of land damage mapping, and properties in the Port Hills and Banks Peninsula have not been given a Technical Category.

8.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in the Figure 1.

Figure 3 Post February 2011 Earthquake Aerial Photography[1]

8.2.6 Summary of Ground Conditions

From the ECan borehole information the ground conditions on Main South Road comprise multiple strata of gravel, sandy gravel and sand, with varying amounts of silt and clay.

8.3 Seismicity

8.3.1 Nearby Faults

There are many faults in the Christchurch region, however only those considered most likely to have an adverse effect on the site are detailed in Table 2.

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system / zone underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

[1] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

Table 3 Summary of Known Active Faults Summary of Known Active Faults[23]

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120	8.3	~300 years
Greendale (2010) Fault	13	7.1	~15,000 years
Hope Fault	100	7.2~7.5	120~200 years
Kelly Fault	100	7.2	~150 years
Porters Pass Fault	54	7.0	~1100 years

8.3.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude 6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

In addition, the ground conditions are anticipated to be Holocene alluvial soils comprising alluvial gravel, sand, and some silt and clay, with bedrock expected to be in excess of 500m deep. Combining this with a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 20023), the ground shaking is expected to be moderate to high.

8.4 Slope Failure and/or Rockfall Potential

The site is located within Hornby, a flat suburb in western Christchurch. Global slope instability risk is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

8.5 Liquefaction Potential

No effects of liquefaction were reportedly observed at the ground surface in Hornby.

Due to the anticipated presence of predominantly gravels and sandy gravels beneath the site, it is considered that liquefaction is less likely to occur at this site than other areas of Christchurch. However, during the inspection undertaken on 18th January, no liquefaction was observed. However, the grain

[2] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002) A New Seismic Hazard Model for New Zealand, Bulletin of the Seismological Society of America, Vol. 92 No. 5, pp 1878-1903, June 2002.

[3] GNS Active Faults Database

size of the sands present is not recorded, and silts are also recorded as present in varying amounts within the gravels. Therefore it is considered possible and likely that liquefaction will occur where sands and silts are present.

8.6 Recommendations

If a more detailed assessment is required to quantify the assessment results then an intrusive investigation comprising of at least one piezocone CPT test to 20m bgl should be undertaken. This will allow a numerical liquefaction analysis to be carried out.

8.7 Conclusions & Summary

This assessment is based on a review of the published geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on stratified alluvial deposits, comprising gravel, sand and silt. Considering these likely anticipated ground conditions the site also has a low-moderate liquefaction potential, the potential, the potential is increased however where sands and/or silts are present.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that an intrusive investigation comprising of at least one piezocone CPT be conducted.

A soil class of D (in accordance with NZS 1170.5:2004) should be adopted for the site.

9. Survey

No level or verticality surveys have been undertaken for these structures at this stage.

10. Initial Capacity Assessment

10.1 Assessment

The towers have had their capacity assessed using the Initial Evaluation Procedure based on the information available. The buildings capacity excluding critical structural weaknesses and the capacity of any identified weaknesses are expressed as a percentage of new building standard (%NBS) and are in the order of that shown below in Table 4. These capacities are subject to confirmation by a more detailed quantitative analysis.

Item	%NBS
Structures excluding CSW's	73

Table 4 Indicative Capacities based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving 73% New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered neither Earthquake Risk or Earthquake Prone as it achieves greater than 67% NBS. This score has not been adjusted when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems.

In addition the structures have been assessed by comparing the wind loading of a relatively frequent wind event to the seismic loading experienced during a less frequent seismic event. The ratio of the lateral loads experienced during a 5 year return period wind event and a 500 year return period seismic event is approximately 10:1 respectively. This would indicate that the towers are most likely wind governed.

10.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS 1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- Return period factor R_u = 2.0 , NZS 1170.5:2004, Table 3.5, Importance level 1 structure with a 50 year design life.

An increased Z factor of 0.3 for Christchurch has been used in line with requirements from the Department of Building and Housing resulting in a reduced % NBS score.

10.3 Expected Structural Ductility Factor

A structural ductility factor of 1.5 has been assumed based on the structural system observed and the date of construction.

10.4 Discussion of Results

The results obtained from the analysis of governing load conditions are consistent with those expected for a pole structure. The design of these pole towers is generally governed by wind forces, this is due to the poles and lighting equipment attached to them being relatively lightweight, whereas the wind area is reasonably large. In addition to this, the design of the poles is predominantly governed by serviceability deflection requirements and not strength. As a result, it is expected that the capacity of the poles and their foundations is more than adequate to resist the ultimate wind and EQ loads.

10.5 Occupancy

The structures do not pose an immediate risk to users and occupants as no critical structural weaknesses have been identified. The structures have not been assessed as being Earthquake Prone. As a result, they can remain in use, as per CCC's policy, and should not be considered when determining occupancy or placard status of the Denton Oval facilities.

11. Initial Conclusions

The structures design is most likely governed by wind loading rather than seismic loading.

The structures have been assessed to have a seismic capacity in the order of 73% NBS and is therefore not potentially Earthquake Prone or a potential Earthquake Risk.

12. Recommendations

The structures have not been assessed as being seismically governed. As a result, they can remain in use as per CCC's policy, and should not be considered when determining occupancy or placard status of the Denton Oval facilities.

The minor damage in the form of partially rusted anchor bolts has been considered in the analysis. The anchor bolts should be monitored, and if loss of section due to rust continues the bolts should be replaced with similar new anchor bolts.

13. Limitations

13.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No analysis of the electrical components has been undertaken.
- No calculations, other than those included as part of the IEP in the CERA Building Evaluation Report, have been undertaken. No modelling of the building for structural analysis purposes has been performed.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this reportrite a specific limitations section.

13.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A

Photographs

Photograph 1 Lighting tower base showing rusting of tension bolts and nuts.

Photograph 2 Lighting tower elevation.

Photograph 3 Lighting tower base showing some rusting of tension bolts.

Photograph 4 Connection between CHC sections on the South East tower.

Photograph 5 Lighting tower at the South East of Denton Oval.

Photograph 6 Base of lighting tower at the North West of Denton Oval showing minor rusting to elements.

Photograph 7 Lighting tower at the North West of Denton Oval.

Photograph 8 Base of the lighting tower to the North East showing rusting of restraining elements.

Photograph 9 Base of the North East lighting tower showing severe rusting to tension bolts and respective nuts.

Photograph 10 Lighting tower at the North East of Denton Oval.

Photograph 11 Cracking to one of the smaller concrete lighting towers around Denton Oval.

Photograph 12 Spalling of concrete of one of the lighting towers at the Eastern end of Denton Oval.

Appendix B

Existing Drawings

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Denton Oval Lighting Towers		Reviewer:	Hamish Mackinven
	Unit	No: Street	CPEng No:	1003941
Building Address:		442 Main South Road	Company:	GHD
Legal Description:	RS 41304 (SO 15376)		Company project number:	513059632
			Company phone number:	(03) 3780900
	Degrees	Min Sec		
GPS south:	43	32 30.64	Date of submission:	06-11-12
GPS east:	172	31 14.11	Inspection Date:	15/02/12
			Revision:	FINAL
Building Unique Identifier (CCC):	BU 0770-001 EQ2		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	Soil Profile (if available):	Former Dump
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	0.00
Storeys below ground	0			
Foundation type:	other (describe)		if Foundation type is other, describe:	Unknown
Building height (m):	22.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	22
Floor footprint area (approx):	3			
Age of Building (years):	38		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL1			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	
Floors:	other (note)	describe sytem	concrete pad
Beams:	none	overall depth x width (mm x mm)	
Columns:	structural steel	typical dimensions (mm x mm)	800 x 800
Walls:			

Lateral load resisting structure

Lateral system along:	welded and bolted steel moment frame	**Note: Define along and across in detailed report!**	note typical bay length (m)	Steel Cantilever CHS Beam
Ductility assumed, μ:	1.50			
Period along:	1.30	1.42 from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	welded and bolted steel moment frame		note typical bay length (m)	Steel Cantilever CHS Beam
Ductility assumed, μ:	1.50			
Period across:	1.30	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:		
Wall cladding:		
Roof Cladding:		
Glazing:		
Ceilings:		
Services(list):		

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green	
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):		
Across	Damage ratio:	0%	
	Describe (summary):		
Diaphragms	Damage?:	no	Describe:
CSWs:	Damage?:	no	Describe: No obvious bracing observed except for precast
Pounding:	Damage?:	no	Describe:
Non-structural:	Damage?:	yes	Describe: Rusting

$$Damage\_Ratio = \frac{(\%NBS(before) - \%NBS(after))}{\%NBS(before)}$$

Recommendations

	Level of repair/strengthening required:	none	Describe:
	Building Consent required:		Describe:
	Interim occupancy recommendations:	full occupancy	Describe:
Along	Assessed %NBS before:	73%	73% %NBS from IEP below
	Assessed %NBS after:	73%	If IEP not used, please detail assessment methodology:
Across	Assessed %NBS before:	73%	73% %NBS from IEP below
	Assessed %NBS after:	73%	

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1965-1976

h_n from above: 22m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

	along	across
Period (from above):	1.3	1.3
(%NBS)nom from Fig 3.3:	6.0%	6.0%

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.20
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final (%NBS)nom:	**7%**	**7%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	0.30
Z_{1992}, from NZS4203:1992	0.8
Hazard scaling factor, **Factor B:**	3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above):	1
Return Period Scaling factor from Table 3.1, **Factor C**:	2.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.50	1.50
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.29	1.29
Ductiity Scaling Factor, **Factor D**:	1.29	1.29

2.6 Structural Performance Scaling Factor:

Sp:	0.850	0.850
Structural Performance Scaling Factor **Factor E**:	1.176470588	1.176470588

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$:	**73%**	**73%**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**1.00**	**1.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**73%**	**73%**
4.4 Percentage New Building Standard (%NBS), (before)		**73%**

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Document Status

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FINAL	Peter O'Brien		Nate Oakes	Hamish Mackinven		09/4/12