Fendalton Community Centre

Quantitative Engineering Evaluation

Functional Location ID: BU 0449-001 EQ2

Address: 170 Clyde Road, Christchurch

Reference: 232643

Prepared for:

Christchurch City Council

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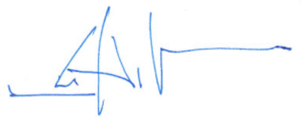
Author Signature		Approver Signature	
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Appendices

Executive Summary

This is a summary of the Quantitative Engineering Evaluation for the Fendalton Community Centre building and is based on the Detailed Engineering Evaluation Procedure document issued by the Engineering Advisory Group on 19 July 2011, visual inspections, available structural documentation and summary calculations as appropriate.

Building Details	**Name**	Fendalton Community Centre			
Building Location ID	BU 0449-001 EQ2			**Multiple Building Site**	N
Building Address	170 Clyde Road, Christchurch			**No. of residential units**	0
Soil Technical Category	TC2	**Importance Level**	3	**Approximate Year Built**	1966
Foot Print (m²)	1000	**Storeys above ground**	2	**Storeys below ground**	0
Type of Construction	A "T" shaped building with two modules: the first one is a double storey high roof building used as a sport and cultural facility; the second one is a double storey building with a mezzanine. In both structures the roof is supported by mixed frames with steel beams and reinforced concrete columns.				
Quantitative L5 Report Results Summary					
Building Occupied	N	The Fendalton Community Centre is not currently in service, except for the Playcentre annex.			
Suitable for Continued Occupancy	Y	The Fendalton Community Centre is suitable for occupation once minor repairs are done. The Playcentre is suitable to continue occupation.			
Key Damage Summary	Y	Refer to summary of building damage Section 3.1 report body.			
Critical Structural Weaknesses (CSW)	Y	Damage to the canopy (south elevation)			
Levels Survey Results	Y	A level survey has been carried out on 8 October 2012.			
Building %NBS From Analysis	50%	Based on an analysis of capacity and demand.			
Approval					
Author Signature		Approver Signature			
Name	Luis Castillo	Name	Lee Howard		
Title	Structural Engineer	Title	Senior Structural Engineer		

1 Introduction

1.1 General

On 8 October 2012 Aurecon engineers visited the Fendalton Community Centre to undertake a quantitative building damage assessment on behalf of Christchurch City Council. Detailed visual inspections were carried out to assess the damage caused by the earthquakes on 4 September 2010, 22 February 2011, 13 June 2011, 23 December 2011 and related aftershocks.

The scope of work included:

- Re-assessment of the nature and extent of the building damage as stated in the previous assessments (see 1.2).
- Visual assessment of the building strength particularly with respect to safety of occupants if the building is currently occupied.
- Assessment of requirements for detailed engineering evaluation including any areas where linings and floor coverings need removal to expose connection details.

This report outlines the results of our Quantitative Assessment of damage to the Fendalton Community Centre and is based on the Detailed Engineering Evaluation Procedure document issued by the Structural Advisory Group on 19 July 2011, visual inspections, available structural documentation and summary calculations as appropriate.

1.2 Previous Assessments

A level 3 assessment was carried out by Opus engineering Consultants on 24 March 2011.

The report dated 20 April 2012 included:

- Damage Assessment and Remedial Works under the Consultancy Services Agreement.

Damages observed in the level 3 assessment have been reviewed during the inspections related to the present quantitative evaluation.

2 Description of the Building

2.1 Building Age and Configuration

The Fendalton Community Centre is a "T" shaped building with two main modules with annexes: the first one is a double storey high roof building (highest point: 6.5m), called Main Hall, used as a sport and cultural facility; the second one is a double storey high building with a mezzanine, called Auditorium[1] (highest point: 6.2m), and includes a seminar room. In both structures the roof is supported by mixed frames with steel rafters and reinforced concrete columns. There is a breaking point in the roof slopes of the Auditorium which is reflected in the geometry of the rafter.

For both modules, light weight roofing iron is supported by timber purlins 8”x2” (200mm x 50mm) at 4’ (1220mm) for the Main Hall and 3’ (915mm) for the Auditorium. These purlins span at approximately

[1] This section is referred as the “Library” in the architectural drawings (*Waimairi County Council*) dated October 1966. However, the room was called “Auditorium” in the level 3 assessment by Opus engineering Consultants which refers to the identification plate on the room’s door. The name “Auditorium" will be used throughout this report.

3.8 m between the transversal frames; which are composed of a steel profile supported by reinforced concrete columns. The steel profiles are typically 300mm deep sections with flanges 165mm wide.

The annex to the Main Hall is a one storey high building with a similar structure, called Playcentre. The Auditorium's annex is a one storey high Workroom with a steel portal frame, timber framed exterior walls and light roof structure.

The two modules are linked together by a timber structure which is attached to the main Hall concrete frames and the Auditorium structure.

The building is approximately 1000 square meters in floor area and is considered to be an importance level 3 structure in accordance with AS/NZS 1170 Part 0:2002. The importance level 3 criteria on Table 3.2 of AS/NZS 1170.0:2002 was confirmed by Christchurch City Council's website[2] according to the maximum capacity of the building. At its maximum capacity the building can receive 330 people, which is higher than the importance 2 criteria limit. The Fendalton Community Centre was built in 1966.

2.2 Building Structural Systems Vertical and Horizontal

The roof gravity loads are supported by the purlins and brought down to the foundation by the steel rafters and the concrete columns. The Workroom annex and the link between the modules are the only exceptions. The Workroom is a steel portal frame (rafter and column) with structural exterior wall and the link is a timber structure attached to the modules concrete frames.

Transverse lateral loads are resisted by the rafter and column moment frames. Lateral loads originate from both the roof structure and the exterior longitudinal masonry walls. These are restrained at the connection between the steel rafters and the concrete column. The column depth varies from 300 mm at the bottom to 460 mm at the top.

Longitudinal roof loads and the loads from the transverse masonry walls are resisted by the concrete frames infill or partially infill with masonry. Reinforced concrete beams span from column to column at the top and at mid height.

2

http://ccc.govt.nz/cityleisure/communityservices/facilitiesforhire/fendaltoncommunitycentre.aspx#jumplink6

2.3 Reference Building Type

A general overview of the reference building type, construction era and likely earthquake risk is presented in the figure below. The Fendalton Community Centre has been constructed in 1966 and according to the figure below, may be seismic prone.

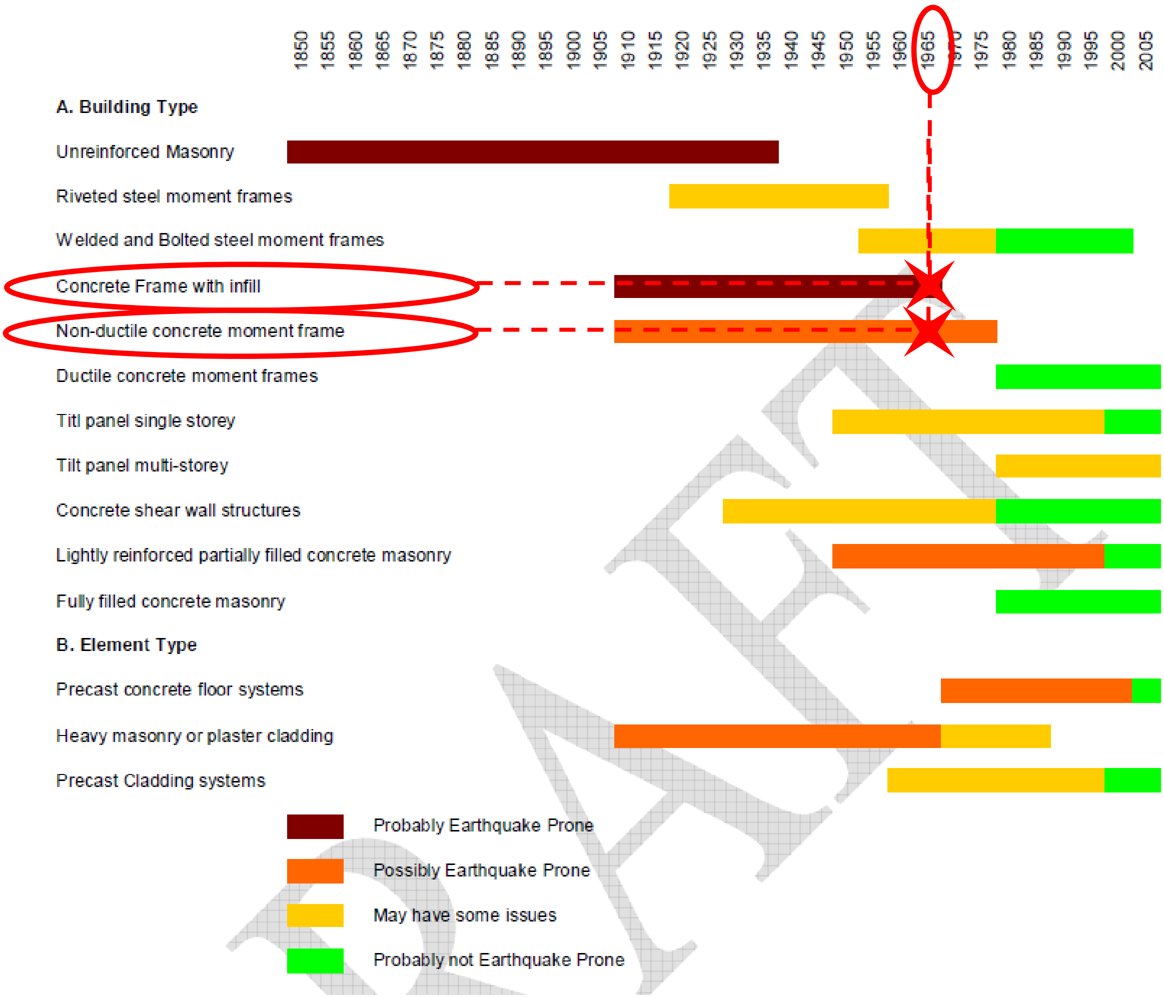

Figure 1: Timeline showing the building types, approximate time of construction and likely earthquake risk. (From the Draft Guidance on DEEs of non-residential buildings by the Engineering Advisory Group)

2.4 Building Foundation System and Soil Conditions

Soil in this area is categorised as technical category 2 (TC2) yellow. According to CERA, TC2 land is considered to “incur minor to moderate land damage from liquefaction and may require specific design for foundations”.

According to the intrusive investigations done (see 3.2 and Appendix C), the foundation system of each module is different. The Auditorium sits on a standard strip footing 300mm deep and the Main Hall system is composed of concrete piles. During the intrusive investigations, 1m of the pile was exposed and its bottom end was not reached. We assumed 1.5m deep piles for calculation purposes.

2.5 Available Structural Documentation and Inspection Priorities

A few architectural drawings and other documents were available.

Electronic copies of the following drawings were provided by CCC on 4 October 2012:

-Proposed Fendalton Community Centre architectural drawing (Waimairi County Council), sheet 1 of 17 (Site Plan) and 3 of 17 (Floor Plan), dated October 1966.

-Proposed Workroom Fendalton Library drawing (Waimairi County Council), sheet 1 of 3 and 2 of 3, dated November 1969.

-Complete Specification (Waimairi County Council), dated December 1966.

No structural calculations or drawings (other than those from the Workroom) were available for review.

2.6 Variation between the drawings and the existing building

The architectural floor plan was considerably modified from the original drawings. Since most of the changes involved only internal walls, the differences do not influence the building's earthquake behaviour.

2.7 Available Survey Information

A floor level survey was undertaken to establish the level of unevenness across the floors. The results of the survey are presented on the attached sketch in Appendix A. All of the levels were taken on top of the existing floor coverings which may have introduced some margin of error.

The Ministry of Business, Innovation and Employment (MBIE) revised guidance "Repairing and rebuilding houses affected by the Canterbury earthquake" recommends some form of re-levelling or rebuilding of the floor

1. If the slope is greater than 0.5% for any two points more than 2m apart, or
2. If the variation in level over the floor plan is greater than 50mm, or
3. If there is significant cracking of the floor.

It is important to note that these figures are recommendations and are only intended to be applied to residential buildings. However, they provide useful guidance in determining acceptable floor level variations.

Code requirements covering acceptability criteria for the floors of buildings are written for new buildings and are not appropriate for older buildings which would have settled with time.

Aurecon performed two Level Surveys: the first is dated 12 October 2012 and the second was issued on the 11 April 2013.

The latest level survey performed indicates that the floor is out of level in the south-west corner of the building. The maximum variation across the building was recorded as 68mm (on the same floor covering) which is outside of the 50mm criteria. The slope of the floor exceeds 0.5% in several locations as shown on the attached drawing S-03 & S-04.

The mezzanine floor is also out of level and from the survey results and it appears that the levels are similar to those of the ground floor below which confirm that the perimeter foundation has settled in the south-west corner of the building causing the mezzanine floor supports to settle by the same amount. The measured slopes exceed the minimum requirement specified by the MBIE criteria and as a result, the floor in the south-west corner of the building will need to be re-levelled to achieve acceptable floor slopes.

3 Structural Investigation

3.1 Summary of Building Damage

The Fendalton Community Centre was not occupied at the time the assessment was carried out and is not currently in use. The only exception is the Playcentre section which was not occupied during the inspection, but is currently in use.

As a reference, the level 3 report helped to target the main areas of damage. The following damages were noticed and reviewed during the inspections of the quantitative assessment;

- Cracked glass panels to the auditorium's upper roof,
- Cracks to gib board in the auditorium at beam/wall connections (typical),
- Hairline cracks to column at its intersection with the mezzanine's beam (typical),
- Horizontal displacement in beam-column connection in the kitchen (±15 mm),
- Cracks in the kitchen's columns and beams,
- Stepped cracks to the masonry wall in the kitchen,
- Diagonal cracks in the concrete panels of the main Hall,
- Horizontal cracks at the base of the main hall's columns, on the east side (typical),
- Cracks to the canopy and its concrete block support,
- Vertical crack on an Hall Entrance column,
- Column cracking at the Hall Entrance, near of its connection with the timber beam,
- Horizontal crack to column at mid height (where the window and the masonry wall connect),
- Cracking to brick veneer on the north/east side of the Workroom.

The damages observed only during the inspections of the quantitative assessment are summarized as follows:

- Cracks in the gib panels starting from opening's upper corners (windows, doors, etc.) (typical) (see photographs, Ref 22),
- Displacement signs at connections between architectural and structural elements (see photographs, Ref 23),
- Small brick damage on the Playcentre's exterior wall (see photographs, Ref 24).

3.2 Record of Intrusive Investigation

Intrusive investigations were carried out on 5 November 2012 in order to observe the type, size and set up of critical structural elements such as rafters, purlins, columns, connection details, structural reinforcement, masonry walls and foundations.

In the auditorium, the roof structure and the rafter-column connections were concealed by wood tiles and the side of the rafter, by wood planks. Small sections of the architectural elements were demolished in order to observe the structural elements. No obvious damage or residual deformations to rafters, rafter-column joints, purlins or cleat were noted. The same observation applies to the Main Hall. Besides, the connections between the rafter and the purlins are different for this module. No cleat is installed and the purlins are only supported by the lower flange. Timber blocking stabilizes purlins laterally.

The presence of a diagonal stiffener was confirmed to the rafter-column joints. Furthermore, a metal detector was used in order to find if the rafter section continues through the column. If it's not, the vertical and horizontal reinforcement spacing can be measured along the columns of the two modules.

Using a rebar scanner on the upper section of the column the readings received were uninterrupted suggesting the presence of a steel profile imbedded in the concrete column. Within approximately 2 metres above the ground steel reinforcement was detected. The demolition of a small portion of concrete cover located near the bottom end of the column, on the exterior of the building, confirmed the presence of stirrups and 25 mm longitudinal bars. No demolition was carried out on the upper section of the columns to avoid incurring any weaknesses, as this rafter-column connection will undergo the most important loads during an earthquake. The location of the end of the steel profile within the column was not detected.

As a conservative assumption, we supposed that maximum moment was developed in the steel section and in the rebars probably welded to the steel section or to a plate. This maximum moment was also compared to the reinforced concrete column capacity as the exact location of the steel section interruption cannot be found.

The metal detector was also used to measure the vertical and horizontal reinforcement spacing in the concrete panels located in the Main Hall (300mm, centre to centre, in each direction) and to find out if the masonry concrete blocks were reinforced or not. No rebar was detected in the blockwork.

Digging works were carried out in order to expose the foundations and observe their characteristics, see 2.4 (Building foundation system and soil conditions).

3.3 Damage Discussion

Most of the damages occurred between the auditorium and the main Hall. Those two stiff modules are built with similar, but partially independent structures. They are linked by a timber structure attached to the concrete elements of the modules. Those concrete elements are basically the continuity of the visible outside concrete frames without masonry infill. During an earthquake, the two structures will have different behaviour in regards to the direction and the amplitude of the displacement. The timber structure between the two modules will act as an expansion gap because of its low relative stiffness. Consequently, damage to this section (Hall Entrance, kitchen and toilet) is more likely to occur.

The masonry infill located in the bays of the structure increases the stiffness of the concrete frames. However, some bays are infill-free (4 frames in a row at the modules link) or partially infill to ensure the openings (windows or doors). As the relative stiffness of those frames is lower, their displacement might be higher and more cracking prone, especially at the top of masonry height if partially infill. At the modules link concrete panels are installed at mid height of the Main Hall and must transfer all shear loads to the columns. As some damages were observed, their design must be analysed.

Furthermore, rocking is more likely to occur where the infill bays ends and the infill-free bays start. This damage results in a vertical (or slight diagonal) cracks at the edge of a frame or panel and was observed in the Hall Entrance to the column located on the gridline intersection axis D and 5. The cracked section of this column must not be considered in the calculations as it no longer helps the frame in compression.
The canopy is located to the other extreme of this infill-free concrete frame row. Displacement signs and damages were also observed to the canopy.

4 Building Review Summary

4.1 Building Review Statement

As no calculations and few drawings or documentations were available, assumptions had to be made in order to complete calculations using current NZ standards (Refer to Appendix B).

4.2 Critical Structural Weaknesses

Critical structural weaknesses were identified to the canopy located on the southern elevation (see photographs, Ref 10 and 11). A temporary structure is currently installed and adequate to prevent hazards.

We feel inclined to highlight our concerns in regards to the purlins in the Main Hall module, even though it may not cause a global failure of the structure. As observed, no cleat connects the purlin and the rafter (see photographs, Ref 29). Consequently, purlins are only supported by the lower flange of the rafter and laterally stabilized by timber blocking. Even if the structural element performed well during the earthquakes, this set up is not considered as good practice and may cause future problems.

5 Building Strength (Refer to Appendix C for background information)

5.1 General

Independently, the two main modules seem to have performed effectively during the Canterbury earthquakes. However, a few signs of concerns have been observed to the link between those two modules. The damages description and photograph are included in the level 3 assessment by Opus.

5.2 Initial %NBS Assessment

The seismic design parameters used to complete this strength assessment are based on current design requirements from NZS1170:2002 and the NZBC clause B1. For this building, the parameters are:

Table 1: Parameters used in the Seismic Assessment

Seismic Parameter	Quantity	Comment/Reference
Site Soil Class	D	NZS 1170.5:2004, Clause 3.1.3, Deep or Soft Soil
Site Hazard Factor, Z	0.30	DBH Info Sheet on Seismicity Changes (Effective 19 May 2011)
Return period Factor, R_u	1	NZS 1170.5:2004, Table 3.5
Ductility Factor for the concrete frame with masonry infill panel in the longitudinal Direction, μ	1	Concrete frame infill with masonry.
Ductility Factor for the steel rafter with concrete columns in the transversal Direction, μ	1.5	Steel beams with concrete columns.
Ductility Factor for timber frame walls	1.5	Timber frame walls

Despite the use of best national and international practice in this analysis and assessment, the values are uncertain due to the many assumptions and simplifications which were made during the assessment (Refer to Appendix B for the limitation and assumptions).

A structural performance summary of the building is shown in the following tables. Note that the values given represent the critical elements in the building. When redistributed, the values can be relied on as these effectively define the building's capacity.

Table 1: Summary of Performance

Structural Element/System	Comments [1]	%NBS Based of Detailed Assessment
LONGITUDINAL DIRECTION		
AUDITORIUM (Library)		
Concrete frame infill with masonry.		**64%**
Sliding shear failure of the masonry wall	This type of failure does not create a complete failure mechanism in the structural system. Once the sliding occurs, the diagonal compression strut is still restrained by the concrete frame corners, which still helps to resist the shear force. Basically, the strut is acting like an X bracing working in compression only. As the lateral force resisting system is still able to act properly, the corresponding %NBS will not be considered for the general building capacity.	16% Not considered for general capacity
Compression failure	When the compression strut exceeds the capacity of the masonry, the failure is brittle.	>100%
Diagonal tension failure of the panel	The cracking capacity of masonry depends on the orientation of the principal stresses with respect to the bed joints. This failure is basically the cracking of the bed joint.	>100%
General shear failure of the panel	It defines the cycling loading effect on the masonry panel.	64%
MAIN HALL + PLAYCENTRE		
Concrete frame infill with masonry.		**65%**
Sliding shear failure of the masonry wall	This type of failure does not create a complete failure mechanism in the structural system. Once the sliding occurs, the diagonal compression strut is still restrained by the concrete frame corners, which still helps to resist the shear force. Basically, the strut is acting like an X bracing working in compression only. As the lateral force resisting system is still able to act properly, the corresponding %NBS will not be considered for the general building capacity.	31% Not considered for general capacity

Structural Element/System	Comments [1]	%NBS Based of Detailed Assessment
Compression failure	When the compression strut exceeds the capacity of the masonry, the failure is brittle.	>100%
Diagonal tension failure of the panel	The cracking capacity of masonry depends on the orientation of the principal stresses with respect to the bed joints. This failure is basically the cracking of the bed joint.	>100%
General shear failure of the panel	It defines the cycling loading effect on the masonry panel.	65%
Concrete column flexural capacity (weak axis) (frames with openings, no masonry infill)	The failure will depend on the type and location of the connection details between the steel sections and the rebars. The structural element was verified assuming that the maximum moment is resisted by the reinforced concrete column. For this case, the maximum stress is located in the middle of the column. If the flexural capacity is exceeded cracking will occur and lead to yielding of rebars. The failure is not brittle but when the columns hinge at the top, a collapse mechanism can form.	68%
Shear failure of the concrete panels (On axis 5, between G & H)	Concrete cracking in the shear plan can lead to eventual yielding of the reinforcement crossing this plan. The failure mechanism itself is not brittle.	93%
WORKROOM		
Steel Portal Frame + Timber frame (exterior wall)		68%
Portal frames flexural capacity (strong axis)	Yielding in flexure of the portal frame's beam and columns. The columns are idealized with "pin" base.	68%
Drift - Longitudinal direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements and premature collapse due to P-Delta effects	76%
Compression capacity of the diagonal timber frame	When the compression capacity is exceeded, the failure can be brittle.	>100%

Structural Element/System	Comments [1]	%NBS Based of Detailed Assessment
TRANSVERSAL DIRECTION		
AUDITORIUM		
Steel / Concrete frame		55%
Steel flexural capacity (strong axis)	Yielding in flexure of the portal frame's beam. The columns are idealized with "pin" base.	100%
Concrete column flexural capacity (strong axis)	The failure will depend on the type and location of the connection details between the steel sections and the rebars. The structural element was verified assuming that the maximum moment is resisted by the reinforced concrete column. Implicitly, the connection is assumed at the upper part of the column. If the flexural capacity is exceeded cracking will occur and lead to yielding of rebars. The failure is not brittle but when the columns hinge at the top, a collapse mechanism can form.	55%
Drift - Transversal direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements and premature collapse due to P-Delta effects	73%
MAIN HALL + PLAYCENTRE		
Steel / Concrete frame		53%
Steel rafters flexural capacity (strong axis)	Yielding in flexure of the portal frame's beam. The columns are idealized with piles which fixe according to the soil	100%
Concrete column flexural capacity (strong axis)	The failure will depend on the type and location of the connection details between the steel sections and the rebars. The structural element was verified assuming that the maximum moment is resisted by the reinforced concrete column. Implicitly, the connection is assumed at the upper part of the column. If the flexural capacity is exceeded cracking will occur and lead to yielding of rebars. The failure is not brittle but when the columns hinge at the top, a collapse mechanism can form.	53%
Drift - Transversal direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements and premature collapse due to P-Delta effects	62%
WORKROOM		
Steel Portal Frame (Out-of-plane) + Timber frame (exterior wall)		50%
Portal frames flexural capacity (Out-of-plane, weak axis)	Yielding in flexure of the portal frame's beam and columns. The columns are idealized as a cantilever.	50%
Compression capacity of the diagonal timber frame	When the compression capacity is exceeded, the failure can be brittle.	>100%

[1] Failure mode, or description of the limiting criteria based on displacement capacity of critical element.

5.3 Results Discussion

Detailed calculations highlighted lower percentages in regards to the concrete columns and out-of-plane Workroom portal frame. Based on the behaviour of these structural elements during the sequence of earthquakes between 2010-2011 and on the inspections made which have shown very few signs of damages, additional strengthening is not considered necessary. Furthermore, the results in regards to the columns of the Main Hall and Auditorium are based on conservative assumptions which may lower the %NBS. However, the result that governs the whole building (Portal frame in Workroom, 50%) comes from structural elements observed on site and shown on drawings (Proposed Workroom Fendalton Library drawing (Waimairi County Council), sheet 1 of 3 and 2 of 3 (west elevation), dated November 1969).

The diaphragm system also appeared to have performed well during the Canterbury Earthquakes to date. Very few observations could be made in regards to this system as no related drawing was available and no roof-purlins nailing patterns were visible. Based again on the behaviour of the roof during the earthquakes and on inspections which have shown no signs of damages, the diaphragm is adequate. It transferred efficiently the loads through the nails, purlins, bolts and weld to the main structure. Furthermore, the Fendalton Community Centre modules, independently considered, are lightweight structures with simple and well defined load paths.

Shear cracking was observed to the concrete panels which link the columns at middle height in the longitudinal direction. These concrete panels are also attached to the problematic timber structure between the two modules which increases the loads transferred to this structural element and may induce torsion as these two sections move independently. Even if the calculated %NBS of that element in shearing is quite high (93%) with conservative assumptions, the torsion may have contribute to the damage. For now, the level of damage to the structural element is not high enough to make a significant impact on the structure behaviour. If the cracks are simply structurally repaired with epoxy, other cracks will occur to this part of the structure, even with less intense earthquakes.

In summary, detailed calculations give a percentage new building standard (%NBS) longitudinally of 64% for the auditorium, 65% for the Main Hall and Playcentre and 68% for the Workroom; transversally of 55% for the Auditorium, 53% for the Main Hall and Playcentre, and 50% for the Workroom, which governs the overall NBS percentage of the building.

6 Conclusions and Recommendations

Given the good performance of the Fendalton Community Centre in the Canterbury earthquake sequence and the lack of foundation damage, **a geotechnical investigation is currently not considered necessary**.

As previously mentioned in *4.2 Critical Structural Weaknesses*, no cleat connects the purlin and the rafter of the Main Hall module. Consequently, we recommend installing angle cleats welded to the rafter and bolted to the purlins.

In regards to the section of the building which links the two modules together, structural modifications should be made in order to allow the independent movement of the modules. These modifications should mainly involve the timber beam-concrete column connections. Then structural repairs to the cracks could be made to the concrete elements and blockwork in the kitchen or Main Hall. In other areas of the building, cracking in concrete elements can be repaired with epoxy. All architecturally related damages can also be repaired without structural concerns.

About the concrete canopy, it is recommended to be completely demolished and rebuilt.

The Fendalton Community Centre (Main Hall, Auditorium and Workroom) is not currently occupied and the building has suffered only minor loss of functionality. In our opinion the Fendalton Community Centre **is suitable for occupation once the recommended works are completed**. The Playcentre is currently occupied and suitable to continue occupation.

7 Explanatory Statement

The inspections of the building discussed in this report have been undertaken to assess structural earthquake damage. No analysis has been undertaken to assess the strength of the building or to determine whether or not it complies with the relevant building codes, except to the extent that Aurecon expressly indicates otherwise in the report. Aurecon has not made any assessment of structural stability or building safety in connection with future aftershocks or earthquakes – which have the potential to damage the building and to jeopardise the safety of those either inside or adjacent to the building, except to the extent that Aurecon expressly indicates otherwise in the report.

This report is necessarily limited by the restricted ability to carry out inspections due to potential structural instabilities/safety considerations, and the time available to carry out such inspections. The report does not address defects that are not reasonably discoverable on visual inspection, including defects in inaccessible places and latent defects. Where site inspections were made, they were restricted to external inspections and, where practicable, limited internal visual inspections.

While this report may assist the client in assessing whether the building should be repaired, strengthened, or replaced that decision is the sole responsibility of the client.

This review has been prepared by Aurecon at the request of its client and is exclusively for the client's use. It is not possible to make a proper assessment of this review without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

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Appendices

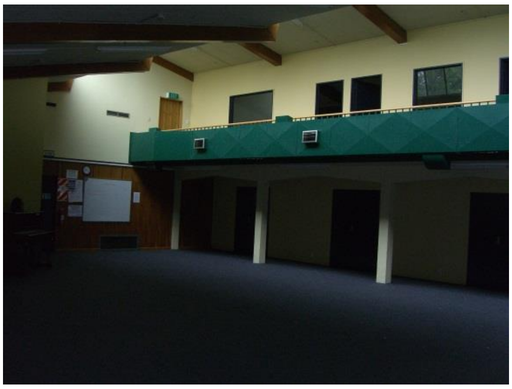
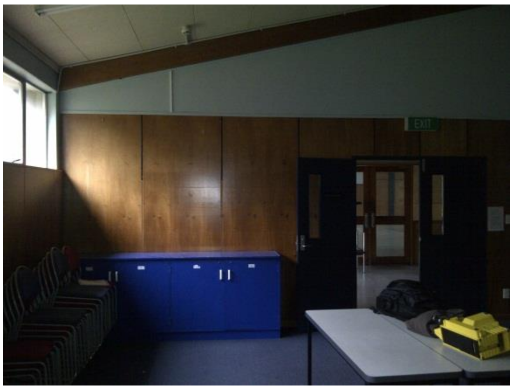
Appendix A
Site Map, Photos, Levels Survey

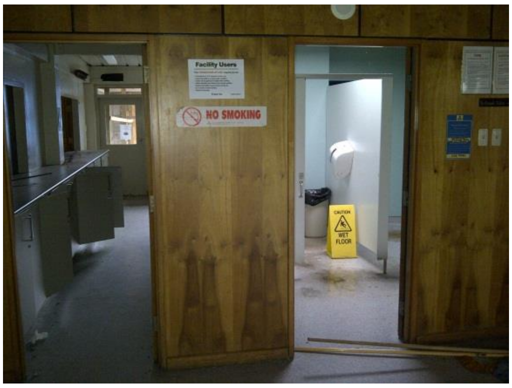
10 January 2012 – Fendalton Community Centre Site Photographs.

Ref	Description	Photographs
1	Main entrance of the Fendalton Community Centre. West façade.	
2	North elevation of the Fendalton Community Centre. North Workroom façade.	
3	North elevation of the Fendalton Community Centre. East façade of the Workroom.	
4	North elevation of the Fendalton Community Centre. Hall Entrance, Playcentre west façade and seminar room north façade.	

Ref	Description	Photographs
5	North elevation of the Fendalton Community Centre. Playcentre north façade.	
6	North elevation of the Fendalton Community Centre. Playcentre north façade.	
7	East elevation of the Fendalton Community Centre. Playcentre East façade and porch.	
8	East elevation of the Fendalton Community Centre. East façade of the Main Hall.	

Ref	Description	Photographs
9	South elevation of the Fendalton Community Centre. South façade of the Main Hall.	
10	South-west elevation of the Fendalton Community Centre. South façade of the Auditorium.	
11	South elevation of the Fendalton Community Centre between the main modules. Lower Part (kitchen, toilet) acting as an expansion gap.	
12	Interior of Auditorium.	

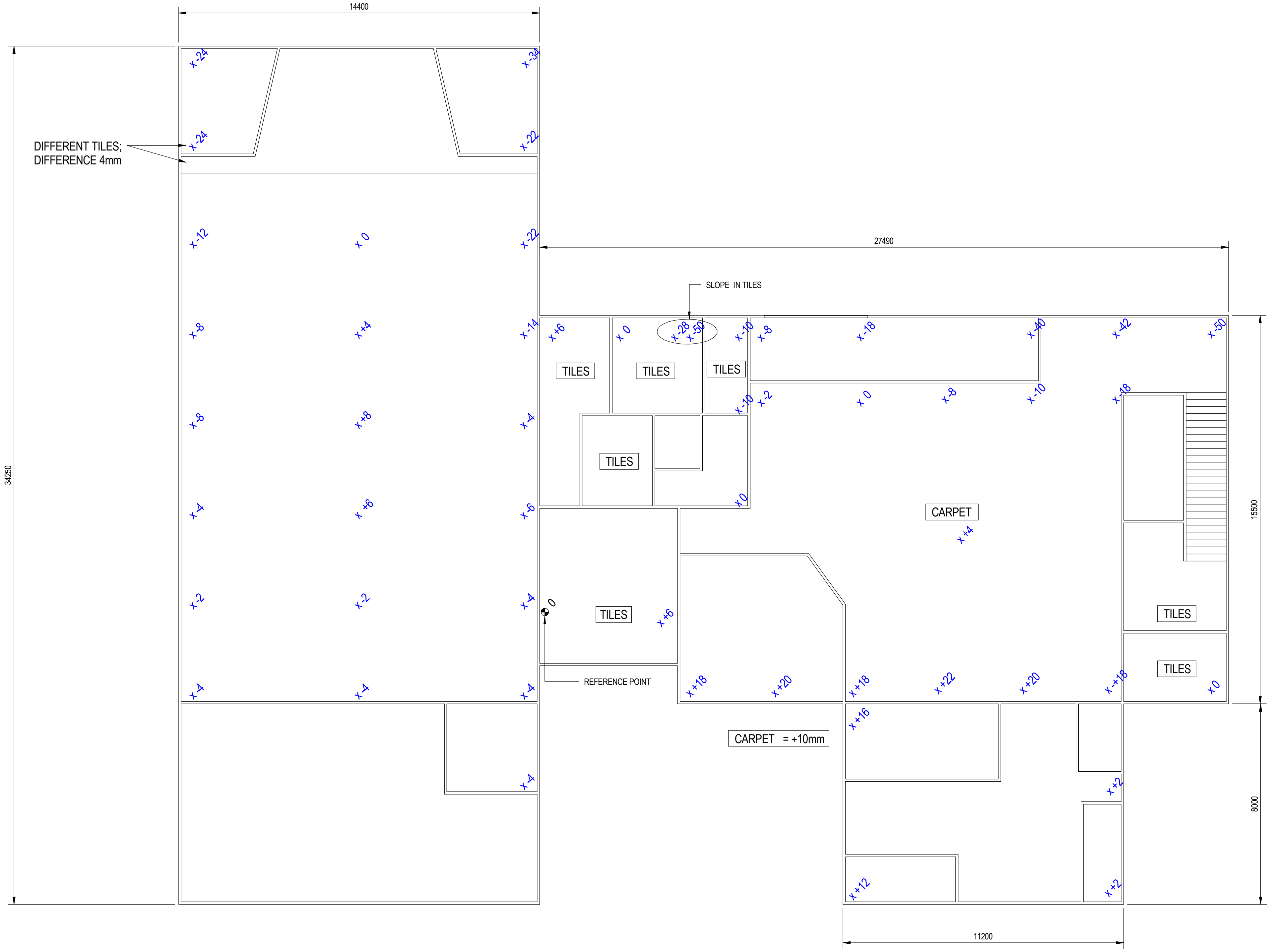
Ref	Description	Photographs
13	Inside the Auditorium.	
14	Auditorium's mezzanine.	
15	Interior of Workroom.	
16	Seminar room between the auditorium and the Hall entrance.	

Ref	Description	Photographs
17	Hall entrance between the two modules.	
18	Kitchen and toilet between the two modules.	
19	Main Hall and stage.	
20	Main Hall from the stage.	

Ref	Description	Photographs
21	Playcentre.	
22	Cracks in the gib panels starting from opening's upper corners (windows, doors, etc.) (typical).	
23	Displacement signs at connections between architectural and structural elements.	
24	Small brick damage on the Playcentre's exterior wall.	

Intrusive Inspections		
Ref	**Description**	**Photographs**
25	Connection between purlins and rafter in the Auditorium.	
26	Blockwork behind architectural wood in the Auditorium.	
27	Foundation below a column of the Auditorium.	

Ref	Description	Photographs
28	Reinforcing bar in the Auditorium's column.	
29	Connection between purlins and rafter in the Main Hall.	
30	Column foundation from Main Hall.	
31	Reinforcing bar in a Main Hall column.	



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CLIENT

REV	DATE	REVISION DETAILS	APPROVAL
A	12.10.12	PRELIMINARY	APP

DRAWN: N.Stanojevic
DESIGNED: G.Lefebvre
CHECKED: L.Castillo
APPROVED: Approver
DATE

PROJECT
Fendalton Community Centre
170 Clyde Road, Christchurch

TITLE
LEVELS SURVEY

PRELIMINARY
NOT FOR CONSTRUCTION

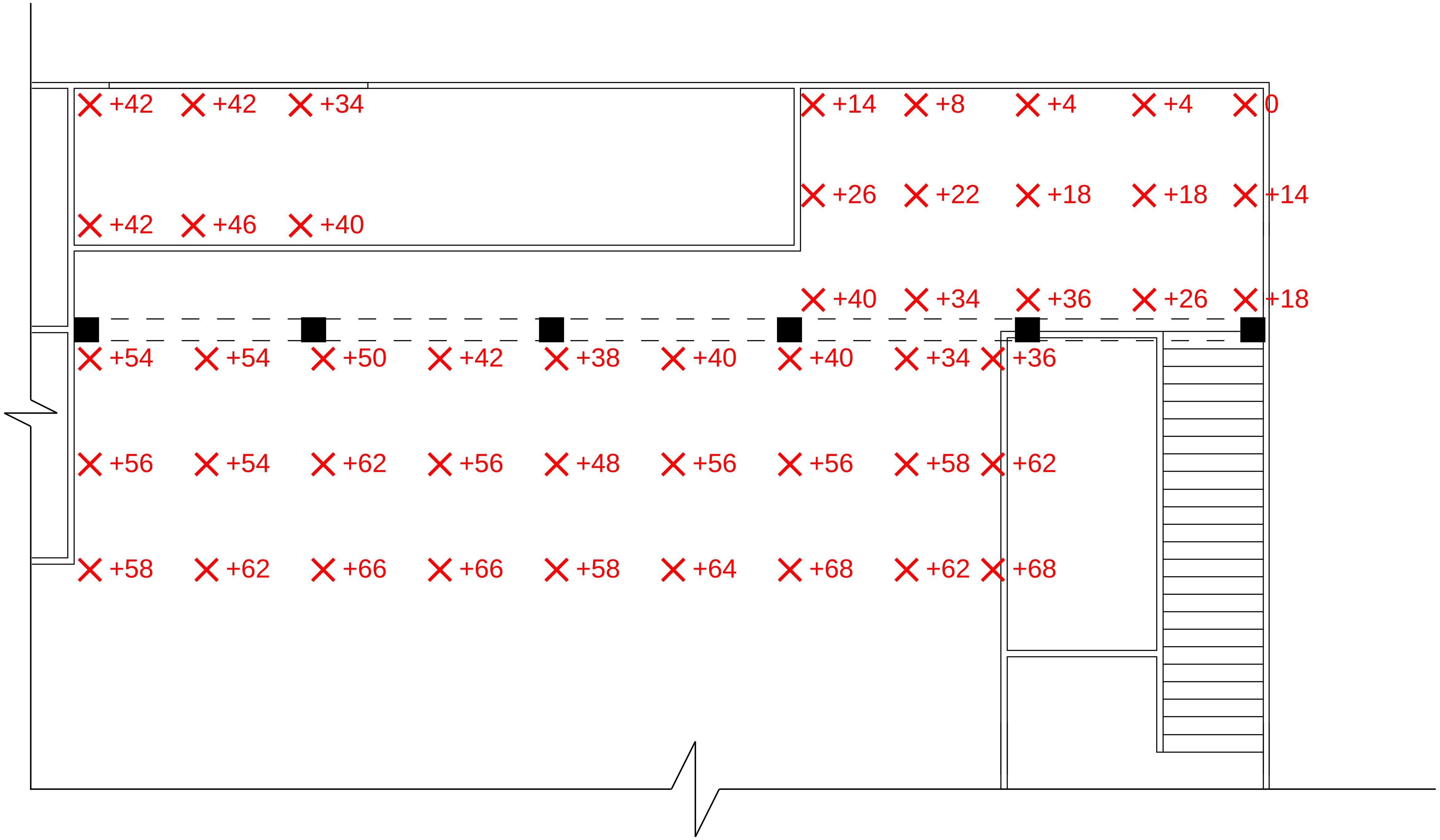
PROJECT No. 232643

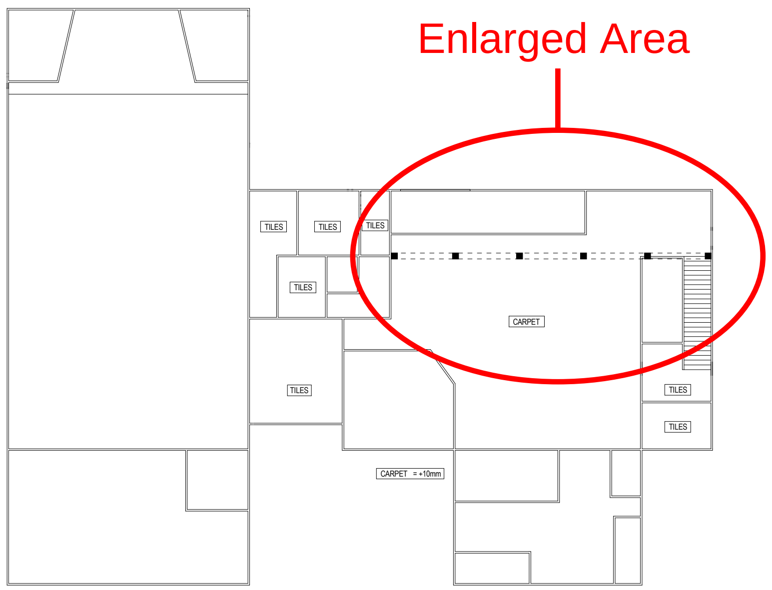
SCALE 1 : 100

SIZE A1

DRAWING No. S-01-00

REV A





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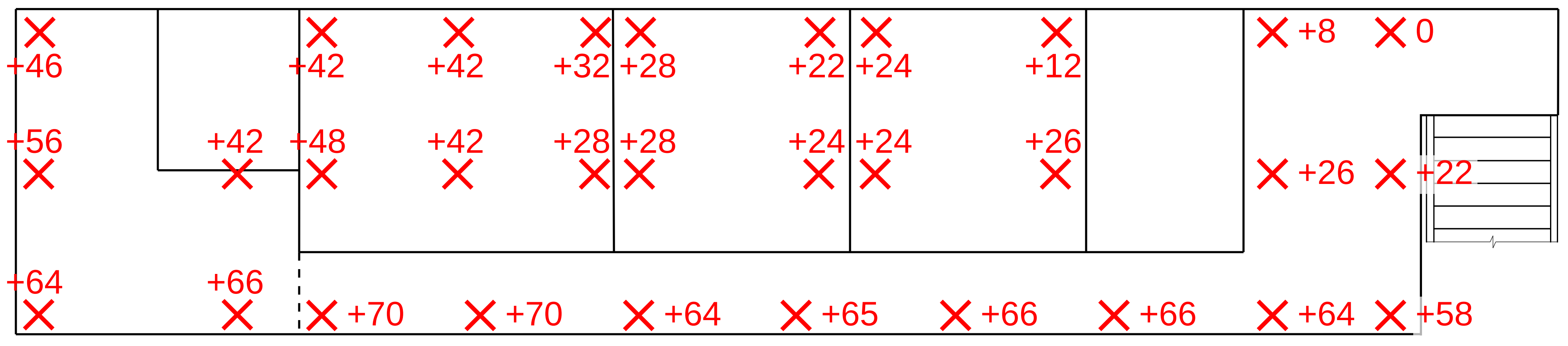
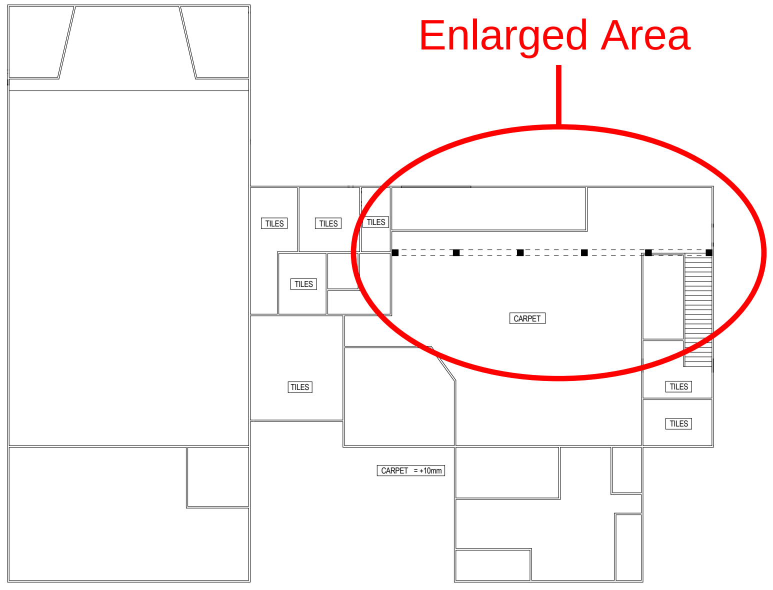
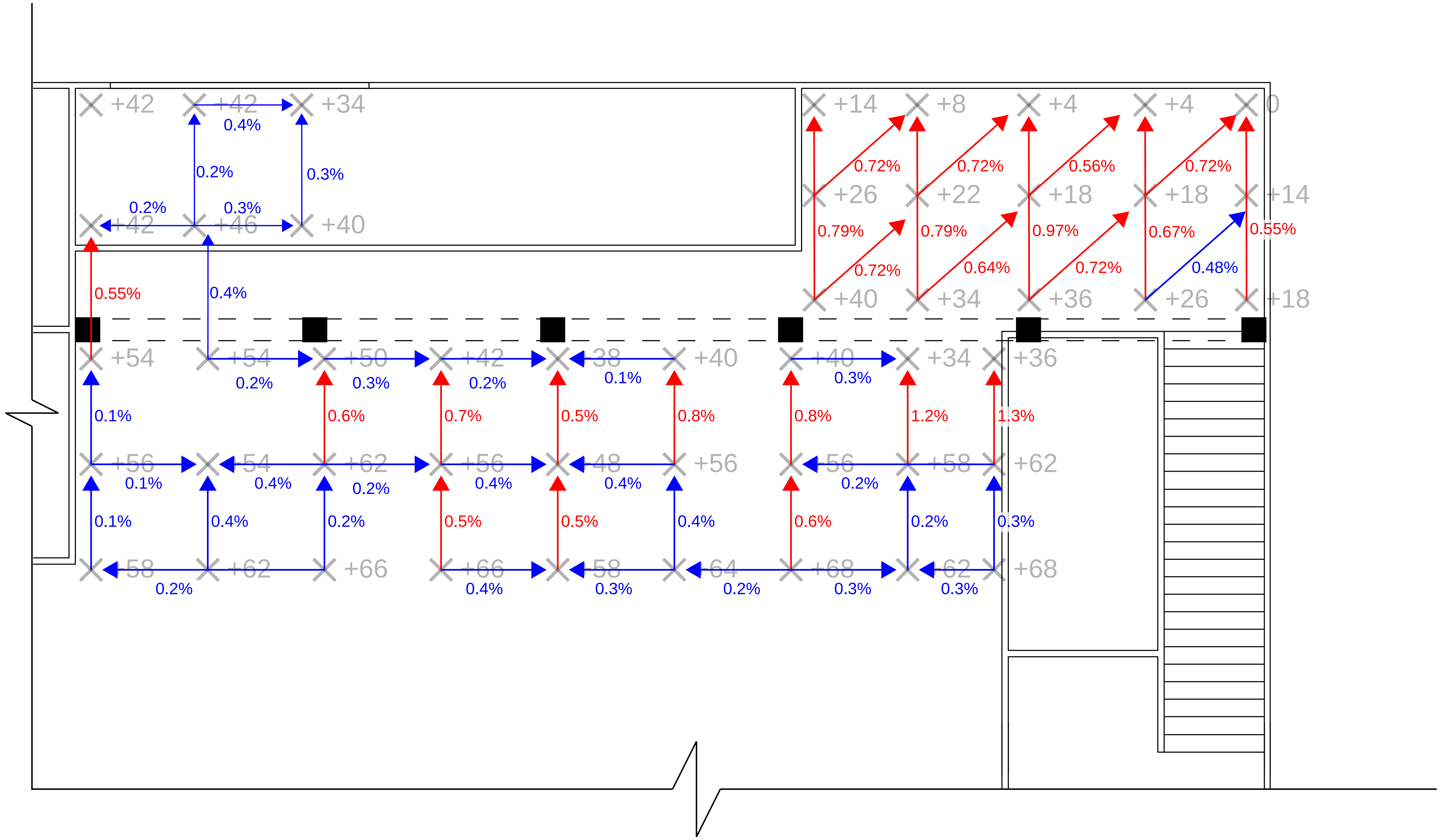
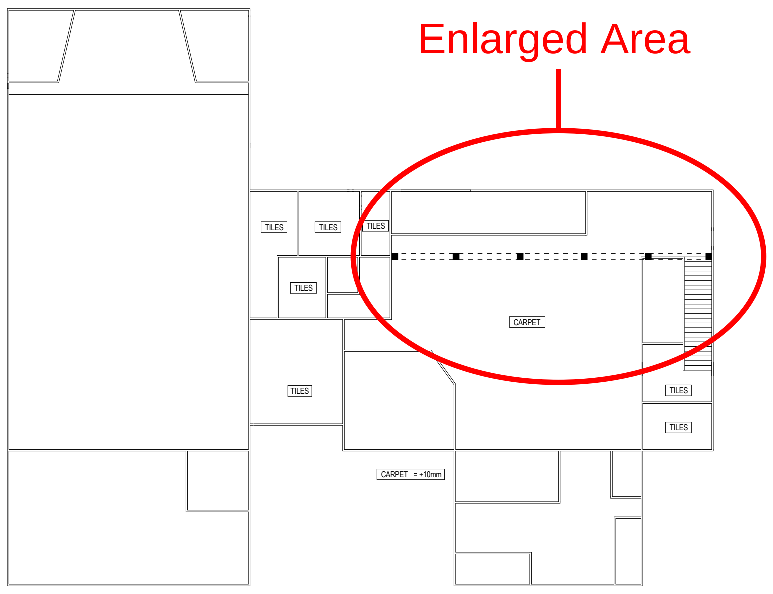
Drawn: D.Goodwin	**Date:** 5/04/2013
Project: Fendalton Community Centre	
Drawing Title: Floor Levels – Ground Floor	
Job Number: 232643	**Drawing:** S-01 **Rev:** 01





Drawn: D.Goodwin	**Date:** 5/04/2013
Project: Fendalton Community Centre	
Drawing Title: Floor Levels - Mezzanine Floor	
Job Number: 232643	**Drawing:** S-02 **Rev:** 01





Slopes less than 0.5%

Slopes greater than 0.5%

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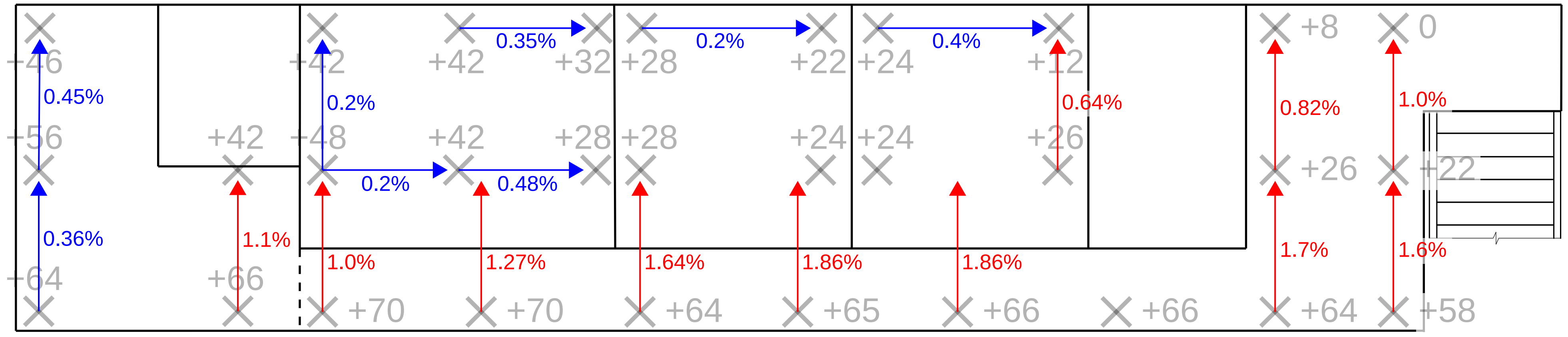
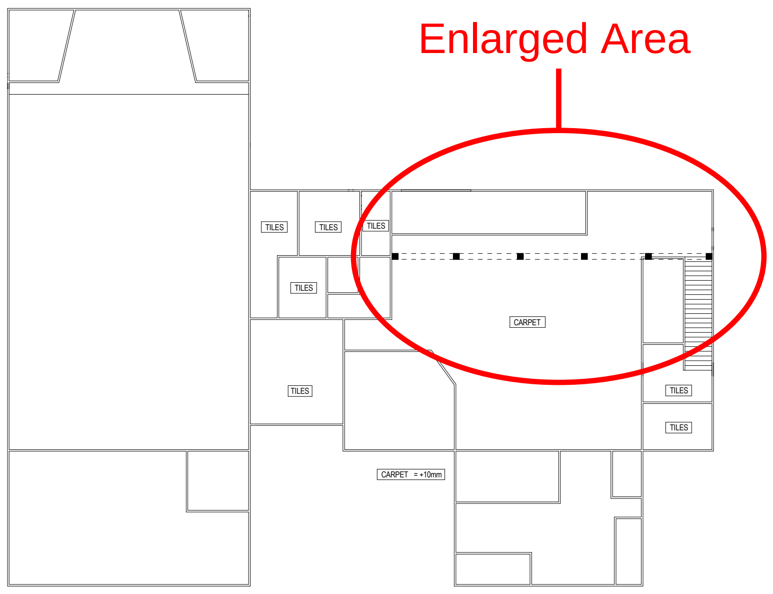
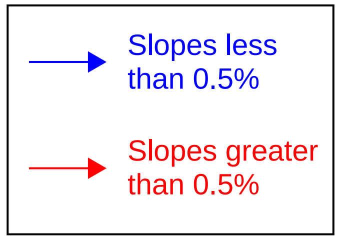
Drawn: D.Goodwin	**Date:** 5/04/2013		
Project: Fendalton Community Centre			
Drawing Title: Floor Slopes - Ground Floor			
Job Number: 232643	**Drawing:** S-03	**Rev:** 01	







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Drawn: D.Goodwin	Date: 5/04/2013
Project: Fendalton Community Centre	
Drawing Title: Floor Slopes - Mezzanine Floor	
Job Number: 232643	Drawing: S-04 Rev: 01

Appendix B

References, Limitation and Assumptions

1. Department of Building and Housing (DBH), "Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence", November 2011

2. New Zealand Society for Earthquake Engineering (NZSEE), "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes", April 2012

3. Standards New Zealand, "AS/NZS 1170 Part 0, Structural Design Actions: General Principles", 2002

4. Standards New Zealand, "AS/NZS 1170 Part 1, Structural Design Actions: Permanent, imposed and other actions", 2002

5. Standards New Zealand, "NZS 1170 Part 5, Structural Design Actions: Earthquake Actions – New Zealand", 2004

6. Standards New Zealand, "NZS 3101 Part 1, The Design of Concrete Structures", 2006

7. Standards New Zealand, "NZS 3404 Part 1, Steel Structures Standard", 1997

8. Standards New Zealand, "NZS 3603, Timber Structures Standard", 1993

9. Standards New Zealand, "NZS 3604, Timber Framed Structures", 2011

10. Standards New Zealand, "NZS 4229, Concrete Masonry Buildings Not Requiring Specific Engineering Design", 1999

11. Standards New Zealand, "NZS 4230, Design of Reinforced Concrete Masonry Structures", 2004

Limitation and Assumptions

The following table resume the limitation and assumptions made in order to complete calculations.

Table 2: Assumptions made

<table>
<tr><th>Assumptions</th><th>Description of the assumptions</th><th>Values</th></tr>
<tr><td>Dead load contributing in seismic calculations.
Auditorium</td><td><table><tr><td>steel structure + timber purlins</td><td>0.15 kPa</td></tr><tr><td>Roofing</td><td>0.15 kPa</td></tr><tr><td>gypsum or suspended ceiling</td><td>0.1 kPa</td></tr><tr><td>Mechanical and electrical services</td><td>0.1 kPa</td></tr><tr><td>Total:</td><td>0.5 kPa</td></tr></table></td><td>0.5 kPa</td></tr>
<tr><td>Dead load contributing in seismic calculations.
Main Hall & Playcentre</td><td><table><tr><td>steel structure + timber purlins</td><td>0.15 kPa</td></tr><tr><td>Roofing</td><td>0.15 kPa</td></tr><tr><td>gypsum or suspended ceiling</td><td>0.1 kPa</td></tr><tr><td>Mechanical and electrical services</td><td>0.1 kPa</td></tr><tr><td>Total:</td><td>0.5 kPa</td></tr></table></td><td>0.5 kPa</td></tr>
<tr><td>Dead load contributing in seismic calculations.
Workroom</td><td><table><tr><td>steel structure + timber purlins</td><td>0.15 kPa</td></tr><tr><td>Roofing</td><td>0.15 kPa</td></tr><tr><td>gypsum or suspended ceiling</td><td>0.1 kPa</td></tr><tr><td>Mechanical and electrical services</td><td>0.1 kPa</td></tr><tr><td>Total:</td><td>0.5 kPa</td></tr></table></td><td>0.5 kPa</td></tr>
<tr><td>f_y of all reinforcing bars.</td><td></td><td>300 Mpa</td></tr>
<tr><td>f_y of all steel sections.</td><td></td><td>300 Mpa</td></tr>
<tr><td>Ductility Factor for the concrete frame with masonry infill panel in the longitudinal Direction, μ</td><td>According to the Section 9-Detailed Assessment of Moment resisting Frame Elements with Masonry Infill Panels, the ductility capacity should be set to $\mu = 1$, unless inelastic structural wall behaviour can be expected.</td><td>1</td></tr>
<tr><td>Ductility Factor for the steel rafter with concrete columns in the transversal Direction, μ</td><td>Steel beams with concrete columns. A nominal ductility is assumed to this type of structure. Conservative value according to AS 1170.4.</td><td>1.5</td></tr>
<tr><td>Ductility Factor for timber frame walls</td><td>Timber frame walls. A nominal ductility is assumed to this type of structure. Conservative value according to AS 1170.4.</td><td>1.5</td></tr>
<tr><td>Thickness of the Main Hall concrete panels.</td><td>Thickness of concrete panels located where the Main Hall and the timber structure connects.</td><td>100 mm</td></tr>
<tr><td>Size of reinforcing bars in the Main Hall concrete panels.</td><td>Size of reinforcing bars in the concrete panels located where the Main Hall and the timber structure connects.</td><td>6 mm mesh
at 300 c/c each direction</td></tr>
<tr><td>Boundary conditions such as foundation fixity.
Auditorium</td><td>Simplifications have been made for analysis. For example, The portal frame’s column is idealized with “pin” base.</td><td></td></tr>
<tr><td>Boundary conditions such as foundation fixity.
Main Hall & Playcentre</td><td>Pile was noted during intrusive inspections. First 1m could be observed and it goes deeper. Pile 1.5 m deep is assumed.</td><td>Pile 1.5m deep</td></tr>
</table>

Boundary conditions such as foundation fixity. **Workroom**	Simplifications have been made for analysis. For example, The portal frame's column is idealized with "pin" base.	
The roof lateral load is evenly distributed in the frames, according to their length.	It is based on the fact that the diaphragm is adequate. By this assumption and the defined load paths, this force is also evenly distributed on the foundation.	
Weight of extremity façades walls in across direction not contributing to the seismic weight when the load is in the transversal direction	The weight from the concrete panels and concrete frames with masonry infill located at the east and west façades for the Auditorium; north and south façades for the Main Hall are not contributing to the seismic weight going through the diaphragm when the seismic load is in the transversal direction. We assumed that the frames bring their own weight directly to the foundation. Consequently, this weight do not affect the steel/concrete frames (Main lateral force resistance system in the transversal direction).	
Weight of all the exterior walls contributes to the seismic weight when the load is in the longitudinal direction	The weight from all the exterior walls (concrete panels and concrete frames with masonry infill) contributes to the seismic weight when the seismic load is in the along direction. Consequently, this weight is evenly distributed in the concrete frames with masonry infill (Main lateral force resistance system in the longitudinal direction).	
The maximum load moment is applied on the reinforced column with the observed rebars.	Maximum load moment is located at the rafter-column joint is developed in the steel section and in the rebars probably welded to the steel section or to a plate. The maximum load moment is compared to the reinforced concrete column capacity as the exact location of the steel section interruption can't be found.	
For the calculation of concrete frame with masonry infill, one layer of block masonry restrains the concrete frames.	All calculations in regards to the concrete frames with masonry infill are made considering only 1 layer (interior layer) of 90mm thick blockwork restraining the frames. However the layer of brick (exterior layer) is considered as additional weight.	
Unbraced length of members in the Workroom Timber frame.	Unbraced length of members in the Workroom Timber frame (L_{ax}= 1.27 m) $L_{ax} = 0.9*\sqrt{(2)} = 1.27m$	L_{ax}= 1.27 m
Approximations made in the assessment of the capacity of each element.	Especially when considering the post-yield behaviour.	

Appendix C
Location of Intrusive Investigations

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aurecon

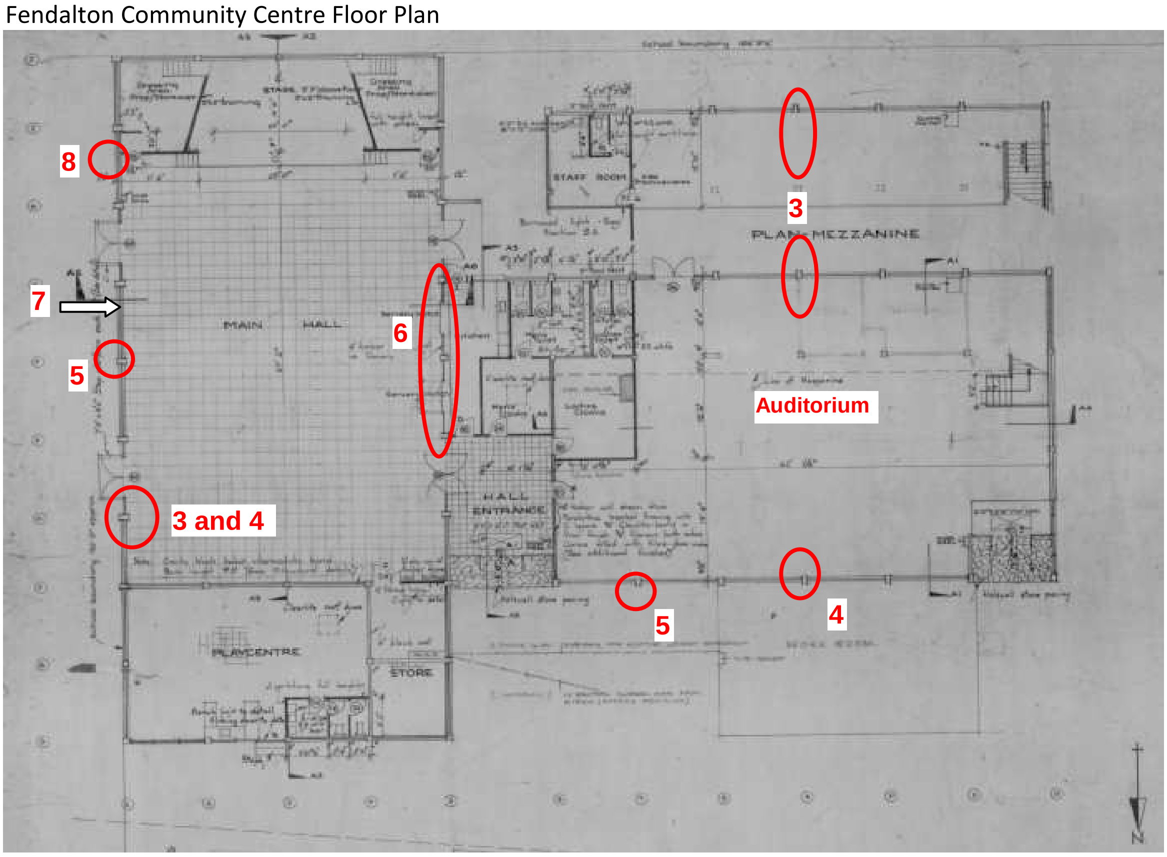
17 October 2012

Mr. Will Rolton
Project Coordinator
Project Management, Capital Programme Group
Christchurch City Council

Dear Will,

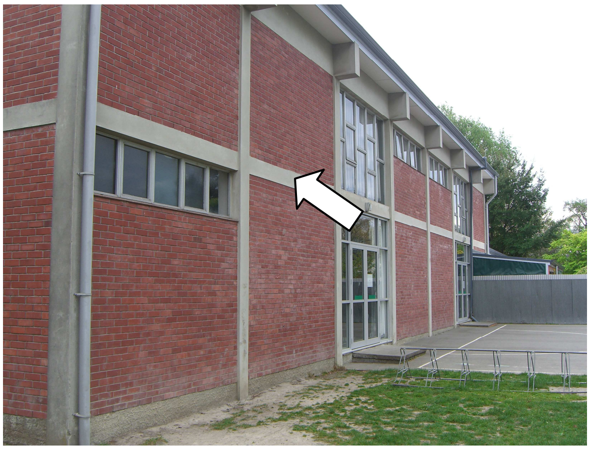
Fendalton Community Centre, 170 Clyde Road, Christchurch – Quantitative Assessment

In regards to the Quantitative Assessment for the Fendalton Community Centre, we would like to request for you to organise with City Care the work in order to conduct intrusive investigation. Several areas of the building will have to be available to observed key structural elements.
The contractor work can be made without an on-site engineer. Contact the structural engineer once the following elements can be inspected.
If concrete demolition is needed to observe the connection detail (#4), wait for the engineer's approval. Below are photos of where we will be conducting the intrusive investigations.

1. Fendalton Community Centre – aerial view.	
2. Fendalton Community Centre Floor Plan 	

3. Remove linings to expose and measure the rafter in both the Auditorium and the Main Hall. (Depth of the section, width and thickness of the flange) (can be made at the same time as #4) Remove the wood panel from the ceiling (for Auditorium) to expose the roof components, to see how the purlins are connected, to measure the distance between each purlins and to see purlin's dimension. Check purlins and rafter directly in the Main Hall. A scissor lift or a scaffolding might be needed and have to be on-site for the engineer inspection. (height: +-6m) (as shown in plan in #2.).	
4. Remove linings to expose connection detail between the rafter and the column. (Can be made to any rafter-column connection in the auditorium and Main Hall, but at least one in each room) Check the reinforcing bar set up if possible and/or type of steel column coated with concrete. Check spacing between vertical and transverse reinforcing with scan (Can be made to any column in the auditorium and Main Hall, but at least one in each room)	 

	 
5. Dig to verify the type and size of foundation (width and depth if possible).	

	
6. Check the reinforcement of the pre-cast panel in the Main Hall. Scan can be used (as shown in plan in #2).	
7. Check the reinforcement of the intermediate concrete beam. Scan can be used (as shown in plan in #2).	

8. Remove bricks locally to observe the wall components (to confirm: probably brick +-100mm + 25 mm space + blocks ?mm)	
9. Remove linings locally to observe the auditorium's wall components (to confirm: probably brick +-100mm + 25 mm space + blocks ?mm)	

Please let me know if you require any further information.

Yours sincerely

Guillaume Lefebvre
Structural Engineer

Luis Castillo
Structural Engineer

Appendix D
Strength Assessment Explanation

New building standard (NBS)

New building standard (NBS) is the term used with reference to the earthquake standard that would apply to a new building of similar type and use if the building was designed to meet the latest design Codes of Practice. If the strength of a building is less than this level, then its strength is expressed as a percentage of NBS.

Earthquake Prone Buildings

A building can be considered to be earthquake prone if its strength is less than one third of the strength to which an equivalent new building would be designed, that is, less than 33%NBS (as defined by the New Zealand Building Act). If the building strength exceeds 33%NBS but is less than 67%NBS the building is considered at risk.

Christchurch City Council Earthquake Prone Building Policy 2010

The Christchurch City Council (CCC) already had in place an Earthquake Prone Building Policy (EPB Policy) requiring all earthquake-prone buildings to be strengthened within a timeframe varying from 15 to 30 years. The level to which the buildings were required to be strengthened was 33%NBS.

As a result of the 4 September 2010 Canterbury earthquake the CCC raised the level that a building was required to be strengthened to from 33% to 67% NBS but qualified this as a target level and noted that the actual strengthening level for each building will be determined in conjunction with the owners on a building-by-building basis. Factors that will be taken into account by the Council in determining the strengthening level include the cost of strengthening, the use to which the building is put, the level of danger posed by the building, and the extent of damage and repair involved.

Irrespective of strengthening level, the threshold level that triggers a requirement to strengthen is 33%NBS.

As part of any building consent application fire and disabled access provisions will need to be assessed.

Christchurch Seismicity

The level of seismicity within the current New Zealand loading code (AS/NZS 1170) is related to the seismic zone factor. The zone factor varies depending on the location of the building within NZ. Prior to the 22nd February 2011 earthquake the zone factor for Christchurch was 0.22. Following the earthquake the seismic zone factor (level of seismicity) in the Christchurch and surrounding areas has been increased to 0.3. This is a 36% increase.

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 C1 below.

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

Figure C1: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table C1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% probability 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% probability of exceedance in the next year.

Table C1: Relative Risk of Building Failure In A

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

Appendix E
Background and Legal Framework

Background

Aurecon has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the building

This report is a Qualitative Assessment of the building structure, and is based 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).

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.

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

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.

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.

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.

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.

Appendix F
Standard Reporting Spread Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Fendalton community centre	Reviewer:	Lee Howard
		CPEng No:	1008889
Building Address:	Unit No: ; Street No: 170 ; Street: Clyde road	Company:	Aurecon NZ Ltd
Legal Description:		Company project number:	232643
		Company phone number:	03 366 0821
GPS south:	Degrees 43, Min 30, Sec 55.28	Date of submission:	Oct-12
GPS east:	Degrees 172, Min 35, Sec 28.30	Inspection Date:	Oct-12
		Revision:	1
Building Unique Identifier (CCC):	BU 0449-001 EQ2	Is there a full report with this summary?	no

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	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):	11.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	11.10
Ground floor split?	no		Ground floor elevation above ground (m):	0.10
Storeys below ground	0			
Foundation type:	raft slab		if Foundation type is other, describe:	
Building height (m):	6.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	3
Floor footprint area (approx):	1000			
Age of Building (years):	46		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL3			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	steel purlins and rafters, concrete column
Floors:	concrete flat slab	slab thickness (mm)	100
Beams:	steel non-composite	beam and connector type	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	255 x 255
Walls:	non-load bearing	0	190

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick	**Note: Define along and across in detailed report!**	note wall thickness and cavity	
Ductility assumed, μ:	1.25			
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 - brick		note wall thickness and cavity	
Ductility assumed, μ:	1.25			
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:	brick or tile	describe (note cavity if exists)	
Roof Cladding:	Metal	describe	0.4mm dimond colour steel
Glazing:			
Ceilings:			
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

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

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	none		Describe:	
	Building Consent required:	no		Describe:	
	Interim occupancy recommendations:	full occupancy		Describe:	
Along	Assessed %NBS before e'quakes:	64%	0% %NBS from IEP below	If IEP not used, please detail assessment methodology:	
	Assessed %NBS after e'quakes:	64%			
Across	Assessed %NBS before e'quakes:	50%	0% %NBS from IEP below		
	Assessed %NBS after e'quakes:	50%			

IEP

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

Period of design of building (from above):	1965-1976	h_n from above:	3m
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:	0.0%	0.0%

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
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

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

Building Importance level (from above):	3
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:	0.925	0.925
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.081081081

2.7 Baseline %NBS, (NBS%)b = (%NBS)nom x A x B x C x D x E

	along	across
%NBSb:	**0%**	**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:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max value =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)	**1.00**	**1.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**0%**	**0%**
4.4 Percentage New Building Standard (%NBS), (before)		**0%**

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