Shirley Library

Quantitative Engineering Evaluation

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Executive Summary

This is a summary of the Quantitative Engineering Evaluation for the Shirley Library 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**	Shirley Library			
Building Location ID	PRO 2215 B001			**Multiple Building Site**	N
Building Address	36 Marshland Road, Shirley			**No. of residential units**	0
Soil Technical Category	TC2	**Importance Level**	2	**Approximate Year Built**	1995
Foot Print (m²)	1100	**Storeys above ground**	1	**Storeys below ground**	0
Type of Construction	Steel warehouse style construction. Light weight roof, cold rolled steel purlins and portal frames, precast concrete walls, concrete floor slab on grade, concrete foundation pads.				
Quantitative L5 Report Results Summary					
Building Occupied	Y	The Shirley Library is currently in service.			
Suitable for Continued Occupancy	Y	The Shirley Library is suitable for continued use.			
Key Damage Summary	Y	Refer to summary of building damage Section 3.1 report body.			
Critical Structural Weaknesses (CSW)	N	No critical structural weaknesses were found			
Levels Survey Results	Y	A level survey has been carried out on 7 may 2012. The results have been reviewed analysed in the current assessment.			
Building %NBS From Analysis	68%	Based on an analysis of capacity and demand.			
Approval					
Author Signature		Approver Signature			
Name	Luis Castillo	Name	Lee Howard		
Title	Senior Structural Engineer	Title	Senior Structural Engineer		

1 Introduction

1.1 General

On 10 September and 20 September 2012 Aurecon engineers visited the Shirley Library 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 geotechnical investigation and any areas where linings and floor coverings need removal to expose connection details.
- Building's drawing analysis.
- Calculation of the building strength including the capacity evaluation of highlighted details in respect of intrusive inspections or drawings.
- Evaluation of the repairing and strengthening needs.

This report outlines the results of our Quantitative Assessment of damage to the Shirley Library 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

Aurecon engineers visited the Shirley Library on 13 January 2012 to carry out a qualitative engineering evaluation.

The report dated 31 January 2012 included:

- Assessment of the nature and extent of the building damage.
- 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 geotechnical investigation, level survey and any areas where linings and floor coverings need removal to expose structural damage.

Damages observed in the qualitative assessment have been reviewed during the inspections related to the quantitative evaluation. They are studied and the relation with the drawing analysis is verified. The analysis includes the calculation and the evaluation of the design/construction methods.

2 Description of the Building

2.1 Building Age and Configuration

The Shirley Library is a large open plan, single story, slab on grade, warehouse style built in 1995. Light weight roofing iron is supported by cold rolled steel purlins (DHS200/12). Purlins span between steel portal frames spaced at 5.7 m intervals for centre spans and 6.5m for east and west extremity spans. These steel portal frames run in the transverse direction and legs are supported on isolated pad foundations. They are typically composed of 360UB45 rafters and columns. The highest point of the roof is typically 4.2m above the finish floor (4.8m above the foundations), with a low point 3.1m above the finish floor. The perimeter pad foundations also support precast concrete cladding panels that form the exterior walls of all façades. The slab on grade is 100mm thick and reinforced with a 668 mesh. The thickness increases close to precast panels to ensure the connection between the slab and the panels.

The Library is approximately 1100 square meters in floor area and is considered to be an importance level 2 structure in accordance with AS/NZS 1170 Part 0:2002. The Library manager Simon Burg was questioned to confirm that the importance level 2 criteria on Table 3.2 of AS/NZS 1170.0:2002 was respected in regards to the building occupancy. At its maximum capacity the building can receive 200 people, which is lower than the 300 people corresponding to the importance level 3 criteria.

2.2 Building Structural Systems Vertical and Horizontal

The roof gravity loads are supported by the purlins and transferred to the foundation by the rafters and columns of the portal frames. An angle is fixed along the concrete panels of the east and west extremity façades and the last bay of purlins spans from this angle to the first portal frame. A part of the gravity loads is therefore supported by these concrete cladding.

The loads from the floor are resisted by the reinforced concrete slab on grade which is founded on 350mm average depth of compacted hard fill.

Transverse lateral loads are resisted by portal frames. Lateral loads originate from both the roof structure and the precast cladding panels. The loads are restrained at the knee of each frame by a welded to frame 150x150x10 angle connection, bolted to couplers which are welded to brackets cast into the cladding panels. This connection provides the out of plane support for the panel. The panels are typically singly reinforced, 150mm thick.

Longitudinal roof loads and the loads from the precast concrete west and east end wall panels are resisted by two bays of double crossed diagonal roof plane bracing (60x60x6 angles) and series of cold formed steel purlins (DHS200/12). These purlins are connected to every frame using a cleat which is welded to the portal frame's rafter. Back to back double purlins are installed at braced bays only. The horizontal bracing is in the between adjacent portal frames two bays in from each end of the library. It transfers loads from the building centre out to the precast concrete side wall panels. Each wall panel will transfer a part of the lateral loads down to the foundation.

2.3 Reference Building Type

The Shirley library is single storey steel portal frame and tilt-up concrete panel warehouse type structure. This type of building that is very common and typically performs well when correctly designed, proportioned and detailed as Shirley Library appears to be, based on visual inspections.

A general overview of the reference building type, construction era and likely earthquake risk is presented in the figure below. The Shirley Library has been constructed in 1995 and according to the figure below may have some issues according to the earthquake loads.

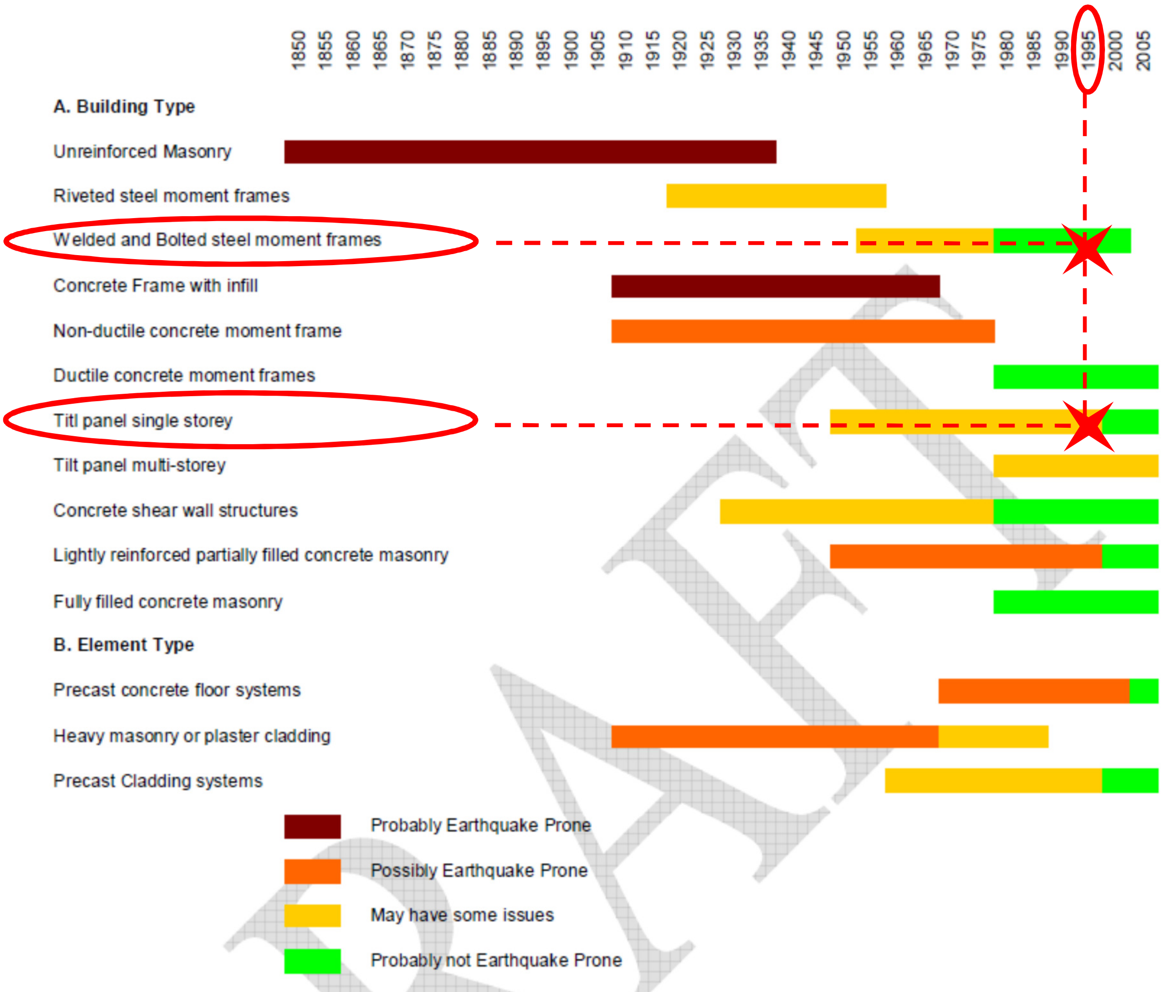


Figure 1: Timeline showing the building types, approximate time of construction and likely earthquake risk.

(From the Draft Guidance on DEEs of non-residential buildings by the Engineering Advisory Group)

2.4 Building Foundation System and Soil Conditions

Drawings indicate that the library floor slab and foundation pads may have been constructed on a number of built up layers of compacted hard fill. Soil in this area is categorised as Technical Category 2 (TC2). According to CERA, TC2 land is considered to "incur minor to moderate land damage from liquefaction and may require specific design for foundations". The Soil Investigation (Soils & Foundations Geotechnical Consulting Engineers) dated 7 July 1995 also confirm this liquefaction possibility. It mentions also that a soil bearing capacity of 100 kPa can be considered for the foundation design. This parameter will be analysed according to the calculated loads transferred to the foundation pads.

2.5 Available Structural Documentation and Inspection Priorities

Original building consent drawings were available and a drawing analysis was carried out.

Electronic copies of the following construction drawings were provided by CCC on 12 September 2012:

-New Library and Service Centre Shirley architectural drawings (Ian Krause, Architects LTD) and structural drawings (Holmes Consulting Group), dated 10 July 1995 and following modifications, dated June 1997.

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses and identify details which required particular attention.

The main potential issues highlighted in the qualitative assessment were the foundation sections indicating multiple layers of compacted hard fill built up below the slab and footings and the lack of a continuous perimeter foundation strip footing between individual pad foundations.

Considering the attention given to the seismic design requirements, seismic load resisting systems and related connections were highlighted in the quantitative assessment.

No structural calculations were available for review, but a copy of the original soil investigation was provided by CCC on 24 September 2012:

-Soil Investigation, Shirley Library (Soils & Foundations Geotechnical Consulting Engineers), dated 7 July 1995.

2.6 Variation between drawings and existing building

The building's inspections and the analysis of the architectural and structural drawings helped to notice two differences between the existing building and the drawings:

- A purlin has been added between the first and second purlin, in the second portal frame span, on the north-east side of the building.
- The north beam of the third portal frame from the east side has been replaced and is connected (weld) to the knee. Welding seems to be full capacity and no damage was observed.

None of these differences influences the building's earthquake behaviour.

2.7 Available Survey Information

2.7.1 Level survey

A floor level survey was undertaken during the qualitative evaluation process and reviewed during the quantitative inspections 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 Department of Building and Housing (DBH) published the "Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence" in November 2011, which 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.

The floor levels for the Shirley Library are considered to be acceptable despite being at the tolerance's limit of the first recommendation in the workroom at building's north-east. In these areas local floor slopes of up to 0.5% have been calculated. The total difference between the slab highest point and the slab lowest point is 30mm.

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

2.7.2 Verticality survey

Even if the code requirements covering the criteria for plumbness in concrete construction are written for new constructions and not for those which may have moved after an earthquake, it provides a good guidance in determining acceptable values with minimal structural impact.

Most of the verticality readings stay within the plumbness tolerance for *in-situ* construction (10 mm) from *concrete construction* NZS3109-1997. However, the readings between N4 and N5 are above this tolerance (23 mm, see verticality survey in Appendix A) which can lead to high damage levels for non-structural elements and an increase of the strain in the portal frame and stability issues due to P-Delta effects. By including this displacement the member strain is only increased by 3%, which is considered to be acceptable. The results also indicate that the widening of the joints did not influence the verticality or stability of the panels. Besides, the effect of verticality deviation is negligible for the in-plane capacity of each panel and no damage to non-structural elements was noted.

3 Structural Investigation

3.1 Summary of Building Damage

The Shirley Library is currently in use and was occupied at the time the assessment was carried out. Library manager Simon Burg was available and was helpful in providing access and assisting with the inspection of critical structural elements.

Taking as a reference, the qualitative report helped to target the main areas of damage. The following damage were noticed in the qualitative assessment and reviewed during the inspections of the quantitative assessment;

- Splitting and partial spalling at the ends of precast concrete panels at bearing points above pad foundations particularly in the north-west and south-east corners.
- Spalling at panel bottom edges, also above foundation pads, on some intermediate panels.
- Widening of control joints in the concrete floor slab both longitudinally and transversely.
- Minor flexural and shear cracking at corners of window openings in precast concrete cladding panels.
- Minor displacement damage to entrance canopy.
- Some evidence of settlement to footpath slabs at the western end of the building.
- Evidence of liquefaction including sink holes and local subsidence in surrounding land. (noticed in qualitative assessment but not in the quantitative)

Other observations or damages noticed during the inspections of the quantitative assessment are summarized as follows:

- Evidence of pull-out actions observed for two connections.
- One flat washer plate not properly installed.

3.2 Investigation Procedures

Ceiling tiles adjacent to the transverse portal frame were lifted making it possible to inspect the condition of most frame knee joints and the precast concrete cladding panel upper connections to the frame (all the accessible elements were inspected). Three knee joints and three upper connections were concealed by a gypsum ceiling.

No obvious damage or residual deformation to either the frame knee joint or the connection of the cladding panel to the frame at knee height was noticeable.
Considering the good overall condition of the structural element, no intrusive inspections were required for the three concealed knees.

Also inspected was the connection of the cold rolled steel purlins to the precast concrete end walls. No damage to the purlins supporting the precast concrete end walls was observed. There was some evidence of displacement of the steel angle supporting the purlins along the end wall. This damage is likely due to in plane movement between adjacent panels.

No damage was observed to the roof bracing. All accessible parts of the bracing were inspected. Special attention was given where the highest stress occurred, which was where the equal angle cross-bracing connects to the portal frame knee. At this location, the angle was welded to the top flange of the portal frame rafter. The connection at this point showed no signs of undue stresses and no sagging of the diagonal bracing.

3.3 Record of Intrusive Investigation

Even though many of the critical structural elements could be observed above the suspended ceiling, some intrusive investigations were carried out to inspect connections between panels and slab, and mid panel to panel connections. The main objectives were to detect signs of movement or yielding in connections and to confirm the proper set up shown in the drawings.

No sign of movement or yielding were observed in the connections during the intrusive investigations. All the elements inspected were as shown on drawings, according to their dimensions and quantity. The presence of bar thread for the bottom connections and the 668 mesh in the slab on grade were also confirmed.

To evaluate if any major slab or panel displacement occurred, the level survey of the previous assessment was analysed (see 2.7.1) and a verticality survey of the panels was carried out (see 2.7.2).

The precast concrete cladding panels were able to be inspected directly from the building exterior. The entrance canopy support structure was not possible to view directly.

3.4 Damage Discussion

The most significant observed damage was splitting and spalling at the ends of precast concrete panels near the connection to the pad and floor slab and widening of the control joints to the concrete floor slab. This damage is likely due to compression failure as the panel rocked under the seismic loads. Panel rocking can generate high localized compression forces at the ends of the panels above the foundation pads. For this type of structure, the highest load generated by rocking occurs at the extremity of the last panel, which is exactly the location of the damage. Although the edges of the corner panels have burst, further rocking is unlikely to cause the damage to progress significantly because compressive forces required to burst further concrete off the panel will be higher than those that have caused the initial damage. Carpet tiles above the interior floor slab adjacent to the areas of damage to the panels were lifted but no damage to the interior concrete slab was observed.

Widening of the floor slab and control joints is of some concern because it may have resulted in a permanent increase in overall slab width. Where it was observed the control joint width was measured at approximately 4 to 10mm. The proportion of the crack width due to earthquake effects can only be estimated as some of the crack width is due to the original saw cut and some due to slab shrinkage. However some is clearly due to earthquake effects and this can be seen in the carpet tiles bridging the cracks. Where the cracks have widened the carpet tiles no longer fit snuggly. Not all control joints were observed and it may be that more than one parallel joint has expanded. This movement may have resulted in the lateral displacement of the foundation pads supporting the portal frame legs and the base of the precast cladding panels although no evidence of this was observed on the building exterior. To make sure no displacement occurred, a level survey was carried out in the previous assessment and a verticality survey was carried out in the present one (see Appendix A).

The evidence of pull-out actions was noticed exclusively to the detail connecting the concrete precast panels to the web of the portal frame's column (see drawing's typical portal details 5, S1-6). The assembly is an angle connected to the precast panels with a plate and dynabolts. The detail is used to stabilize the portal frame in case of an out-of-plane drift or torsion of the portal's column, which may have occurred during the earthquakes. However, the level of damage is very low, and it doesn't influence on the general stability of the building, so as for the not properly installed washer.

Other damage mentioned in the building damage summary section includes displacement damage to the entrance canopy and evidence of settlement and liquefaction in the near vicinity of the library. The aerial photo in Appendix A was taken on February 24 and clearly shows evidence of ground disturbance and liquefaction. Other residual evidence of ground movement was noted at the time of the inspection. The entrance canopy supporting structure was not observed however structural drawings indicate that the design is robust.

4 Building Review Summary

4.1 Building Review Statement

Because most of the critical structural components of this building were assessable by lifting the ceiling panels, a visual inspection was carried out. Only the entrance canopy support structure and the foundations were not able to be directly reviewed. From the minor nature of the observed displacement damage and the robust design documented on the original construction drawings the performance of the entrance canopy support structure has been inferred as adequate.

4.2 Critical Structural Weaknesses

No specific critical structural weaknesses were identified as part of the building quantitative or qualitative assessments.

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

5.1 General

With well distributed walls and good detailing, the building has performed well in the Canterbury earthquake.

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 panels in the along Direction, μ	1.5	Tilt-up concrete precast panels
Ductility Factor for the connections between the tops of the precast wall panels and the portal frames, μ	1	Connections between the tops of the precast wall panels and the portal frame. Shear at bottom connection.
Ductility Factor for the steel frames in the across Direction, μ	3	Portal frames (moment frames)

Despite the use of best national and international practice in this analysis and assessment, the values contain uncertainty due to the many assumptions and simplifications which are made during the assessment (Refer to Appendix B for the limitation and assumptions). Furthermore, no original structural calculations were available for review.

A summary structural performance 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.

A structural analysis was carried out to verify if the concrete walls were able to resist lateral loads.

Table 1: Summary of Performance (concrete wall 1)

Structural Element/System	Comments [1]	%NBS Based of Detailed Assessment
LONGITUDINAL DIRECTION		
Precast concrete wall		**100%**
Precast concrete wall, along direction seismic performance. (South wall panels line C: P1, P2, P3, P4, P5, P6, P8, P9; North wall panels line A: P14, P15, P16, P17, P18, P19, P20, P21, P22)		
In-plane shear capacity of the bottom connections	Yielding of the anchor bars (YD-12) from the thickened ground floor slab to the precast concrete panels. The failure mechanism itself is not brittle. Considering the important amount of bars for connection per panel (18).	>100%
Weld capacity from the panel connection angle to the portal frame	Brittle failure of the weld during an earthquake	>100%
In plan shear capacity of the panel	Concrete cracking in the shear plan can lead to eventual yielding of the reinforcement crossing this plan. The failure mechanism itself is not brittle.	>100%
In plan shear at the last panel's extremity	Assuming no rebar is perpendicular to the shear plan. It's a compression failure as the panel rocks under seismic loads. Although the edges of the corner bursts, further rocking is unlikely to cause the further damage because compressive forces required to burst further concrete off the panel will be higher than those that have caused the initial damage.	100%
Precast concrete wall, along direction seismic performance (South wall panels line C: P7)		
In-plane shear capacity of the bottom connections	Yielding of the dowels (YD-20 dowels) connected to the precast concrete panels on both sides and the portal frame. The failure mechanism itself is not brittle. Considering the important amount of reinforcing bars for connection per panel (18).	>100%
Precast concrete wall, across direction seismic performance. (East wall panels line 10: P10, P11, P12, P13; West wall panels line 1: P23, P24, P25, P26)		
In-plane shear capacity of the bottom connections	Yielding of the anchor bars (YD-12) from the thickened ground floor slab to the precast concrete panels. The failure mechanism itself is not brittle. Considering the important amount of bars for connection per panel (18).	>100%
In plan shear capacity of the side connections between panels	Concrete cracking in the shear plan can lead to eventual yielding of the reinforcement crossing this plan. The failure mechanism itself is not brittle.	>100%
In plan shear capacity of the panel	Assuming no rebar is perpendicular to the shear plan. It's a compression failure as the panel rocks under seismic loads.	>100%

Structural Element/System	Comments [1]	%NBS Based of Detailed Assessment
TRANSVERSAL DIRECTION		
Portal Frames		**85%**
Portal frames flexural capacity (strong axis)	Yielding in flexure of the portal frame's beam and columns. The columns are idealized with "pin" base.	85%
Drift-across direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements and premature collapse due to P-Delta effects	100%
Steel frame stability and diaphragm		**68%**
Horizontal roof bracing	Yielding in axial tension of the bracing angles. The failure mechanism itself is not brittle. The bracing is assumed to be in tension only (conservative).	68%
Weld capacity of the horizontal bracing weld to the portal frame	Brittle failure of the weld during an earthquake.	>100%
Combined bending and compression capacity of the purlins in the horizontal bracing bays	Compression failure is not brittle and the	94%
Foundations		**84%**
Foundation dimensions according to soil bearing capacity	Under-dimensioned foundations can increase the risk of settlement when fully loaded, which can eventually lead to other structural disorder. Assuming that the information included in the July 1995 soil investigation is still applicable.	84%

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

5.3 Results Discussion

Detailed calculations highlighted moderate percentages in regards to the horizontal roof bracing. Based on the behaviour of the roof during the earthquakes and on the many inspections made which have shown no signs of damages, the diaphragm is adequate. It transferred efficiently the loads through the purlins, bracing, bolts and welds to the main structure. Furthermore, the Shirley Library is a symmetrical, single story, lightweight structure with simple and well defined load paths. This is a building type and configuration that can be resilient and appears to have performed well during the Canterbury Earthquakes to date.

As for the qualitative study, our opinion remains the same. Although the precast concrete exterior panels are damaged it is our opinion that the damage is not sufficient to significantly reduce the capacity of the building to resist lateral loads. The tops of the panels are well secured to the frames. No damage to panel frame connection was observed and no obvious damage or residual deformation to the supporting moment frame knee joint was observed.

Widening of the control joints in the floor slab may have resulted in a permanent increase in overall slab width but this will not have reduced the lateral load capacity of the building itself.

Construction plans are stamped Sept 1995 indicating that this building was designed to the New Zealand Standard NZS 4203:1992 loadings code. Detailed calculations give a percentage new building standard (%NBS) of 85% transversely, 100% longitudinally, 84% in regards to the foundations and 68% for the steel frame stability and diaphragm, which governs the overall NBS percentage of the building.

6 Conclusions and Recommendations

The land below the Shirley Library is zoned TC2 and as such, has been identified as somewhat prone to liquefaction and settlement. The original soil investigation (1995) confirms this statement. A level survey was carried out within the Shirley Library to determine the extent of any differential settlement, but the results are within the acceptable limits. Furthermore, the local evidences of liquefaction in the surrounding land have not been included in the quantitative assessment. Based on the good overall performance of the building shown in calculations, its good behaviour during the earthquakes, the limited differential settlement observed and the acceptable plumbness, the geotechnical investigation recommended in the previous assessment is no longer needed.

Additional strengthening is not required in any of the structural systems. Visible cracks and spalling can be repaired using epoxy-based coating and patching mortar. As the structural strength of the concrete elements has not been reduced in the earthquakes, the repairing objective will be to prevent concrete degradation and the rebar corrosion as a result of the minor damage.

The building is currently occupied and in use as a library. Additionally, the building has suffered no loss of functionality and in our opinion the Shirley Library **is considered suitable for continued 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.

To carry out the structural review, existing building drawings were obtained (where available) from the Christchurch City Council records. We have assumed that the building has been constructed in accordance with the drawings.

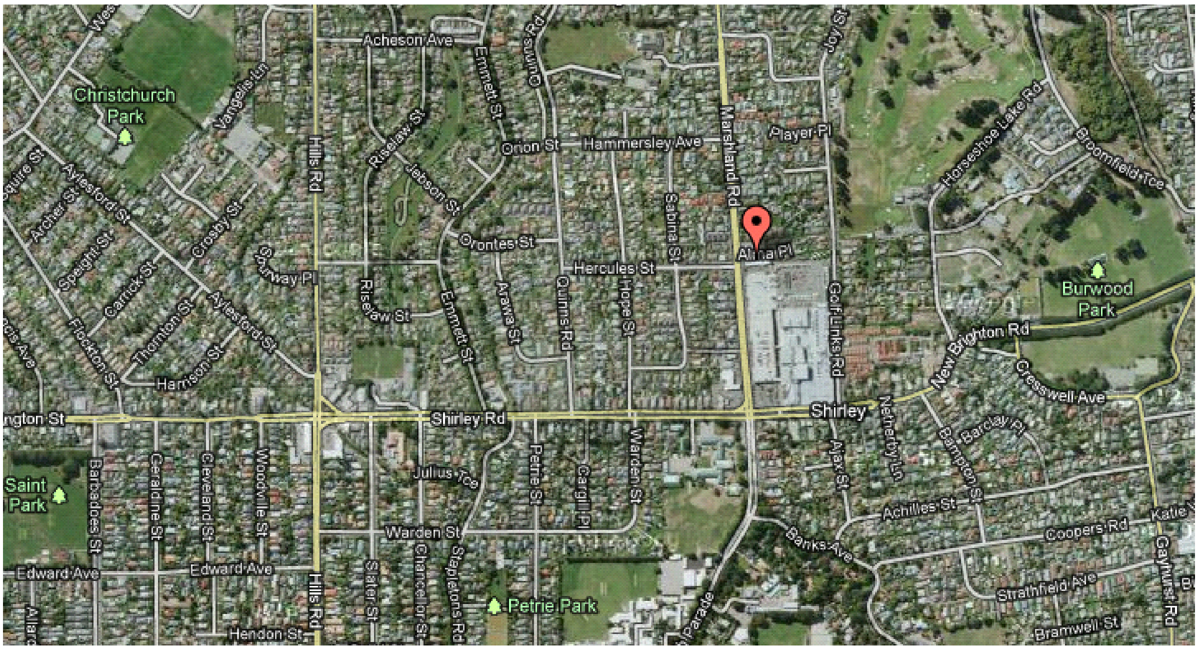
While this report may assist the client in assessing whether the building should be repaired, strengthened, or replaced that decision is the sole responsibility of the client.

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.

Without limiting any of the above, Aurecon's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited as set out in the terms of the engagement with the client.

Appendices

Appendix A
Site Map, Photos, Levels Survey, Verticality Survey, Original Soil Investigation



10 January 2012 – Shirley Library Site Photographs.

South elevation of the Shirley Library.	
North elevation of the Shirley Library.	
Oblique view of the West elevations of the Shirley Library.	
Oblique view of the East elevations of the Shirley Library.	

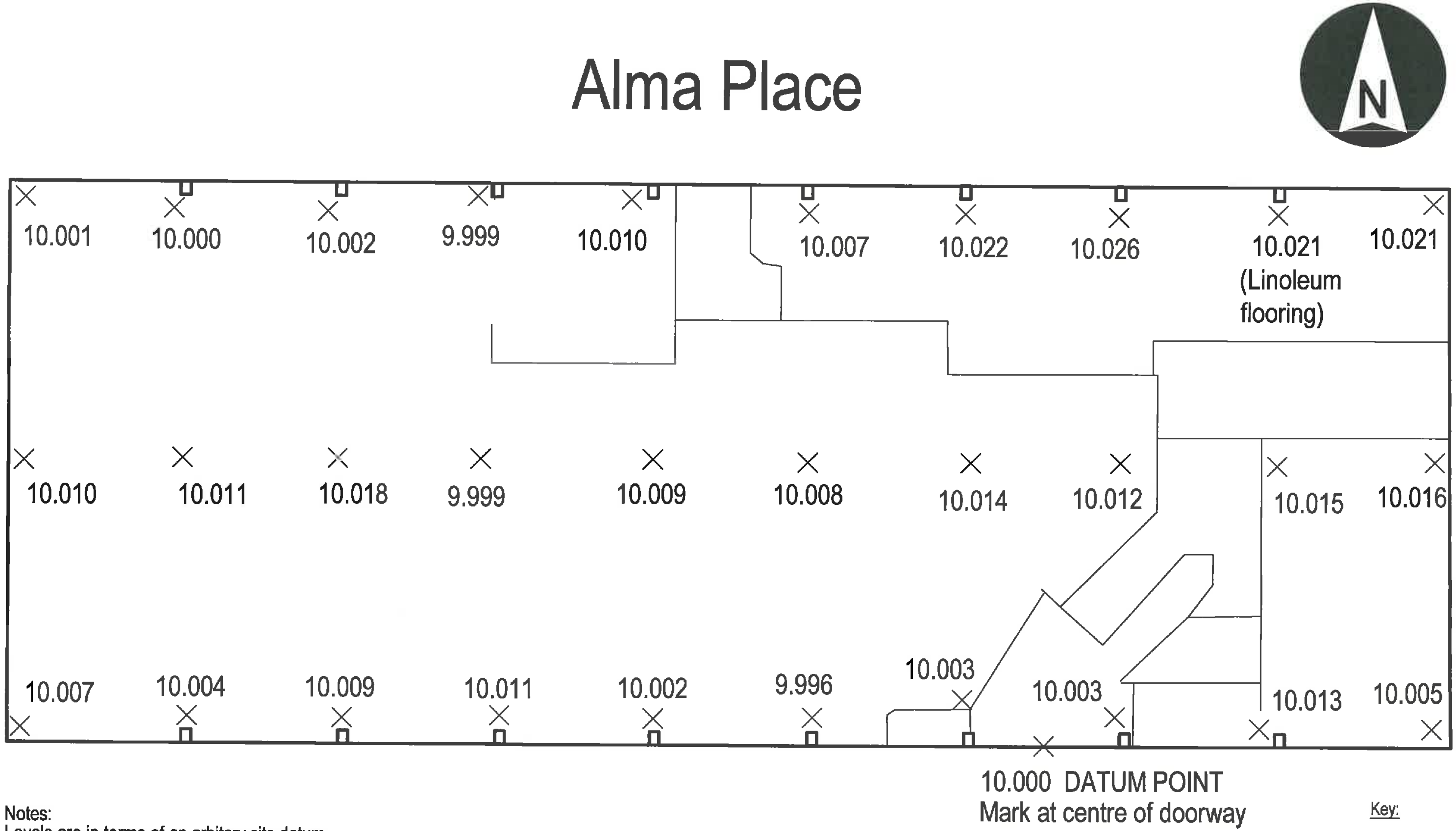
Vertical splitting at end of south-west precast concrete corner panel.	
Concrete spalling at connection between adjacent panels above foundation pad (on west facade).	
Evidence of pull-out actions in some connections.	
Connection’s washer not properly installed.	

No evidence of damage to floor slab on interior at panel joint.	
Important widening of slab control joint by up to 10.0mm.	
Important widening of slab control joint by up to 10.0mm.	
View of portal frame knee joint above ceiling.	

View of portal frame rafter.	
Precast concrete panel to panel connection at corner of building.	
Equal angle steel stringer supporting purlins on west elevation.	
Differences between the existing construction and the drawing. Rafter welded to the knee.	

Differences between the existing construction and the drawing. Purlin added.	
Intrusive inspection of the bars connecting the precast panels and the slab.	
Intrusive inspection of the bars connecting the precast panels and the slab.	
Intrusive inspection of the middle connection plates connecting two precast panels and the portal frame partially concealed.	

Alma Place



Notes:
Levels are in terms of an arbitary site datum
All levels were taken on carpet unless otherwise noted.
The linoleum flooring is 5mm lower than carpet heights.
Relative level accuracy ±2mm.
Date of Survey: 07/05/2012

Key:
- Internal Walls
- External Walls
- Columns
- Level location

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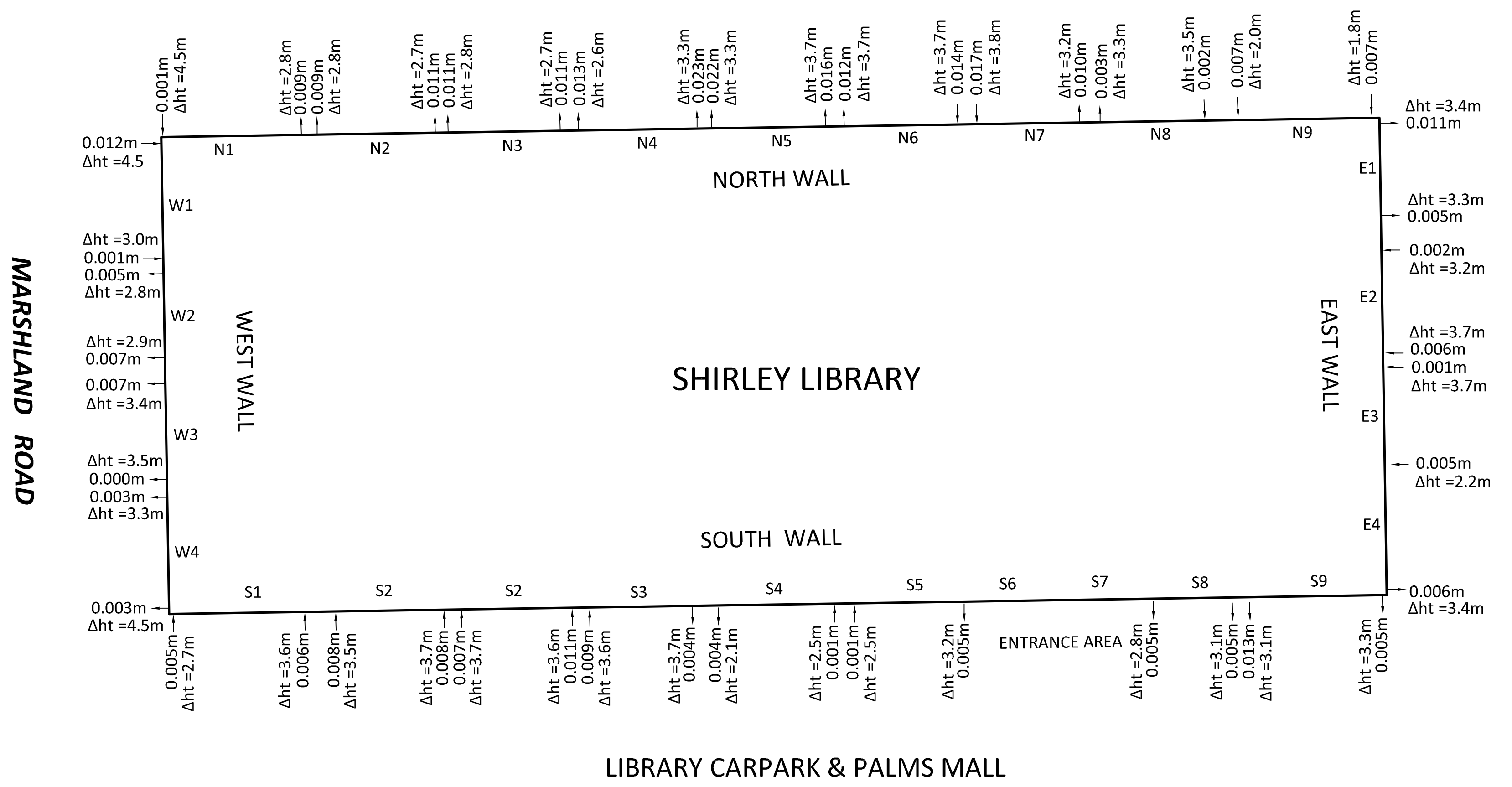
CLIENT	PROJECT	
CCC	Shirley Library, Marshland Road	NOT FOR CONSTRUCTION

DRAWN	SURVEYED	CHECKED	APPROVED	DATE	TITLE	PROJECT No.	SCALE	SIZE	DRAWING No.	REV
TMurray	TMurray	RHope	RHope		Floor Level Survey	227255	N.T.S	A4	SU.01	A

REV	DATE	REVISION DETAILS	APPROVED
A	[illegible]	[illegible]	



DATA QUALITY STATEMENT

SURVEYED DATA

SURVEYED DATA HAS BEEN CAPTURED USING SURVEY EQUIPMENT, TO A RELATIVE ACCURACY WITHIN ±5mm

0.012m → HORIZONTAL DISPLACEMENT (AND DIRECTION) FROM BOTTOM TO TOP OF PRECAST PANELS. MEASUREMENTS TAKEN TO BOTTOM AND TOP CORNERS OF CONCRETE PANELS.

2.5 0 5 10m

SCALE 1:300

10/10/12

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CLIENT

REV	DATE	REVISION DETAILS	APPROVED
A	16.10.12	VERTICALITY INFORMATION	RH

PROJECT
36 MARSHLAND ROAD
SHIRLEY
CHRISTCHURCH

TITLE
SHIRLEY LIBRARY
BUILDING VERTICALITY

DRAWN	DESIGNED	
R DAWSON	K RITSON	INFORMATION NOT FOR CONSTRUCTION
CHECKED		PROJECT No.
K ASHBY		227255
APPROVED	DATE OCT 2012	SCALE 1:300 / SIZE A4
R HOPE		DRAWING No. SU 02 / REV A

29810

SOILS & FOUNDATIONS

Geotechnical Consulting Engineers

Soils & Foundations (1973) Ltd
Ernst & Young House
227 Cambridge Tce
PO Box 13-052
Christchurch, New Zealand
Facsimile 0-3-366 7780
Telephone 0-3-379 8432

7 July 1995
92739/00

DISTRIBUTION	
RGW	G
BJW	
HJH	
TMB	

Tim Bradford
C/- Holmes Consulting Group
PO Box 701
CHRISTCHURCH

Dear Sir

RE: SOIL INVESTIGATION, SHIRLEY LIBRARY

Soils and Foundations have carried out a soils investigation on the above property as requested.

Five handaugers and six scala penetrometer tests were performed at the locations shown on the site plan 92739/1 attached. Borelogs and scala penetrometer test results are also attached.

The borelogs show similar soil profiles of topsoil to between 0.1m and 0.5m overlying silts and sandy silts to between 0.5m and 1.2m where fine to medium sands were encountered. Generally drilling was stopped due to collapsing sands at depths ranging between 1.1m and 1.8m. Borehole HA4 found a deposit of organic fill located at ground surface down to a depth of 0.6m. Drilling was stopped prematurely in borehole HA5 at a depth of 0.5m due to an unidentified solid object.

Apart from HA4 and HA5, the soil profiles on this site are found to be consistent with other nearby sites we have investigated.

The water table was found to be at approximately 1.5m at the time of the investigation.

The scala penetrometer test results show that generally firm or compact conditions exist below 0.5m within all layers. However around the north-east corner of the site scala SC7 found very soft fill down to a depth of 1.4m. No borehole was drilled at this location, although HA3 was located 1-2m away and showed firm natural soils.

Bearing capacities for footings founded below the topsoil layers can generally support loads of up to 100kPa. The extent of all filled areas should be located during foundation excavation. These areas should then be excavated and backfilled with compacted pit-run material.

While on site, a discussion held with a local of the area lead us to believe that there may be an old infilled swimming pool located to the north-west of the site, (near HA1).

If you have any questions regarding the above, please don't hesitate to contact this office.

Yours faithfully
SOILS & FOUNDATIONS
per:

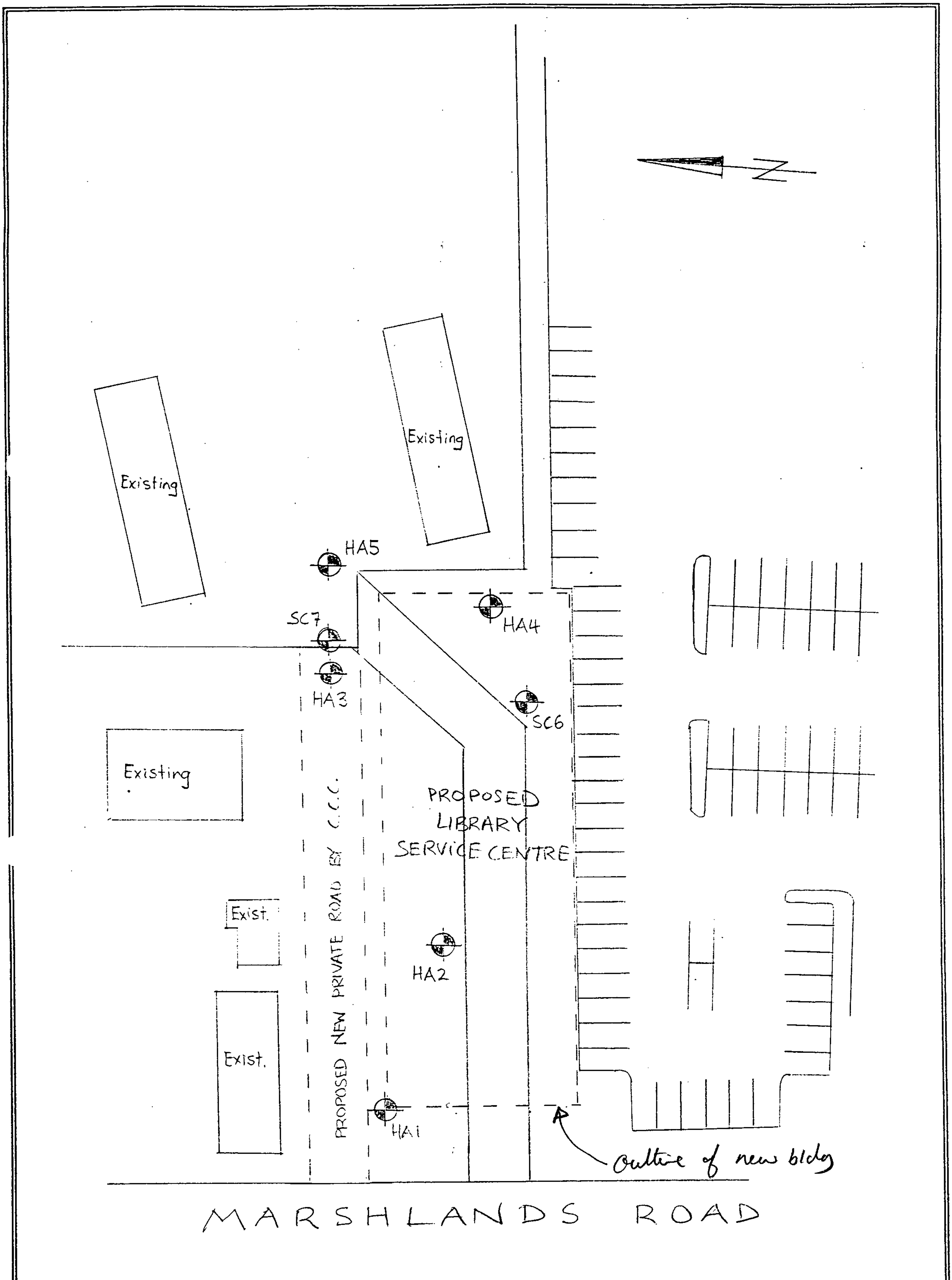
JBGarratt

J B Garratt

JBG:IMcC

"This report has been prepared solely for the benefit of Holmes Consulting Group and the Christchurch City Council. No liability is accepted by this firm or by any principal, or director, or any servant or agent of this firm, in respect of its use by any other person. Any other person who relies upon any matter contained in this report without consultation with and agreement by Soils and Foundations as to its applicability to that persons intentions, does so entirely at their own risk.

This disclaimer shall apply notwithstanding that the report may be made available to any person other than the above client and territorial authority in connection with any application for permission or approval, or pursuant to any requirement of law."



SITE PLAN OF PROPOSED LIBRARY SERVICE CENTER	Drawn JBG	SOILS & FOUNDATIONS
	Date 7-7-95	Figure 92739/1

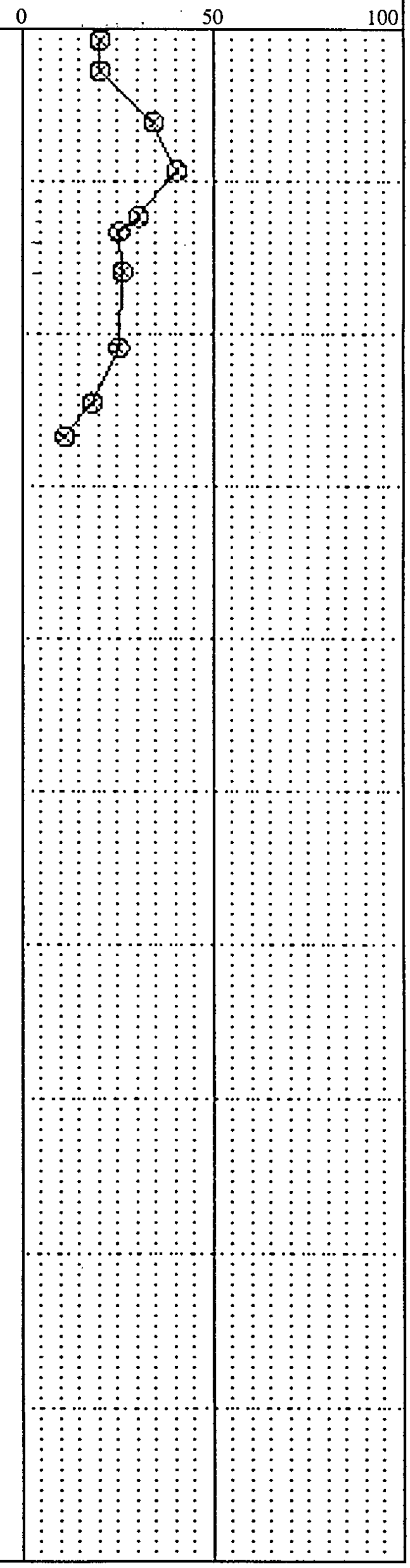
S O I L S & F O U N D A T I O N S L T D.

Project: **SHIRLEY LIBRARY**	Project Number: **92739**	Boring ID. **HA1**
Client: **HOLMES CONSULTING GROUP LIMITED**		Sheet No. **1** of **1**

Boring Depth (m.): **1.8**	Elevation:	Engineer: **G. CHESTERMAN**	Starting Date: **5/7/95**
Datum/Notes: **/**			Ending Date: **5/7/95**

Elev. (m)	Depth (m)	Lith-ology	**Material Description**	Well Detail	Comments
			TOPSOIL.		
			SILT with minor sand. -moist, firm, some iron staining		
	1		Grey SILT. -dry to moist -firm -heavy iron staining		
			Fine grey SAND. -wet -loose -clean		
	2		NO FURTHER PROGRESS DUE TO COLLAPSING SAND.		
	3				
	4				

Scala Penetration (mm/blow)

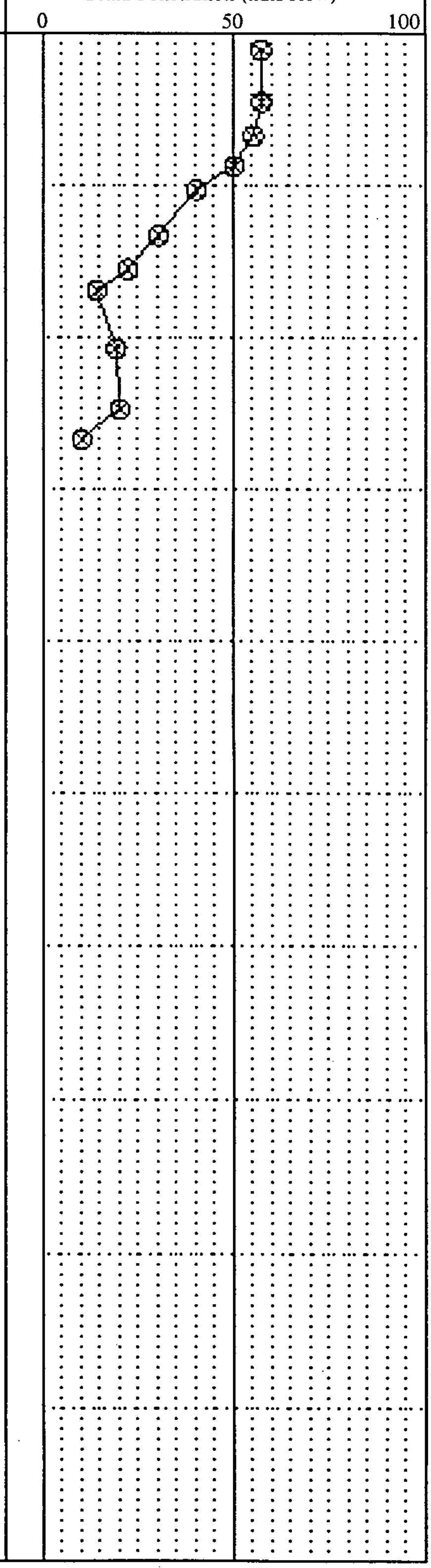


SOILS & FOUNDATIONS LTD.

Project: SHIRLEY LIBRARY	Project Number: 92739	Boring ID. HA2
Client: HOLMES CONSULTING GROUP LIMITED		Sheet No. 1 of 1
Boring Depth (m.): 1.8	Elevation:	Engineer: G. CHESTERMAN
Starting Date: 5/7/95	Ending Date: 5/7/95	
Datum/Notes: /		

Elev. (m)	Depth (m)	Lith-ology	Material Description	Well Detail	Comments
			TOPSOIL.		
			SILT with minor sand. -dry -soft -heavy iron staining		
	1		Medium grey SAND. -moist -firm -wetter with depth		
	2		NO FURTHER PROGRESS DUE TO COLLAPSING SAND.		
	3				
	4				

Scala Penetration (mm/blow)



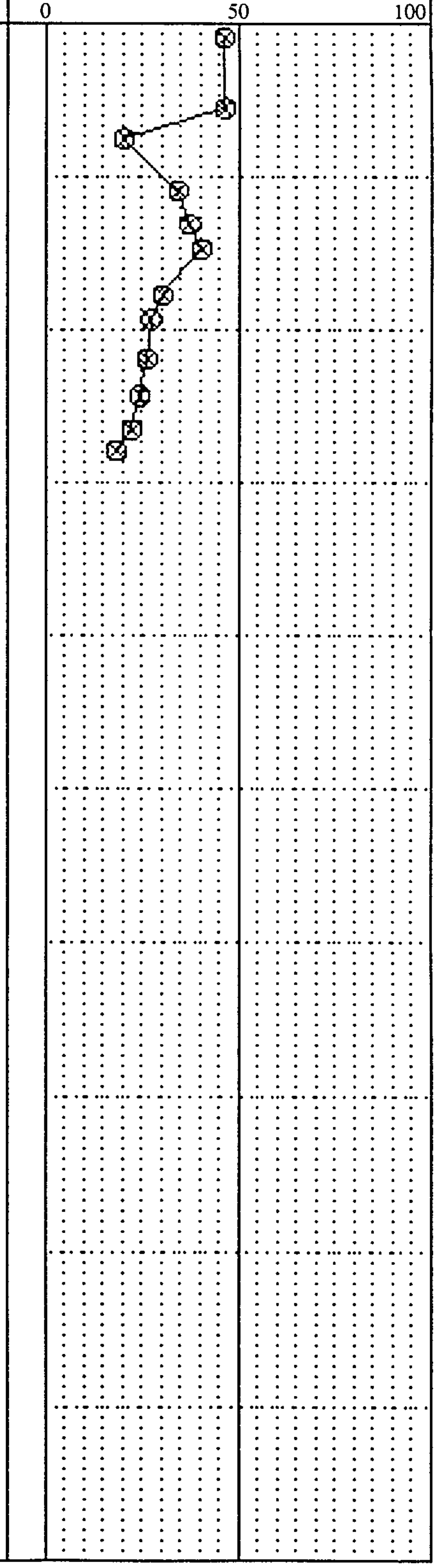
SOILS & FOUNDATIONS LTD.

Project: **SHIRLEY LIBRARY**	Project Number: **92739**	Boring ID. **HA3**
Client: **HOLMES CONSULTING GROUP LIMITED**		Sheet No. **1** of **11**

Boring Depth (m.): **1.4**	Elevation:	Engineer: **G. CHESTERMAN**	Starting Date: **5/7/95**
Datum/Notes: /			Ending Date: **5/7/95**

Elev. (m)	Depth (m)	Lithology	**Material Description**	Well Detail	Comments
			TOPSOIL.		
			Grey SANDY SILT. -moist -firm -some iron staining.		
	1		Medium grey SAND. -moist -firm -clean -some iron staining with depth -becoming wetter with depth		
			NO FURTHER PROGRESS DUE TO COLLAPSING SAND.		
	2				
	3				
	4				

Scala Penetration (mm/blow)

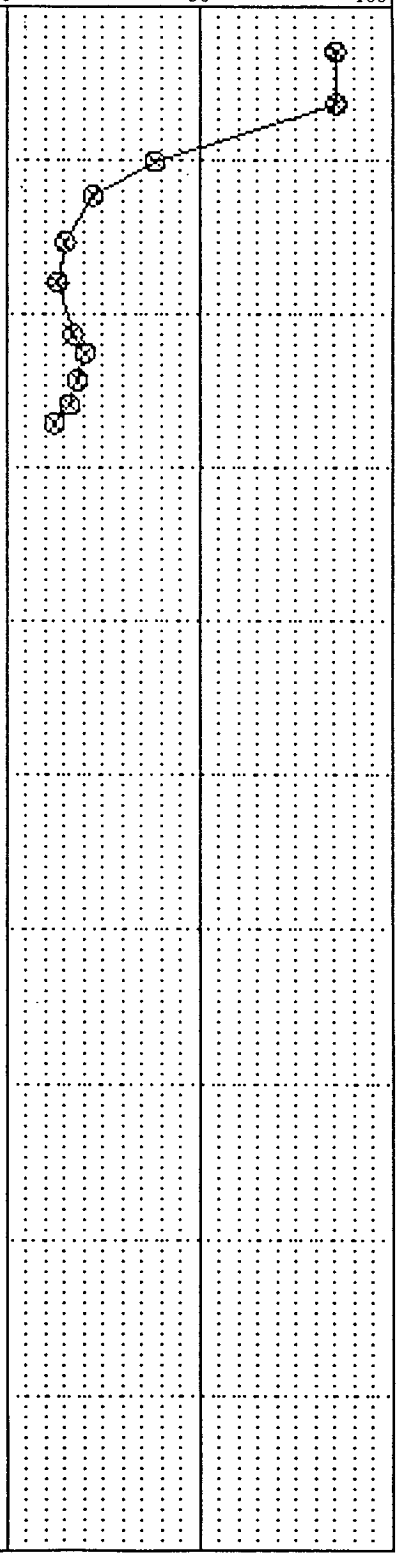


SOILS & FOUNDATIONS LTD.

Project: SHIRLEY LIBRARY	Project Number: 92739	Boring ID. HA4
Client: HOLMES CONSULTING GROUP LIMITED		Sheet No. 1 of 1

Boring Depth (m.): 1.4	Elevation:	Engineer: G. CHESTERMAN	Starting Date: 5/7/95
Datum/Notes: /			Ending Date: 5/7/95

Elev. (m)	Depth (m)	Lith- ology	Material Description	Well Detail	Comments	Scala Penetration (mm/blow) 0 50 100
			Dark brown organic FILL. -moist -firm			
			Grey SILT with minor sand. -moist -firm -mottled orange -becoming siltier and softer with depth -More iron staining with depth			
	1		Medium grey SAND. -clean, wet, compact			
			NO FURTHER PROGRESS DUE TO COLLAPSING SAND.			
	2					
	3					
	4					



SOILS & FOUNDATIONS LTD.

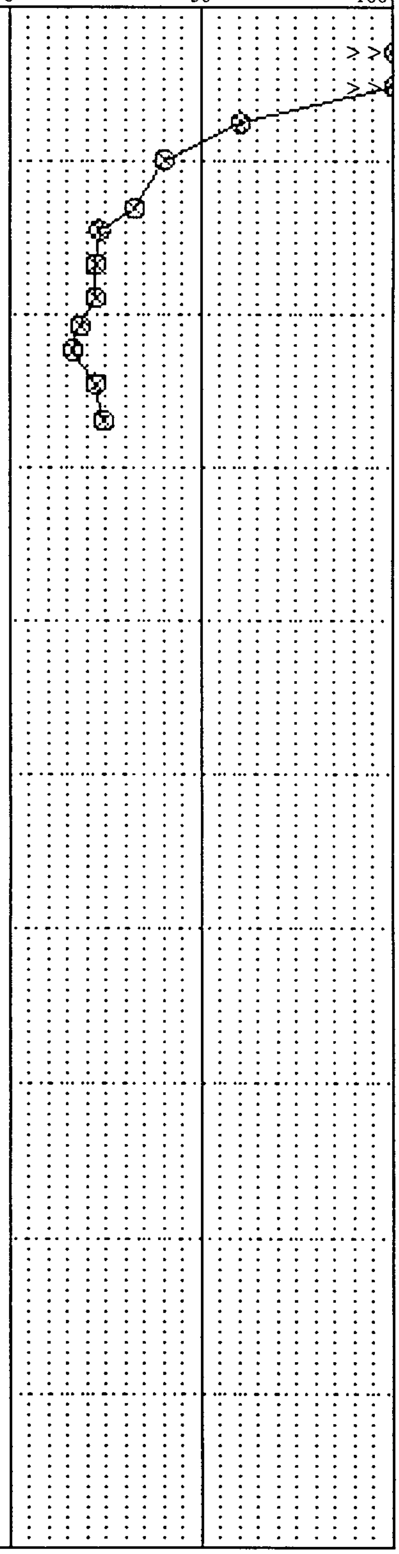
Project: **SHIRLEY LIBRARY**	Project Number: 92739	Boring ID. HA5
Client: **HOLMES CONSULTING GROUP LIMITED**		Sheet No. 1 of 1
Boring Depth (m.): 0.5	Elevation:	Engineer: **G. CHESTERMAN**	Starting Date: 5/7/95
Datum/Notes: /		Ending Date: 5/7/95

Elev. (m)	Depth (m)	Lith-ology	Material Description	Well Detail	Comments	Scala Penetration (mm/blow) 0 50 100
			TOPSOIL			
			SILT with minor sand. -dry -firm -heavy iron staining			
			NO FURTHER PROGRESS DUE TO HARD OBJECT.			
	1					
	2					
	3					
	4					

SOILS & FOUNDATIONS LTD.

Project: **SHIRLEY LIBRARY**	Project Number: **92739**	Boring ID. **SC6**
Client: **HOLMES CONSULTING GROUP LIMITED**		Sheet No. **1** of **1**

Boring Depth (m.): **1.4**	Elevation:	Engineer: **G. CHESTERMAN**	Starting Date: **5/7/95**
Datum/Notes: /			Ending Date: **5/7/95**

Elev. (m)	Depth (m)	Lith-ology	**Material Description**	Well Detail	Comments	Scala Penetration (mm/blow) 0 50 100
	1 2 3 4		NO BORE LOG TAKEN.			

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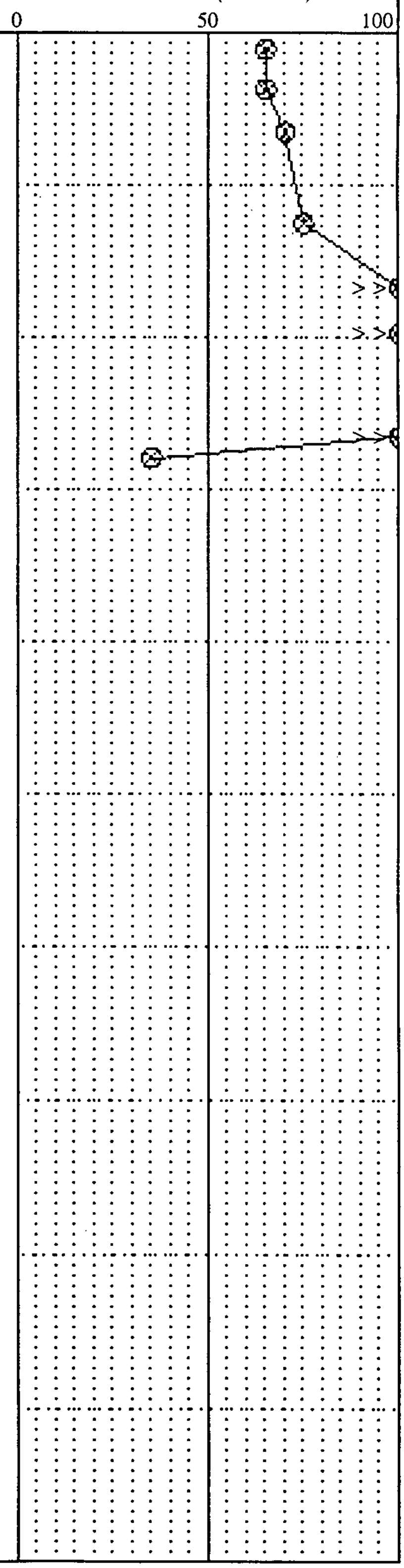
Project: SHIRLEY LIBRARY	Project Number: 92739	Boring ID. SC7
Client: HOLMES CONSULTING GROUP LIMITED		Sheet No. 1 of 11
Boring Depth (m.): 1.4	Elevation:	Engineer: G. CHESTERMAN	Starting Date: 5/7/95
Datum/Notes: /		Ending Date: 5/7/95

Elev. (m)	Depth (m)	Lith-ology	Material Description	Well Detail	Comments	Scala Penetration (mm/blow)
	1 2 3 4		NO BORE LOG TAKEN.			

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SOILS & FOUNDATIONS

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Telephone 0-3-379 8432

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BJW	
HJH	
HMB	✓

FACSIMILE TRANSMISSION

DATE 6/7/95

FAX NO. 379-2169

TO Holmes Consulting Group Ltd.
Attn Tim Boadford

FROM R Wise.

JOB NO. 92739.

NO OF PAGES (INCLUDING THIS PAGE) 3

PLEASE ADVISE IMMEDIATELY IF THE NUMBER OF PAGES RECEIVED IS LESS THAN THE NUMBER OF PAGES STATED.

Tim.

Shirley Library + Service Centre

Preliminary results of soils investigation as follows:-

* Most soils are silts (to ≈ 1.0m) over compact sands. (Topsoil to depths of ≃ 0.5 - 0.65m)
* Therefore most footings will require excavation to 0.5 → 0.65m. (shallow footings OK).
* Bearing capacities for strip footings below topsoil are 100kPa or better in most instances. (better with depth)

However.

* Soft fill was located (to a depth of 1.32m) in the area of SC7 (No hand auger just scala) But Not in HA3 (see site plan to follow)
* Some sort of hard fill located at 0.5m in HA5 (No further progress with auger).
* Possible infilled swimming Pool in area (to south of) HA1. Information from local talked to on site.

Therefore

* Exact locations of fill will need to be determined during construction.
* Excavation of fill and back fill with compacted Pitrun will be required.

SOILS & FOUNDATIONS	Shirley Library + Service Centre.	Job No.
Computed R Wise		Sheet of
		Date

I hope this helps. full report to follow ASAP.

Note that most of the site is paved and sealed so access for Handaugers was difficult ∴ some areas have not had subsurface investigation. (ie South west of site).

Also, the natural soils found are consistent with those found at Shirley shopping centre.

Sands below 1m are potentially liquefiable but are dense so should not be a problem.

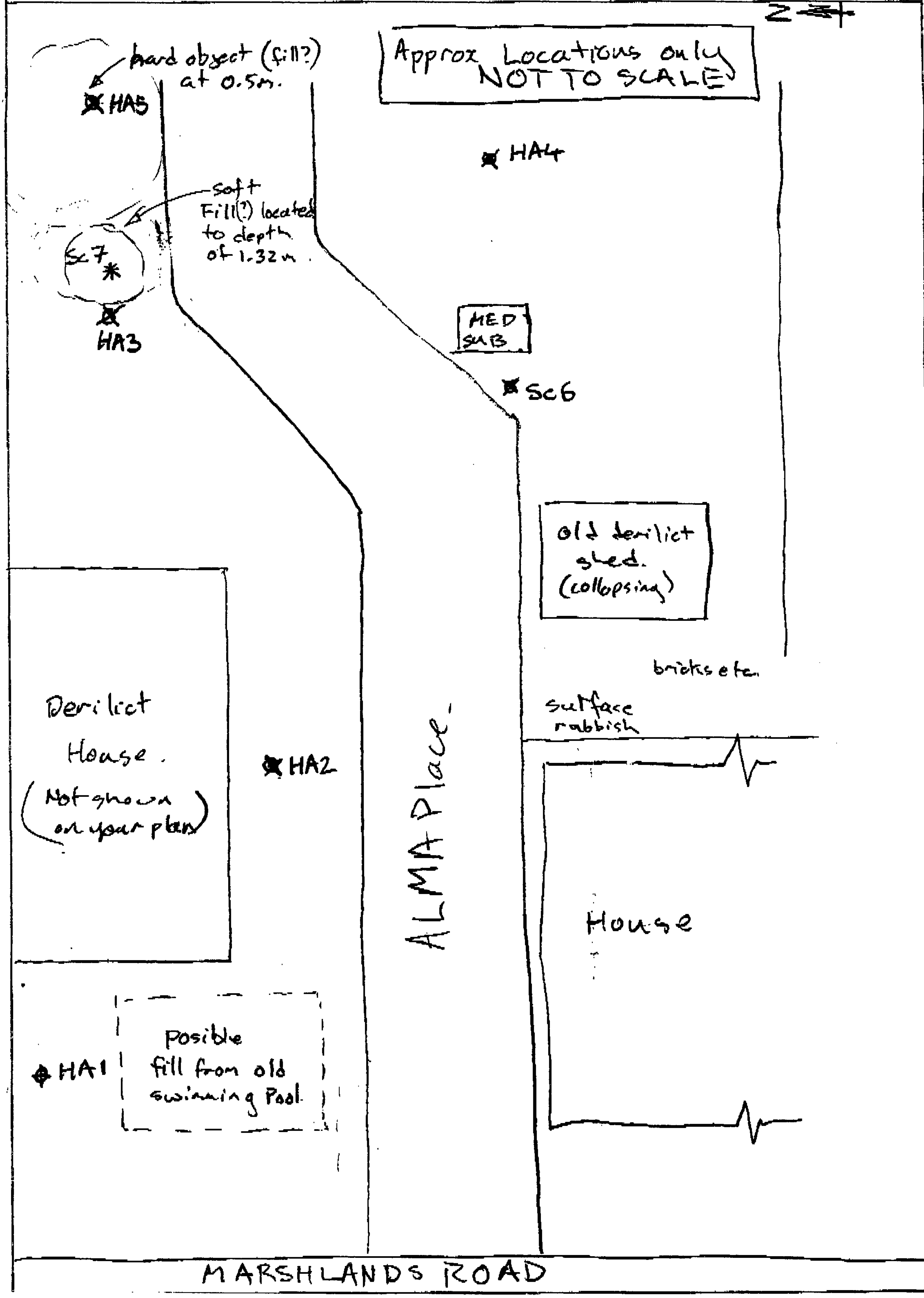
Water table is at about 1.5m below ground level.

If you have any questions please Ring.

Regards.

R Wise

SOILS & FOUNDATIONS		Job No.
		Sheet of
Computed		Date



Appendix B
References

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</td><td><table>
<tr><td>steel structure (including purlins and permanent steel weight)</td><td>0.25 kPa</td></tr>
<tr><td>Roofing</td><td>0.2 kPa</td></tr>
<tr><td>gypsum or suspended ceiling</td><td>0.13 kPa</td></tr>
<tr><td>Mechanical and electrical services</td><td>0.1 kPa</td></tr>
<tr><td>Total</td><td>0.68 kPa</td></tr>
</table></td><td>0.68 kPa</td></tr>
<tr><td>Specified compressive strength of concrete (f'_c)</td><td></td><td>20 MPa</td></tr>
<tr><td>Boundary conditions such as foundation fixity.</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>The roof lateral load is evenly distributed in the panels, according to their length.</td><td>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 each foundation pad.</td><td></td></tr>
<tr><td>Approximations made in the assessment of the capacity of each element.</td><td>Especially when considering the post-yield behaviour.</td><td></td></tr>
</table>

Appendix C
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	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 D
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 E
Standard Reporting Spread Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Shirley Library	Reviewer:	Simon Manning
	Unit / No: 36 / Street: Marshland Road	CPEng No:	
Building Address:	36 Marshland Road	Company:	Aurecon
Legal Description:		Company project number:	227255
		Company phone number:	(03) 3660821
	Degrees / Min / Sec		
GPS south:	43 / 30 / 18.51	Date of submission:	27/05/2013
GPS east:	172 / 39 / 46.97	Inspection Date:	10/09/2012
		Revision:	2
Building Unique Identifier (CCC):	PRO 2215 B001	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
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):	6.00

Building

No. of storeys above ground:	1	single storey = 1 / Ground floor elevation (Absolute) (m):	3.30
Ground floor split?	yes	Ground floor elevation above ground (m):	0.30
Storeys below ground			
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	4.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	2
Floor footprint area (approx):	1100		
Age of Building (years):		Date of design:	
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):	public		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	Cold Rolled Purlins on Steel Portal Frames
Floors:	other (note)	describe sytem	Concrete Pad Foundation
Beams:			
Columns:			
Walls:			

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	single level tilt panel		note total length of wall at ground (m):	53
Ductility assumed, μ:	1.50		wall thickness (m):	0.15
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	5		estimate or calculation?	estimated
Lateral system across:	welded and bolted steel moment frame		note typical bay length (m)	6
Ductility assumed, μ:	3.00			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	35		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	35		estimate or calculation?	estimated

Separations:

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

leave blank if not relevant

Non-structural elements

Stairs:			
Wall cladding:	precast panels	thickness and fixing type	150 thick Bolted to frames
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	light tiles		
Services(list):			

Available documentation

Architectural	full	original designer name/date	Ian Krause 1997
Structural	full	original designer name/date	Holmes Consulting Group 1997
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	
Liquefaction:	2-5 m²/100m³	notes (if applicable):	photographic records
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:	slight	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?:	yes	Describe:	Cracks at base of precast panels

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

Recommendations

	Level of repair/strengthening required:	none		Describe:	
	Building Consent required:	no		Describe:	
	Interim occupancy recommendations:	full occupancy		Describe:	
Along	Assessed %NBS before:	100%	0% %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	100%			
Across	Assessed %NBS before:	68%	0% %NBS from IEP below		
	Assessed %NBS after:	68%			

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):	0	h_n from above:	2m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		not required for this age of building	

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		
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

		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
	Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1
2.3 Hazard Scaling Factor	Hazard factor Z for site from AS1170.5, Table 3.3:		0.30
	Z_{1992}, from NZS4203:1992		0.8
	Hazard scaling factor, **Factor B:**		3.333333333
2.4 Return Period Scaling Factor	Building Importance level (from above):		2
	Return Period Scaling factor from Table 3.1, **Factor C**:		1.00
2.5 Ductility Scaling Factor	Assessed ductility (less than max in Table 3.2)	1.50	3.00
	Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.00	1.00
	Ductiity Scaling Factor, **Factor D**:	1.00	1.00
2.6 Structural Performance Scaling Factor:	Sp:	0.850	0.700
	Structural Performance Scaling Factor **Factor E**:	1.176470588	1.428571429
2.7 Baseline %NBS, (NBS%)$_b$ = (%NBS)$_{nom}$ x A x B x C x D x E	**%NBS$_b$:**	**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	significant	0.7

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

		Along	Across
3.6. Other factors, Factor F	For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
	Rationale for choice of F factor, if not 1	1	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

3.7. Overall Performance Achievement ratio (PAR)		**0.70**	**0.70**
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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