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Civil & Structural
Seismic Risk Analysis
Building Compliance

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Detailed Seismic Assessment

For

Woburn House
40-44 Bloomfield Terrance
Lower Hutt

Date: 22/12/2015
Revision: 1

J40166

Executive Summary

Certa Engineering Ltd. has been engaged to undertake a detailed seismic assessment, of the Woburn House at 40-43 Bloomfield Terrace, Lower Hutt.

The assessment is based on the available architectural and structural drawings and a walkthrough visual building inspection undertaken on 26th March 2014. It should be noted that discrepancies between the structural drawings and the actual building configuration have been noticed on site.

The building is a four storey reinforced concrete frame building designed by Smith Leuchars Limited in 1986. It has a rectangular footing print (i.e. 30meters wide by 34 meters long) with the South-West corner chamfered.

The engineer's design calculation records indicated that the building has been designed to comply with the design code at the time and the capacity design principle has been adopted.

We have adopted the Modal Response Analysis (i.e. MRA) as required in the current loading standard: NZS1107:2005 to calculate the seismic capacity of the building with respect to that standard. A 3 dimensional building model was created in the computer structural analysis software SPACE GASS to assist our evaluation. The outcome of our assessment is summarised in table 1 below. The seismic strength of the building is given as a percentage of the new building standard (i.e. %NBS).

Table 1: Results

Structural Elements	% New Building Standard	Comments
North-South Perimeter frames	72%NBS	Ground storey column hinging, some beam-column joints are not fully protected against beam hinging to develop their over strength capacity
East-West Perimeter frames	77%NBS	
South-East Corner frame	25%NBS	Column hinging at multi floor level
Concrete Stair	20%NBS	limited movement tolerances
Concrete block wall infill	30%NBS	insufficient seismic separation
Gravity-load bearing columns	33%NBS	Lack of confinement from column stirrup reinforcement

Note: If a seismic strengthening regime is to be implemented then there are other features which have only marginally more strength than the above that may also require strengthening . Refer Section 7 for a comprehensive list and Section 8 for strengthening concepts.

Table of Contents

1.0 Scope of Our Engagement

Certa Engineering Ltd. has been engaged to undertake a Detailed Seismic Assessment, of the building located at the above address. This evaluation has been carried out in accordance with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Buildings in Canterbury" Draft prepared by the Engineering Advisory Group; Revision 6 dated 16 May 2012, which is considered the appropriate reference document for evaluations of this nature throughout New Zealand.

The assessment is made with regard to Clause B1 – Structure of the New Zealand Building Code. No other Building Code Clauses have been assessed by this report.

This structural assessment is based on information provided to us, which includes:-

- Structural Drawings and design calculations

This structural assessment is also based on the visual evidence & indications present at the time of inspection. The findings of this report may be subject to revision pending the discovery of more detailed information.

We have not made any Geotechnical assessment of the soils on the site.
This report is provided solely for use by Woburn House Body Corporate, and the Hutt City Council and shall not be relied on by any other parties without written approval from Certa Engineering Ltd.

2.0 The Building

Address:	40-44 Bloomfield Tce
Overall Plan Dimensions:	34m by 30m
Number of Storeys:	4
Total Plan Area:	4080 m^2
Building History: (i.e. Previous Additions and Alterations)	Office building with numerous alterations (mostly non-structural) since its construction.
Occupancy:	Office
Importance Classification: (AS/NZS 1170.0:2002: Table 3.2)	IL=2 (i.e. Normal Building)
Other Classification:	Non-Heritage Building

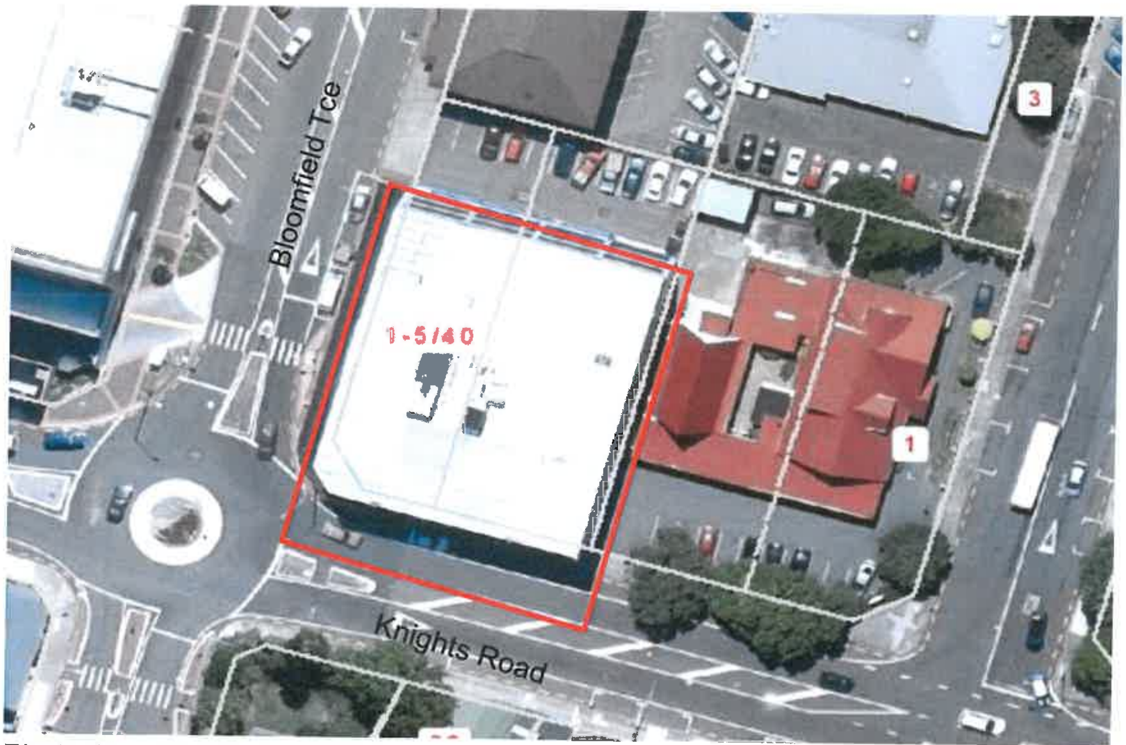

Photo 1: Aerial Photo of Woburn House at 44-45 Bloomfield Terrace, Lower Hutt

Photo 2: Front View from Bloomfield Terrace.

Photo 3: Side view from Knights Road

3.0 Site

The site has been assessed as Category D (i.e. deep and soft) subsoil classification as described in NZS1170:2005. This may be subject to review on the receipt of more detailed geotechnical information. As shown in the photo 1 the building located at corner of the Bloomfield Tce and the Knights Road. A single storey church building is located at the Eastern end of the building. Our site inspection of the building revealed that a seismic gap of approximately 90mm is present.

3.1. Ground Conditions

The ground is flat and paved. No obvious signs of subsidence was present at the time of our inspection on 26th March 2014.

3.2. Geotechnical Investigation

We have not undertaken any geotechnical investigation of the building site and we have been unable to obtain any geotechnical records of the building site from local council archive. However the local council *Seismic Hazard Maps* indicate the site subsoil is likely to be composed of alluvial silts and may have moderate to high liquefaction risks. Please note that the information presented in the council Seismic Hazard maps are regional in scale and could vary significantly for a specific building site.

4.0 Drawing and Document Review

The detailed seismic assessment of the building is primarily based on the information present on the structural drawings obtained from the Hutt City Council archive titled ***Woburn House For Petron Properties Ltd*** numbered 1793-1 to 1793-12. The drawings were produced by Smith Leuchars Limited and dated December 1985.

During our walk-through inspection of the building we noticed two significant discrepancies in the building's contrstuction compared to that represented on the drawings.
(1) The building suspecnded concrete flooring comprises 200mm deep double T floor units with 65mm topping instead of the 200mm thick Dycore unit as specified on the drawings.
(2) The building possesses a fully covered upper storey instead of the partially covered roof shown on the drawings. This may have been an alteration carried out more recently.

We also noted that the set of structural drawings of the building from city council were labelled as preliminary drawings. This could suggest that the building design has been altered in the final design stage of the project. This could account for the observed discrepancies outlined above. However we were unable to obtain a copy of an 'as-built' set of construction drawings from Hutt City Council. This is not unusual for a building of this vintage. Please note that we have not carried out any Ferroscan of the building to verify the correctness of the reinforcing steel details described in the drawings but we consider these likely to be accurate.

We have also obtained a copy of the original design calculations for the building. The design of the building was carried out in accordance with the relevant design standards at the time, namely NZS4203:1984, NZS3101:1982 and NZS3404:1982.

5.0 Building Arrangement

The building is a four storey reinforced concrete office building. It comprises reinforced concrete perimeter moment resisting frames supporting precast concrete double T floor units (200mm thick T units with 65mm topping). The building's Eastern perimeter frame contains 20 series reinforced masonry block infills from ground level to the third floor level. The stair and lift core is located at the centre of the building as shown in Figure 1 below:

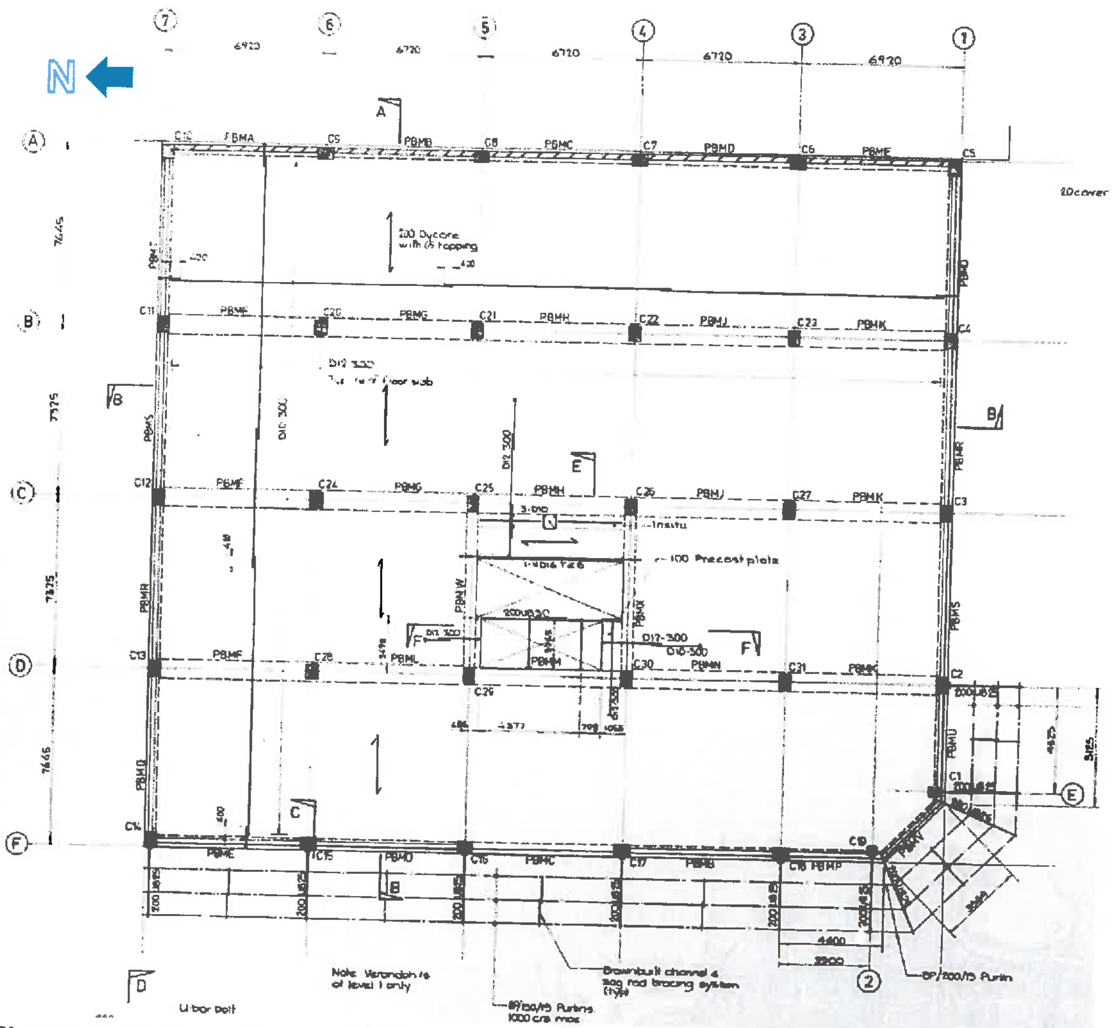

Figure 1: Woburn House 1st Floor Plan (other floors similar).

5.1. Structural System

The primary lateral-load resisting elements in the building are the perimeter moment resisting frames shown in Figure 1. These seismic frames typically comprises of 500mm wide by 600mm deep columns and 400mm wide by 700mm deep beams. The typical bay width is 6.7 metres and 7.3 meters for the frames in the "Along" (i.e. North-South) and "Across" (i.e. East-West) directions respectively.

Unlike other perimeter frames, the Eastern perimeter frame comprises reinforced block infill panels from the ground level up to the third floor level. The likely reason for adopting the infill panels is to provide effective protection against spread of fire. All infill panels are separated from the concrete framing on both sides and the top of the panels by 25mm and 10mm respectively.

The building comprises a total of three internal gravity-load carrying frames along Grids B, C and D as shown in Figure 1. The beams in the gravity frame are typically 800mm wide by 425mm deep and the gravity columns (i.e. Column C20 to C31) are 400mm wide by 600mm deep.

5.2. Foundation System

The building foundation comprises a total of forty-three 500mm diameter concrete piles, approximately 6.0m deep. All piles are reinforced by 6 HD20 longitudinal reinforcements and hand drawn wire spiral reinforcement with a pitch of 112mm. The council records indicate that the perimeter frames are supported by a single pile under each column while double piles are used under the internal gravity columns. The ground floor of the building consists of 125mm thick concrete slab reinforced with HD16 bars at 300mm centres in both principal directions.

5.3. Generic Configuration Issues and potential structural weaknesses

Based on the details presented on the council drawings, we have identified the building has plan irregularity as defined by the current loading standard NZS1170:2005. This implies that the building likely to carry additional torsion (i.e. twist) demand generated from earthquake.

In additional to the above outlined generic configuration issues, we have also identified some potential structural weaknesses of the building and they are:

(1) The concrete stair flights are restrained at both ends between floor levels 1 and 3. Only limited movement allowance is provided (7mm) at the base of the ground floor stair flight. Without adequate movement tolerance, the concrete stair would attract very large seismic induced forces and are likely to be severely damaged in the event of a large earthquake. Loss of function as a post-earthquake means of access/egress is a strong possibility.

(2) The gravity columns (column C20 to C31) and the columns on the chamfered end (i.e. column C1 and C19) have limited ductility due to a lack of confinement (transverse or horizontal) reinforcement.

(3) Block infill wall panel lack adequate clearance to the surrounding concrete frame and are likely to attract large unanticipated forces in the event of a strong earthquake. This also has the effect of reducing the building's overall seismic capacity.

6.0 Building Modelling and Analysis Method

6.1. Method

The building has been identified as being “irregular” in accordance with NZS1170:2004 therefore a Modal Response Spectrum (MRS) method has been selected to assess the probable seismic strength of the building and its components. A three dimensional model of the building has been set up in structural software: SPACEGASS version10.85b (Figure 2) and analysed under full design earthquake loading as required by the current design standard.

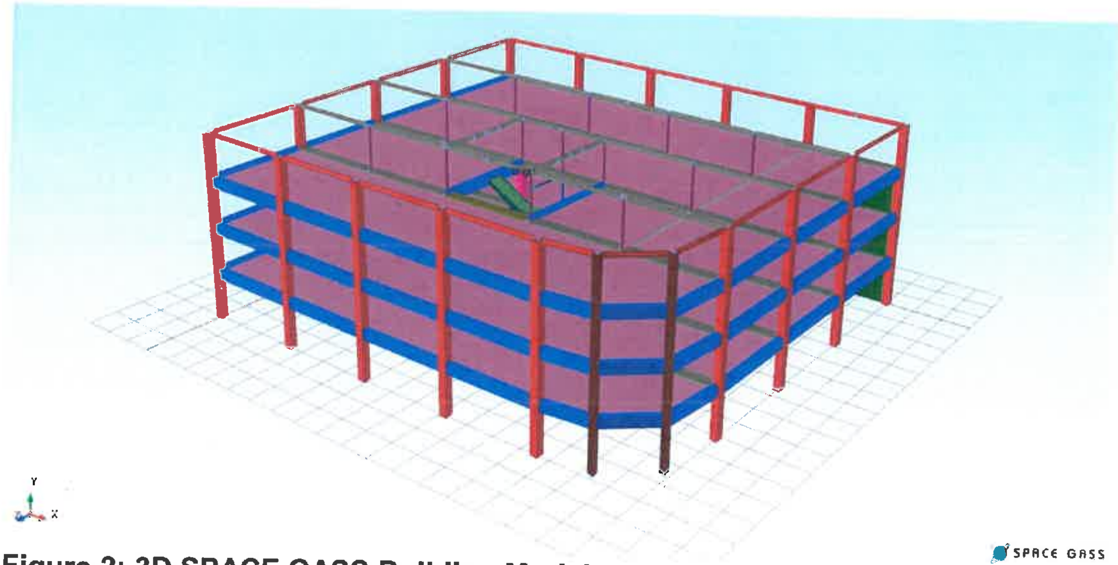

Figure 2: 3D SPACE GASS Building Model.

Space Gass Model Building

Input parameters for MRA adopt in SPACE GASS modelling:

Location:	Nelson
Subsoil category:	D (i.e. Deep and soft)
Building importance level	IL=2 (Normal building)
Structural ductility	μ=3 (ductile structure)
Base shear force scale factor:	100% of equivalent static base shear force
Modal combination method:	Complete Quadratic Combination (CQC)
Number of modes used	8

Effective section properties:

- Floor Beams: I eff = 35% Ig
- Columns: I eff = 55% Ig

Note: *Stair flights are modelled as 200mm thick concrete plate elements in the building.*

Summary of MRA out-puts:

T_1 = 0.65 sec

V_{base} = 5874 kN

Mass participation:

100% in X direction

99% in Z direction

6.2. Modelling and Assessment Assumptions

To facilitate the modelling and analysis of the building the following assumptions have been made:

- Assumptions on the material properties of the building as shown in Table 2 below:

Table 2 – Material Assumptions

Tensile reinforcement yield strength (f_y): • Grade D bars • Grade HD bars • Grade HR stirrups	 f_y = 300MPa f_y = 430MPa f_y = 380MPa
Concrete compressive strength of the building (current).	f'_c =40MPa
Block wall shell compressive strength	f'_c =12MPa
Block wall concrete fill compressive strength	f'_c =17.5MPa

- Due to ambiguities in some of the existing drawings, assumptions have been made for some column reinforcement and beam-column joint reinforcement content. Details of these assumptions are given in the appended calculations.
- Columns at the base of the building are assumed to have fixed connections and the effects of soil structural interaction and soil liquefaction has not been considered.
- Based on the council drawing records, the ends of the gravity beams where they are connected to their supporting columns are detailed in such way that limits the amount of bending that can be transferred to the column. We have therefore modelled these as pinned connections.
- The precast double T floor is modelled as rigid diaphragm adopting the "Master and Slave" function of the SPACE GASS program.

7.0 %NBS (New Building Standard)

We have undertaken an assessment of the structural seismic capacity with respect to New Building Standard (%NBS) using the Modal Response Spectrum Method as recommended by the current structural design loadings standard NZS1170.5:2004. Detailed engineering calculations of the building's probable seismic strength are given in Appendix B. Our results are summarised below:

The primary seismic-lateral load resisting frames ***excluding*** the chamfered corner bay on the South-West corner (i.e. between Grid 3 and 1 and Grid D and F) are:

- North/South Direction.......72% NBS
- East/West Direction.........77% NBS

However the overall probable seismic strength of the building is limited by several potential structural weaknesses shown below:

South-West Corner Perimeter Frame	
• Column C1 and C19:	**25%NBS**
• Beam PBMP:	**30%NBS**
• Beam PBMV:	62%NBS
• Beam PBMU:	**30%NBS**
Non-Ductile columns (Column C20 to C31)	**33%NBS**
Concrete stair seating	**20%NBS**
Grid A (Infilled concrete frame)	
• Infill between Ground and level 1	44%NBS
• Infill between level 1 to level 2	**30%NBS**
• infill between level 2 to level 3	46%NBS
• Infill frame column strength:	71%NBS
Seismic frame Beam-Column Joint Strength	100%NBS*
Seismic gap width	100%NBS
Concrete pile foundation bearing strength	100%NBS
Floor diaphragm	100%NBS
Perimeter column (A3 to A 6 and F3 to F6) to slab tie-connections	**<33%NBS**

Note: *= beam-column joints in the seismic frames may not have sufficient shear strength to allow beams to develop their maximum overstrength.

Base on the above results the building's main seismic frame have a probable seismic strength level of 72%NBS but overall building seismic strength is limited by the potential critical structural weaknesses of some individual elements to 20%NBS. Consequently in our opinion the building should be classified as an ***earthquake prone building.*** Mandatory strengthening to a level of over 34%NBS is required by the Building Act 2004. To further reduce the building's seismic risks, our recommendation is for any building strengthening be targeted to achieve 72%NBS.

8.0 Seismic Retrofit Concept

The building's seismic strength could be improved by addressing the individual potential structural weaknesses identified in the previous paragraph. A possible way to retrofit these structural elements is outlined below. Structural sketches of the proposed retrofit concept are given in Appendix A, they are conceptual only and not for construction. Detailed design, working drawings and Building Consent would need to be obtained prior to commencing this work.

8.1. Concrete stairs

The concrete scissor stairs above the ground floor are effectively tied at both ends to the floor slabs and hence there is no movement tolerance provided to the stair to accommodate the earthquake induced inter-storey drift demands. A simple way to improve the seismic performance of the stair is to install a seismic gap along the stair flight possibly at the mid-height landings to allow the stairs to move in both principal directions.

8.2. South-West corner frame

One way to improve seismic strength of the beams (i.e. Beams PBMP PBMV and PBMU) and the columns (in particular columns C1 and C19) around the South-West corner is to cut slots (approximately 20-25mm wide) at bottom of the beams. This will limit the bending demand transferred from the main seismic frame to the corner and reduce the demands place on those elements. Additional steel angle cleats could be installed at bottom of the slotted beams to supplement the shear strength lost by the cutting procedure. Because the corner columns are eccentrically located to the floor beams, we recommend the installation of additional steel UC or SHS posts to provide supplementary support under the beam; this will eliminate the effects of the eccentricity and also provide added redundancy to the corner frame.

8.3. Block Infill panels

The seismic performances of the block infill panels may be improved by enlarging the existing seismic gaps between the panels and the columns.

8.4. Non-Ductile Columns

Fibre reinforced polymer wrapping (FRP) could be applied to the potential plastic hinge regions (usually at base and tops of the columns) to improve the ductility capacity of the gravity load bearing columns (columns C20 to C31).

The provided recommendations and/or sketches shall not be used for construction and will also be consentable works with the need for the associated documentation to be submitted to Hutt City Council as part of the normal building consent application process.

8.5. Perimeter column-slab connections

Reid threaded rods can be installed across the building to tie the perimeter columns A3 to A6 and F3 to F6 together to satisfy the requirements set out in the current concrete structural standard NZS3101:2006, clause 10.3.6.

9.0 Continued Occupancy Recommendations

We consider that continued occupancy is acceptable as long as the building is to be retrofitted within a short time frame (we recommend 1 to 2 years as being appropriate). Priority should be given to the low %NBS elements such as the stairs, the corner frame, and the non-ductile gravity load bearing columns in the building.

10.0 Conclusions:

The building's seismic strength has been evaluated using the Modal Response Method as per current design code NZS 1170. Our calculations indicate that the building's primary seismic frames along both principle directions have a strength level of 72%NBS. However the overall building strength is limited by various individual element weaknesses to 20%NBS. Consequently in our opinion the building should be classified as an ***earthquake prone building*** being one that possesses less than the minimum threshold of 34%NBS. Seismic retrofit of the building is therefore required. The New Zealand Society for Earthquake Engineering and most territorial authorities recommend a minimum strengthening target of 67%NBS.

Because the building's main seismic frames have been demonstrated to have an existing capacity of at least 72%NBS, we believe this would be a realistic and practical strengthening target. Achieving a level higher than the capacity of the primary frames would involve a significantly greater amount of construction works and unlikely to be economically feasible.

Report prepared by:
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Report reviewed by:
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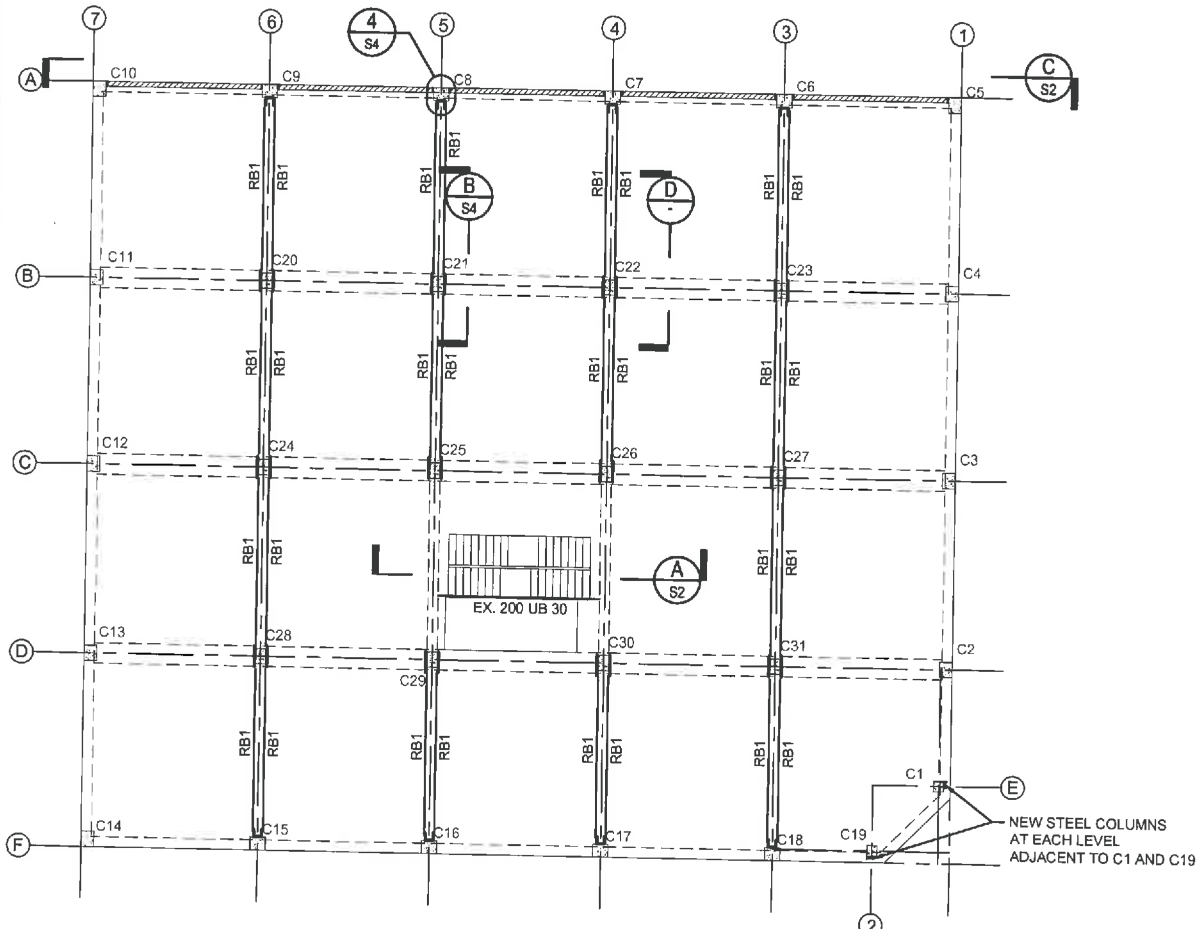
Appendix A

- Drawings of Strengthening Work.

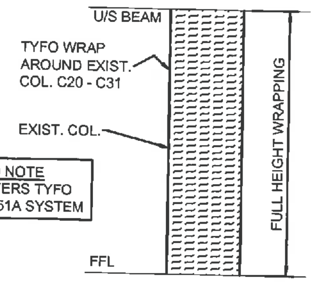


LEVEL1-3 STRUCTURAL FLOOR PLAN

SCALE 1:200

RB1 = Ø16 REIDBRACE INSTALLED AS PER MANUFACTURERS DETAILS.

FRP WRAPPING TO COLUMNS C20-C31 IN ACCORDANCE WITH MANUFACTURERS DETAILS AND SPECIFICATION - REFER DETAIL.



TYFO NOTE
3 LAYERS TYFO
SEH-51A SYSTEM

COLUMN WRAPPING DETAIL

1:50

CLIENT	WOBURN HOUSE
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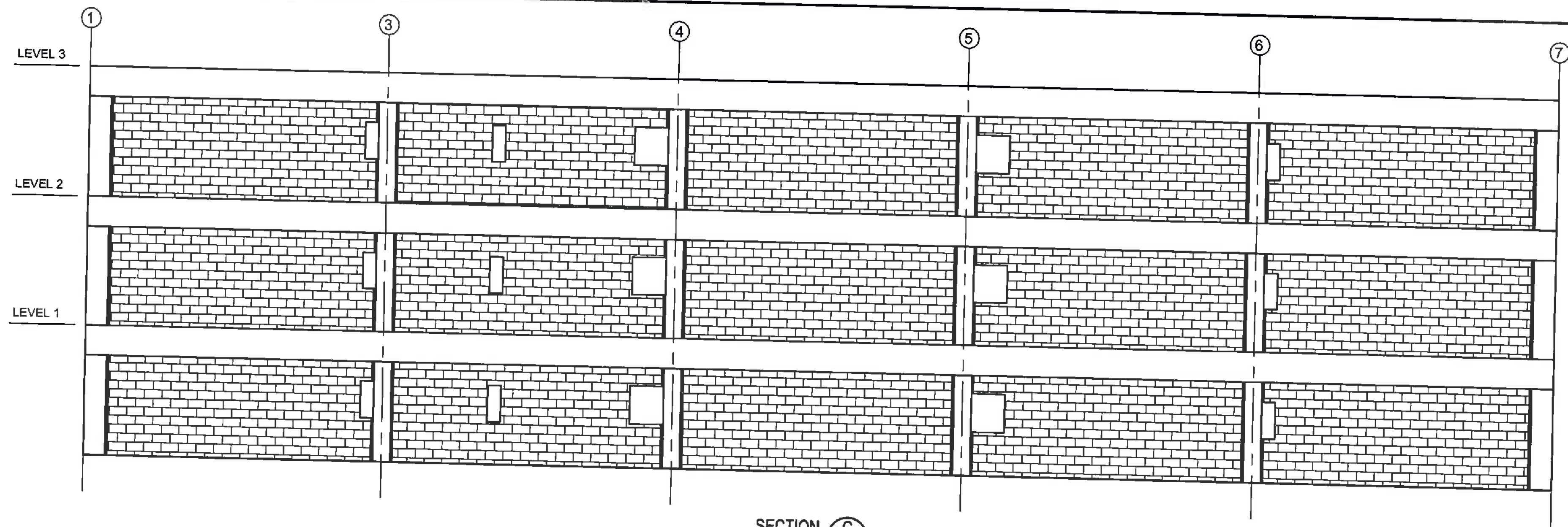
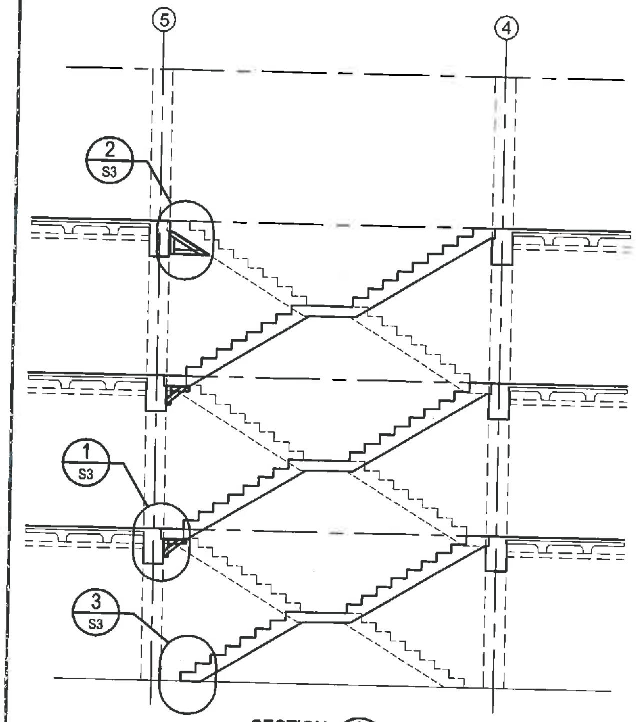


SECTION C/S1
SCALE 1:100



SECTION A/S1
SCALE 1:100

- - - - - SAW CUT OUT EXISTING BLOCK WORK AT COLUMN INTERFACE. PROVIDE 50mm CUT OUT. SEAL JOINT WITH IN ACCORDANCE WITH NZBC C/AS1 PART 7.

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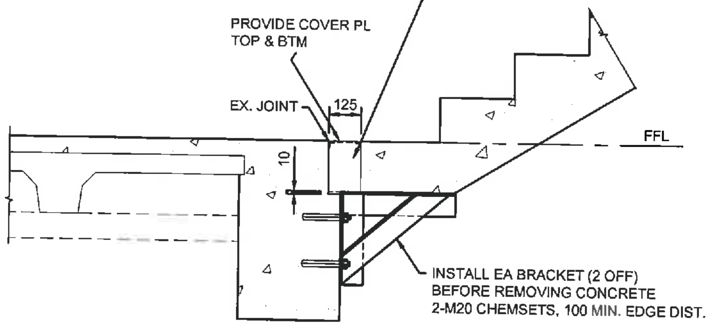
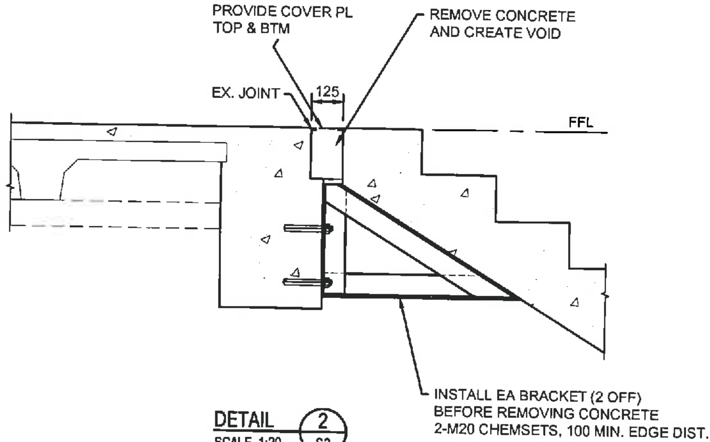
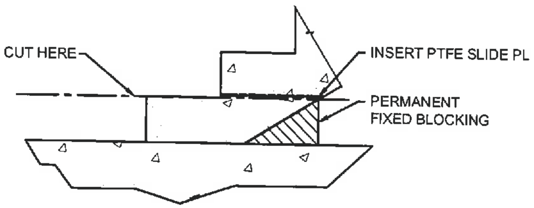


DETAIL 1/S2
SCALE 1:20
TYPICAL DETAIL



DETAIL 2/S2
SCALE 1:20
TYPICAL DETAIL



DETAIL 3/S2
SCALE 1:20

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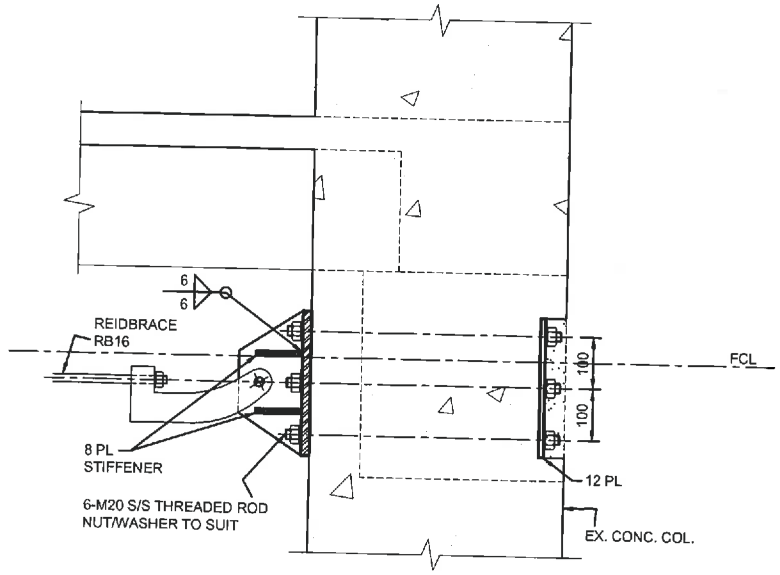
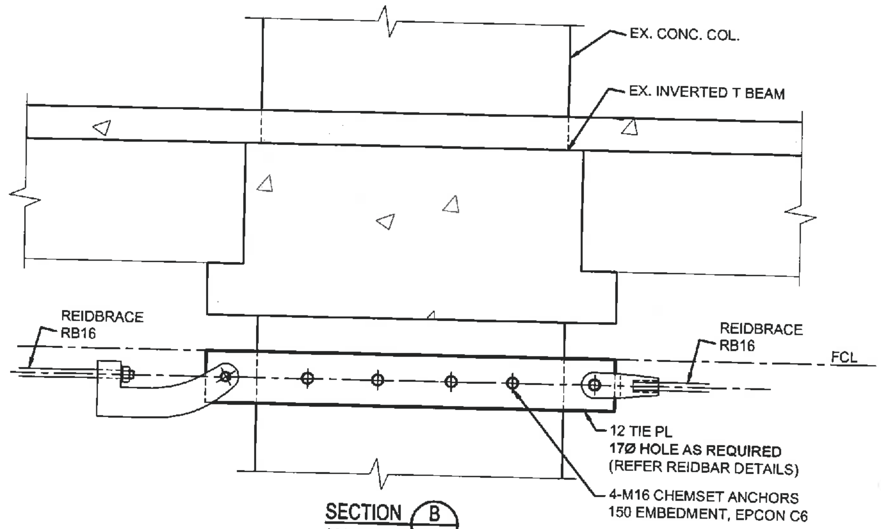
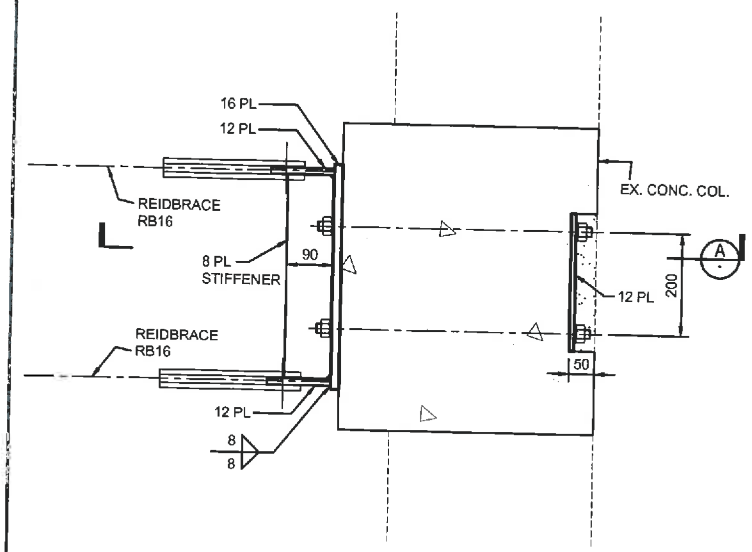


SECTION A
SCALE 1:10



SECTION B
SCALE 1:10 S1

TYPICAL DETAIL



DETAIL 4
SCALE 1:10 S1

TYPICAL DETAIL

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