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# Edmonds Phone Booth Poplars Reserve
# PRK 1927 BLDG 002 EQ2

Detailed Engineering Evaluation

Qualitative Report

Version FINAL

Corner of Madras Street, Chester Street and Oxford Terrace

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

# Edmonds Phone Booth Poplars Reserve
# PRK 1927 BLDG 002 EQ2

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

Corner of Madras Street, Chester Street and Oxford Terrace

Christchurch City Council

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**Date**
12th December 2013

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# Qualitative Report Summary

**Edmonds Phone Booth Poplars Reserve**

**PRK 1927 BLDG 002 EQ2**

**Detailed Engineering Evaluation**

**Qualitative Report - SUMMARY**

**Version FINAL**

**Corner of Madras Street, Chester Street and Oxford Terrace**

### Background

This is a summary of the Qualitative report for 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 and visual inspections on 14 September 2012.

### Building Description

The Edmonds Phone Booth in Poplars Reserve is located on the corner between Madras Street, Chester St and Oxford Terrace in Christchurch. The date of construction is 1929 as indicated by a plaque on the interior of the building. The sole use of the building is a phone booth. The phone booth was restored by Telecom in 1999.

The small building consists of a relatively thick (approximately 650mm) reinforced concrete roof slab with a carved stone veneer supported by unreinforced stone masonry walls in both the longitudinal and transverse directions of the building. The stone masonry walls are shown in Photograph 5. The window frames are constructed from of carved stone. The walls are likely to be supported on concrete strip footings. The floor consists of a concrete slab-on-grade.

### Key Damage Observed

Some cracking in the reinforced concrete roof was observed. The cracking has occurred along joints in the carved stone veneer in sections of the roof that overhang the supporting stone masonry walls.

### Critical Structural Weaknesses

The layout of the lateral load resisting stone masonry walls in the building is irregular. There is a thick stone masonry wall on the north-eastern side of the building with two perpendicular walls extending from it to form the phone booth enclosure. There is no wall along the south-western side of the building in the transverse direction. The resultant stiffness eccentricity may induce torsion in the building during an earthquake. The stiffness eccentricity has been assessed as a ‘significant’ plan irregularity Critical Structural Weakness in accordance with NZSEE guidelines.

## Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the building has been assessed to be in the order of 5% NBS and post-earthquake capacity also in the order of 5% NBS. The buildings post-earthquake capacity excluding critical structural weaknesses is in the order of 8% NBS.

The building has been assessed to have a seismic capacity in the order of 5% NBS and is therefore potentially Earthquake Prone.

## Recommendations

It is recommended that a quantitative assessment of the building be undertaken to determine the seismic capacity and to develop potential strengthening concepts.

# 1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Edmonds Phone Booth in Poplars Reserve.

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 have not been made available. The building description below is based on our visual inspections only.

# 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 |
|---|---|---|---|---|
| Low Risk Building | A or B | Low | Above 67 | Acceptable (improvement may be desirable) |
| Moderate Risk Building | B or C | Moderate | 34 to 66 | Acceptable legally. Improvement recommended |
| High Risk Building | D or E | High | 33 or lower | Unacceptable (Improvement |

| Improvement of Structural Performance | |
|---|---|
| Legal Requirement | NZSEE Recommendation |
| 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 |
| | Not recommended. Acceptable only in exceptional circumstances |
| 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

The Edmonds Phone Booth in Poplars Reserve is located on the corner between Madras Street, Chester St and Oxford Terrace in Christchurch. The date of construction is 1929 as indicated by a plaque on the interior of the building. The sole use of the building is a phone booth. The phone booth was restored by Telecom in 1999.

The small building consists of a relatively thick (approximately 650mm) reinforced concrete roof slab with a carved stone veneer supported by unreinforced stone masonry walls in both the longitudinal and transverse directions of the building. The stone masonry walls are shown in Photograph 5. The window frames are constructed from of carved stone. The walls are likely to be supported on concrete strip footings. The floor consists of a concrete slab-on-grade.

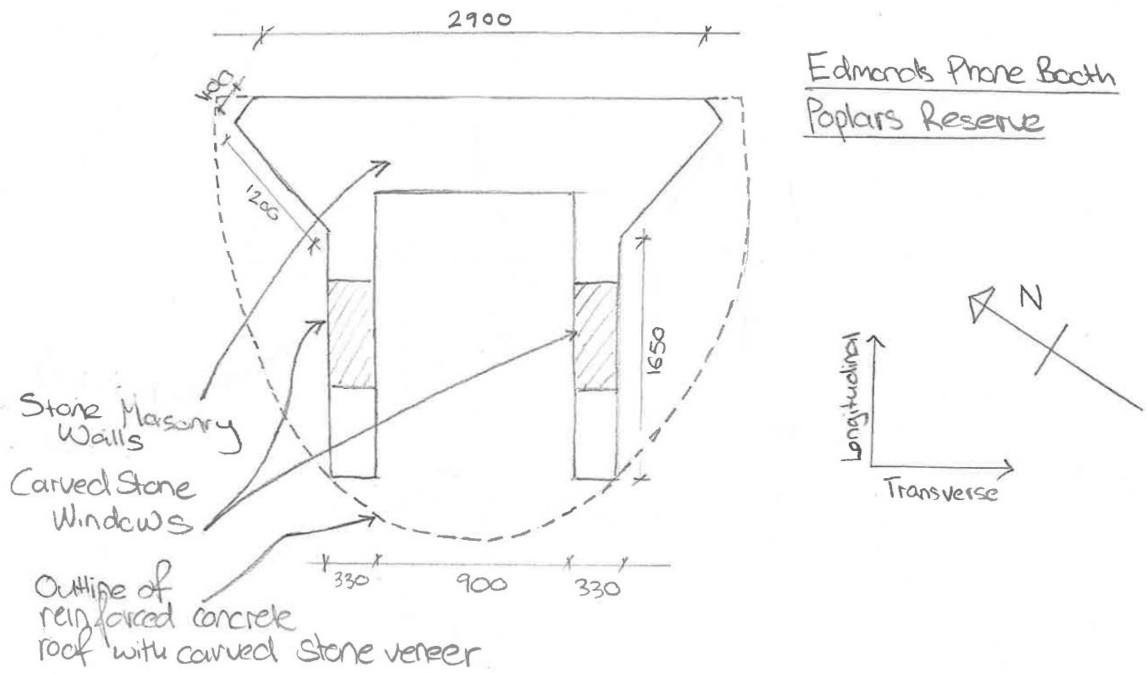



**Figure 2 Plan Sketch Showing Key Structural Elements**

The building is approximately 3.4 m in length by 2.4 m in width with a height of 2.85 m. The building occupies a footprint of approximately 8 $m^2$. The relatively flat site is approximately 25m south of the Avon River.

No plans of the building were made available.

## 4.2 Gravity Load Resisting System

Gravity loads acting on the building are resisted by load bearing stone masonry walls. Gravity loads from the reinforced concrete roof slab are transferred to the stone masonry walls. The gravity loads are transferred through the stone masonry walls to the concrete strip footings where they are distributed into the ground. Floor gravity loads are transferred through the concrete slab to the underlying ground.

## 4.3 Lateral Load Resisting System

Lateral loads are resisted primarily by panel action of the stone masonry walls in both the transverse and longitudinal directions of the building. Given the thickness of the roof slab and the corresponding large seismic mass at roof level, lateral seismic loads are likely to be significant. The walls may be subject to additional forces as a result of torsion, due to the stiffness eccentricity caused by the larger wall on the north-eastern side of the building.

Diaphragm action of the 650mm thick reinforced concrete roof slab distributes in-plane seismic forces in the roof to the lateral load resisting stone masonry walls. The stone masonry walls then transfer lateral loads to the foundations through in-plane shear and bending.

# 5. Assessment

An inspection of the building was undertaken on the 14 September 2012. Both the interior and exterior of the building were inspected.

The inspection consisted of scrutinising the building to determine the structural systems and likely behaviour of the building 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 building in both structural and non-structural elements.

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 building.

## 5.1 Damage Assessment

### 5.1.1 Surrounding Buildings

The clock tower to the north-east of the phone booth in Poplars Reserve has suffered earthquake damage. The clock tower has been partially deconstructed. The top section of the clock tower has been removed and placed on the ground. The removed section can be seen in the background in Photograph 1. The clock tower was constructed at the same time and from the same materials as the phone booth.

### 5.1.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

Some cracking in the reinforced concrete roof was observed. The cracking (shown in Photograph 2) has occurred along joints in the carved stone veneer in the sections of the roof that overhang the supporting stone masonry walls.

### 5.1.3 Ground Damage

No ground damage was observed in the immediate vicinity of the building or in the area surrounding the building.

### 5.1.4 Floor Level Survey

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

# 6. Critical Structural Weakness

## 6.1 Short Columns

No significant short columns are present in the structure.

## 6.2 Lift Shaft

The building does not contain a lift shaft.

## 6.3 Roof

There is no critical structural weakness in the roof structure. The large concrete roof slab is likely to have sufficient stiffness to transfer in-plane seismic forces in the roof structure to the supporting walls. Given the thickness of the roof slab and the large seismic mass, seismic inertia forces associated with the roof are likely to be significant.

## 6.4 Staircases

The building does not contain a staircase.

## 6.5 Site Characteristics

Following the geotechnical appraisal it was found that the site has a moderate susceptibility for liquefaction. For the purposes of the IEP assessment of the building and the determination of the %NBS score, the effects of soil liquefaction on the performance of a building of this type and size has been assessed as an 'insignificant' site characteristic in accordance with the NZSEE guidelines.

## 6.6 Plan Irregularity

The layout of the lateral load resisting stone masonry walls in the building is irregular. There is a thick stone masonry wall on the north-eastern side of the building with two perpendicular walls extending from it to form the phone booth enclosure. There is no wall along the south-western side of the building in the transverse direction. The resultant stiffness eccentricity may induce torsion in the building during an earthquake. The stiffness eccentricity has been assessed as a 'significant' plan irregularity Critical Structural Weakness in accordance with NZSEE guidelines.

# 7. Geotechnical Consideration

## 7.1 Site Description

The site is situated in the Central Business District, in Christchurch. It is relatively flat at approximately 5m above mean sea level. It is approximately 25m south of the Avon River, and 7.5km west of the coast (Pegasus Bay).

## 7.2 Published Information on Ground Conditions

### 7.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by:

- Dominantly sand and silt overbank deposits, being alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age.

Brown & Weeber (1992) indicates that groundwater is likely within 1m of the ground surface.

### 7.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that eight boreholes with lithographic logs are located within 200m of the site (see Table 2).

These indicate the area is underlain by interbedded layers of sand silt and sandy gravels. Varying amounts of clay and peat are also indicated to be present.

**Table 2 ECan Borehole Summary**

| Bore Name | Log Depth | Groundwater | Distance & Direction from Site |
|---|---|---|---|
| M35/2162 | 47.6m | 1.4m bgl | 130m SW |
| M35/1974 | 30.4m | Artesian | 150m S |
| M35/5873 | 21.0m | Not Recorded | 170m W |
| M35/2210 | 128.0m | 10.0m bgl | 170m SW |
| M35/1913 | 128.0m | 10.0m bgl | 180m SW |
| M35/5874 | 15.0m | Not Recorded | 180m SW |
| M35/8726 | 18.5m | 2.5m bgl | 190m SW |
| M35/2162 | 47.6m | 1.37m | 190m SW |

It should be noted that 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.

[1] Brown, L. J. & Weeber, J.H. (1992): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. IGNS Limited: Lower Hutt.

### 7.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Central Business District[2]. Two investigation points were undertaken within 200m of the site, as summarised below in Table 3.

**Table 3 EQC Geotechnical Investigation Summary Table**

| Bore Name | Orientation from Site | Depth (m bgl) | Log Summary |
|---|---|---|---|
| CPT CBD 42 | 130m SE | 0.0 – 1.5 | Pre-drilled |
| | | 1.5 – 5.0 | SAND mixtures, loose to medium dense |
| | | | (GW at 2.0m bgl) |
| CPT CBD OC01 | 150m S | 0.0 – 1.5 | Fill |
| | | 1.5 – 4.5 | SAND mixtures, loose to medium dense |
| | | 4.5 – 6.0 | SILT mixture, loose |
| | | 6.0 – 10.0 | SAND, dense |
| | | | (GW at 7.0m bgl) |

Initial observations of the CPT results indicate the soils are fine to medium grained, and are loose to dense.

### 7.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land is expected to perform in future earthquakes.

The site has been categorised as “N/A – Urban Non-residential”. However, neighbouring residential properties have been classified TC3 (blue) zone[3]. This means that moderate to significant land damage from liquefaction is possible in future significant earthquakes.

### 7.2.5 Post February Aerial Photography

Aerial photography taken following the 14-15 June 2011 earthquake shows signs of liquefaction adjacent to the site, as shown in Figure 3. There are no signs of liquefaction fro previous earthquakes.

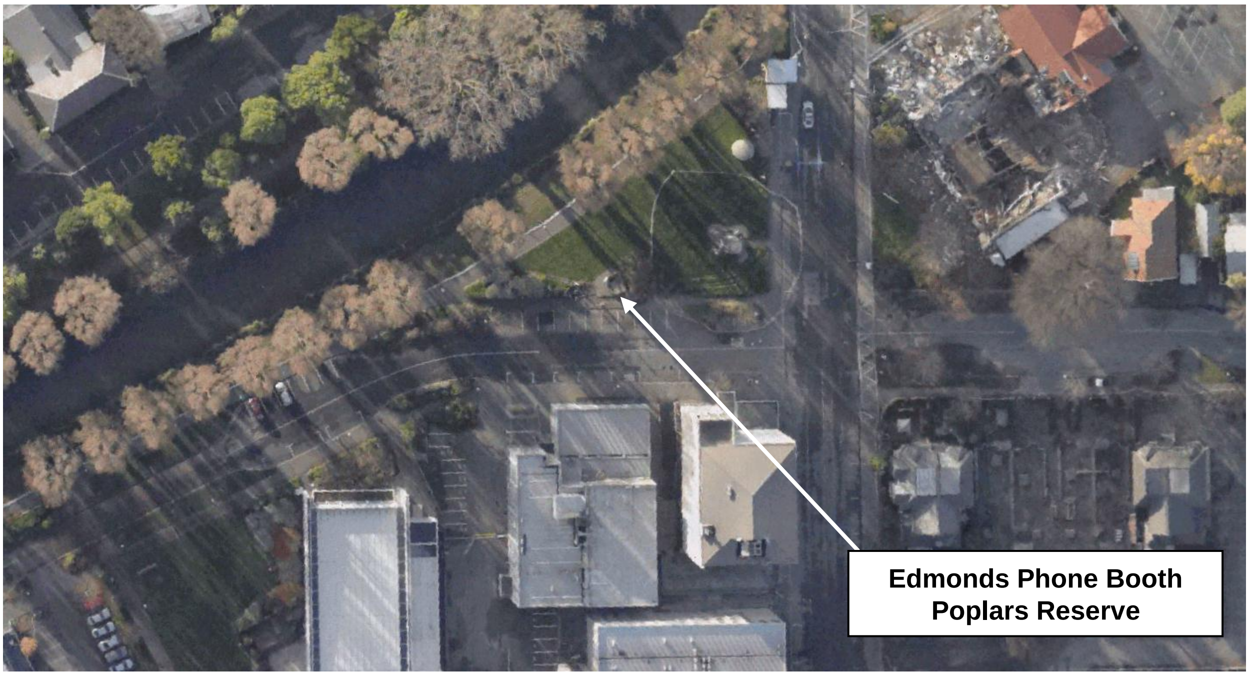
Information from the Canterbruy Geotechnical database[4] indcates 10 to 50mm ground cracking is present within 10m of the site.

[2] Tonkin & Taylor Ltd., 2011: Christchurch Earthquake Recovery, *Geotechnical Factual Report, Central Business District.*

[3] CERA Landcheck website, http://cera.govt.nz/my-property

[4] Canterbury Geotechnical Database (2012) "Aerial Photography", Map Layer CGD0100 - 1 June 2012, retrieved [date] from https://canterburygeotechnicaldatabase.projectorbit.com/

**Figure 3 Post June 2011 Earthquake Aerial Photography[5]**




### 7.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise Interbedded strata of sand silt and sandy gravel. Groundwater is expected to be between 1.0m and 10.0m bgl.

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

## 7.3 Seismicity

### 7.3.1 Nearby Faults

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

**Table 4 Summary of Known Active Faults[6][7]**

| Known Active Fault | Distance from Site | Direction from Site | Max Likely Magnitude | Avg Recurrence Interval |
|---|---|---|---|---|
| Alpine Fault | 130 km | NW | ~8.3 | ~300 years |
| Greendale (2010) Fault | 22 km | W | 7.1 | ~15,000 years |
| Hope Fault | 105 km | N | 7.2~7.5 | 120~200 years |
| Kelly Fault | 110 km | NW | 7.2 | ~150 years |
| Porters Pass Fault | 63 km | NW | 7.0 | ~1100 years |

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including 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.

### 7.3.2 Ground Shaking Hazard

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.

The recent 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.

## 7.4 Slope Failure and/or Rockfall Potential

The topography surrounding the site suggests that rockfall is not a potential hazard. However, given its proximity to the Avon River, and evidence from the recent earthquakes, the site may be susceptible to lateral spreading. In addition, any retaining structures or embankments nearby should be further investigated to determine the site-specific local slope instability potential.

[6] 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.

[7] GNS Active Faults Database

## 7.5 Liquefaction Potential

The site is considered to be moderately susceptible to liquefaction, due to the following reasons:

- Observations of liquefaction following the June Earthquake;
- The classification of neighbouring properties as TC3; and,
- Anticipated presences of saturated sands and silts beneath the site.

## 7.6 Conclusions & Recommendations

This assessment is based on a review of the 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 sand, silt and gravel. Associated with this the site also has a moderate liquefaction potential, in particular where sands and/or silts are present.

The site has a potential for lateral spreading due to its proximity to the Avon River and liquefaction potential.

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

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

# 8. Initial Capacity Assessment

## 8.1 % NBS Assessment

The building has had its capacity assessed using the Initial Evaluation Procedure based on the information available. The buildings capacity excluding critical structural weaknesses is in the order of 8% NBS. The assessed capacity including Critical Structural Weaknesses is in the order of 5% NBS. These capacities are subject to confirmation by a more detailed quantitative analysis.

Following an IEP assessment, the building has been assessed as achieving 5% New Building Standard (NBS). The building is considered potentially Earthquake Prone as it achieves less than 33% NBS.

## 8.2 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 $R_u$ = 1.0 NZS 1170.5:2004, Table 3.5, Importance level 2 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.

## 8.3 Expected Structural Ductility Factor

A structural ductility factor of 1.0 has been assumed based on the unreinforced stone masonry wall system observed. It is unlikely that the walls are reinforced given the appearance and the age of the building. As a result, the walls are expected to have very limited ductility.

## 8.4 Discussion of Results

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type. The building was built in 1929 and a result it is unlikely that the building was designed to a seismic loading standard. The design loads, if any, considered in the original design of this structure are likely to have been significantly less than those required by the current loading standard.

Combined with the increase in the seismic hazard factor for Christchurch to 0.3 and the presence of a critical structural weakness, it is reasonable to expect that a building of this construction type, built in 1929, would achieve less than 33% NBS.

# 9. Initial Conclusions & Recommendations

The building has been assessed to have a seismic capacity in the order of 5% NBS and is therefore potentially Earthquake Prone.

A quantitative assessment of the building should be undertaken to determine the seismic capacity and to develop potential strengthening concepts.

# 10. Limitations

## 10.1 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 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.

## 10.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and 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 View from Chester Street**

**Photograph 2 Cracking in carved stone veneer on reinforced concrete roof**

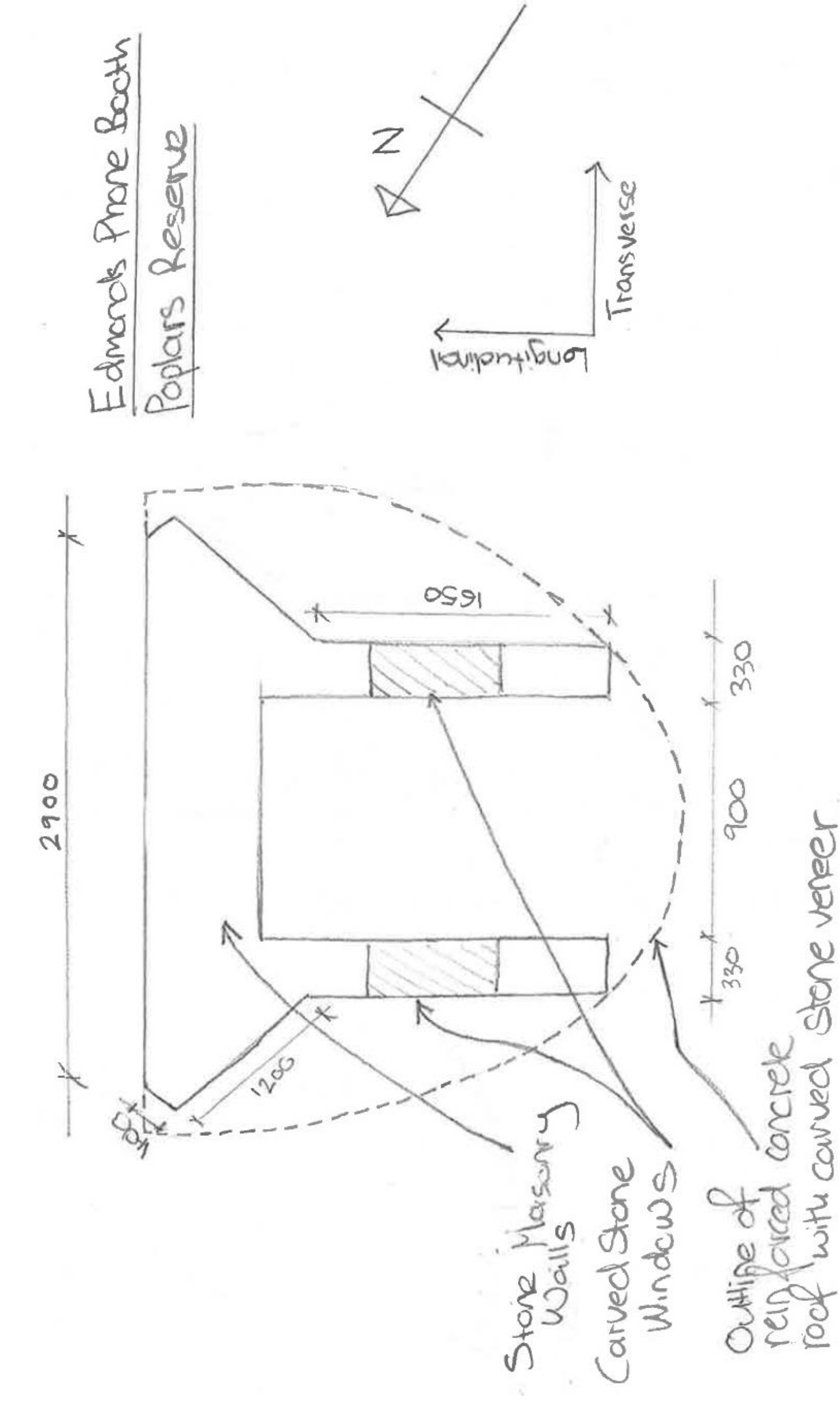

**Photograph 3 View from Oxford Terrace**

**Photograph 4 Phone booth interior**


**Photograph 5 Stone masonry walls**

Appendix B

# Sketch


Edmonds Phone Booth
Poplars Reserve
N
Longitudinal
Transverse
2900
1650
330
900
330
1200
Stone Masonry Walls
Carved Stone Windows
Outline of reinforced concrete roof with carved stone veneer

## Appendix C

# CERA Building Evaluation Form

**Detailed Engineering Evaluation Summary Data** V1.11

## Location

| | | | |
|---|---|---|---|
| Building Name: | Edmonds Phone Booth Poplars Reserve | Reviewer: | Stephen Lee |
| | Unit / No: / Street | CPEng No: | 1006840 |
| Building Address: | Cnr Mardras St/Chester St/Oxford T | Company: | GHD |
| Legal Description: | | Company project number: | 513090270 |
| | | Company phone number: | 04 472 0799 |
| | Degrees / Min / Sec | | |
| GPS south: | | Date of submission: | |
| GPS east: | | Inspection Date: | 14-09-12 |
| | | Revision: | FINAL |
| Building Unique Identifier (CCC): | PRK_1927_BLDG_002 EQ2 | Is there a full report with this summary? | yes |

## Site

| | | | |
|---|---|---|---|
| Site slope: | flat | Max retaining height (m): | |
| Soil type: | | Soil Profile (if available): | |
| 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): | 5.00 |

## Building

| | | | | |
|---|---|---|---|---|
| No. of storeys above ground: | 1 | single storey = 1 | Ground floor elevation (Absolute) (m): | 5.10 |
| Ground floor split? | no | | Ground floor elevation above ground (m): | 0.10 |
| Storeys below ground | 0 | | | |
| Foundation type: | strip footings | | if Foundation type is other, describe: | |
| Building height (m): | 2.85 | | height from ground to level of uppermost seismic mass (for IEP only) (m): | 2.85 |
| Floor footprint area (approx): | 8 | | | |
| Age of Building (years): | 83 | | Date of design: | Pre 1935 |
| Strengthening present? | no | | If so, when (year)? | |
| | | | And what load level (%g)? | |
| Use (ground floor): | other (specify) | | Brief strengthening description: | |
| Use (upper floors): | | | | |
| Use notes (if required): | Phone Booth | | | |
| Importance level (to NZS1170.5): | IL2 | | | |

### Gravity Structure

| | | | |
|---|---|---|---|
| Gravity System: | load bearing walls | | |
| Roof: | concrete | slab thickness (mm) | 650 |
| Floors: | other (note) | describe sytem | Slab on grade |
| Beams: | | | |
| Columns: | | | |
| Walls: | load bearing stone | #N/A | |

### Lateral load resisting structure

| | | | | |
|---|---|---|---|---|
| Lateral system along: | unreinforced masonry bearing wall - stone | **Note: Define along and across in detailed report!** | | |
| Ductility assumed, μ: | 1.00 | | note wall thickness and cavity | |
| Period along: | 0.40 | 0.40 from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | estimate or calculation? | |
| maximum interstorey deflection (ULS) (mm): | | | estimate or calculation? | |

| | | | |
|---|---|---|---|
| Lateral system across: | unreinforced masonry bearing wall - stone | | |
| Ductility assumed, μ: | 1.00 | note wall thickness and cavity | |
| Period across: | 0.40 | 0.00 estimate or calculation? | estimated |
| 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: | Good | 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: | | | |
| Along | Damage ratio: | 0% | Describe how damage ratio arrived at: | |
| | Describe (summary): | Minor damage. Less than 5%. | | |
| Across | Damage ratio: | 0% | | |
| | Describe (summary): | Minor damage. Less than 5%. | | |
| Diaphragms | Damage?: | no | Describe: | |
| CSWs: | Damage?: | no | Describe: | |
| Pounding: | Damage?: | no | Describe: | |
| Non-structural: | Damage?: | no | Describe: | |

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

**Recommendations**

| | | | | | |
|---|---|---|---|---|---|
| | Level of repair/strengthening required: | | | Describe: | |
| | Building Consent required: | | | Describe: | |
| | Interim occupancy recommendations: | | | Describe: | |
| Along | Assessed %NBS before e'quakes: | 5% | 5% %NBS from IEP below | If IEP not used, please detail assessment methodology: | |
| | Assessed %NBS after e'quakes: | 5% | | | |
| Across | Assessed %NBS before e'quakes: | 5% | 5% %NBS from IEP below | | |
| | Assessed %NBS after e'quakes: | 5% | | | |

**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): Pre 1935

$h_n$ from above: 2.85m

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): | 0.4 | 0.4 |
| (%NBS)nom from Fig 3.3: | 2.9% | 2.9% |

| | |
|---|---|
| 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 | 1.00 |
| 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) | 0.8 |

| | along | across |
|---|---|---|
| **Final $(\%NBS)_{nom}$:** | **2%** | **2%** |

**2.2 Near Fault Scaling Factor**

| | along | across |
|---|---|---|
| Near Fault scaling factor, from NZS1170.5, cl 3.1.6: | 1.00 | |
| 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): | 2 |
| Return Period Scaling factor from Table 3.1, **Factor C**: | 1.00 |

**2.5 Ductility Scaling Factor**

| | along | across |
|---|---|---|
| Assessed ductility (less than max in Table 3.2) | 1.00 | 1.00 |
| Ductility scaling factor: =1 from 1976 onwards; or $=k_{\mu}$, if pre-1976, fromTable 3.3: | 1.00 | 1.00 |
| Ductiity Scaling Factor, **Factor D**: | 1.00 | 1.00 |

**2.6 Structural Performance Scaling Factor:**

| | along | across |
|---|---|---|
| Sp: | 1.000 | 1.000 |
| Structural Performance Scaling Factor **Factor E**: | 1 | 1 |

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

| | along | across |
|---|---|---|
| **$\%NBS_b$:** | **8%** | **8%** |

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

| | | |
|---|---|---|
| **3.1. Plan Irregularity, factor A:** | significant | 0.7 |
| **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 |

| **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** |

Therefore, Factor D: 1

| 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 |

**3.5. Site Characteristics** insignificant 1

| | | Along | Across |
|---|---|---|---|
| **3.6. Other factors, Factor F** | 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

| | | Along | Across |
|---|---|---|---|
| **3.7. Overall Performance Achievement ratio (PAR)** | | **0.70** | **0.70** |
| **4.3 PAR x (%NBS)b:** | PAR x Baselline %NBS: | **5%** | **5%** |
| **4.4 Percentage New Building Standard (%NBS), (before)** | | | **5%** |

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