MEMO

To: Mayor and Councillors

From: Liveable City Programme Manager

CC: General Manager Regulation and Democracy Services

Date: 5 October 2010

Re: Council meeting 6 October 2010: Additional information to accompany heritage reports

Further to requests for engineering reports relating to the six heritage reports on the Council agenda Monday 4 October 2010, please find enclosed the following further information for each building.

160 Manchester Street

Ground penetrating radar survey report, 28 September 2010

Memo regarding Council's engineering report, 4 October 2010-10-05

192 Madras Street

Structural engineer's report by Lewis and Barrow Limited, 30 September 2010 (on behalf of the owner)

Structural engineer's heritage peer review by Holmes Consulting Group, 27 September 2010 (on behalf of the Council)

461-469 Colombo Street

Structural engineer's heritage peer review by Holmes Consulting Group, 1 October 2010 (on behalf of the Council)

456 Colombo Street

Structural engineer's heritage peer review by Endel Lust Civil Engineering Limited, September 2010 (on behalf of the Council)

580 Ferry Road

Structural engineer's heritage peer review by Ruamoko Solutions, 30 September 2010 (on behalf of the Council)

Ohinetahi, 31 Teddington Road, Governors Bay,

Structural engineer's report by Holmes Consulting Group, 14 September 2010 (on behalf of the owner)

Earthquake damage report by Sir Miles Warren

Structural engineer's report by Ruamoko Solutions, 17 September 2010 (on behalf of the owner)

DETECTION SERVICES
WATER LOSS MANAGEMENT

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Fiona Wykes
Christchurch City Council
PO Box 237,
Christchurch 8140

28th September, 2010

GPR Survey 160 Manchester Street

Dear Fiona,

Detection Services was engaged by the Christchurch City Council to conduct Ground Penetrating Radar survey on the external walls of the structure at 160 Manchester Street. This was achieved using a cage suspended from a crane to survey the north and west wall structure, in accordance with the methodology provided by Holmes Consulting Group.

The survey was conducted on the 25th and 26th September 2010. Operations had to wait while wind conditions were acceptable.

Survey of the east wall could be completed from the roof top of the adjacent building at 186 Hereford Street. This was very successful and identified a series of targets which, due to their regular grid pattern, indicate a steel framing construction. Further verification should be conducted to confirm the exact elements.

Two main areas of the columns on the north face were surveyed in detail. The GPR showed a dense brick construction of the columns between floors. The returns tended to indicate mortar lines and air gaps between bricks. However, due to the nature of steel returns looking very similar in appearance to an air gap, at larger depths the presence of steel is possible. Maximum depth of penetration for the Structure MiniScan GPR unit utilised in the survey is 400mm, actual column depth is over 550mm.

At floor joist level and in the lintel area above the windows, strong steel reflections were observed.

Further minor investigative work should be conducted to ascertain the interior construction between floor levels. This would be simple enough to achieve given a detailed methodology.

We thank you for the opportunity to be involved in a historic building survey.

Regards,

Chris Pickering
General Manager
Detection Services Limited

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Ground Penetrating Radar (GPR) Survey

160 Manchester St,

Christchurch

25th/26th September, 2010

Background

Christchurch City Council are trying to assess the stability of the seven story building located on the corner of Manchester and Hereford Streets in Christchurch, address 160 Manchester St. This is a heritage building and efforts may be made to restore the building in the future.

Post earthquake the building was deemed unsafe and no access has been allowed to the interior of the building. It was decided to use Ground Penetrating Radar (GPR) to examine external elements of the building to ascertain the internal construction, or at least the structural elements within the walls.

For geographical reference, the Hereford Street face of the building is deemed the North side as referenced in this report.

Initial Survey

The East face of the building is adjacent to 186 Hereford Street. Access was gained to the rooftop of 186 Hereford St allowing an initial assessment of level 6 East face. The GPR survey data indicated targets at regular intervals in both the vertical and horizontal survey directions.

A window is present in this wall at the corresponding height to the external survey so the depth measurements could be verified.

Targets showed at depths of:

95 - 100 (Horizontal)

180 (Vertical)

195 (Horizontal)

290 (Vertical and Horizontal)

305 (Vertical and Horizontal)

The East wall differs in construction to the North and West walls as it has a full brick face from Floor 3 through to Floor 7. Images 1 and 2 illustrate the targets within the wall.

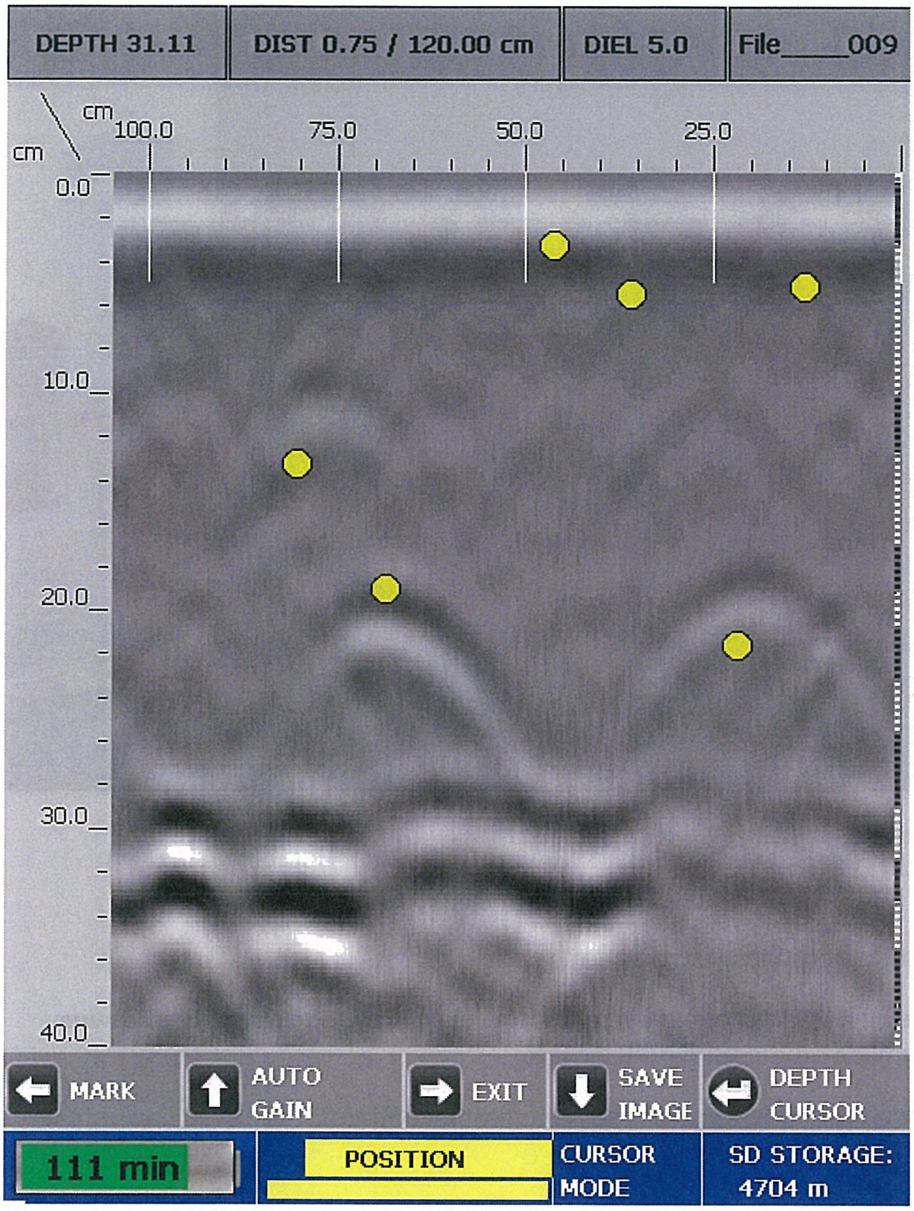

Image 2. East Wall Vertical – showing internal wall lining at the bottom around 300mm.

Main Survey

Site Setup

A large crane was utilized to hold a small two man cage just off the face of the building (Image 4). The vertical columns on the North and West faces of the building were surveyed using GPR from the confines of the cage. Light wind conditions were required to ensure the cage did not swing around, and could be held clear of the building.

Holmes Consulting provided a methodology and plan of areas to be surveyed. Initially this was 12 segments of the brick columns; six on the North face and six on the West.

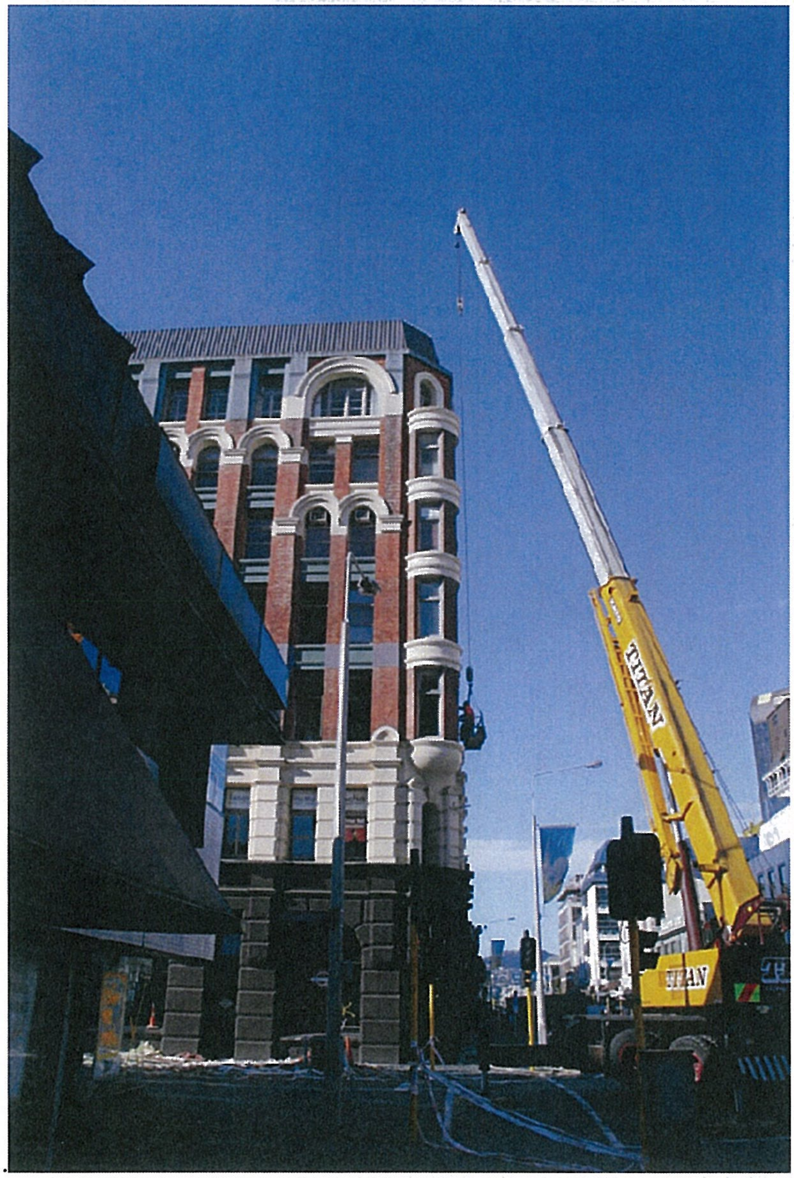

Image 4. Crane positioned in intersection.

Two vertical lines were surveyed either side of the column and the depths marked on the front face. Image 6 illustrates the consistency of the brick depth, with the initial distance between adjacent brick runs being 110-130mm and the third at approx 375mm. Deeper targets at 350-370mm may indicate the presence of a structural steel component; these reflections may also be airgaps or mortar lines. Image 3 illustrates a depth of four brick layers.

Photo 2. Marking of column on North face. (Time reference 7:15)

Crucifix Crack Area of Column

One of the columns on the North face exhibits a large crucifix type deep crack. Image 2 is the survey image taken across the crack running vertically, the crack centre being marked with a 'X'. In looking through the crack openings, it was possible to confirm at least a triple brick construction of the outer part of the column. There is a possibility of some kind of steel reinforcing in the centre closer to the interior wall, however this could not be verified by GPR.

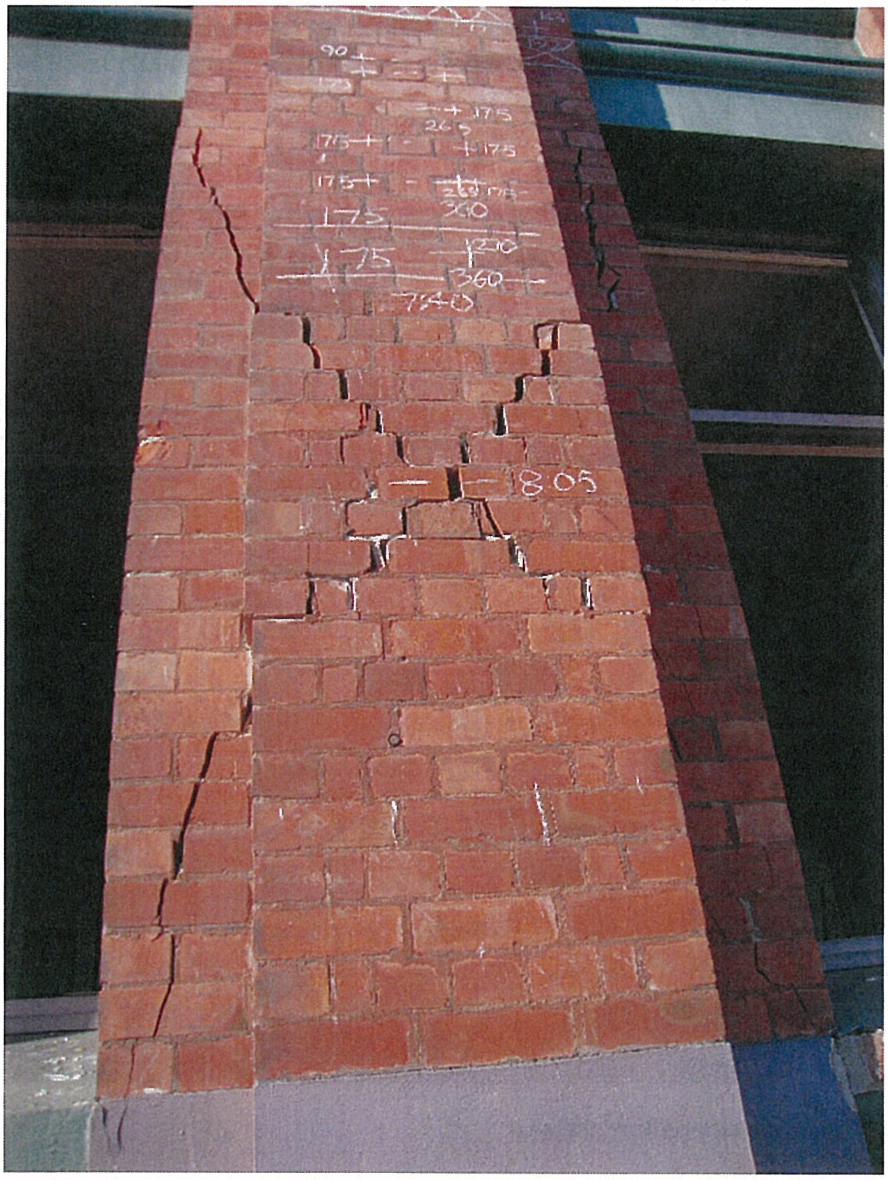

Image 8. Crucifix crack in column, North face.

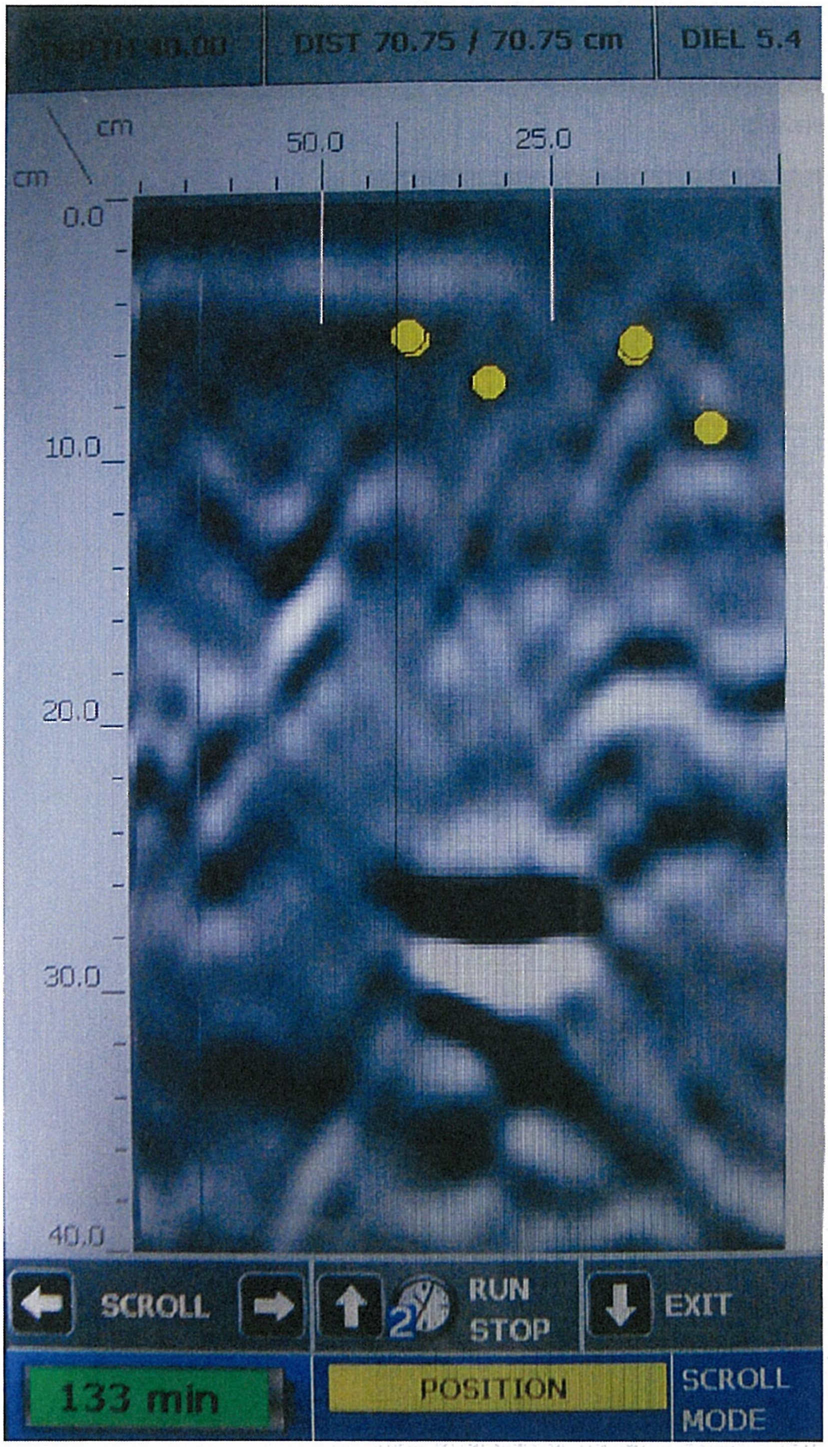

Image 10. Vertical extent of plate at floor joist level.

Conclusions

GPR survey has provided some good baseline data showing the presence of many of the structural elements in the areas surveyed. The East wall provides for the overall 'grid' pattern of construction, giving good reflections off the front surfaces of components. A structural engineer should be consulted as to a more precise interpretation of component size.

The column construction on the North and East face showed a predominant presence of at least a triple brick construction. At floor level there are indications of a steel plate surface some 200mm wide (vertically)and located 280mm from the front surface.

A good reflection was obtained from another plate above the window on the East face. This object is quite narrow indicative of a steel lintel of some kind.

In many areas, good small reflections were obtained, however these would tend to indicate either voids between brickwork. It is possible that some of these reflections are ties or standards but nothing definite can be stated.

Recommendations

The GPR survey was conducted primarily to identify the presence of steel structural members. The areas that have been surveyed should now be cross checked to verify the exact construction.

Once the next series of cross checking has been completed, and internal access is possible, the internal shear wall construction should be verfified using GPR. Access from both sides of the wall will be possible.

MEMO

To: Peter Mitchell, General Manager, Regulation and Democracy Services

From: Fiona Wykes, Heritage Team

CC: Carolyn Ingles, Programme Manager, Liveable City

Date: 4 October 2010

Re: 160 Manchester Street, Council's engineering report

Following a request to understand what report Council have commissioned on the above building and what it says:

The main question regarding this building is whether or not it has an internal steel frame/structure of some sort, which would have implications for any works carried out on the building . Were this known an engineer could then write a report detailing potential options for the building in terms of demolition or retention. The knowledge might impinge on costs of these works as well.

No full engineering report has been provided for/by Council for this building for the following reasons:

- Initially building owners were concerned and unclear about the legal process and their liability surrounding Council undertaking a survey.
- A ground penetrating radar survey was commissioned as the least invasive way to ascertain what internal structure there might be (rather than drilling through columns to ascertain whether there was metal inside)
- This survey was carried out, from a crane, but the results were inconclusive
- This being the case, the engineer was unable to write anything for Council, other than stating that the results were inconclusive and that further investigative work would need to be undertaken to confirm the results of the radar which would be either 'an invasive study by drilling through the piers or by undertaking a visual inspection from within the building, including the removal of linings adjacent to the piers.'
- Programme Manager Liveable City decided no further investigations would be undertaken due to time constraints and managing any risks associated with an invasive technique.

Christchurch City Council

Warren R. Lewis BE (Hons) MIPENZ CPEng. ANZIM
Stephen W. Barrow BE (Hons) MIPENZ

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30 September, 2010

File No. 18476

REPORT ON

EARTHQUAKE DAMAGED BUILDING

AT

192 MADRAS STREET, CHRISTCHURCH

Prepared by

Warren Lewis
BE (Hons) MIPENZ
CP ENG (Civil and Structural) INTPE (NZ)

(i) Major propping of the building to prevent it collapsing in further earthquakes or high winds including the whole west wall. This propping, to be done safely requires the construction of a very heavy steel safety shield to protect workmen working near the base of the wall. Fixing these braces against the building would need to be carried out from a man cage suspended by a crane.

(ii) The whole building would need to be scaffolded with structural braces out onto adjoining ground, some of which is under separate ownership. If permission for this is not granted then work will not be able to proceed.

(iii) New shear wall foundations will need to be excavated out and constructed around the perimeter of the building including the south light well. This will require unusual and costly protective work as it undermines the existing foundations.

(iv) Stainless steel twist ties (Helifix) ties would need to be installed in all the above brick walls from inside to 40mm short of the outside. This is required to hold the various wyths (widths of bricks) together. The ties would be left long inside to be bent over and tied to a new internal wall. These ties would need to be placed at an average of 400mm crs each way to be sure that isolated brick could not fall out in future. These ties alone have been costed at over $200,000 + GST.

(v) New reinforced concrete walls would then be installed by boxing or shotcrete. Once the new walls are up to the height of the first floor, the existing first floor could be removed and a new prestressed ribbed concrete floor installed. The wall concreting would then continue up to the top of the parapets. Some cost saving could be made by reducing the height of the parapets.

(vi) Considerable rebuilding of brickwork is required to repair the damaged walls. Even where not damaged the exterior faces of the building need to have the mortar raked out to a depth of at least 40mm and be repointed with a 12.5MPa mortar. (Existing compressive strength of mortar is less than 0.5MPa).

(vii) The roof structure would then be rebraced and structurally tied into the new composite concrete/brick wall.

The reason why the building is structured in the method set out above is that such a building will shed bricks if it is not rigid and elastically responding in a design earthquake. Steel frames would not be stiff enough unless they were very large and obtrusive. Timber and ply diaphragms are not as stiff or as reliable as a concrete floor. These lessons were learnt in Padang after this city experienced similar shaking as Christchurch in two earthquakes in 2007 and 2009. Knowledge gained by the NZSEE reconnaissance team found that strengthened regular brick buildings with concrete floors performed better than those with timber floors.

An alternative to the above restructuring is to use base isolation techniques on new foundations and lesser structures above. However, this is likely to be more expensive.

8. LIMITATIONS

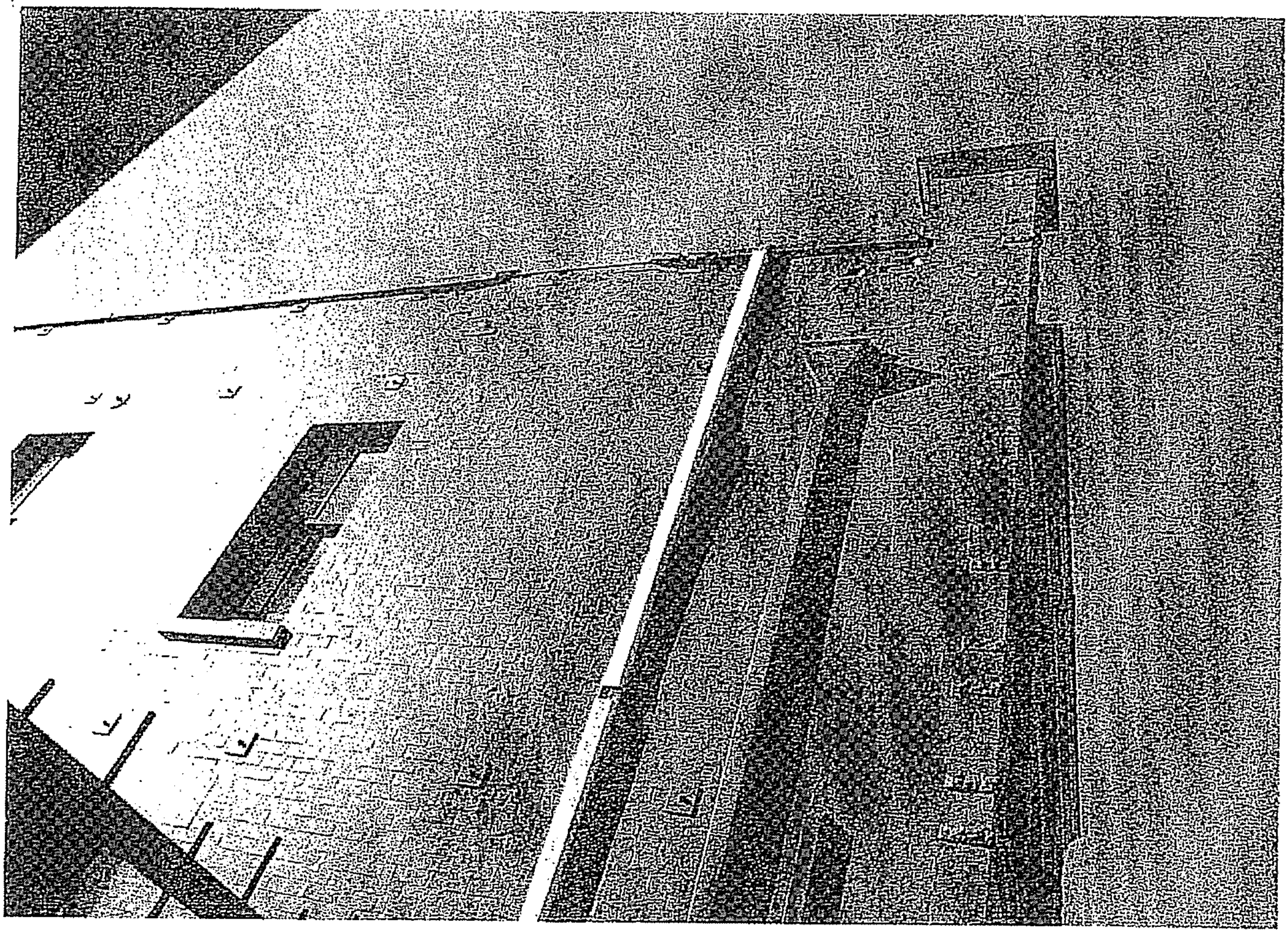
This report has been prepared for the owner of the building so that he can present the options and costs to the Christchurch City Council so that they may decide whether the building should be saved or not. The reliance by other parties on the information or opinions contained in this report shall, without our prior review and agreement in writing be at such parties' sole risk.

APPENDICIES

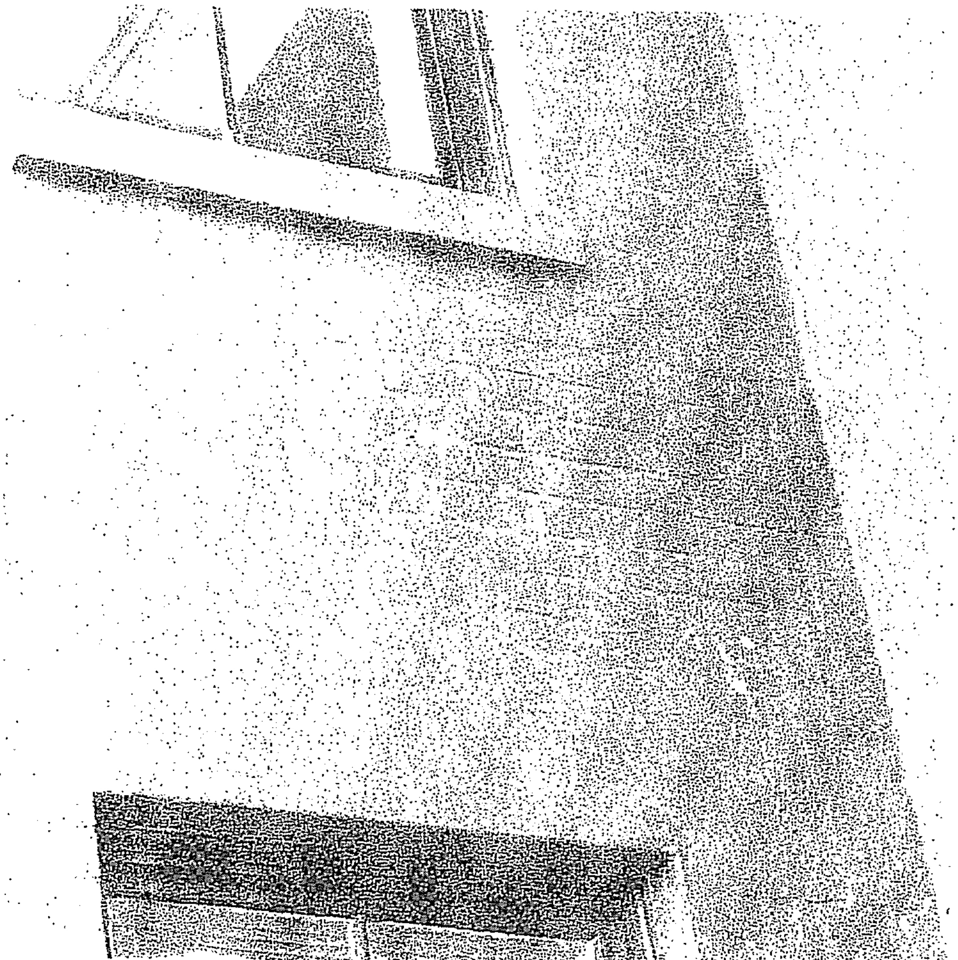
I. Photos taken 27/9/10.
II. Photos taken 30/9/10.
III. Report by John Hare.
IV. Report by Warrick Weber

References:

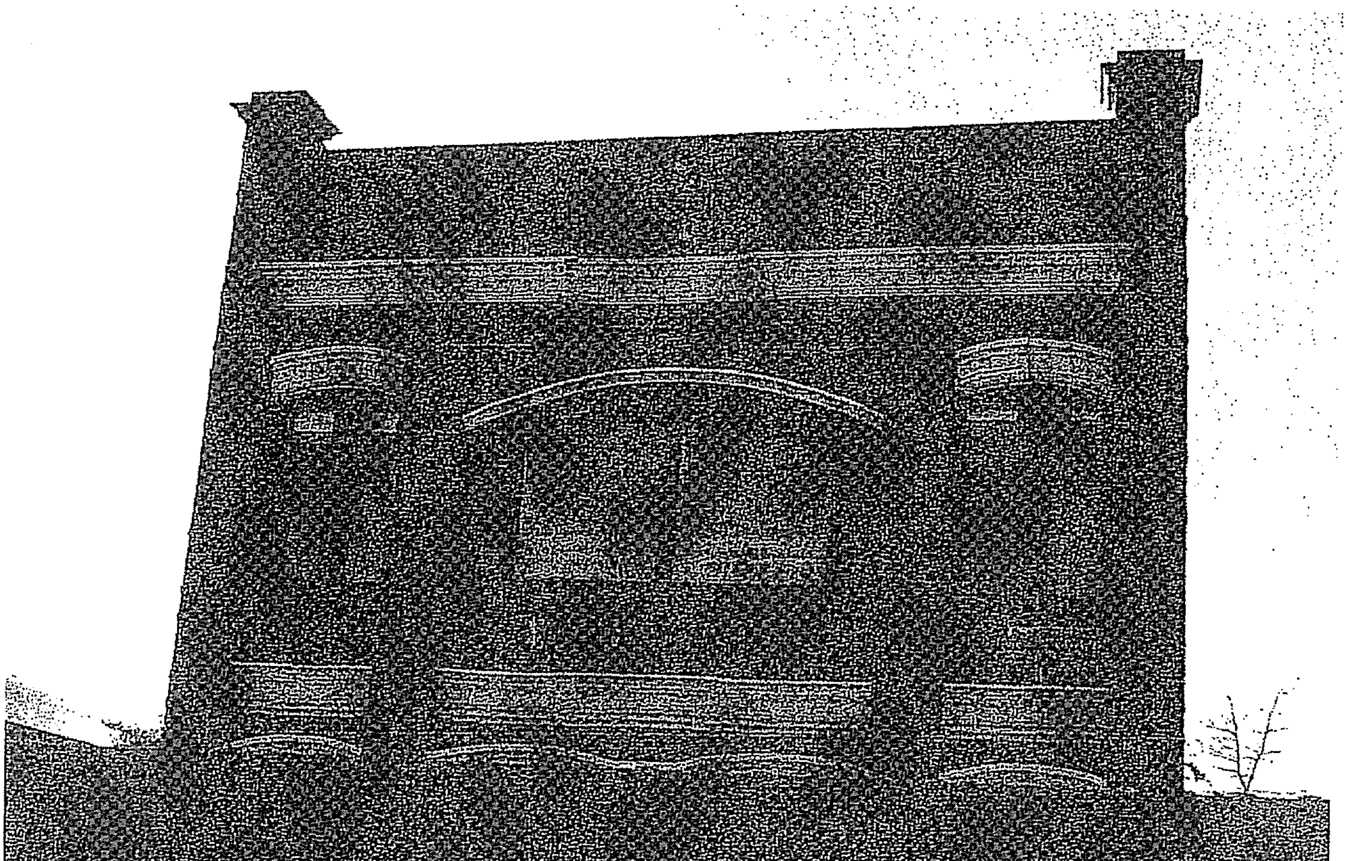
1. Draft NZSEE Reports on Padang Earthquake 2010.
2. NZSEE – June 2006 – Assessment and Improvement of Structural Performance of Buildings in Earthquakes.
3. AS/NZ1170 Parts 0, 1, 2, 3 & 5 Structural Design Actions.
4. Rawlinsons NZ Construction Handbook.
5. The NZ Building Economist.
6. Builders Price Guide.



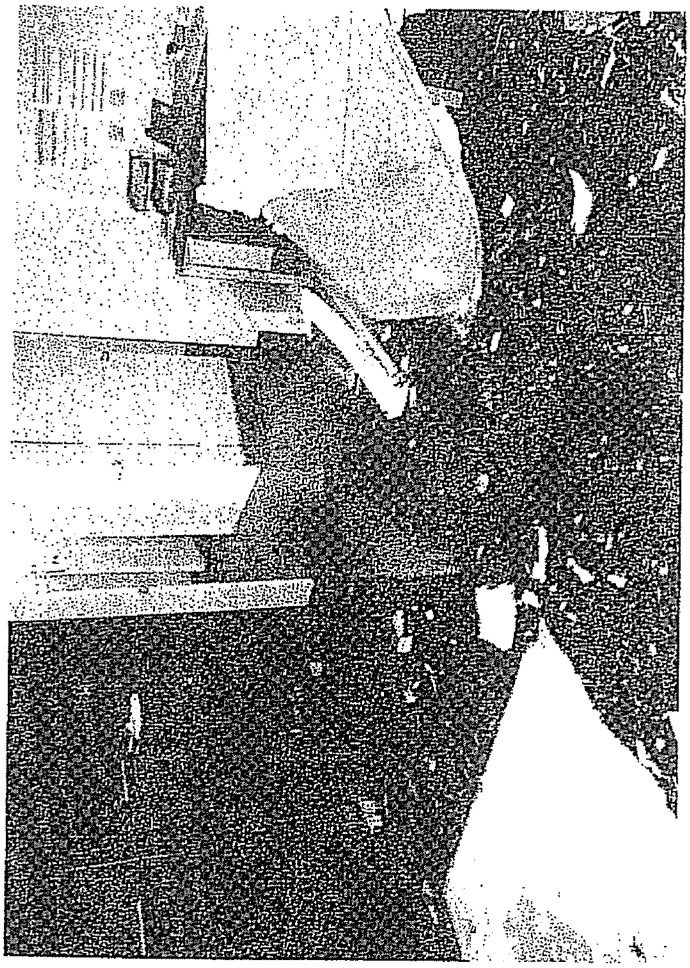
192 Manchester St
Nurse Maude Building
27/9/10

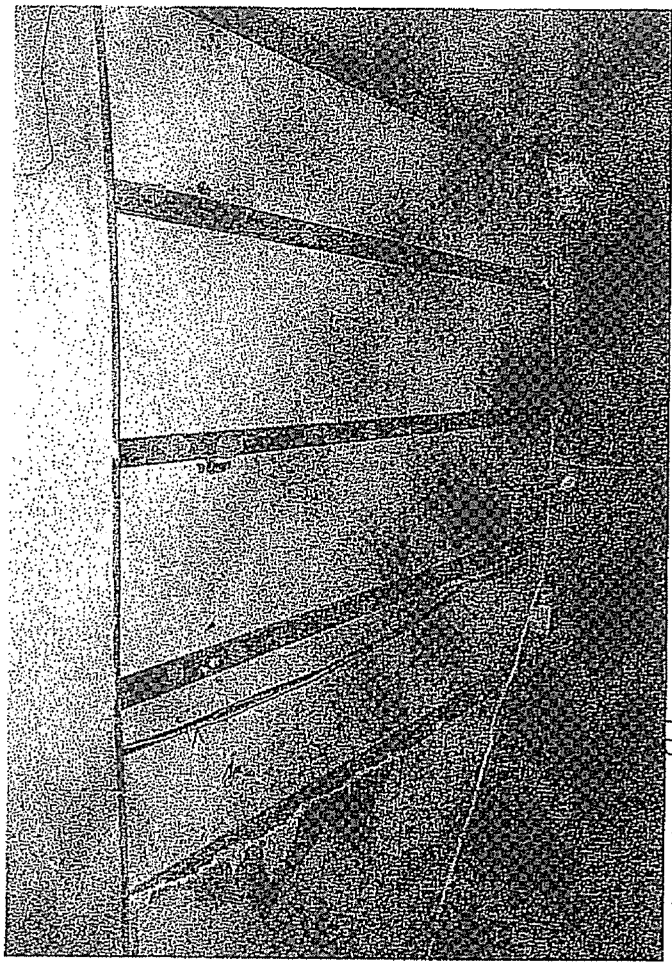


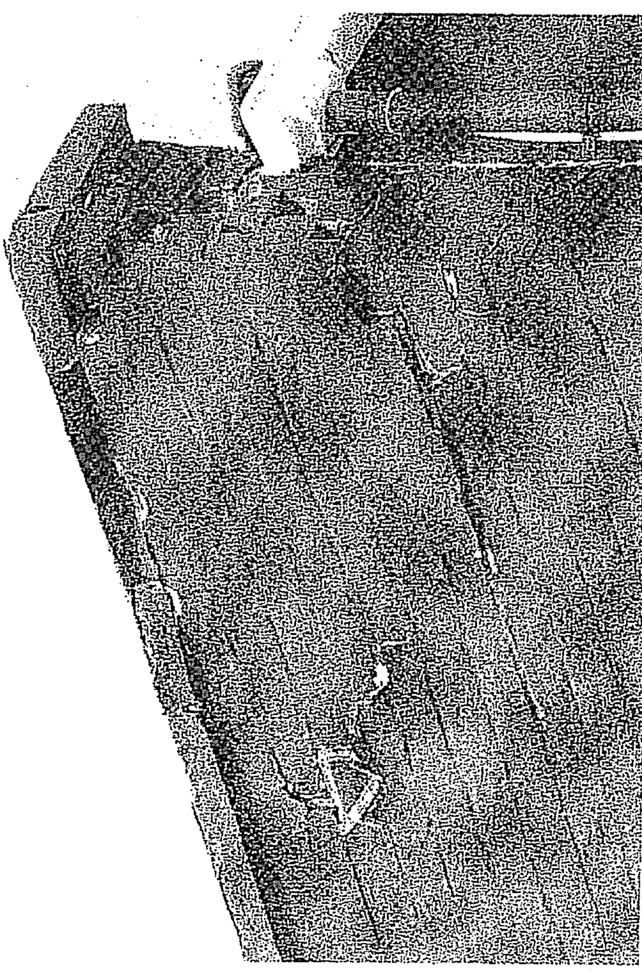
192 Madras St.
27/9/10



27/9/10
192 Madras St







192 Manchester St.
30/9/10

APPENDIX 3

Report by John Hare

Limitations

Findings presented as a part of this project are for the sole use of the Christchurch City Council in its evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our inspections have been visual only and neither calculations nor other analyses have been performed. Our inspections have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed. 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 this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Building Evaluation

The following are our observations of the building reviewed, and our conclusions as to its condition and seismic load resisting capacity.

Existing Building Description

The existing building at 192 Madras Street dates from 1918. It is a Group 3 heritage building in hte City Plan, and appears to have no classification with the Historic Places Trust.

The building is generally of two stories, with two small penthouse structures, one at the top of the stairs, and the other towards the rear of the building. The main body of the building is approximately 21m x 8.5m in plan dimensions, with a lightwell towards the rear. There is a 6.5m long single storey section at the rear of the building. There is a parking space at rear which was formerly partially covered, with access via a driveway on the north side of the building. To the south, there is a drive access for the adjacent student accommodation.

The exterior walls and a number of interior walls are of solid brick construction, generally 335 to 215mm thick (ground and first respectively). The west wall (facing Madras Street) has extensive openings over both levels. The north wall and the walls facing onto the small lightwell also have significant openings.

There is a tall parapet (approximately 2000mm at highest, above the roof level) over the west facade. The remainder of the parapets are in the order of 1400mm high.

to the diaphragms, as well as possibly adding plywood overlays to either floor or ceiling.

4. Reduction of the parapet height, subject to a review of the existing parapet strengthening.

5. Reduction in the height of the brick penthouse structure over the stair, and removal of the other structure. These add a significant amount of weight to the building for only relatively little gain in lettable area, particularly in the eastern-most small room.

6. General repair of damaged surfaces and fittings throughout the building.

Recommendations

We recommend that a detailed strengthening plan is prepared in order to provide an independent cost estimate for the building, including all necessary repairs and remedial work.

Report Prepared by:-

John Hare
DIRECTOR

105421.02RS2709.001.DOC

APPENDIX 4

Report by Warrick Weber

Construction Description

Building construction is unreinforced brick masonry with wooden floors and a light weight roof.

Walls are load bearing solid masonry brick. These start at a 3 brick thickness and step inwards as they move up the building – 2 bricks at first floor and 1 brick at parapet level.

The front Façade has many openings. The side walls are almost completely solid with only a few openings. On the south side there is step back into the building which includes the fire escape stairs. The rear wall has some openings.

The building was earthquake strengthened in 1998. This consisted of tying the side walls to 1st floor and roof levels only. There was no front façade strengthening observed. It should be noted that the primary objective of earthquake strengthening of unreinforced masonry buildings is to preserve lives. Strengthening work does not guarantee that the building fabric is maintained.

Damage Observations and Assessment

The building has suffered significant structural damage to all elevations. Shear failures of critical elements has occurred.

<u>South and North Elevations (Side walls)</u>

Transverse shear failures in both walls have occurred as observed from outside the building. Internally cracks extent into the stairway walls on the south side.

These walls are unstable under lateral loading.

<u>West Elevation (Front Façade)</u>

The front Façade has suffered the most severe damage. All spandrel panels' elements have failed.

The façade is extremely unstable.

<u>East Elevation (Rear wall)</u>

The rear wall has a number of transverse share failures.

The wall is unstable under lateral loading.

Stability Assessment and Options

Given the heritage significance of the building it is important that a robust analysis of potential stabilization methods is considered.

If there is a political will to try and save the building then very significant time and resources will be required.

- Seismic Emergency Gravity Support -essentially an internal frame which provides supplementary seismic emergency columns.

Another option, although probably not economically viable, would be to take down the building and rebuild in the same external appearance but with a modern seismic resistance frame internally.

To bring the building up to 2/3 of code would require careful consideration of seismic ductility issues and building regularity. Global FEA modeling combining new and existing elements would be difficult. Modal response spectrum analysis is recommended.

Demolition Options

Demolition shall be undertaken to ensure the safety and protection of the general public, workers and surrounding property.

A detailed demolition plan from the Contractor shall be required prior to demolition commencing.

Health and Safety

Both Strengthening and Demolition options present significant health and safety risks. Well considered Health and Safely plans must be required prior to construction/demolition work commencing and take into account the extraordinary risks the site poses.

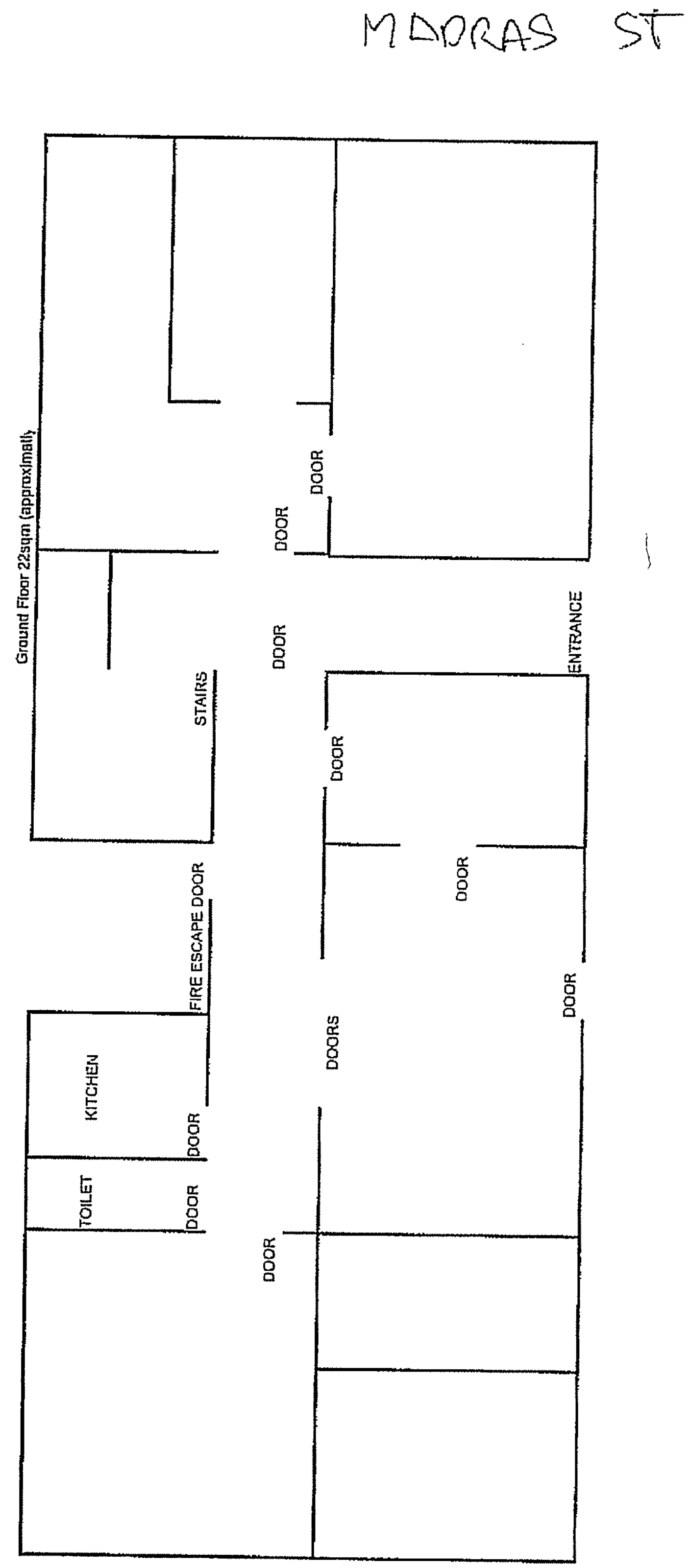
Conclusions and Recommendations

The extent of earthquake damage is significant and the building is considered unsafe and unstable under lateral loading.

Strengthening of the structure would require rebuilding of major sections of the building. This would be a significant and costly construction project with its own challenges. Most if not all failed walls would require reconstruction. Any remaining elements would require strengthening. Overall stability, ductility and regularity would be significant design issues which may make any efforts to save the structure and strengthen to 67% of code difficult.

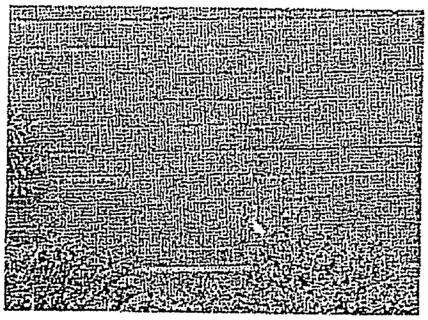
If there is a public willingness to save the building by strengthening then this will require significant time and resources. This option has higher exposure to risk.

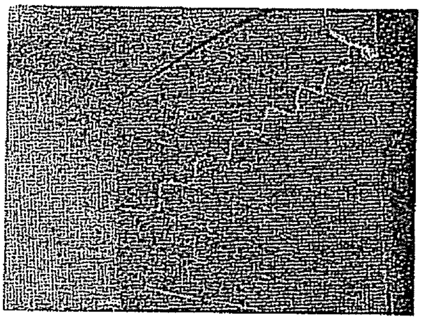
Without consideration to heritage or political issues it is recommended that it is not a feasible proposition to strengthen this severely earthquake damaged building.

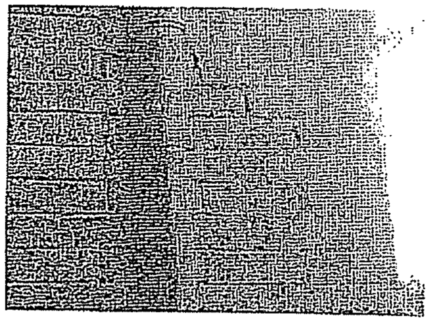

MADRAS ST
Ground Floor 22sqm (approximatly)
DOOR
DOOR
DOOR
STAIRS
ENTRANCE
DOOR
DOOR
FIRE ESCAPE DOOR
DOOR
DOORS
KITCHEN
DOOR
TOILET
DOOR
DOOR

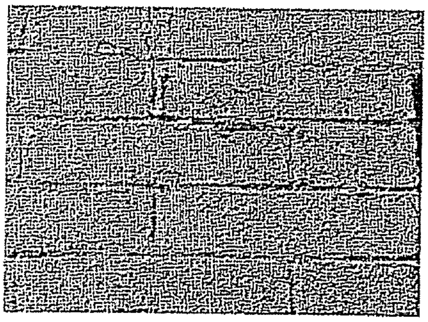
Appendix 2

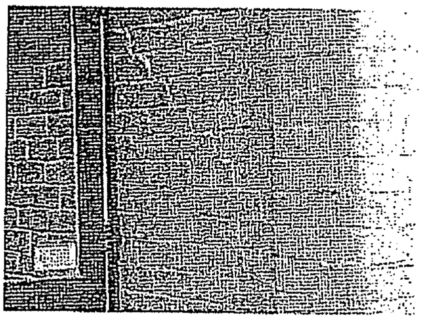
Schematic Ground Floor Plan

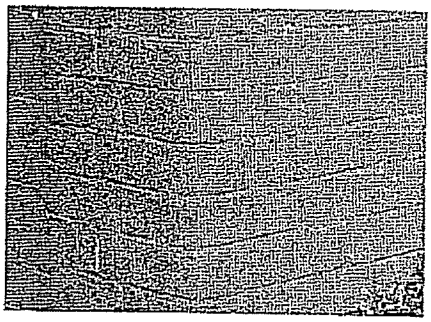


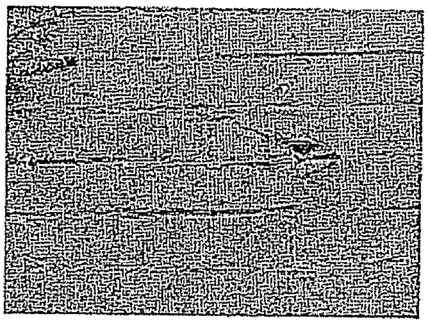


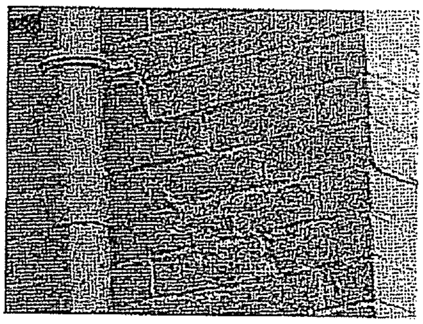


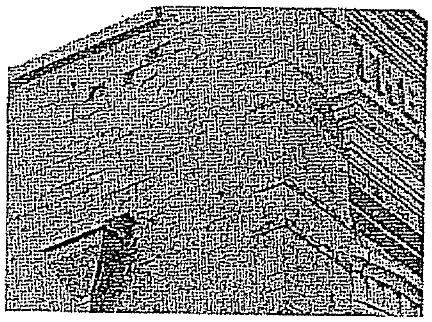


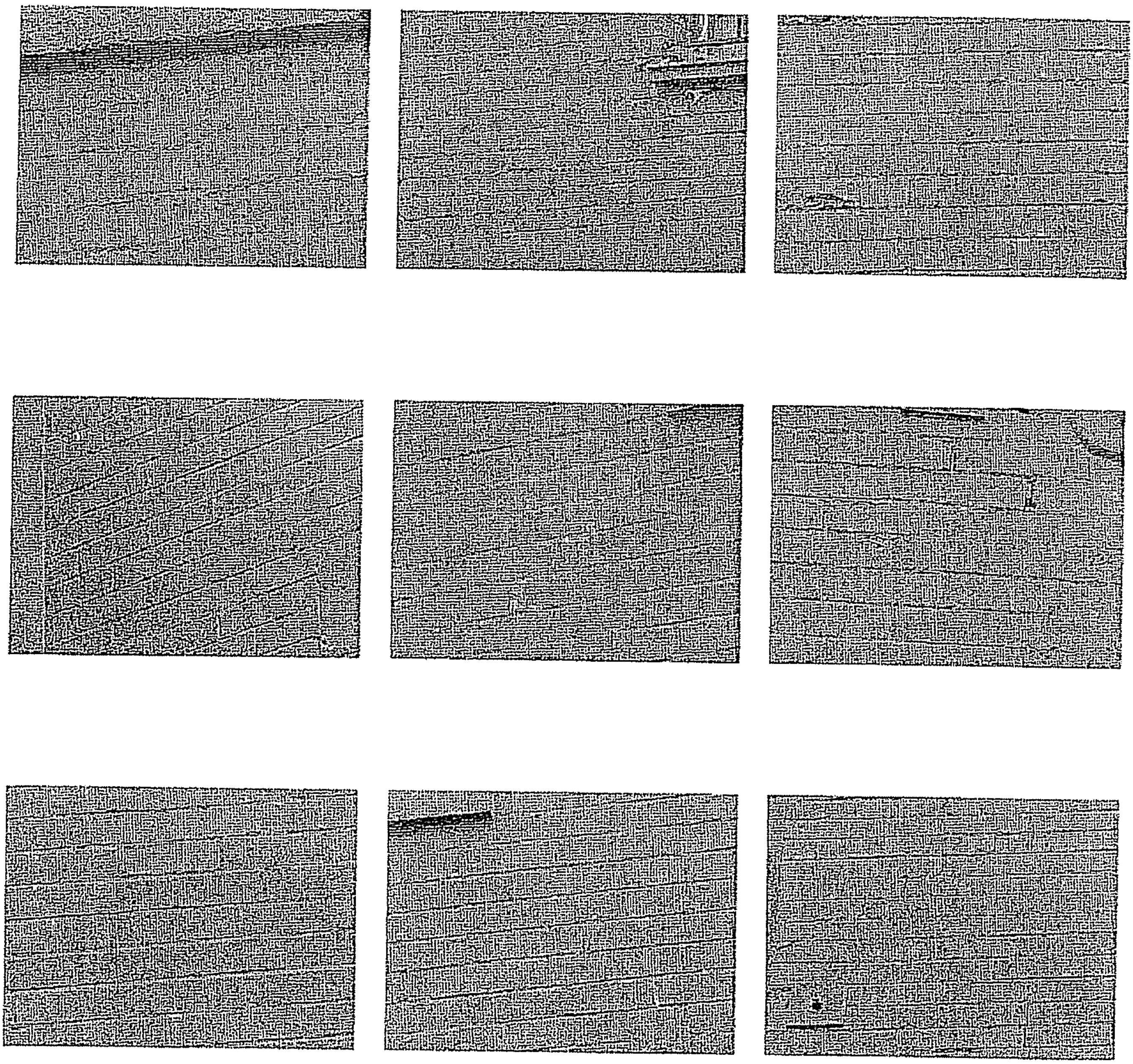


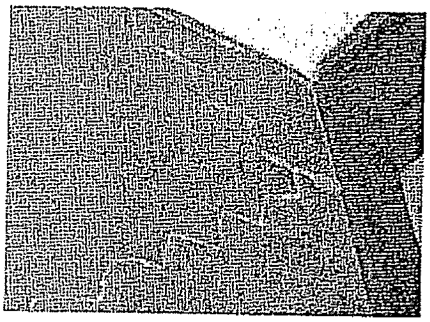


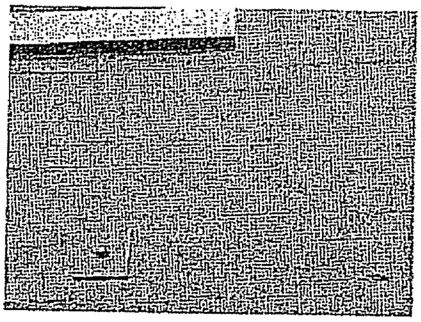


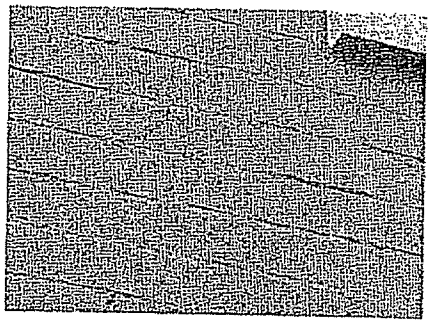


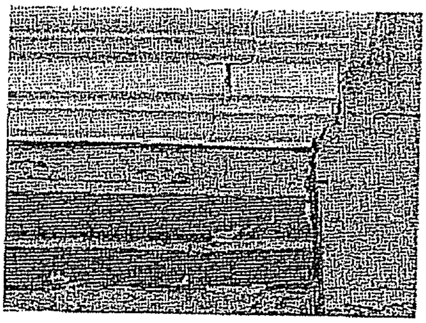


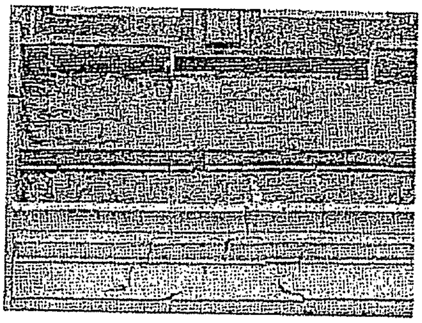


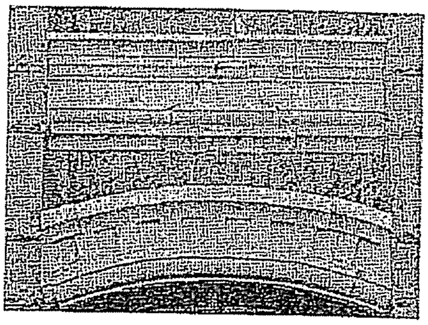


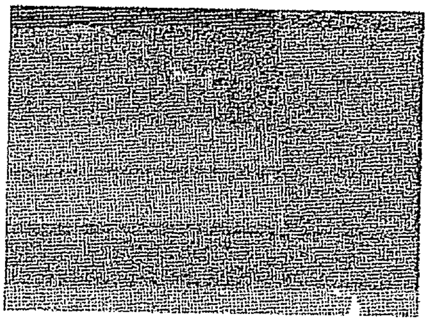












HolmesConsultingGroup

STRUCTURAL AND CIVIL ENGINEERS

REPORT

CCC Heritage Reviews

192 Madras Street

PREPARED FOR

Christchurch City Council

27/9/10

Christchurch

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Introduction

Holmes Consulting Group has been engaged by Christchurch City Council Heritage to review the demolition report prepared by Gridline Ltd, dated 13 September 2010.

The Gridline Limited report summarises observed damage to the building and recommends demolition.

The purpose of our review is to provide an independent assessment of the building, and conclude whether we would support the demolition, and if not, to provide a summary of what work may be required to the building.

The Richter Magnitude 7.1 earthquake of 4:36 am on September 4 was approximately 30 km west of Christchurch, at a depth of 10 km. The earthquake subjected the building to strong ground motions, which are currently estimated to have been between 30% -100% across Christchurch, with the wide variation due to local ground conditions and topography.

Scope of Work

The scope of work for this project included the following:-

1. Review the Gridline Limited report for the building.

2. Walk-down the building to determine the nature of any damage.

3. Assess the apparent seismic performance of the building.

4. Report on our findings and recommendations.

The building has been recently upgraded by the current owner, and appears to have had some limited seismic upgrading (floor ties are apparent on the north and south walls). The mortar and/or pointing is in reasonable condition.

Building Condition

The building has sustained some damage from the strong shaking.

Most of the damage is to the west facade, where the lintels have suffered severe cracking. It is apparent that the damage has increased significantly through the aftershocks that have occurred. In addition to lintel damage, there is a slight displacement of the piers to each side, and the entire parapet above roof level appears to have offset slightly to the west.

Of the remaining brick walls, most have minor cracking, up to 1.5mm width, with the exception of the east wall of the lightwell, which has a crack of approximately 10mm maximum width, above the fist floor level, accompanied by a slight offset of the wall over, to the south.

There is a significant amount of water in the building, apparently due to a broken pipe.

The north parapet has lost some brick at the top, just behind the west parapet, but for the most part, the parapets are otherwise intact.

Most of the interior doors that were investigated are straight and swing unimpeded, implying that there is no residual offset in the building.

Conclusions

Although the west facade has been extensively damaged, the damage is localised to the spandrel elements. The remainder of the walls have sustained only relatively light damage that is readily repairable.

It is our opinion that the building is readily repairable, although the cost of repairs must be balanced against practicality. Likely repairs include:

1. Reconstruction of the brick spandrel elements to the west facade, and facing at rear with reinforced concrete, to form a new concrete frame behind.

2. Repair of the cracked brick walls, by raking out and repointing the joints, and possibly strengthening key walls with composite overlays.

3. Adding further strengthening to the floors and/or ceilings, to create full seismic diaphragms. The strengthening is likely to include additional ties

HolmesConsultingGroup

REPORT

STRUCTURAL AND CIVIL ENGINEERS

CCC Heritage Reviews

461-469 Colombo Street

PREPARED FOR

Christchurch City Council

1/10/10

Introduction

Holmes Consulting Group has been engaged by Christchurch City Council Heritage to review the status of the building at 461-469 Colombo Street, Sydenham.

A brief report was available prepared by TM Consultants, based on a visit of 21/9/10.

The purpose of our review is to provide an independent assessment of the building, and conclude whether we would support the demolition, and if not, to provide a summary of what work may be required to the building.

The Richter Magnitude 7.1 earthquake of 4:36 am on September 4 was approximately 30 km west of Christchurch, at a depth of 10 km. The earthquake subjected the building to strong ground motions, which are currently estimated to have been between 30% -100% across Christchurch, with the wide variation due to local ground conditions and topography.

Scope of Work

The scope of work for this project included the following:-

1. Review the documentation supplied for the building.
2. Visit the building to determine the nature of any damage – note that the interior of the building was inaccessible.
3. Assess the apparent seismic performance of the building.
4. Report on our findings and recommendations.

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There was a moderate height parapet assumed to be over the north and east facade.

No seismic strengthening or securing appears to have been done to any of the building.

Building Condition

The building has sustained severe damage from the strong shaking.

The north wing and the northeast corner have completely collapsed above the first floor, with the first two bays of the east facade having collapsed into the street. The lower level has remained mostly intact along the north facade, although it has sustained damage below the floor level, and is severely cracked in parts. The remainder of the east facade above the first floor has lost its parapet, and is leaning outwards for at least one bay, by as much as 100mm. This inclination reduces to the south. It appears that the dividing walls between hte units may be intact, but no internal inspection was possible.

The south wall above first floor has delaminated and partially collapsed onto the roof of the neighbouring property.

Parts of the west wall above the first floor have apparently collapsed, but not all of this wall was visible. It appears that part of this wall may have been either re-clad or reconstructed with lightweight material.

No internal inspection was possible.

Figure 1: View from north

Conclusions

Although the internal (party) walls may be in reasonable condition, the upper level has otherwise been severely damaged, particularly at the northern end. Although four out of six bays of the upper level east facade have remained standing, at least half of this length is leaning and it is to be assumed that none of it is secured in any way and is therefore unstable.

It is my opinion that the building is not readily repairable, although it may be feasible to retain part only of the eastern facade with a steel gantry placed in Colombo Street footpath. This would have to remain in place until a new structure could be built to support it. This would add considerable time and expense to the rebuilding process. :

Recommendations

We recommend that the building be demolished.

Report Prepared by:-

JHare

John Hare
DIRECTOR

105421.02RS101001_461 COLOMBO.DOC

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Post Earthquake Report
On 456 Colombo Street
For Christchurch City Council

1. Preliminary

An assessment of the structure, post earthquake, of the building at the above address has been commissioned. The building has undergone significant damage and an independent 'second opinion' has been requested in response to an application for demolition.

The building was inspected on 28 September 2010.

The building has a Group 4 Heritage Classification in the Christchurch City Council City Plan. The design is attributed to Hurst Seagar and the original building is dated at 1880.

2. Background

This Consultancy carried out an inspection of the building in 2002 to assess the 'seismic strength' of the structure and to outline remedial work to secure the building. The building was assessed as 'earthquake prone' with main deficient elements being; the brick chimney, the parapets – particularly the decorative elements to the front parapet, lack of ties to the brickwork at the roof level, the state of the exposed brick mortar courses and a concern that the North and/or South walls could be prone to 'rotate' on their foundations.

No securing work has been carried out in the interim.

3. Building & Damage

The original building is relatively small (approx 4.5M wide x 11M long) and comprises a two storey brick structure. There are brick firewalls with parapets to the North and South boundaries. The 'shop front' is open to the West with a brick façade and parapet over.

Around 1987/88 the 'rear' wall was removed with the Ground Floor opened up to the 'Old Sydenham Town' Mall and the First Floor was closed in with a timber frame wall with corrugated steel cladding. The ground floor has been changed to concrete and this was probably done at about this time.

Columns comprising concrete encased structural steel sections were also installed at about this time. These columns are situated approx half way along the North and South walls with columns at the rear of the building but none at the 'front'. The columns extend up to the First floor and no further and are not connected to the brick boundary walls.

A canopy across the footpath is supported by steel rods anchored into the front wall and these rods in turn support 'railway iron' beams within the canopy.

RUAMOKO solutions CONSULTING STRUCTURAL ENGINEERS

STRUCTURAL INSPECTION REPORT

Building Location: 580 Ferry Road, Christchurch

Date of Inspections: 30th September
Inspecting Engineer: Grant Wilkinson
Contact Phone of Inspector: 021 331741

Scope and Limitations of Report:

Ruamoko Solutions Ltd was engaged by Christchurch City Council (Amanda Ohs, Heritage Planner) to inspect the property at the above address which is subject to an application for demolition lodged and under consideration by the Council.

Our conditions of contract are the standard ACENZ/IPENZ "Short Form Agreement for Consultant Engagement" March 2010 version. These conditions can be downloaded from the ACENZ website www.acenz.org.nz. Ruamoko Solutions is liable to Christchurch City Council only to the extent of that agreement.

This report is for the use of the Christchurch City Council to help with their deliberations on the application for demolition. The findings shall not be used by any other parties other than the Christchurch City Council, the building owners and their insurers.

A brief walkover assessment was undertaken with Amanda Ohs, Dave Pearson, conservation architect: Noel Unwin, Owner: Michael Pile, Arrow International: Gavin Ryan, Cooks Commercial et el on 30 September 2010

The primary purpose was to assess the damage arising from the earthquake that occurred on Saturday 4th September 2010 (and subsequent aftershocks up to the time of the inspection) against the application for demolition. The inspection was visual only for the inside and outside of the building and no drawings or documents were reviewed, and no calculations or other analyses were carried out. Electrical and mechanical equipment, gas connections, water supplies and sanitary facilities were not inspected, and secondary elements (partitions, windows, fittings and furnishings) were not considered.

Note that a more comprehensive inspection may reveal other safety hazards and that any damage shall be monitored closely. It is also possible that additional aftershocks and ground movement will result in additional cracking damage, particularly over the next few weeks and non-urgent repairs shouldn't be made until the aftershocks have abated. For any worsening of the cracking and damage noted, a structural engineer shall be notified to determine whether structural safety hazards exist.

Comments regarding this building are limited by the nature of the inspection, and are not intended to be used outside the context of this earthquake event, and re-inspection may be required following further aftershocks. The findings are based on best engineering judgement and are not intended to be used outside this context.

Previous Reports and Data Supplied

We have been supplied with:

- The Demolition of Earthquake Damaged Buildings – Application Worksheet dated 15 09 10

There was minor cracking to other brick walls particularly in the vicinity of doorways, lintels and the like.

Earthquake Insurance

We understand, from the owner, that the building has 'replacement value' insurance. We further understand that a mortgage has been secured against a group of properties including this property. The owner considered that he will control the insurance payout rather than the mortgagor.

Seismic Upgrading

If the building(s) is to be repaired and reused the Christchurch City Council new policy is that it be seismically strengthened to 67% of the seismic strength of an equivalent new building. That is the 'aim' of the policy. If that cannot be achieved (because it is uneconomic or for some other valid reasons) then the building can be seismically strengthened to at least 33% of the seismic strength of an equivalent new building.

For the purpose of this report we have assumed that 67% strength may not be economically achievable (or paid for by the insurer) so we have considered what will be required to achieve the 34% strength so that is no longer regarded as an 'earthquake prone building'.

While we haven't considered the detail of a 33% strengthening scheme we have designed many other buildings to be successfully seismically strengthened to that level. Ruamoko Solutions seismic strengthening CV is available upon request.

The single storey section of this building is likely to be easier to seismically strengthen than many other buildings we have strengthened as the north-south steel roof beams can be easily modified to act as portal frames with the addition of portal legs and foundations. In the east-west direction there is ample length of double or triple brick walls to resist the code shears loads.

Extra bracing will be needed in the plane of the roof steel beams and to created braced diaphrams to gather up and distribute the seismic load to the resisting elements.

Of course other seismic strengthening elements will be required to support gables, parapets and the like.

Similarly, the two storey section is quite straightforward. It has generous brick wall lengths on three sides and a very deep and stiff steel beam that supports the west side at first floor level. Clearly, the west side at ground floor level will require portal legs (and perhaps a central UB post) to achieve the required strength and stiffness to achieve 33% strength...but that is quite straightforward. The floor and roof diaphrams will need to be strengthened and connected to the walls and parapets will need to be propped as usual.

On a scale of 1 to 10 (1 being the least difficult to strengthen) I rank this building at 3. It is likely to be quite straightforward.

Approx Costs to Seismically Strengthen to 33% Code

I have researched the costs of strengthening two buildings, The two storey Armson Historic Warehouse and the 4 Storey Mayfair Building on the corner of Poplar Lane and Tuam St.

Armson was strengthened to 40% code and Mayfair was designed for 67% code and estimated by a QS.

RUAMOKO solutions
CONSULTING STRUCTURAL ENGINEERS

STRUCTURAL INSPECTION REPORT

- Repairing the earthquake damage
- Compliance costs for egress, fire alarms, disabled access and the like
- Refurbishment costs
- Demolition costs
- Rebuilding the tenants services and equipment in a new building
- The cost consequence of the time taken to deliver a new building or a strengthened building.
- Contingencies
- Professional fees
- Financial assistance eg heritage grants

Report prepared by

Grant Wilkinson
BE (Hons) (Civil), FIPENZ (Structural),
CPEng
STRUCTURAL ENGINEER
30 09 10

HolmesConsultingGroup

STRUCTURAL AND CIVIL ENGINEERS

REPORT

Ohinetahi

Earthquake Damage - Structural review

PREPARED FOR

Sir Miles Warren

14/9/10

Christchurch

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Introduction

Holmes Consulting Group has been engaged by Sir Miles Warren to review the damage that has occurred to the existing historic house at Ohinetahi.

The Richter Magnitude 7.1 earthquake of 4:36 am on September 4 was approximately 35 km west of Ohinetahi, at a depth of 10 km. The earthquake subjected the building to strong ground motions, which may have been close to current design code levels. The house has sustained severe damage, dropping heavy stone masonry from the upper level through the roof below.

Ohinetahi is listed in the Historic Places Trust as a Category I building (from June 1990); and also in the Banks Peninsula District Plan Appendix IV as a Category I protected building.

This report discusses the damage to the house and gives recommendations for future work.

Limitations

Findings presented as a part of this project are for the sole use of the Sir Miles Warren and the Christchurch City Council in the evaluation of the subject propertiy. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses. 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 this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Although there is no existing record of levels prior to the earthquake, it appears that there has been some settlement of the existing foundations.

Figure 1: Northeast corner

Figure 2: Southeast corner

Ohinetahi

Draft Demolition Methodology Statement

15 September 2010

Prepared by C Lund & Son Ltd

Introduction

C Lund & Son Ltd were approached on 9 September 2010 by Holmes Consulting on behalf of Sir Miles Warren to initially structurally secure the damaged central section of the Ohinetahi Homestead at Governors Bay following damage sustained during the 4 September 2010 earthquake. We have subsequently been approached to provide a brief methodology statement for the demolition of the upper portion of level one of the central stone section.

Initial Securing Work Carried Out 13/14 September 2010

The remaining loose stone materials to the upper section of the four gable ends of the central stone section were removed by hand using a man-basket suspended from a 25t crane. The removed stones were placed on seven pallets and remain on the property. There were two bins of loose rubble removed from the interior of the gable ends and removed from the premises.

Horizontal slimshor walers were placed along the northern and southern walls near the level of the top of the level one windows and reid-bar ties placed right through the upper level of the structure to temporarily tie the south wall walers to the north wall walers.

Proposed Demolition Methodology

It is proposed that the upper level of the central stone structure is to be demolished down to the level of the upper level window sills.

This will need to be carried out with care, and structural engineers input will be required for each stage to ensure the structure remains safe during the demolition.

The proposed sequence of demolition works is as follows:

1. Remove roof cladding to the central section of the lower level balconies along the north and south faces and store cladding for possible later re-use.
2. Erect exterior scaffold to the four faces of the stone central core.
3. Remove the stone to the parapets by hand. A crane will be used to lower bins of removed cladding stone to the ground. Cladding stone will be placed onto pallets and the rubble removed off site.
   The pallets of cladding stone will be either stored on site or removed to an agreed storage location off-site.

Ohinetahi

Earthquake Report

Earthquake Damage

1. The four stone gables and two chimneys on the east and west elevations collapsed on to the iron roofs of the single storey blocks, the stone blocks smashing the roof structure and falling into the library, cloakroom, laundry and green room.

2. The stone north face at ground and first floor has bowed out by 100mm. The heads of the two ground floor windows and the door up to the sills of the first floor windows are severely cracked with the worst damage in the north east corner. The central voussoir in the east ground floor window in one of the after shocks fell out and smashed the floor of the verandah. The first floor cornice, though held by the steel tie, has bowed out. The parapet held by the ply strengthened roof is straight but cracked.

3. The eaves of the verandah have bowed out by 150mm and the poorly built stone face has in part collapsed inwards.

4. The south stone wall appears to be in relatively good order up to the first floor window sills. The south-east corner quoins and adjacent wall have cracked and have moved out at the top by approximately 50mm.

   The head of the eastern most first floor window is badly damaged, cracked and with stones out of line.

5. The east and west stone walls, hidden by the timber blocks, cannot be checked. The stone faces below the attic windows appear to lean out. From inside the drawing room the wall above the east wall is badly cracked.

6. The double gabled roof, strengthened with a ply diaphragm to tie the weak parapets, is in good order. But the roof in the south east was holed by the falling chimney. The head of the one window on the west wall is cracked.

7. The northern part of the east block, the former green room, is the oldest part of the building and the worst built. It stands on a stone walled basement with stone piers but on a clay floor. It has not been possible to check these walls because the stone foundations of the perimeter verandah have collapsed inwards. The outer edge of the sagging verandah on the east and west faces has been propped up by temporary posts.

8. Interior Damage:

   The worst room is the drawing room. The movement of the north wall has opened up a wide crack between wall and ceiling. The plaster and cornice have fallen out. The same applies to the hall. There are cracks above the east wall and at the southwest and southeast corners and above and each side of the fireplace.

Below that level, the still vertical south, east and west walls should be strengthened by removing in sections the interior stone and rubble and constructing reinforced concrete walls, say a minimum 150mm thick, against the back of the sandstone face, then battened and lined with gib board.

3. The badly cracked and bowed north wall will have to be carefully demolished with dressed and moulded stones of the jambs and head of the one door and two windows, the dressed quoins, and the courses around the windows other than 300mm high separately identified the whole rebuilt to the level of the window sills as described in 2 above.

4. The central chimney stack will be brought down to first floor level and the interior brick walls, if found to be stable, may be strengthened with reinforced concrete.

5. The first floor above the drawing room has a central steel beam running north-south and 300x50 joists at 450mm centres parallel to the north wall. When the building was restored in 1976 it was found that the joists were simply built into the stone wall with no other fixing. The perimeter joists were blocked and bolted to the adjacent stone. With the loose stones this proved to be ineffective.

6. The amount of stone to be demolished is thus:

Height of walls	x	Length	x	Thickness		
4.5	x	46	x	0.5	=	103.50
Less windows		1.89x1.4x6			=	15.12
						88.38m3
1m3 = 1.8 tons		1.8x 88.30			=	159 tons

One pallet can hold 2 tons.

Sir Miles Warren

poorly cemented stone rubble fill. Interior walls were all plastered but likely to be triple brick or double brick construction.

While the exterior pointing appeared to be reasonably recent the lime mortar between stone (and brick) had little strength and could easily be scratched out.

This masonry section of the house had some limited seismic strengthening comprising a plywood roof diaphragm that was attached to the parapets and an external steel rod tie to all four sides just below the stone cornice level.

All floors are timber framed construction and most of the ground floor internal walls are thick masonry.

The south and west walls sit on shallow stone foundations and the north and east walls sit on taller stone foundations as the natural ground dips diagonally under the house.

Seismic Damage to the Central Section of the House.

There is widespread and extensive damage to the central section of the house.

All four gables fell onto the roofs of the timber framed single storey portions of the house to the east and west of the central section.

The brick chimney that was part of the south west gable also fell.

The stone exterior walls have bowed outwards. The north wall has bowed approx 100mm outwards at first floor window head level and about 40mm outwards at first floor level. The south wall has bowed approx 50mm outwards at first floor window head level.

The plywood roof diaphragm appears to have held the stone cornice and parapets in position meaning that the top section of the north wall has slid outwards by approx 100mm from under the stone cornice. It is likely that diaphragm prevented the complete collapse of the central section of the house.

All the window and door heads have shallow stone arch lintels. In most cases, the seismic shaking has caused the arch stones to drop downwards by approx 20 to 25mm which has in turn caused the wall sections to be pushed sideways and stones immediately above the lintel arches dropping down or in one case, falling out of the wall. Individual jamb stones to windows and doors had displaced sideways by up to approx 20mm.

The sideways movement of the stone is most extreme at the corners of the building above first floor level where the quoin stone and other stones in the vicinity of the corners have been rattled quite loose and shifted sideways by up to up to approx 40mm.

The dry stacked stone forming the verandah foundation/screen wall has been dislodged in many locations and has fallen in other locations.

RUAMOKO solutions
CONSULTING STRUCTURAL ENGINEERS

STRUCTURAL INSPECTION REPORT

Window lintel arches fell approx 2omm displacing stones above and walls alongside

Approx 100mm outward displacement of north wall

Seismic Strength for the Repairs and Rebuilding

The house will be used as a public display as a Cat 1 heritage building. It is arguable that in that use it could be regarded as being subject to The Building Act requirements for Earthquake Prone Buildings. That would mean that the building would be required to be strengthened to at least 34% of the seismic strength of an equivalent new building. The September 2010 CCC Earthquake Prone Building Policy requires that owners of EPBs damaged in this earthquake should aim to strengthen their buildings to 67% of the seismic strength of an equivalent new building.

Conclusion/Short-term Recommendations:

The central section of the house has clearly been seriously damaged by the recent earthquake and is unsafe for occupation and should only be entered by building professionals and contractors that understand the care required when entering or working in damaged buildings.

The severe distortion of the upper sections of the walls mean that they will have to be dismantled and rebuilt to their original alignment. The west, east and south walls could be dismantled down to approx first floor window sill level to get well clear of the dislodged stone but the north wall may need to be