DETAILED ENGINEERING EVALUATION – QUALITATIVE AND QUANTITATIVE REPORT

SCARBOROUGH CLOCK TOWER

June 2012

Capital Programme Group

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Revision History

Revision	Prepared By	Description	Date
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DETAILED ENGINEERING EVALUATION – QUALITATIVE REPORT

SCARBOROUGH CLOCK TOWER
(Scarborough Park, 147 Esplanade Rd, Sumner)

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June 2012

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EXECUTIVE SUMMARY

Structural inspections of the clock tower undertaken follow the 4th of September 2010, 22th of February earthquakes and subsequent aftershocks revealed minor new cracking and the opening of existing damage which has not compromised its load resisting capacity.

The building has been assessed initially using the NZSEE Initial Evaluation Procedure (IEP), and has been assessed to be 25% New Building Standard (NBS) and therefore a full detailed engineering evaluation (DEE) has been undertaken.

The Detail Engineering Evaluation indicates the structure to have a seismic capacity of 66% NBS. It is therefore not considered Earthquake Prone. The building is therefore considered low to moderate risk and can remain in use.

This report recommends that the structure requires maintenance to reduce the corrosion of the concrete reinforcement and spalling of the concrete surface in some parts of the structure.

If the client desires to upgrade the structure to 100% of the New Building Standard, further geotechnical investigations and complex analytical modelling will be required.

Strengthening of the building is likely to involve intrusive work to the foundations and superstructure which may need to be given careful considerations due to the structure's heritage status.

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2. BACKGROUND

Capital Programme group has been engaged by the Transport and Green space Unit to undertake a detailed engineering evaluation of the Scarborough clock tower.

This report comprises two type of analysis: the first analysis is the initial Qualitative Assessment of the building structure based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. The second is a detail assessment based on the rocking mechanism of the structure.

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 detail assessment is a more realistic approach based actual material testing, load distribution analysis and computer modelling to represent the actual building condition.

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).

3. 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 time of writing.

3.1. CANTERBURY EARTHQUAKE RECOVERY AUTHORITY

CERA was established on 28th March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18th 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

3.2 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, i.e. the 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.

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.

3.3 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 1st 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.

3.4 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.

4. 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 4a below. Figure 4b (extracted from AISPBE) 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.

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 4a NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE

Building Grade	Percentage of New Building Strength (*%NBS*)	Approx. Risk Relative to a New Building	Risk Description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 80	2 to 5 times	low or medium risk
C	33 to 67	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

Figure 4b %NBS compared to the relative risk of failure.(Refer Table 2.1 of the NZSEE 2006 AISPBE)

5. BUILDING DESCRIPTION

5.1 GENERAL

The clock tower consists on 2 storey reinforced concrete building with stone veer cladding with a total height of 13.8m height and square shape of 6.2m long with a floor foot print area of 38.5m2 and was built circa 1934.

The gravity and lateral resistance is provided by two long resisting walls in the North-South direction while four short walls connected by a archway are provided in the East-West direction to achieve the desired global displacement demand. Copies of the architectural plan are shown in Appendix C of this report. These drawings were provided following a building survey as part of the detail engineering evaluation process.

Figure 5a – Plan view showing location of clock tower

5.2 GRAVITY AND SEISMIC LOAD RESISTING SYSTEM

The gravity loads acting on the structure correspond to the self weight of the concrete wall and floor elements in addition to the stone veneer on the external faces of the walls.

Gravity loads from the reinforced concrete roof and 1st floor slab are transferred through a monolithic connection between the slab and the walls. The loads are then transferred into a reinforced concrete foundation. It should be noted that the foundations for the building were not inspected, however a desktop investigation based on copy of the original architectural drawings gives an estimate of their shape and size.

Lateral loads acting on the structure are resisted by the reinforced concrete walls all directions of the building on the lower floor and then transferred into the foundations. During an earthquake the building is expected to behave in a relatively stiff manner due to the size and arrangement of walls in both directions. The geometric nature of the structure is likely to initiate a rocking mechanism under dynamic loading conditions above a certain threshold which helps to dissipate the seismic energy.

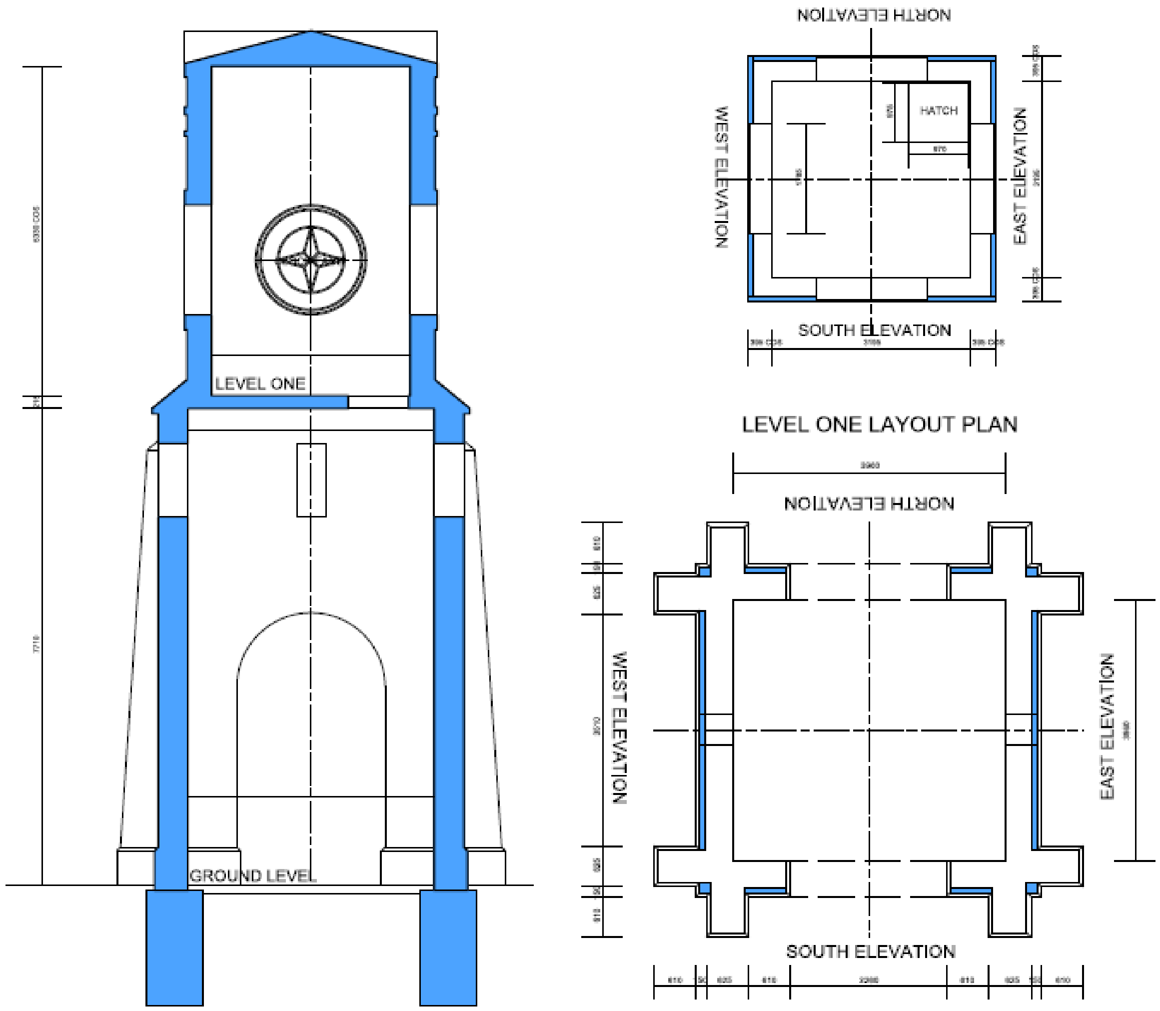


Figure 5b – Elevation and plan view of the structure

6. ASSESSMENT

An inspection of the building was undertaken on the 24th of February 2011 and subsequent inspections following a major aftershocks. The latest inspection compressed a full external and internal inspection and was carried out on the 10th of March 2012.

Inspections did not reveal a civil defence placard on the building, and investigations with the Christchurch City Council revealed no record of the placard of this structure either. The building was assumed to have a green placard in place as no restriction on access were enforced. The clock tower operator had mentioned that the clock stopped operating after the quakes and access to fix the clock was not prohibited.

A large portion of the main structural components of the building were able to be viewed since they are uncovered. The concrete walls are unlined internally but have a stone work veneer on much of the external face limiting direct observation of that side. No inspection of the foundations of the structure was undertaken. Reinforcing bars within the concrete walls were scanned to find their spacing and concrete strength testing was undertaken using a Schmidt Hammer.

The site was assessed for damage, including observing the ground conditions, checking for damage in areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in structural and non- structural elements.

The %NBS score determined for the building is based on the IEP procedure described by the New Zealand Society of Earthquake Engineering (NZSEE) from information obtained by visual observation of the building only as no record drawings are available.

7. DAMAGE ASSESSMENT

General observations are as follows:

- The clock tower is not in close proximity to any buildings.
- Its general condition is not considered to pose an immediate risk to the public.
- Some cracking of the superstructure appears to be related to corrosion of the underlying reinforcement due to water ingress. This is also evident at some of the more "recent" patch repairs where the recent seismic activity has caused damage. Overall the cracking has likely worsened and there is some new cracking which can be attributed to the earthquakes (Figure 7a).

Figure 7a – Vertical crack on the inside of the north face arch

- All four interior clock face rebates and surrounding walls have vertical and horizontal cracks (max size approximately 0.4mm). Some ingress of water was observed indicating that water might be corroding the underlying reinforcing bars and potentially damaging the internal clock equipment (Figure 7b).

Figure 7b – Interior of the north and East faces showing the vertical cracks and moisture inside the clock tower

- The tower appears to be vertical, but there is evidence of some lateral movement due to slab repairs adjacent to the sea wall. The cracks in the slab around the perimeter of the tower are most likely due to the tower foundations moving behaving differently from the shallower slab and step footing (Figure 7c).

Figure 7c – Cracks around the slab around the perimeter of the tower

- An inspection of the top of the tower revealed damage to the concrete roof. A closer inspection of this area indicated that there was no immediate danger from loose concrete. The concrete spalling is due to deterioration of the concrete and corrosion of the reinforcement. Further investigations to determine the extent of the issue and specify suitable repairs is required (Figure 7d).

Figure 7d – Concrete spalling at the roof

- A horizontal crack was observed in the side of the penetration through the 1st floor slab

Figure 7e – Crack in concrete slab at 1st floor

8. CRITICAL STRUCTURAL WEAKNESS

8.1 SHORT COLUMNS

No short columns were observed in the building.

8.2 PLAN AND VERTICAL IRREGULARITY

The building is square with no irregularities in plan or height and therefore for the purposes of the IEP assessment of the building, and determination of the %NBS score, no reduction has been assessed in accordance with the NZSEE guidelines.

8.3 STAIRCASE

There is no staircase in the structure. Access to the 1st floor is provided by ladder.

8.4 ROOF DAMAGE

No critical structural weaknesses were identified in the roof structure.

9. GEOTECHNICAL CONSIDERATION

9.1 SITE DESCRIPTION AND GEOTECHNICAL INVESTIGATION

The site is relatively flat and is situated adjacent to the Sumner beach elevated approximately two meters above mean sea level. No liquefaction was observed from the post 22nd February 2011 Aerial survey (Fig 9a).

Information from Environment Canterbury (ECan) indicates one borelog in close proximity to the clock tower. Classified as N36/0161, the borelog was taken in 1971 to a depth of 4.57m from ground level. The site geology described indicates sand down to 2.7m and blue sand from 2.7 to the end of borelog refusal 4.57m.

It should be noted that the purpose of the boreholes and well logs from Ecan were for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording is variable and may not be representative. Soil strength data was not recorded.

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

Fig 9a. post 22nd February 2011 aerial survey and location of Ecan borelog (indicated by the star)

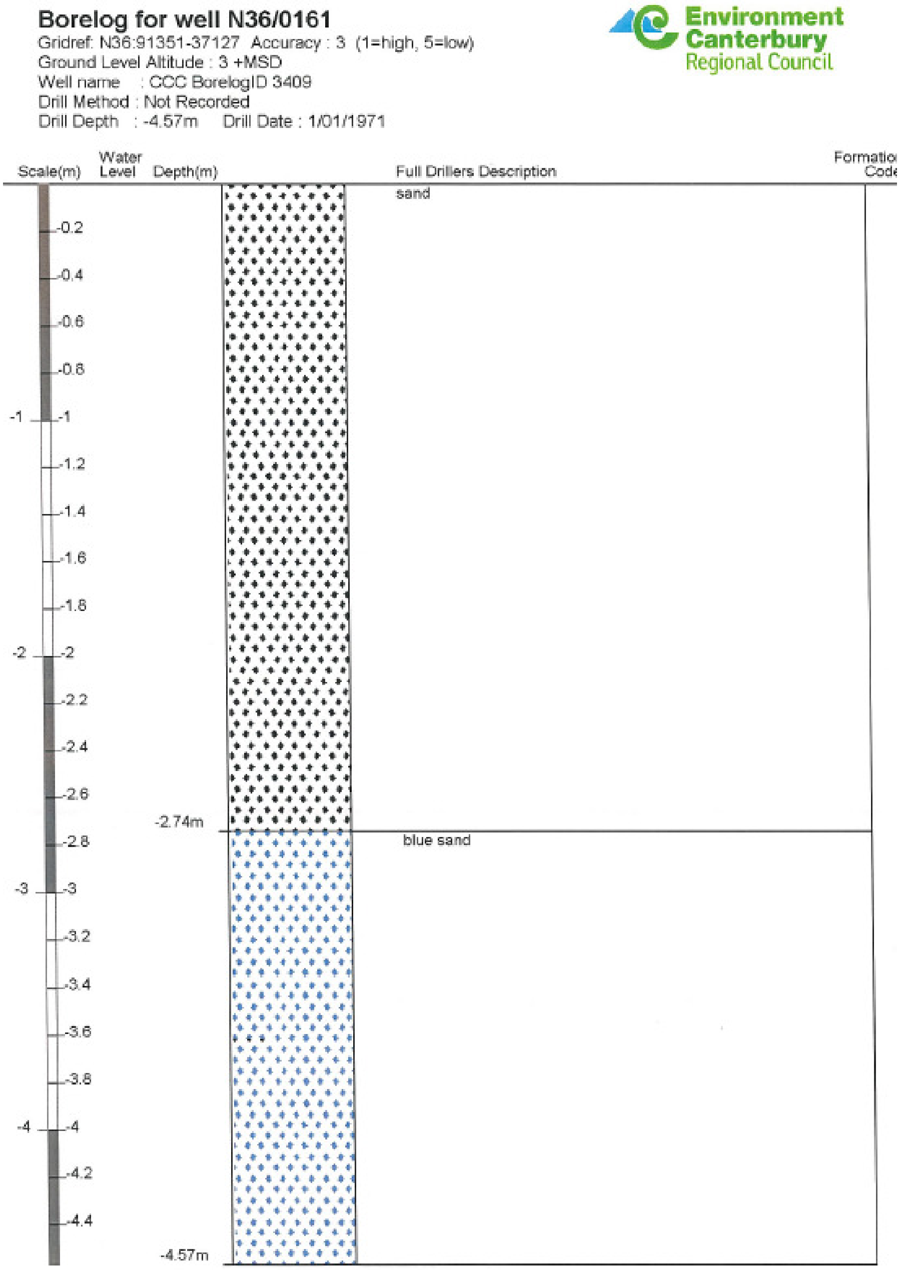


Fig 9a. Location of Ecan borelog investigated (indicated by the star)

The site appears to be situated on hard sand deposits, and potentially gravel at depth. The site has been identified as having moderate liquefaction potential, in particular where sands and/or silts are present. Isolated lithologies may be susceptible to liquefaction; however this is not anticipated to have significant detrimental effects on structures and amenities at the ground surface. It should be noted that little or no liquefaction occurred at or around the site following the major earthquakes.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that an intrusive investigation comprising of a minimum of two piezocone CPT be conducted. A soil class of D (in accordance with NZS 1170.5:2004) has been adopted for the site for the purposes of the detail analysis.

9.2 SLOPE FAILURE AND ROCKFALL POTENTIAL

The site is located within Sumner near to the sea and away from potential rockfall locations. Global slope instability risk is considered negligible and the site is within a green zone as identified by CERA.

10. SURVEY

No level or verticality surveys were conducted for this building. However, the tower appears to be vertical. There is evidence of some lateral movement of the tower due to recent slab repairs adjacent to the sea wall.

11. INITIAL STRUCTURAL CAPACITY ASSESSMENT

11.1 %NBS ASSESSMENT

The building's capacity has been assessed using the Initial Evaluation Procedure based on the information available. The building's capacity excluding critical structural weaknesses, and the capacity of any identified weaknesses are expressed as a percentage of New Building Standard (%NBS) are shown in Table 11a.

Table 11a Indicative Building and Critical Structural Weaknesses Capacities based on the NZSEE Initial Evaluation Procedure (IEP)

Item	%NBS
Building excluding CSW's	25

The building is therefore considered potentially Earthquake Prone as it achieves less than 33% 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.

11.2 DISCUSSION OF RESULTS

The IEP score is generally considered as a guide for the buildings expected performance in a seismic event and is based upon general parameters.

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type founded on Class D soils. The original building was constructed circa 1934 and there was no earthquake design loading standard current at the time (other than for buildings constructed in Wellington).

Since only minor damage was observed and the structure appears to have performed much better than the IEP score would suggest a detailed capacity assessment was undertaken to establish a more refined score.

12. DETAIL CAPACITY ASSESSMENT

12.1 MATERIAL PROPERTIES AND MEMBER STRENGTHS

An assessment of the material concrete strength was evaluated on site using a Schimitt hammer rebounding test. This measures the hardness of the concrete surface and can be calibrated to provide an estimate of the concrete strength. A representative number of tests were performed and it was found that a f'c=30 MPa can be used conservatively, which seems reasonable.

Non intrusive location and depth testing was undertaken for the steel reinforcement using a HILTI –bar scanner. This determined that two layers of longitudinal reinforcing at 550mm spacing and 600mm for the vertical reinforcing bars was within the concrete walls. The scan indicated the depth of the bars but did not provide the bar diameters. It was assumed round R12 were used for the walls as bar of this diameter were exposed where the concrete surface had spalled. The NZTA manual section 5.1 suggests that for this age of structure a tensile strength of 210 MPa can be used for the steel reinforcing. Figure 12.1 shows a plan cross section and the steel bar layout assumed within the walls, based on the site measurements.

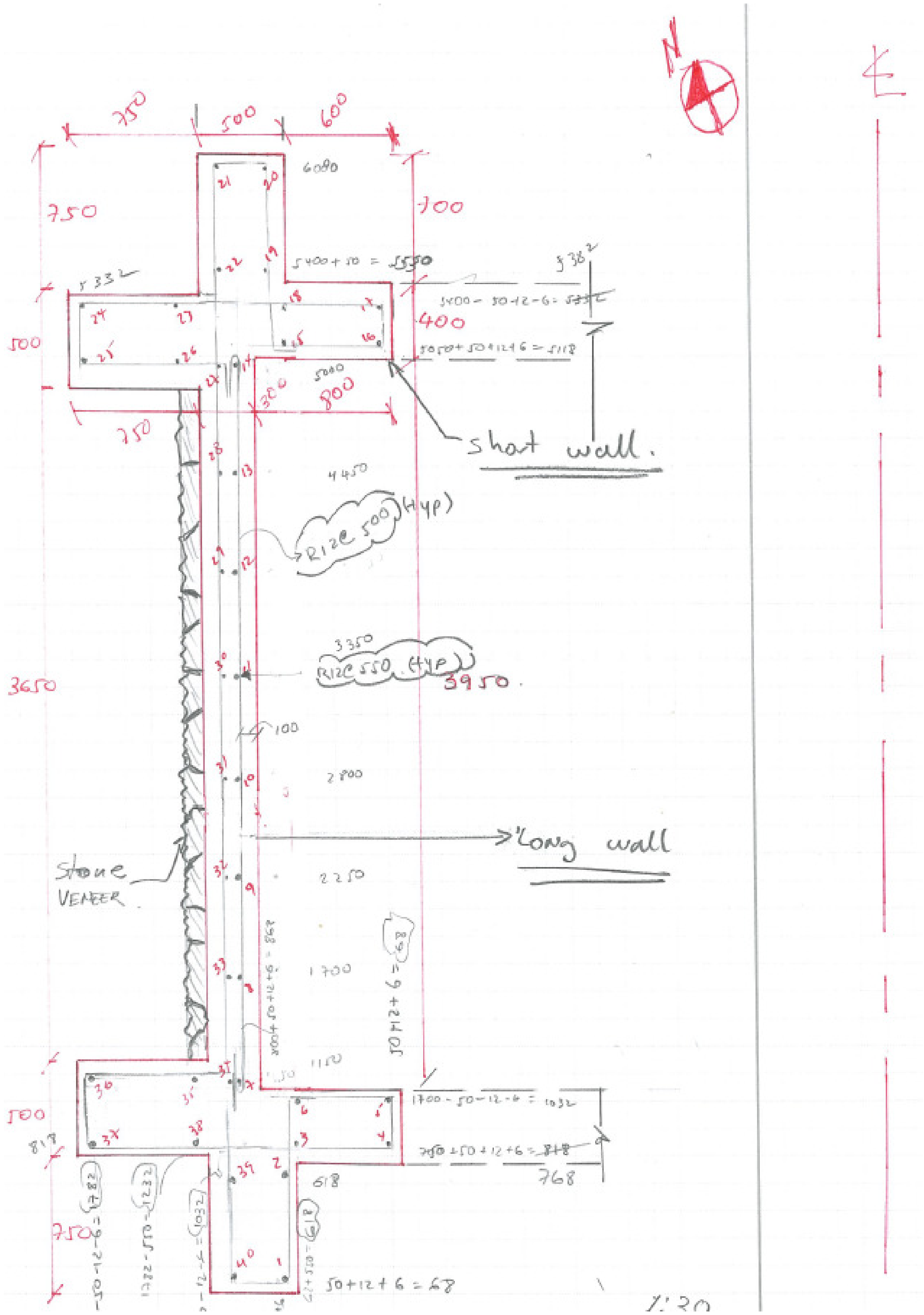


Fig 12. Cross section of walls and reinforcement

12.2 SEISMIC PARAMETERS

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil (assumed)
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011. The increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations (effectively resulting in a reduced % NBS score compared to IEP's undertaken prior to that date).
- Return period factor Ru = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life (note that the clock tower was constructed 78 years ago).
- A structural ductility factor of 1.0 has been assumed in both the long and short direction at both Serviceability limit state (SLS) and ultimate limit state (ULS) of the building based on engineering judgment (construction and detailing age of the structure). This ductility factor can be increased to 1.25 to make the study less conservative.
- The seismic coefficient was calculated as 0.21g and 0.63g for SLS and ULS respectively. This equates that total mass participating in the inertial forces during an earthquake.

12.3 LOADS

Total weight of the structure was calculated as 2528kN including stone veneer cladding and the assumed foundation size. No live load was accounted in the tower as no access to the tower is allowed, and there is a negligible load due to the clock mechanism.

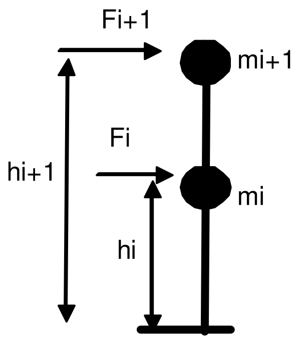
The total seismic weight was calculated as 1592 kN. It is assumed that two long walls resist the seismic forces in the East-West direction (796 kN total shear for each long wall) and four short walls in the North-South direction (398 kN per short wall).

Mass redistribution for the first floor was calculated assuming that the mass of the first floor is the contribution (based on half of the first and second floor height), while the mass contribution of the second floor was based on half of the first floor height. Total floor weight was calculated as 1402 kN and 575 kN for the first and second floor respectively.

12.4 STRUCTURAL ANALYSIS METHOD

Equivalent static method analysis was used to obtain the lateral load distribution according to AS/NZ 1170.5 Earthquake Actions. Total base shear demand at ULS was estimated to be 796 kN and overturning moment of 8442 kNm at the base of the structure for the long walls direction. For the short walls, the total base shear was 398 kN and overturning moment of 4221 kNm. Table 12.a shows the lateral loads applied at each floor.

Table 12a Equivalent static method Procedure



				LONG WALL DIRECTION			
				SLS		ULS	
LEVEL	mi(kN/frame)	H (m)	miH (kNm)	Fi(kN)	FiHi(kNm)	Fi(kN)	FiH(kNm)
2	287.5	13.80	3967.5	124	1710	372	5130
1	701.0	7.80	5467.8	142	1104	425	3312
TOTAL			9435	265	2814	796	8442

				SHORT WALL DIRECTION			
				SLS		ULS	
LEVEL	mi(kN/frame)	H (m)	miH (kNm)	Fi(kN)	FiHi(kNm)	Fi(kN)	FiH(kNm)
2	143.8	13.80	1983.75	124	1710	186	2565
1	350.5	7.80	2733.9	142	1104	212	1656
TOTAL			4718	265	2814	398	4221

12.5 STRUCTURAL ASSESSMENT METHOD

An initial section capacity of the structural components assuming fixed foundation determined that the walls in both direction do not have sufficient capacity to resist the seismic forces generated by lateral load at 100% NBS. In particular the tension forces in the walls due to the high overturning moments were in excess of the wall capacity.

However, since there was no evidence that the walls had been overstressed (by the presence of significant horizontal cracking) and there was pavement cracking around the structure, it suggests that the dynamic response of the clock tower was a rocking motion subjected to uplifting.

Rocking motion is a commonly observed phenomenon with complex and nonlinear behaviour. It can be shown that by allowing rocking to take place in structures, the resulting accelerations and hence forces can be significantly reduced. This approach when applied correctly acts as an effective isolation mechanism for structures against severe ground motion. A rocking structure normally enhances the seismic resistance and their post-earthquake serviceability such as in this case.

A prevalent sentiment amongst practitioners is that although the philosophy behind rocking as a seismic isolation solution is logical, it is prudent not to implement it for high seismic areas until the system performance has been tested by an actual major earthquake in addition to the lack of available guidelines for engineers.

The assessment of the clock tower's capacity has been undertaken in accordance with a paper recently published by the Structural Engineering Society new Zealand in April 2011 "Tentative Seismic Design guidelines for Rocking Structures" By Trevor Kelly.

This paper predicts the rocking uplifting structures by defining a foundation size and calculates spring soil stiffness given the shear modulus and Poisson's ratio from the soil characteristics and assesses the wall rocking strength, seismic displacements, drift and general performance of the walls.

The analysis presented in the paper is applicable for walls with relatively small ductility factors (DF) with a rocking strength (static restoring moment) of the one-quarter or more of the elastic demand (i.e. DF<4.0) which is applicable to this structure.

12.6 %NBS DETAIL ASSESSMENT

An assumed foundation width of 0.9m for the long wall and 1.9m (this is taking into consideration the continuity of the wall acting as a T section) for the short wall directions were used to assess the capacity of the rocking structure. Soil properties were evaluated by a parametric analysis using upper, medium and lower values of dense to medium sand. Shear modulus, Poisson's ratio soil failure stress and ultimate capacities where are given in Table 12.b.

Table 12b Range of soil properties analyzed

Soil	**Type**	**Shear Modulus** (kPa)	**Posson's Ratio**	**ULS (kPa)**
Dense Sand	Upper	80000	0.4	650
Medium Sand	medium	60000	0.35	550
Soft Sand	Low	40000	0.3	450

The building had been evaluated by top drift and shear and are found to be within the drift limit and capacity of the concrete wall with the reinforcement. Table 12c shows the results and compared with the current standards.

It can be seen that as the structure is allowed to rock (i.e increase the drift), the total shear demand decreases and the structure is stiffer in the long walls direction than the short walls.

In terms of drift demand the structure has a total capacity between 66% to 104% of the New building standard (NBS). Similarly, the shear wall capacities are well in excess of the demand. However, the spacing of horizontal and vertical shear reinforcement, and the horizontal shear reinforcement in the walls and the ratio of vertical reinforcement area to gross concrete are of horizontal section do not comply with the NZ3101:2006 Concrete Structure Standards.

Table 12c Indicative Building capacity based on the DEE

LONG WALL				
Soil	**Top drift (%)**	**%NBS** (2.5% limit)	**V* (kN)**	**ϕVn (kN)**
Dense Sand	2.0	104	236	646
Medium Sand	2.6	96	193	646
Soft Sand	3.5	71	61	646
SHORT WALL				
Dense Sand	3.1	81	88	228
Medium Sand	3.5	71	77	228
Soft Sand	3.8	66	65	228

Medium dense sand with ultimate soil capacity of 550 Mpa, with shear modulus of 60000 kPa and passion's ratio of 0.35 is an average value for this assessment and represents the actual conditions for the structure. Therefore, with this assumption, the structure will possess a minimum of 71% NBS.

13 CONCLUSIONS

The damage to the structure during recent seismic activity in Christchurch has caused minor crack damage and open existing damage due to ageing. The building suffered insignificant damage that would not compromise the load resisting capacity of the existing structural systems.

The building is therefore not considered potentially Earthquake Prone as it achieves more than 66% NBS. This score is based on assumed foundation size based on construction/photograhs and geotechnical parameters based on a near by borehole log and seems to be appropriated 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. As a result it is recommended that the building remain in use.

Minor repairs are required to prevent corrosion on the reinforcing steel and concrete spalling.

14 RECOMMENDATIONS

This report recommend as follows:

- The structure requires maintenance to reduce the corrosion of the bars and spalling of the concrete in some parts of the structure.
- If the client want to upgrade the structure to a higher level of the new building standard in order to reduce drift levels it will requires to strengthening foundation by using screw piles in combination of strengthening walls by using FRP or similar products. This will require geotechnical investigations and model complex computer analysis of the structure.
- Repair slab and steps

15. LIMITATIONS

15.1 GENERAL

This report has been prepared subject to the following limitations and assumptions:

- Consented drawings of the building were not available.
- The information contained in this report has been obtained from visual inspections of the building, non destructive testing and available literature review of model analysis.
- The foundations of the building were not inspected.
- 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.

It is noted that this report has been prepared at the request of the Client and is intended to be used for their purposes only. The author accepts no responsibility for any other party or person who relies on the information contained in this report.

15.2 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 the Client 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. The author accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal and limited desktop study. 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 the author. The author accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

16 APPENDIX A INITIAL EVALUATION PROCEDURE

etailed Engineering Evaluation Summary Data

V1.11

ocation

Building Name:	New Brighton Clock Tower
Building Address: Unit / No: / Street	147A Esplanade Road
Legal Description:	Clock Tower
GPS south: Degrees / Min / Sec	
GPS east: Degrees / Min / Sec	
Building Unique Identifier (CCC):	
Reviewer:	Lloyd Greenfield
CPEng No:	78784
Company:	CCC
Company project number:	
Company phone number:	03 9418756
Date of submission:	03/12/2013
Inspection Date:	10/03/2012
Revision:	B
Is there a full report with this summary?	yes

te

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

ilding

No. of storeys above ground:	2 (single storey = 1)
Ground floor split?	no
Storeys below ground	0
Foundation type:	strip footings
Building height (m):	13.80
Floor footprint area (approx):	25
Age of Building (years):	80
Ground floor elevation (Absolute) (m):	
Ground floor elevation above ground (m):	
if Foundation type is other, describe:	
height from ground to level of uppermost seismic mass (for IEP only) (m):	13.8
Date of design:	Pre 1935
Strengthening present?	no
If so, when (year)?	
And what load level (%g)?	
Brief strengthening description:	
Use (ground floor):	public
Use (upper floors):	other (specify)
Use notes (if required):	clock maintenance only
Importance level (to NZS1170.5):	IL2

avity Structure

Gravity System:	load bearing walls		
Roof:	concrete	slab thickness (mm)	200
Floors:	concrete flat slab	slab thickness (mm)	210
Beams:	none	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	load bearing concrete	#N/A	

teral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	concrete shear wall		note total length of wall at ground (m):	6.15
Ductility assumed, μ:	1.00		wall thickness (m):	0.355
Period along:	0.67	0.68 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:	non-ductile concrete moment frame		note typical bay length (m)	3
Ductility assumed, μ:	1.00			
Period across:	1.13	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

leave blank if not relevant

north (mm):	
east (mm):	
south (mm):	
west (mm):	

Non-structural elements

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

Available documentation

Architectural	partial	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:	slight	notes (if applicable):	ageing cracks due to construction defects

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:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	

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

Recommendations

Level of repair/strengthening required:	minor non-structural	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	Access required for clock only

Along	Assessed %NBS before:	100%	### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Refer to DEE quantitative Assessment (66%
	Assessed %NBS after:	100%			
Across	Assessed %NBS before:	66%	### %NBS from IEP below		
	Assessed %NBS after:	66%			

EP **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): Pre 1935

h_n from above: 13.8m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.67	1.13
(%NBS)nom from Fig 3.3:		

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
Note 2: for RC buildings designed between 1976-1984, use 1.2
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final (%NBS)nom:	**0%**	**0%**

2.2 Near Fault Scaling Factor

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

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =k_μ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, (NBS%)b = (%NBS)nom x A x B x C x D x E **%NBSb:** **#DIV/0!** **#DIV/0!**

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

3.1. Plan Irregularity, factor insignificant 1

3.2. Vertical irregularity, Fainsignificant 1

3.3. Short columns, Factor C insignificant 1

3.4. Pounding potential Pounding effect D1, from Table to right
Height Difference effect D2, from Table to right

Therefore, Factor D: 0

3.5. Site Characteristics insignificant

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	1.5	1.5
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)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

17 APPENDIX B SPREADSHEET CALCULATIONS

Long Wall hard soil

WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	0.58	Ve (kN)	355		
L (m)	4.65	V_E (kN)	1105	Selected	0.9		T upper limit (sec)		0.439
W_D (kN)	1754	V_R (kN)	126	DF	8.8				
Qc (kPa)	650	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.150		G =	80000	Kend	910667			
L2 (m)	0.725		v =	0.4	Kmid	97333			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.075	-2.250	136600	344	26	691538			
2	0.513	-1.813	70567	178	91	231823			
3	1.238	-1.088	70567	178	220	83456			
4	1.963	-0.363	70567	178	349	9273			
5	2.688	0.363	70567	178	477	9273			
6	3.413	1.088	70567	178	606	83456			
7	4.138	1.813	70567	178	735	231823			
8	4.575	2.250	136600	344	1573	691538			
		0	696600	1754	4078	2032178			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	3.00			Cm	1.0	Fy (kN)	126	Cy	0.072

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	0.439	0.630	8.772	3.852	8.772
2	2.146	0.212	2.946	1.294	0.603
3	1.720	0.267	3.714	1.631	0.948
4	1.675	0.273	3.806	1.672	0.998
5	1.674	0.274	3.810	1.673	1.000
6	1.673	0.274	3.810	1.673	1.000
7	1.673	0.274	3.810	1.673	1.000
8	1.673	0.274	3.810	1.673	1.000
9	1.673	0.274	3.810	1.673	1.000

SEISMIC DISPLACEMENT				TOP WALL DISPLACEMENT					
2 upper bound (mm)		190		2 upper bound (mm)		229			
2 lower bound (mm)		0		2 lower bound (mm)		0			
DUCTILITY FACTOR				DYNAMIC AMPLIFICATION EFFECT					
DF =	8.8			Avn	0.1	min limit	1.88	max lim	2.5
TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
2kdm	1.2	drift upper limit(%)		2.0	OK	drift lower limit (%)		0.0	OK
Therefore the foundation width of			0.9	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	236	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						212	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

Long Wall medium soil

WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	0.69	Ve (kN)	355		
L (m)	4.65	V_E (kN)	1105	Selected	0.9		T upper limit (sec)		0.528
W_D (kN)	1754	V_R (kN)	84	DF	13.1				
Qc (kPa)	550	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.150		G =	60000	Kend	630462			
L2 (m)	0.725		v =	0.35	Kmid	67385			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.075	-2.250	94569	344	26	478757			
2	0.513	-1.813	48854	178	91	160493			
3	1.238	-1.088	48854	178	220	57777			
4	1.963	-0.363	48854	178	349	6420			
5	2.688	0.363	48854	178	477	6420			
6	3.413	1.088	48854	178	606	57777			
7	4.138	1.813	48854	178	735	160493			
8	4.575	2.250	94569	344	1573	478757			
		0	482262	1754	4078	1406893			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	3.54			Cm	1.0	Fy (kN)	84	Cy	0.048

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	0.528	0.620	12.894	6.806	12.894
2	3.667	0.101	2.108	1.113	0.303
3	2.390	0.190	3.941	2.080	0.871
4	2.235	0.204	4.230	2.233	0.999
5	2.234	0.204	4.232	2.234	1.000
6	2.234	0.204	4.232	2.234	1.000
7	2.234	0.204	4.232	2.234	1.000
8	2.234	0.204	4.232	2.234	1.000
9	2.234	0.204	4.232	2.234	1.000

SEISMIC DISPLACEMENT				TOP WALL DISPLACEMENT					
≅ upper bound (mm)		253		≅ upper bound (mm)		303			
≅ lower bound (mm)		0		≅ lower bound (mm)		0			
DUCTILITY FACTOR				DYNAMIC AMPLIFICATION EFFECT					
DF =	12.9			Avn	0.1	min limit	2.29	max lim	2.5
TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
≅kdm	1.2	drift upper limit(%)		2.6	NG	drift lower limit (%)		0.0	OK
Therefore the foundation width of			0.9	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	193	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						173	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

Long Wall soft soil

WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	0.84	Ve (kN)	355		
L (m)	4.65	V_E (kN)	1105	Selected	0.9		T upper limit (sec)		0.671
W_D (kN)	1754	V_R (kN)	24	DF	45.4				
Qc (kPa)	450	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.150		G =	40000	Kend	390286			
L2 (m)	0.725		v =	0.3	Kmid	41714			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.075	-2.250	58543	344	26	296373			
2	0.513	-1.813	30243	178	91	99353			
3	1.238	-1.088	30243	178	220	35767			
4	1.963	-0.363	30243	178	349	3974			
5	2.688	0.363	30243	178	477	3974			
6	3.413	1.088	30243	178	606	35767			
7	4.138	1.813	30243	178	735	99353			
8	4.575	2.250	58543	344	1573	296373			
		0	298543	1754	4078	870933			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	4.33			Cm	1.0	Fy (kN)	24	Cy	0.014
			UPPER BOUND						
	Ti	C(Ti)	Re	Te	Te/Ti				
1	0.671	0.550	39.588	26.558	39.588				
2	13.614	0.067	4.795	3.216	0.236				
3	8.415	0.067	4.795	3.216	0.382				
4	5.816	0.067	4.795	3.216	0.553				
5	4.516	0.067	4.795	3.216	0.712				
6	3.866	0.091	6.565	4.404	1.139				
7	4.135	0.079	5.724	3.840	0.929				
8	3.988	0.085	6.114	4.102	1.029				
9	4.045	0.083	5.954	3.994	0.988				
SEISMIC DISPLACEMENT				TOP WALL DISPLACEMENT					
≅ upper bound (mm)		336		≅ upper bound (mm)		403			
≅ lower bound (mm)		0		≅ lower bound (mm)		0			
DUCTILITY FACTOR				DYNAMIC AMPLIFICATION EFFECT					
DF =	39.6			Avn	0.1	min limit	4.96	max lim	2.5
TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
≅kdm	1.2	drift upper limit(%)		3.5	NG	drift lower limit (%)		0.0	OK
Therefore the foundation width of			0.9	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	61	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						55	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

Short Wall hard soil

WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	0.73	Ve (kN)	71		
L (m)	1.85	V_E (kN)	552	Selected	1.925		T upper limit (sec)		0.742
W_D (kN)	877	V_R (kN)	44	DF	12.6				
Qc (kPa)	650	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.321		G =	80000	Kend	910667			
L2 (m)	0.201		v =	0.4	Kmid	97333			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.160	-0.765	292172	365	59	170800			
2	0.422	-0.503	19602	24	10	4969			
3	0.623	-0.302	19602	24	15	1789			
4	0.824	-0.101	19602	24	20	199			
5	1.026	0.101	19602	24	25	199			
6	1.227	0.302	19602	24	30	1789			
7	1.428	0.503	19602	24	35	4969			
8	1.690	0.765	292172	365	617	170800			
		0	701956	877	811	355513			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	0.70			Cm	1.0	Fy (kN)	44	Cy	0.050
			UPPER BOUND						
	Ti	C(Ti)	Re	Te	Te/Ti				
1	0.742	0.509	10.193	7.568	10.193				
2	4.155	0.079	1.577	1.171	0.282				
3	2.663	0.170	3.402	2.526	0.949				
4	2.595	0.174	3.484	2.587	0.997				
5	2.591	0.174	3.489	2.590	1.000				
6	2.591	0.174	3.489	2.591	1.000				
7	2.591	0.174	3.489	2.591	1.000				
8	2.591	0.174	3.489	2.591	1.000				
9	2.591	0.174	3.489	2.591	1.000				
SEISMIC DISPLACEMENT				TOP WALL DISPLACEMENT					
Ξ upper bound (mm)		291		Ξ upper bound (mm)		349			
Ξ lower bound (mm)		0		Ξ lower bound (mm)		0			
DUCTILITY FACTOR				DYNAMIC AMPLIFICATION EFFECT					
DF =	10.2			Avn	0.1	min limit	2.02	max lim	2.5
TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
Ξkdm	1.2	drift upper limit(%)		3.0	NG	drift lower limit (%)		0.0	OK
Therefore the foundation width of			1.9	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	88	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						199	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

Short Wall medium soil

WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	0.86	Ve (kN)	71		
L (m)	1.85	V_E (kN)	552	Selected	1.925		T upper limit (sec)		0.892
W_D (kN)	877	V_R (kN)	39	DF	14.2				
Qc (kPa)	550	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.321		G =	60000	Kend	630462			
L2 (m)	0.201		v =	0.35	Kmid	67385			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.160	-0.765	202273	365	59	118246			
2	0.422	-0.503	13571	24	10	3440			
3	0.623	-0.302	13571	24	15	1238			
4	0.824	-0.101	13571	24	20	138			
5	1.026	0.101	13571	24	25	138			
6	1.227	0.302	13571	24	30	1238			
7	1.428	0.503	13571	24	35	3440			
8	1.690	0.765	202273	365	617	118246			
		0	485969	877	811	246124			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	0.83			Cm	1.0	Fy (kN)	39	Cy	0.044

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	0.892	0.443	9.966	8.893	9.966
2	4.893	0.067	1.499	1.337	0.273
3	3.115	0.141	3.166	2.826	0.907
4	2.970	0.152	3.412	3.045	1.025
5	3.008	0.149	3.359	2.997	0.996
6	3.002	0.150	3.368	3.005	1.001
7	3.004	0.149	3.365	3.003	1.000
8	3.003	0.150	3.366	3.004	1.000
9	3.004	0.150	3.366	3.003	1.000

SEISMIC DISPLACEMENT			TOP WALL DISPLACEMENT				
Ξ upper bound (mm)	335		Ξ upper bound (mm)	402			
Ξ lower bound (mm)	0		Ξ lower bound (mm)	0			
DUCTILITY FACTOR			DYNAMIC AMPLIFICATION EFFECT				
DF =	10.0		Avn	0.1	min limit	2.00	max lim	2.5

TORSIONAL INCREASE IN DISPLACEMENT							
not including in this spreadsheet							
ASSESSMENT OF PERFORMANCE							
Ξ_{kdm}	1.2	drift upper limit(%)	3.5	NG	drift lower limit (%)	0.0	OK
Therefore the foundation width of		1.9	is satisfactory provided the walls design shears are satisfactory				
Vx (kN)	78	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag				175	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT							

Short Wall soft soil

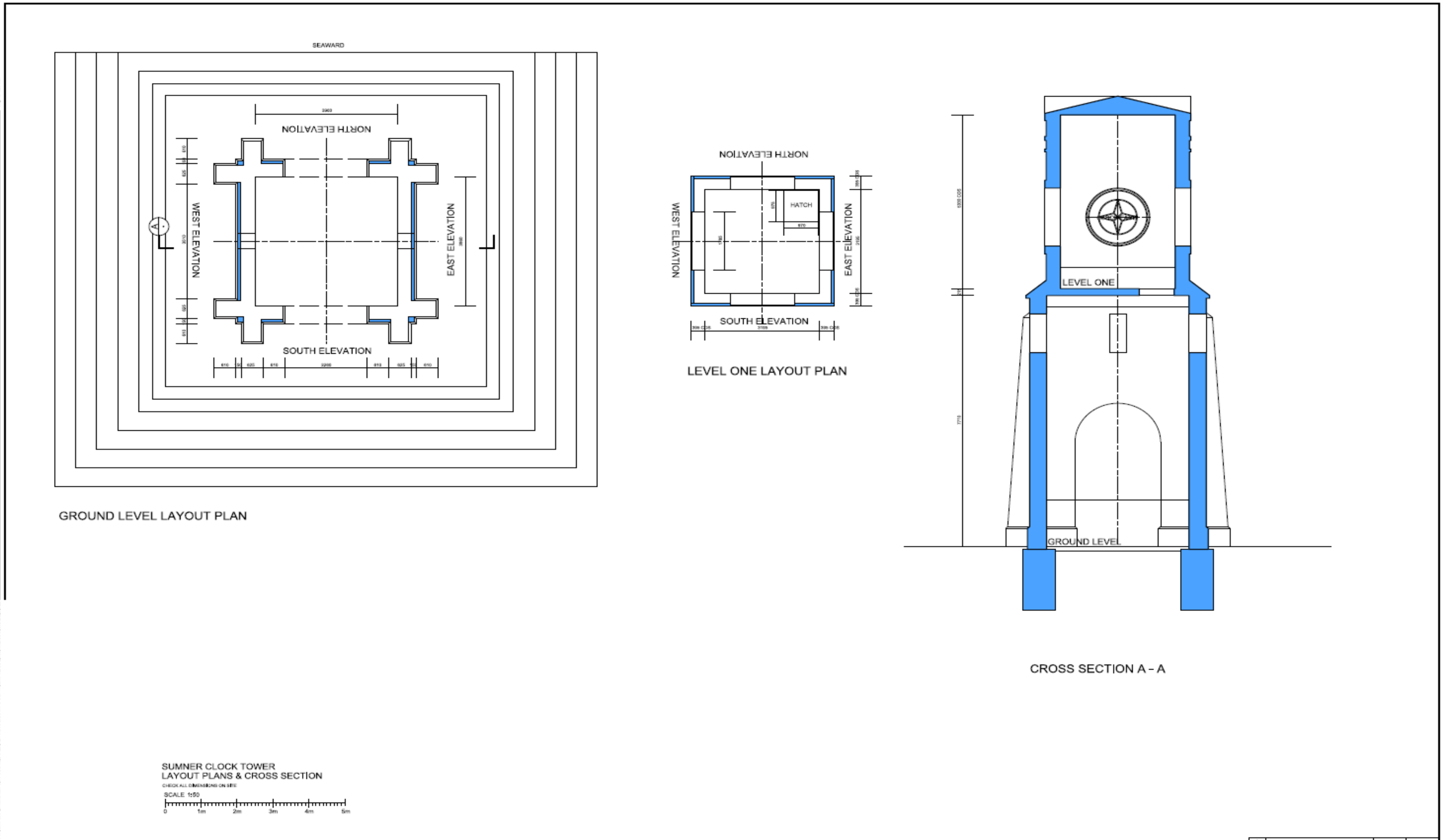
WALL GEOMETRY									
H (m)	13.80	Co	1.2	B (m)	1.05	Ve (kN)	71		
L (m)	1.85	V_E (kN)	552	Selected	1.925		T upper limit (sec)		1.134
W_D (kN)	877	V_R (kN)	32	DF	17.3				
Qc (kPa)	450	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.321		G =	40000	Kend	390286			
L2 (m)	0.201		v =	0.3	Kmid	41714			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.160	-0.765	125217	365	59	73200			
2	0.422	-0.503	8401	24	10	2129			
3	0.623	-0.302	8401	24	15	767			
4	0.824	-0.101	8401	24	20	85			
5	1.026	0.101	8401	24	25	85			
6	1.227	0.302	8401	24	30	767			
7	1.428	0.503	8401	24	35	2129			
8	1.690	0.765	125217	365	617	73200			
		0	300838	877	811	152363			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	1.01			Cm	1.0	Fy (kN)	32	Cy	0.036

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	1.134	0.378	10.368	11.759	10.368
2	6.447	0.067	1.828	2.073	0.322
3	4.260	0.075	2.061	2.338	0.549
4	3.299	0.126	3.461	3.925	1.190
5	3.612	0.104	2.863	3.247	0.899
6	3.430	0.116	3.175	3.601	1.050
7	3.516	0.109	3.000	3.402	0.968
8	3.459	0.113	3.112	3.529	1.020
9	3.494	0.111	3.035	3.442	0.985

SEISMIC DISPLACEMENT			TOP WALL DISPLACEMENT				
Z upper bound (mm)	335		Z upper bound (mm)	402			
Z lower bound (mm)	0		Z lower bound (mm)	0			
DUCTILITY FACTOR			DYNAMIC AMPLIFICATION EFFECT				
DF =	10.4		Avn	0.1	min limit	2.04	max lim	2.5

TORSIONAL INCREASE IN DISPLACEMENT							
not including in this spreadsheet							
ASSESSMENT OF PERFORMANCE							
Z_{kdm}	1.2	drift upper limit(%)	3.5	NG	drift lower limit (%)	0.0	OK
Therefore the foundation width of		1.9	is satisfactory provided the walls design shears are satisfactory				
Vx (kN)	65	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag				147	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT							

APPENDIX C DRAWINGS



Christchurch City Council
CAPITAL PROGRAMME GROUP

DATUM C.D.D.
BENCH MK.
RL
SURVEY
SURVEY LB
DRAWN. REF.
SAP WBS

NAME SIGNED DATE
DESIGNED Design Name
DES. REVIEW Review Name
DRAWN MAR 2012
DRW. CHECK Check Name
FILE LOCATION

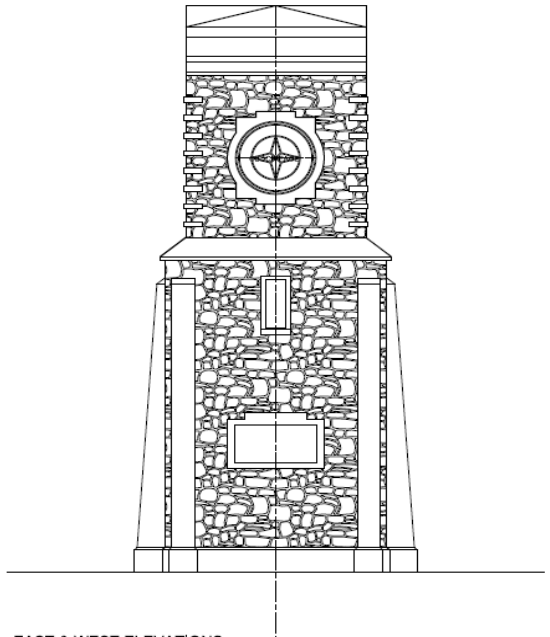
APPROVED
FOR TENDER
DATE SIGNED
FOR CONSTRUCTION
DATE SIGNED

CONSULTANT
CONSULTANT PROJECT REF.
CONSULTANT FILE REF.

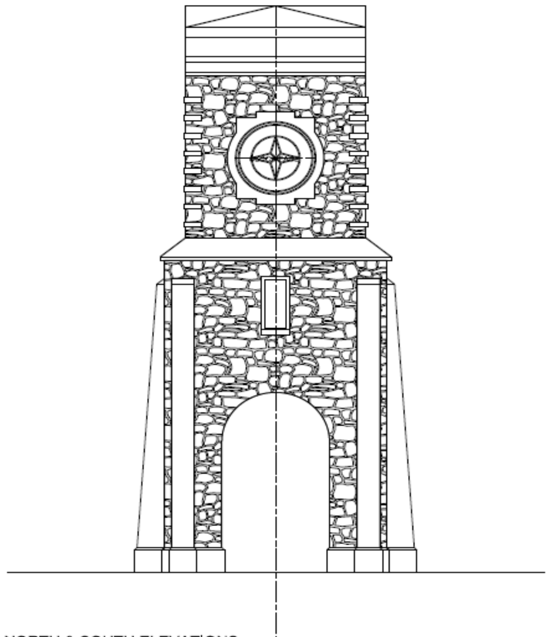
PROJECT TITLE
SUMNER CLOCK TOWER

DRAWING TITLE
PLAN & X SECTION

CONTRACT NUMBER ?
CAD DRAWING FILE REF. AD157301
ORIGINAL SHEET SIZE A1
SCALES 1: 50
CPG PROJECT FILE NUMBER CP501638
SHEET 1 OF 1



EAST & WEST ELEVATIONS



NORTH & SOUTH ELEVATIONS

SUMNER CLOCK TOWER
ELEVATIONS
CHECK ALL DIMENSIONS ON SITE

SCALE 1:50

0 1m 2m 3m 4m 5m

ISSUE	AMENDMENTS	SIGNED	DATE
A			

Christchurch City Council
CAPITAL PROGRAMME GROUP

DATUM	C.D.D.		NAME	SIGNED	DATE
BENCH MK.		DESIGNED	Design Name		
RL		DES. REVIEW	Review Name		
SURVEY		DRAWN	MJKITT		MAR 2012
SURVEY LB		DRW. CHECK	Check Name		
		FILE LOCATION			
DRAWN. REF.		[illegible]			
SAP WBS	[illegible]	PRINTED ON [illegible] BY [illegible]			

APPROVED
FOR TENDER
DATE SIGNED
FOR CONSTRUCTION
DATE SIGNED

CONSULTANT

CONSULTANT PROJECT REF.
CONSULTANT FILE REF.

PROJECT TITLE
SUMNER CLOCK TOWER

DRAWING TITLE
ELEVATIONS

CONTRACT NUMBER ?
CAD DRAWING FILE REF. AD157301
ORIGINAL SHEET SIZE A1
SCALES 1: 50
CPG PROJECT FILE NUMBER CP501638
SHEET 2 OF 2