Report Number 1520

Yaldhurst Memorial Hall

BU 1643-001 EQ2

Detailed Seismic Assessment Report

Stage 2 – Quantitative Report

CHRISTCHURCH CITY COUNCIL

Report Number 1520

Yaldhurst Memorial Hall
Detailed Engineering Evaluation
Stage 2 – Quantitative Report

Corner of Yaldhurst Road and Pound Road, Yaldhurst, Christchurch

Christchurch City Council

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Date: September 2012
Reference: 6-QUCCC.44
Status: Final

Report No. 1520

Yaldhurst Memorial Hall
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Detailed Engineering Evaluation
Quantitative Report – SUMMARY
Final

Corner of Yaldhurst Road and Pound Road, Christchurch

Background

This is a summary of the Quantitative report for the Yaldhurst Memorial Hall building, and is based on the Detailed Engineering Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspection on 22 February 2012, limited intrusive investigations on 11 April 2012 and available drawings. No original structural calculations are available for this building.

Key Damage Observed

The following damage to structural elements has been observed:

- Cracks between RC columns and masonry panels all elevations
- Vertical cracks to blockwork on north gable
- Horizontal and vertical cracks on porch at north end
- Fine stepped cracks in blockwork to west elevation
- Fine horizontal cracks to RC columns above and below window openings west elevation.
- Hairline cracks to RC columns at low level.
- Horizontal crack to lean-to store south elevation.
- Localised horizontal cracking below windows east elevation.
- Cracks to RC chimney stack at centre of property.
- Cracks to some corbels supporting roof trusses.
- Crack across floor slab in porch.
- Some horizontal and stepped cracking on URM infill panels internally.

Indicative Building Strength

Based on the information available, and from undertaking a quantitative assessment, the building's capacity has been assessed to be less than 34%NBS along and across the building. The main limitations are the out of plane strength of the blockwork panels, the flexural capacity of the RC columns and the overturning capacity of the column foundations. The building's post-earthquake capacity is in the order of 6%NBS being the minimum value that can be attributed to the infill blockwork panels. The transverse capacity of the RC columns is about 24% NBS for both the columns and the foundations.

The building has been assessed to have a seismic capacity of less than 34%NBS and is therefore classed as earthquake prone.

Recommendations

It is recommended that:

a) The CCC reviews the on-going occupancy of this building until such time that any strengthening works have been undertaken.

b) A strengthening scheme be developed to increase the overall capacity of the building to at least 67%NBS.

c) Provide a cordon around the full perimeter of the building.

Contents

1 Introduction

Opus International Consultants Ltd has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Yaldhurst Memorial Hall.

This report is a Stage two, Quantitative assessment of the building structure, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Engineering Society (SECOC) on 19 July 2011.

The purpose of the assessment is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

A qualitative assessment report was issued on 7 March 2012. The qualitative assessment noted that the building had a seismic capacity of 10%NBS.

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 to 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 (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

We anticipate that any building with a capacity of less than 34% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

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 the 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)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if,

1. 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:
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. 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 loads 33% of those 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 on 4th September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented 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 required under Act)

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 Guidelines

Table 1 below 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.

Table 1: %NBS compared to relative risk of failure

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

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

– The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being EPB's. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof) until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

– Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

– Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

– It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings

4 Building Description

4.1 Background Information

Exterior and interior photographs of notable facets of the Yaldhurst Memorial Hall complex are presented in Appendix A.

Drawings of the building have been provided. The more important drawings reviewed as part of this assessment are included in Appendix B of this report.

[1] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

A post February Earthquake site visit was undertaken on 11 March 2011 and there were concerns about the stability of some of the masonry, particularly on the high north gable wall. A further visit was made on 21 February 2012, to ascertain the accuracy of dimensions and overall layout of the building and to obtain as much information as possible regarding the construction of the building. The survey confirmed that the record drawings were accurate in terms of layout. A further, partially intrusive investigation was carried out at this property on 11 April 2012. In this investigation a sample of the foundations were exposed with small trial pits and the masonry infill panels were inspected to ascertain their make-up, thickness and to assess if wall ties are present.

Some assumptions have been made in the detailed assessment where information was unavailable.

4.2 Building Description

The Yaldhurst Memorial Hall was constructed in 1954 – there is a dedication plaque adjacent to the main entrance confirming this. The building is in four parts, a floor plan and elevation are provided in Appendix B.

1. Entrance Lobby: At the north end of the Memorial Hall is a small single storey flat roofed section forming the main entrance lobby, which was apparently added at a later date, as it is not shown on the drawings. This is constructed with rendered concrete block masonry (CMU) and has external concrete steps leading up to it and a concrete floor.

2. Memorial Hall: Immediately to the south of the main entrance is the main part of the building. The hall is comprised of timber roof trusses sitting on a reinforced concrete beam which in turn rests on 300mm reinforced concrete columns with reinforced concrete pad footings below each column and strip foundations between to support the in-fill walls. The panels between the columns are in-filled with cavity Unreinforced Masonry (URM) wall panels and most column bays have windows extending for all but a short length of the column bay width. The investigations show the inner leaf of the walls to be 100mm thick Concrete Masonry Units (CMU) and the outer leaf to be 110mm thick Unreinforced Brickwork (URM).

3. This main section of the building complex (the main hall) houses a hall and facilities with a stage across the south end. At the north end of the building there is a partial mezzanine first floor accessed by a timber staircase leading up from the west side. The main hall has a maximum eaves height of 4.5m and is approximately 35m long and 11m wide.

4. Attached to the south of Memorial Hall is a building of similar construction but of a lower height and shorter length. This houses the 'supper room', committee room and a kitchen. This section of the building has an eaves height of approximately 3.0m and is 10m long by 11m wide. At the step in roof level, between the main hall and this section there is a reinforced chimney serving an open fire-place within the supper room.

5. To the south of this a door leads into a small lean to type construction store room, which is not shown on the drawings, and was therefore probably added at a later date. This is formed with CMU block walls and has a mono-pitched roof sloping down towards the south.

4.3 Gravity Load Resisting System

The gravity load bearing system for this property comprises:-

The tied timber roof trusses are at approximately 3.2m centres, supporting timber purlins at approximately 1.0m centres, over which is fixed a lightweight profiled metal roof covering. Timber cross-bracing is provided in the plane of the rafters.

A reinforced concrete frame with a ring beam at eaves level transfers the roof and ceiling loads, via approx. 300x300mm square section columns below roof truss positions which are assumed (from details on the available drawings) to be supported on reinforced concrete pad foundations approximately 1.5m wide. Intermediate reinforced strip foundations support the cavity masonry infill panels between columns and the floor construction and are (likewise based on the drawing details) assumed to be 600mm wide.

The ground floor of the main hall is assumed to be a suspended timber floor construction with joists spanning in the direction of the width of the hall, with timber floorboards over. The drawings indicate two intermediate strip footings along the length of the building, breaking the floor span into thirds of the width of the building.

The floor finishes of the other rooms with floor coverings was not investigated. The ground floor of the main entrance area is a reinforced concrete slab.

4.4 Seismic Load Resisting System

4.4.1 Longitudinal – North to South Direction

Longitudinal seismic loads are resisted by the moment connection between the columns and the ring beams at eaves level on the east and west elevations and the foundations at ground level, with the support of the masonry infill panels below the window openings, which act in in-plane shear. Due to the typical window openings (most bays) the system is reliant on the columns for the transfer of loads from eaves level to the masonry and frame below. The infill panels of the longitudinal walls can be seen to be painted blockwork externally and are assumed to be of similar construction on the inner leaf. The diagonal timber roof bracing between roof trusses in the plane of the rafters, in conjunction with the purlins will provide some limited resistance to seismic loads in this direction. The ceiling battens will provide no significant diaphragm action. It is assumed that the hardboard ceiling finishes will not provide any diaphragm action.

The two ridges, of the different levels of duo-pitched roof, are not directly connected, but are intersected by the chimney construction and partial gable end wall.

4.4.2 Transverse – East/ West Direction

Transverse seismic loads are resisted by the moment connection between the columns and the beams of the gable elevations and the internal wall at the step in roof level, along with the moment connection between the columns and the foundations at these same locations at ground level. Support is provided by the in-plane shear resistance of the masonry panels between the columns at the gables and the internal wall along the line of the chimney.

The purlins will provide some transfer of seismic loads between roof trusses in this direction. The timber ceiling battens and hardboard ceiling finishes are assumed to provide no significant diaphragm action.

At the front elevation, it could be seen from inside the building that the inner leaf is 100mm blockwork and the outer leaf clay brick with a cavity between. This is rendered externally. With access to the cavity given at the time of the survey only by a narrow gap between the masonry and concrete frame for inspection, there did not appear to be any connections tying the masonry panels of this elevation to the reinforced concrete frame. The drawings also provide no evidence of such connections, or of cavity ties.

It should be noted that for the north gable elevation, the outer bay of the frame on each side is a completely infilled with masonry at high level, but the central bay comprises mainly openings over its full height. Loads can therefore be transferred only by the beams of the frame from one side of the gable wall to the other, including the transfer of in-plane seismic loads.

5 Damage Assessment

A damage assessment survey of internal and external structural elements was carried out by Opus on 21 February 2012. The inspection included a limited external and internal visual inspection of readily visible structural elements.

Key damage observed includes:

- Cracking to the elevations between the reinforced concrete frame and the masonry infill panels
- Diagonal stepped shear cracking in the masonry wall panels on the east and west elevations, between the reinforced concrete columns
- Cracking of the reinforced concrete columns at construction joints
- Cracking of the porch construction masonry
- Cracking of the masonry of the lean-to store structure to the south elevation
- Cracking on the reinforced concrete chimney where it is exposed between the two roof levels.

5.1 North Elevation (main entrance)

There is vertical cracking to the gable elevation either side of the central column locations, suggesting a separation between the concrete frame and the masonry infill panels.

There is vertical and diagonal cracking to the masonry of the porch construction below the window opening.

There is horizontal cracking to the wing walls of the porch construction just above the internal floor level.

5.2 South Elevation (rear gable)

There is a significant horizontal crack in the lean-to store room, extending from the south west corner of the building at lintel height and extending to the masonry panel between the window openings one block course below lintel height.

Similar to the North elevation, cracks are evident between the concrete frame and the masonry panel, indicative of separation

5.3 West Elevation (facing car park)

There are some fine stepped cracks in the infill cavity panels below the ground floor windows. These run mainly through the joints.

At the reinforced concrete columns there are some fine horizontal cracks above and below the window openings at what appear to be construction joint locations.

Some columns have a number of hairline horizontal cracks in their lower section, in the zone of the infill masonry panel contiguity, suggesting out-of-plane flexure.

That window opening, located in the second infill wall bay from the north gable, has a diagonal crack extending upwards and away from the opening.

There are generally cracks between the columns and the adjoining masonry panels, suggesting a degree of separation between the two.

There is a diagonal stepped crack at the south corner of the building, extending from foundation level at the door opening upwards towards lintel height at the corner of the building.

5.4 East Elevation

Generally, this elevation shows little sign of damage, but there is localised horizontal cracking below some window openings.

Within the roof attic space, it was noted that there were cracks present in the lower face of the corbelled pad-stones at column supports for the second and third roof trusses from the north gable. (Not all pad-stones were inspected.) This may be due to spalling of the concrete through damage, or due to poor compaction during construction.

A limited inspection of the construction of the north gable wall cavity was possible because of a convenient void which had opened up due to mortar loss between the inner leaf of the masonry infill panel and the concrete frame, No mechanical connections between the frame and the infill panels, either horizontally or vertically, and no wall cavity ties between the two leaves (wythes) of masonry were visible at this location (but ties were observed elsewhere during the intrusive inspection.)

The floor slab in the porch is cracked across its width (north to south)

There is horizontal cracking in the URM cavity infill wall panels, readily visible in the toilet area, at the north west corner of the building, at or close to lintel height.

6 Detailed Seismic Assessment

6.1 Critical Structural Weakness

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 7 May 2011, the term ‘Critical Structural Weakness’ (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of the building.

With the level of information currently available the following potential CSW’s were identified during the qualitative stage and checked during this quantitative assessment.

6.1.1 Cavity Walls

The cavity infill unreinforced masonry walls have no mechanical connection to their surrounding beams or columns and so represent a falling hazard for occupants and pedestrians in that they may "pop-out" of the building during a significant seismic event.

6.1.2 Short Columns

The reinforced concrete columns between the windows on the three exposed elevations may be behaving as "short columns" due to deflection constraint provided by the infill cavity wall panels. The concrete columns are not adequately reinforced with steel to resist the redistributed forces to which they could be subject to in a large seismic event. The longer columns of the main hall will tend to redistribute a proportion of the seismic load to the stiffer short columns of the lower hall. However, depending upon the principal direction of actual seismic loading it is possible that the infill masonry panel would fail in an out-of-plane mode prior to the "short column effect" mechanism causing potential building collapse. Notwithstanding this the "short column" mechanism must be protected against in the event that it is decided to repair the infill walls as part of a seismic retrofit strategy.

6.1.3 Chimney

The reinforced concrete chimney effectively forms a "short column" between the two ridge lines of the stepped roof levels, as evidenced by the cracking patterns. The chimney has sustained significant damage, with vertical cracking and spalling of concrete and horizontal cracking at the mid height between the ridges

6.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170 for this building are:

- Site soil class: D – Soft Soil, clause 3.1.3 NZS 1170.5:2004
- Importance Level 2 structure (for a building where no more than 300 people can congregate) with a 50 year design life
- Site hazard factor, Z = 0.3, SESOC Christchurch Seismic Design Load levels Interim Advice, Building Code B1/VM1 amendment, August 2011,
- Return period factor Ru = 1.0 from table 3.5 NZS1170.5:2004, for an importance level 2 building. (Note: should the building be identified as being an importance level 3 structure where more than 300 people can congregate, then Ru = 1.3).

Based on our assessment of the structural drawings, our initial estimates for the expected minimum structural ductility factors for the main reinforced concrete frame seismic resisting systems are:

- $\mu max = 1.25$, Transverse (East to West direction)

and

- $\mu max = 1.25$, Longitudinal (North to South direction)

The ductility factor is restricted in the transverse direction because no concrete beams are designed in this direction parallel to the roof trusses (except at the building ends and change of roof pitch at chimney), so the only moment resisting capacity is located at column to footing connection, which is of marginal capacity.

The out-of-plane capacity of the infill URM cavity walls governs the building capacity over-all, and for this action, a ductility factor of 1.25 was assessed as most appropriate.

The CMU compressive strength was assumed as $f'_b = 20$ MPa

6.3 Quantitative Assessment Results

A summary of the structural performance of the building is shown in the table below. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building will have significantly greater capacity when compared with the governing elements.

The results are tabulated as follows:-

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode or description of limiting criteria based on elastic capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Cavity Wall, out of plane at 4.5m. section at entrance	Out-of-plane instability due to excessive deflection.	Yes	6-11% NBS
Cavity Wall out of plane at 3.0m section between windows	Out-of-plane instability due to excessive deflection.	Yes	8-15% NBS
R.C. Columns loaded in transverse direction	Flexural failure of the reinforced 300mm concrete columns that must support the mass of the cavity walls.	Yes	24% NBS
Footings of transversely loaded R.C. columns	Overturning failure of the pad footing beneath the columns	Yes	24% NBS

6.4 Discussion of results

Based on the information available, the building has been assessed as having a seismic capacity of 6% of new building standard (%NBS), using the detailed engineering evaluation process (New Zealand Society for earthquake engineering, "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, 2006)[2]"

The overall capacity was limited by the out of plane strength of the masonry infill panels. The RC columns are rated at 24%NBS in the transverse direction due to possible flexural failure or overturning failure of the foundations.

As the building has a capacity of less than 34% NBS it is defined as earthquake prone in accordance with the Building Act 2004. The building therefore has a relative risk of failure of over 25 times that of a building constructed to the new building standard. We recommend that the CCC review the on-going occupancy of this building until such time that any strengthening works have been undertaken. It is recommended that a cordon to 1.0 times the building height be placed around all URM walls.

6.5 Limitations on Assumptions and Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity;
- Assessments of material strengths based on limited drawings, specifications and site inspections;
- The normal variation in material properties which change from batch to batch;
- Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

7 Geotechnical Assessment

The Opus Christchurch geotechnical group have made a desktop study of this area and consider that from the site photos reviewed, and a brief site visit and local shallow excavations, there is no evidence of ground damage at this site. Also, the ECan Solid Facts map suggests the site has low liquefaction potential. No liquefaction was observed near the site, the nearest location of liquefaction was 4.5km east.

A class D soft soil category was assumed but if a structural retrofit of the building complex is to be undertaken then further investigations will be required before any building repairs, in order to confirm bearing capacity and classification.

8 Conclusions

(a) The results obtained from the quantitative engineering calculations indicate that the building has a seismic capacity between 6%-24%NBS with a seismic grade E risk.

(b) The seismic capacity is limited by the capacity of the cavity walls in both directions and the RC columns and foundations in the transverse direction.

(c) Strengthening work is required to increase the overall building capacity to at least 67%.

(d) Earthquake related damage has been noted on a number of structural elements.

(e) Based on the calculated seismic capacity of the building and the observed damage it is recommended that the CCC review the on-going occupancy of the building and provide a cordon around the building.

9 Recommendations

(a) The building is classed as earthquake prone and it is recommended that the CCC review the ongoing occupancy of the building.

(b) It is recommended that a cordon be installed around the full perimeter of the building.

(c) The building should be strengthened to at least 67%NBS.

10 Limitations

(a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is mentioned but this is not intended to be a comprehensive list of non-structural items.

(b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at the time.

(c) This report is prepared for the CCC to assist with assessing remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

11 References

[1] NZS 1170.5:2004, Structural Design Actions, Part 5 Earthquake Actions New Zealand, Standards New Zealand.

[2] NZSEE: 2006, Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering.

[3] NZSEE 2011- Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance, February 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix A – Photographs

1. North Elevation

2. West Elevation

3. East Elevation

4. South Elevation

5. Main Hall – looking North

6. Main Hall – looking South

7. Supper Room – looking North towards Chimney Breast

8. Supper Room – looking South towards Committee Room and Kitchen

9. Kitchen – looking South West towards side entrance

10. Lean-to Store at South end of Building – looking South

11. Wing wall at Entrance (North/West Elevation) - Horizontal cracking

12. North Elevation – Vertical Cracks either side columns of concrete frame

13. West Elevation – Stepped Cracks in blockwork

14. West Elevation – separation cracks between columns and masonry panels

15. West Elevation – Horizontal cracks to columns at construction joint locations below windows

16. West Elevation – stepped cracks in masonry panels

17. West Elevation - Stepped cracking in masonry wall panel

18. Chimney damage

19. West Elevation – south corner – stepped crack in masonry

20. South Elevation – Horizontal cracks in masonry at lintel height

21. Roof construction

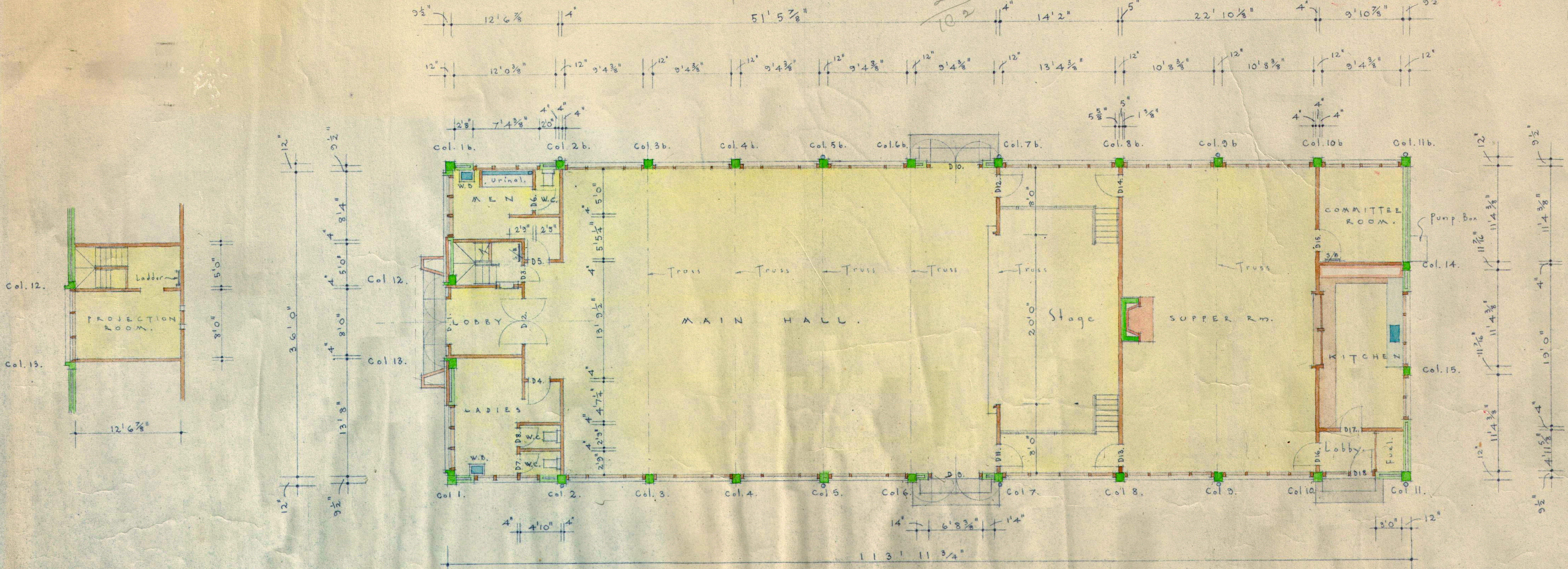
22. Truss supports and eaves beam

23. Eaves beam

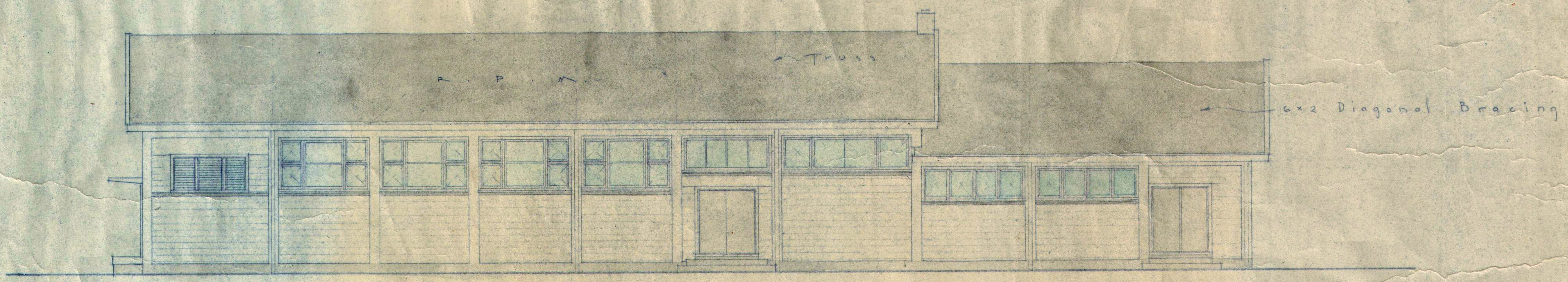
24. Dedication Plaque at front entrance

Appendix B – Drawings

J Hewlett



PLAN



WEST ELEVATION

SET NUMBER 4.
DRAWN 20.3.53
BY L. G. CHILDS

YALDHURST MEMORIAL HALL
PLAN AND ELEVATION

SCALE - 1/8" - 1 FOOT.
DRAWING No 1.

Appendix C – CERA Data Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Yaldhurst Memorial Hall	Reviewer:	Alistair Boyce
	Unit / No: / Street	CPEng No:	209860
Building Address:	Cnr Yaldhurst and Pound Roads	Company:	Opus International Consultants Limited
Legal Description:		Company project number:	6-QUCCC.44
		Company phone number:	
	Degrees / Min / Sec		
GPS south:		Date of submission:	13-Sep-12
GPS east:		Inspection Date:	22-Feb-12
		Revision:	Final
Building Unique Identifier (CCC):	BU 1643-001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	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):	40.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):
Ground floor split?	no		Ground floor elevation above ground (m):
Storeys below ground	0		
Foundation type:	pads with tie beams		if Foundation type is other, describe:
Building height (m):			height from ground to level of uppermost seismic mass (for IEP only) (m): 5.8
Floor footprint area (approx):			
Age of Building (years):	58		Date of design: 1935-1965
Strengthening present?	no		If so, when (year)?
			And what load level (%g)?
Use (ground floor):	public		Brief strengthening description:
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	timber truss	truss depth, purlin type and cladding	2.60m; timber; profiled metal sheeting
Floors:	timber	joist depth and spacing (mm)	not known
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	various - see drawings
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	mostly 300mm sq some 380x355
Walls:	non-load bearing	0	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	concrete frame with infill		note total length of wall at ground (m):	
Ductility assumed, μ:	1.25		wall thickness (m):	
Period along:	0.34	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)		describe system	Concrete columns and timber roof trusses
Ductility assumed, μ:	1.25			
Period across:	0.22	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

leave blank if not relevant

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	Profiled Metal sheeting
Glazing:	timber frames		
Ceilings:			
Services(list):			

Available documentation

Architectural		original designer name/date	
Structural	partial	original designer name/date	E.G.S Powell
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	None visible
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:	yellow		
Along	Damage ratio:		Describe how damage ratio arrived at:	IEP and DEE
	Describe (summary):			
Across	Damage ratio:	#DIV/0!		
	Describe (summary):			
Diaphragms	Damage?:		Describe:	No real diaphragm present
CSWs:	Damage?:	yes	Describe:	Identified in report
Pounding:	Damage?:	no	Describe:	building stands alone
Non-structural:	Damage?:	yes	Describe:	

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	Cavity walls, Rc columns and footings
Building Consent required:	yes	Describe:	removeURM strengthen cols and founds
Interim occupancy recommendations:	do not occupy	Describe:	provide cordon 1.5X height

Along	Assessed %NBS before:		##### %NBS from IEP below
	Assessed %NBS after:	6%	
Across	Assessed %NBS before:		##### %NBS from IEP below
	Assessed %NBS after:	6%	

IEP

Period of design of building (from above): 1935-1965

h_n from above: 5.8m

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):	0.336	0.224
(%NBS)nom from Fig 3.3:		
Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25		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)		1.0
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		
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:	0.30
Z_{1992}, from NZS4203:1992	
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)	1.25	1.25
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.14	1.14
Ductiity Scaling Factor, **Factor D**:	1.14	1.14

2.6 Structural Performance Scaling Factor:

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

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ — **$\%NBS_b$:** **#DIV/0!** | **#DIV/0!**

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

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	significant	0.7
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
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)	**0.00**	**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!**

OPUS