INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE OF

GALLERY APARTMENTS - 62 GLOUCESTER STREET

FOR

Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes

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Spencer Holmes Limited

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Introduction

This report has been commissioned by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes to review the performance of the building known as Gallery Apartments at 62 Gloucester Street, Christchurch, during the Canterbury earthquake sequence.

The report is based on documentation provided by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes and a limited inspection of the building on the 14th September 2011. A list of documentation provided is appended to this report in Appendix 3.

Location of Building

The building is located at 62 Gloucester Street, Christchurch, between Montreal Street and Cambridge Terrace, with Gloucester Street to the north.

The Avon River is located to the east of the building, approximately 120m away.

The location of the building in the Christchurch CBD is shown on an aerial photo of Christchurch included in Appendix 1, together with the direction from the epicentre of the main earthquakes.

Geotechnical Site Assessment

Prior to construction of the building, a geotechnical investigation was undertaken on the site by Geotech Consulting Limited. The report is dated 30th September 2005 and was lodged with the Christchurch City Council on 16th December 2005 with the building consent application for the foundations. At the time the investigation was undertaken the site was partly covered by two existing buildings and a sealed carpark. Investigations on the site were undertaken with two direct push/dual tube boreholes drilled at each end of the site with standard penetration testing (SPT) at 1m intervals. The holes were taken through gravel until looser conditions were encountered in DT2 (north end) and refusal was reached in DT1 (south end).

Cone penetration tests (CPT) were conducted through the push tube holes and along side them as well as an additional test halfway down the site. The CPT test in the base of DT1 reached 9.9 metres depth (the hole had collapsed due to the casing being withdrawn following the high SPT test).

The water table was noted in the geotechnical report as likely to be at approximately 2.7 metres depth (RL 13.0m). CPT 05 was open and dry to 3.5 metres depth, which indicated that the water table was lower than an RL of 12.8 metres at the time of investigation. Foundation designs were based on an assumed groundwater RL of 14.0 metres.

The report identified that there was a liquefaction hazard on the site based on the analysis of the CPT profiles. CPT01 indicated that there were a number of thin lenses of loose sand below the gravel that could liquefy. Due to the depth of these layers (at greater than 10m depth) the report notes that this liquefaction is insignificant for the building site. High SPT values at an 11 metre depth at the south side of the site meant that there was also potential for liquefaction, however it is noted that at these depths the effects would be small. Analysis of CPT04 indicated that there were liquefiable soils above the gravel, which was deeper at that end. The approximate thickness of liquefied sand at each CPT site is shown on table 4.3 of the geotechnical report.

The report noted the following in regard to the seismic category:

The relatively deep alluvial formations underlying this site defines this site as Class D – deep or soft soil site – in terms of the seismic design requirements of NZS 1170:2004.

The geotechnical report included recommendations for foundation design using pad and strip footings bearing directly onto the gravel, with piles the preferred options at the southern end of the site due to the liquefiable sands above the gravel. The ultimate pile capacities were provided in tables 5.1 and 5.2 of the report.

Description of Building

The Gallery Apartments building at 62 Gloucester Street is a 14-storey building that has been constructed as two towers, north and south. The lower 7 levels of the south tower incorporate car parking with the floor plate size reducing in size by approximately half above this level. Services consisting of two passenger lifts, a car lift and the main stairwell, are located at the north end of the southern tower. The two blocks have seismic separation between the southern face of the north tower and the services core, which is part of the south tower. The buildings precast concrete panels provide the exterior cladding along with a curtain wall glazing system.

The building was designed by Wilson & Hill Architects Limited and LSC Consulting, Structural and Civil Engineers. We have reviewed the engineering documentation set, a schedule of which is provided in Appendix 3.

Gravity System
The floor system is constructed using Inter-span units, pre-cast ribs with timber in-fills and a reinforced concrete topping slab. Cold drawn wire mesh has been used as reinforcing for the concrete slabs. The Inter-span floors are connected to the pre-cast concrete wall panels using a 310mm deep by 225mm wide insitu concrete element, which interconnects the Inter-span units to the pre-cast panels and inter-joins the pre-cast panels. Starters from the concrete wall panels extend into the insitu beam.

In-situ concrete columns and beams have been used at the car lift entry to provide additional support to the concrete floor slab of the south tower in this area.

The gravity loads from the structure are transferred to the building foundations using insitu reinforced concrete beams that are connected to reinforced concrete piles. Concrete piles range in diameter from 0.9m to 1.2m and have been founded to depths that vary between 4m and 7m below ground level.

Seismic System
The seismic system consists of pre-cast concrete panel shear walls in both the north-south and east-west directions, varying in thickness between 175 and 325mm. The individual panels are

typically two storeys in height, and have been detailed at horizontal joints by splicing the vertical reinforcing with grouted couplers.

The LSC Consulting Design Features Report of June 2006 states the following with regards to Seismic Design:

Seismic design is according to Part 5 of NZS 1170.

A Response Spectrum Analysis was carried out for the four positions of accidental mass eccentricities.

The building has been designed for a ductility factor μ= 3, Z=0.22, R=1, Soil Classification=D, and N (T,D) determined by fundamental period of the building on the four directions corresponding to mass eccentricities.

Fundamental Periods are tabled below:

Load Direction & Eccentricity	***North Block (Seconds)***	***South Block (Seconds)***
East-West (N ecc)	*3.96*	*2.06*
East – West (S ecc)	*3.44*	*1.96*
North – South (W ecc)	*3.65*	*2.00*
North South (E ecc)	*3.75*	*2.82*

Deflection:

Load Eccentricity & Deflection Direction	***North Block***		***South Block***	
	Serviceability (mm)	***Ultimate (mm)***	***Serviceability (mm)***	***Ultimate (mm)***
East-West (N ecc)	*115*	*345*	*136*	*408*
East – West (S ecc)	*132*	*396*	*150*	*450*
North – South (W ecc)	*122*	*366*	*122*	*366*
North South (E ecc)	*118*	*354*	*118*	*354*

Maximum drift under ultimate load is 1.07%

Secondary Elements

The main stairs are located in the services core and are pre-cast concrete elements that have been seated on a 100mm thick shelf at each level. An insitu concrete stitch has been used to connect the stairs to the concrete floor slab at each level/landing.

External balconies supported by steel SHS posts are located on the northwest corner of the north tower.

Compliance

The Christchurch City Council issued a building consent for the project in three stages:

ABA10061822	Stage 1 – Foundation / ground floor slab / subfloor services
ABA10063897	South tower inc. carparking levels
ABA10065788	North tower

The compliance documentation appears to be in order and the Christchurch City Council has issued the following Code Compliance Certificates:

ABA10061822	11 July 2007
ABA10063897	27 July 2007

The Christchurch City Council appear to have relied on producer statements PS1-design issued by the design firm in issuing the building consents for both towers. In the consent process for the south tower, the council specifically requested, *"calculations or a Producer Statement…providing verification that structural design complies with the New Zealand Building Code."* A producer statement was provided by the designer, however there are no structural calculations in the consent documentation listed in Appendix 3. The council also specifically requested a producer statement PS1-design for the connection of the inter-span ribs to the support beam, which was provider by the designer.

We have not seen a copy of a Code Compliance Certificate or the supporting documentation associated with ABA10065788.

Events Subsequent to 4th September 2010 Earthquake

There are no records of any damage sustained to the building from the 4th September 2010 and 26th December 2010 earthquakes or in any aftershocks prior to the 22nd February 2011 earthquake, but the building was severely damaged by the 22nd February 2011 earthquake.

Damage Report

The Canterbury Earthquake Royal Commission has been forwarded a post earthquake report titled *"Preliminary Damage Review, Gallery Apartments, Revision 1, May 20 2011"* by Holmes Consulting Group.

This report was prepared following a rapid structural assessment and visual inspection on 11 April 201 and primarily records the damage following the magnitude 6.3 earthquake on 22 February 2011 and aftershocks that had occurred up until the date of inspection.

Holmes Consulting Group commented on the building's condition as follows:

- *The Column to the north-west corner of the tower appears to have settled. This can be seen from the street and is observable in the suspended floors when walking around the apartments.*
- *Failure of the pre-cast panel connections on the western elevation of the northern tower. Significant shear cracking is apparent. USAR engineers have advised the*

reinforcing bars have fractured over significant lengths of these walls. External steel straps have been provided by contractors as a temporary securing measure.

- *Failure of the internal shear wall at ground floor level in the northern tower. Significant shear cracking is apparent. USAR engineers have advised the reinforcing had buckled in the end of the wall. Prior to our visit contractors have provided a steel jacket to the end of the wall to provide some containment. Steel straps are welded to the jacket and fixed through the wall to provide some shear capacity and hold the jacket in place. There is a vertical crack in the end of the wall above the steel jacket which may indicate the reinforcing steel is beginning to buckle in this area also.*
- *The level 7 beam supporting the south side tower (where the southern tower steps in) lost its support and apparently dropped approximately half a metre. This had been jacked back into place and re-supported temporarily at the time of our inspection.*
- *Uneven floor surface at the two internal shear wall locations in the northern tower. Floors are raised in these areas suggesting either settlement of the side walls or elongation of the shear walls due to yielding of reinforcing steel.*
- *Damage to the floor outside the passenger lifts and at the seismic joint between the two towers is evident. The exact extent of this can't be determined without stripping back the finishes.*
- *Shear cracking of the front wall (northern towr0 at level 6 has occurred. "Gib" plasterboard has popped at this level and allowed inspection, other levels were not inspected.*
- *Flaking of paint to SHS balcony column, possibly due to flexure (level 30.*
- *Failure of car lifcore panel to steelwork connection has occurred in a number of areas at panel to panel and panel to steelwork connections. The panels separating the car and passenger lifts have displaced horizontally relative to each other.*
- *There appears to be a lack of grout in the reid couplers at front of car lift which should be investigated further to confirm. If so, the tension capacity of these reinforcing bars cannot be relied upon.*
- *Shear cracking has occurred in the insitu concrete floor stitch beams at panel openings. Some concrete spalling of the Inter-span rib adjacent to the support connection was also apparent in one location.*
- *Movement is visible between the pre-cast stair units at the insitu concrete stitchs and at the pre-cast unit to landing connection.*

In conclusion, the building has sustained severe damage. Although it is not considered an immediate overall collapse hazard, the building is not safe to spend any significant amount of time in due to the extent of structural damage and the high possibility for unknown damage still not identified.

Concrete Testing

The Canterbury Earthquake Royal Commission has requested and obtained a post earthquake report titled *"Report 107267-1 v1.1, Materials Testing in Buildings of Interest, November 2011"* by Holmes Solutions which is attached in Appendix 6.

The report was prepared through destructive testing of concrete cores and non-destructive Schmidt Hammer testing to determine the compressive and tensile properties of the concrete. Holmes Solutions comments on the building's concrete properties were as follows:

Concrete strength results for Gallery Apartments indicated that the walls had compressive strengths of 46MPa to 56MPa, with associated tensile strengths ranges from 3.4MPa to 2.6MPa respectively.

Structural Performance

Design Standards
The Gallery Apartments building was designed to the now current building code NZS/AS 1170 and the previous concrete code NZS 3101 Part 1 1995, the current concrete design code having been issued in 2006.

A preliminary comparison of the following design standards was made;

NZS 3101:Part 1:1982
NZS 3101:Part 1:1995
NZS 3101:Part 1:2006

All three standards could be regarded as modern, describing concepts of ductility and capacity design. The primary difference between the three standards is the additional specifics included for how to detail to achieve that ductility. It is obvious that each update of the standard has been strongly based on the previous version, only with additions being made and the majority of existing clauses retained or slightly altered.

Analysis of North Block
An equivalent static analysis was undertaken by Spencer Holmes Limited on the 14-storey north tower. The seismic weight for each of the typical floor levels was calculated to be approximately 1500kN. The equivalent static actions were calculated in accordance with NZS 1170.5 with the following site parameters;

Site Subsoil Class = D (Deep or soft soil)
Hazard Factor, Z = 0.22
Near Fault Factor, N = 1.0 (assumed distance, D to fault > 20km)

The equivalent static method was used to apply the seismic loads. The seismic weight of the roof level was included as part of level 13 (the top concrete level), and 8% of the base shear was applied to level 13 (top concrete level).

Transverse Direction - Panels
The effective concrete panel cracked section properties were approximated as $0.34I_g$. The period in the transverse (east-west) direction was found to be 2.97 seconds. The foundations were assumed to be rigid for these analyses.

The distribution of direct shear and torsion into the concrete panels was determined. The transverse direction was found to be highly torsional as the main transverse panels are located on the south side (grids 3, 4 and 5). These torsion loads were found to be more critical on the grid B and F panels than the direct shear applied to them in the longitudinal direction.

The transverse panels on grids 3 and 4 were specifically analysed at ground floor level;

Height	- 39.0 metres high, 3.0 metres per level
Dimensions	- 3680mm long, 250mm thick
Concrete	- 30MPa (specified minimum compressive strength at 28 days)
Reinforcing	- RB12-440 EF vertically, HD12-400 EF horizontally, No end region confinement
Detailing	- Horizontal panel joins with grouted ducts, typically at floor levels
Axial Load	- Self-weight of panel, floor spans parallel to panel

The vertical and horizontal reinforcement were found to be approximately 75% of the minimum ratio requirements of NZS 3101.

The panels do not have enlarged end regions and do not have any additional confinement reinforcing in the end regions. We would therefore only expect nominally ductile performance from the panels. The horizontal design action coefficient for nominal ductility was calculated to be 0.069. At a ductility of 1.25 we found the flexural yield capacity of the panels to be 16%, and the shear capacity to be over 100% of that required by current code. It is important to note that the flexural capacity is sensitive to the axial load. For these analyses gravity loads from self-weight and tributary widths were taken as the axial load, and no vertical accelerations were assessed.

The design features report states the building was assessed to have a transverse period of 3.96 seconds and was designed to a ductility of 3. We have assessed the equivalent static loads under these parameters, and without accounting for p-delta effects, we found the flexural yield capacity of the transverse grid 3 and 4 walls is approximately 93% of that required. This indicates that limited ductile behaviour was assumed without accounting for p-delta effects or adequate detailing.

The transverse requirements of shear walls to achieve limited and full ductile behaviour in earthquakes is detailed in NZS 3101 cl 11.4.6. In the case of these panels, a 350mm length from each end would be required for confinement under compression loads. The maximum spacing of the transverse confinement reinforcing would be 120mm to achieve limited ductility, and 70mm to achieve full ductility.

Enlarged end regions are also typically required for ductile performance in walls. In the case of these panels, a minimum thickness of 250mm is required for limited ductile performance, and 350mm for fully ductile performance.

The equivalent static deflections were calculated in accordance with NZS 1170.5. P-delta effects were approximated using method A. P-delta effects were found to be applicable under limited ductile performance with a factor of 1.44 at every level. Under nominally ductile performance p-delta was still applicable, but the effects were substantially less, only having a factor of 1.01. The maximum inter-storey drift was calculated to be 3.2% under nominal ductility, with levels 6-13 exceeding the 2.5% limit of cl 7.5.1.

The potential plastic hinge zone of the wall was assumed to be 6.5 metres high. The maximum inter-storey deflection in this region was calculated to be 26mm, which equates to a drift of approximately 0.9%. This is acceptable for a doubly reinforced concrete walls detailed to achieve limited ductile performance, however as previously stated, we would only expect nominally ductile performance from these panels. The curvature of the panels in levels 6-13 is very low as the levels are above the potential plastic hinge zone, which is typical of concrete shear wall behaviour.

Longitudinal Direction - Panels

The effective concrete panel cracked section properties were approximated as $0.39I_g$. The period in the longitudinal (north-south) direction was found to be 3.05 seconds. The foundations were assumed to be rigid for these analyses.

The distribution of direct shear and torsion into the concrete panels was determined. The longitudinal direction was found to be less torsional than the transverse direction. This is due to the panels on grids B and F being located relatively equally either side of the centre of mass and rigidity.

A longitudinal panel on grid F was specifically analysed at ground floor level;

Height	- 39.0 metres high, 3.0 metres per level
Dimensions	- 4300mm long, 325mm thick
Concrete	- 30MPa (specified minimum compressive strength at 28 days)
Reinforcing	- RB12-460 EF vertically, HD12-400 EF horizontally, No end region confinement
Detailing	- Horizontal panel joins with grouted ducts, typically at floor levels
	- In-situ vertical panel join in centre, typical throughout height
Axial Load	- Self-weight of panel, floor tributary width

The vertical and horizontal reinforcement were found to be approximately 55% of the minimum ratio requirements of NZS 3101.

The panels do not have enlarged end regions and do not have any additional confinement reinforcing in the end regions. We would therefore only expect nominally ductile performance from the panels. The horizontal design action coefficient for nominal ductility was calculated to be 0.066. At a ductility of 1.25 we found the flexural yield capacity of the panels to be 16%, and the shear capacity to be over 100% of that required by current code. It is important to note that the flexural capacity is sensitive to the axial load. For these analyses gravity loads from self-weight and tributary widths were taken as the axial load, and no vertical accelerations were assessed.

The design features report states the building was assessed to have a longitudinal period of 3.75 seconds and was designed to a ductility of 3. We have assessed the equivalent static loads under these parameters, and without accounting for p-delta effects, we found the flexural yield capacity of the longitudinal grid F wall is approximately 79% of that required. This indicates that limited ductile behaviour was assumed without accounting for p-delta effects or adequate detailing.

The transverse requirements of shear walls to achieve limited ductile and full ductile behaviour in earthquakes is detailed in NZS 3101 cl 11.4.6. In the case of these panels, a 500mm length from each end would be required for confinement under compression loads. The maximum spacing of the transverse confinement reinforcing would be 120mm to achieve limited ductility, and 70mm to achieve full ductility.

Enlarged end regions are also typically required for ductile performance in walls. In the case of these panels, a minimum thickness of 250mm is required for limited ductility performance, and 350mm for fully ductile performance.

The equivalent static deflections were calculated in accordance with NZS 1170.5. P-delta effects were approximated using method A. P-delta effects were found to be applicable under limited ductile performance with a factor of 1.46 at every level. Under nominally ductile performance p-delta was still applicable, but the effects were substantially less, only having a factor of 1.01. The maximum inter-storey drift was calculated to be 3.4% under nominal ductility, with levels 6-13 exceeding the 2.5% limit of cl 7.5.1.

The potential plastic hinge zone of the wall was calculated to be 6.5 metres high. The maximum inter-storey deflection in this region was calculated to be 23mm, which equates to a drift of approximately 0.8%. This is acceptable for a doubly reinforced concrete walls detailed to achieve limited ductile performance, however as previously stated, we would only expect nominally ductile performance from these panels. The curvature of the panels in levels 6-13 is very low as the levels are above the potential plastic hinge zone, which is typical of concrete shear wall behaviour.

Concrete Tension Contribution
The effect of the concrete tension was analysed within the potential plastic hinge region of the grid 3 and 4 panels;

Steel:	RB12-440 EF vertically	
	A_s	$= 514mm^2/m$
	f_y	= 500MPa
	ΦN_{ts}	= 231kN/m
Concrete:	A_c	$= 250{,}000\ mm^2/m$
	f'_c	= 30MPa (specified minimum strength at 28 days)
	f_{ct}	= 1.97MPa
	ΦN_{tc}	= 419kN/m
	N_{tco}	= 650kN/m (tested tensile strength, f_{ct} = 2.6MPa)

Due to the low ratio of vertical reinforcement, the tension capacity of the concrete substantially exceeds the tension capacity of the reinforcing. Using the design compressive strength of 30MPa, the concrete would be likely to have a tension capacity approximately 180% higher than that of the reinforcing steel. The concrete testing from the Holmes Solutions report indicated compressive strengths of 46MPa to 56MPa, with associated tensile strengths ranges from 3.4MPa to 2.6MPa respectively. This is substantially higher than the design properties, and would result in the concrete having a tension capacity of 280% to 370% higher than that of the reinforcing steel. This is expected to affect the behaviour of the potential plastic hinge zone as the concrete provides initial resistance in the tension region without any ductile yielding.

For the purposes of this report it is assumed that the concrete acts in tension to its capacity, at which point it forms a single major crack in the PPHZ equivalent to that of multiple splayed small cracks that would typically be expected in ductile shear wall design. The curvature of the wall was approximated with 50mm lateral displacement over the 6.5 metre height of the PPHZ. Assuming a single crack forms at the mid-height of the PPHZ, the vertical width of a single crack required to achieve the 50mm lateral deflection is 22.8mm. The induced axial tension in the reinforcement over the width of the crack was then calculated using Hooke's Law. The induced axial load in the bars within 500mm of the panel end (4-RB12) exceeds their ultimate tension strength and could result in a brittle failure. The induced axial load in the remaining bars in the tension region is at or below their calculated yield strength.

Pre-cast Floor Support

The inter-span ribs are supported eccentrically off the face of the panels with an insitu concrete edge beam. On the drawings, the ribs appear to be fixed to the insitu concrete beam with reinforcing that has not been annotated. The damage found on-site indicates that this reinforcing is in fact the pre-stressing strands of the inter-span ribs. The edge beam is typically 310mm deep by 225mm wide, reinforced with 4-HD12 bars and HR10 stirrups at 450mm centres. The beam is tied to the panels with hooked R10 starters at 450mm centres.

The performance of the connection of the edge beam to the panel was investigated under the combined effects of gravity load, rib eccentricity, and transfer of seismic shear into the panel via diaphragm action. The connection was found to be sufficient under ultimate limit state gravity only loads, however, was found to be approximately 40% of current code requirements when gravity loads are combined with seismic shear loads. This was primarily due to the R10 starters at the top of the beam being unable to resist the tension load from the eccentricity combined with the seismic shear.

The performance of the edge beam is likely to be further compromised under seismic loads due to the inter-storey drifts of the building. The edge beams experience high curvature, and hence material strains, due to them being fixed directly along the panels and free between the panels over the openings. It appears from the drawings that the designer has attempted to reduce the curvature demands in some locations by specifying un-bonded sections to the panel in order to lengthen the free edge beam. This condition is especially critical in levels 6-13 where the drifts range from 2.5-3.4%. Our calculations indicate that the edge beams would experience approximately 3 times their maximum allowable curvature under these levels of drift, and therefore an excessive amount of spalling of the concrete can be expected to occur. We estimate that the maximum allowable inter-storey drift to restrict the curvature of the edge beams within acceptable levels is 1.0%.

It is important to note that the edge beams are not detailed with reinforcement that would be typically expected of a ductile or limited ductile concrete beam. No calculations have been done on the bending or shear demand in the edge beams, however, we would expect stirrup spacings to be in the order of 60-70mm if hinging was expected in a beam of this dimension.

Foundations

The bending and shear capacity of the foundation beams under the grid 3 and 4 concrete panels was calculated. The foundation beams are detailed as 1500mm deep by 1000mm wide, reinforced with 6-RB32 bars top and bottom and 2 sets of HR16 stirrups at 300mm and 600mm centres. The beams were assessed under nominally ductile loads applied with a meshed equivalent static model of the concrete panel walls above. The bending strength of the beams was calculated to be sufficient however shear strength was calculated at 90% where stirrups are at 300mm centres, and 50% where stirrups are at 600mm centres.

A preliminary analysis of the building displacements was done to determine the sensitivity of the structure to vertical movement in the foundations. The equivalent static model was provided with vertical spring reactions at the pile supports. The spring stiffness was limited to achieve a maximum vertical pile load of 2680kN (from geotechnical report table 5.1 for the more conservative south end values) under gravity and seismic loads. The lateral displacement at the top of the structure was found to have increased by 58% due to the base rotation caused by vertical movements of the piles. It is important to note that this calculation is extremely dependent on the spring stiffness and is only intended as an indicator of sensitivity.

Issues Arising from Review

A review of the documentation supplied by the Canterbury Earthquake Royal Commission identifies that the Gallery Apartments at 62 Gloucester Street is a 14-storey reinforced concrete building designed in 2005. The building was completed midway through 2007.

The post-earthquake report provided was prepared by Holmes Consulting Group, and reports on the damage following the 22nd February 2011 earthquake. The report concludes that the building is not safe to spend any significant amount of time in due to the severe structural damage it has sustained.

A seismic analysis was undertaken by Spencer Holmes Limited on the 14-storey north tower, with a view to assessing the specific aspects of damage observed following the 22nd February 2011 earthquake, and how these aspects are addressed in current design standards.

1. Compliance Review

The consent documentation has been provided for the building and the compliance requirements appear to have been approved by the Christchurch City Council. In processing the building consents for both towers, the Christchurch City Council has relied on producer statements PS1-design issued by the designer.

- In the interests of public safety and good industry practice, acceptance of a Producer Statement-PS1 Design as the sole criteria for establishing compliance may not provide an adequate basis for the issuing of a building consent.

- Building Consent Authorities must be satisfied on reasonable grounds that a design is compliant. It is suggested that guidelines for Building Consent Authorities to determine the appropriate level of structural review and audit be reviewed.

2. Foundation Movement

A reasonably comprehensive site investigation was undertaken prior to commencement of construction. The building suffered foundation settlement at the column at the northwest corner of the north tower, possibly due to the loss of skin friction through liquefaction of the supporting soils.

The investigation identified the presence of liquefiable material below the gravel layer on which the piles were founded. It is possible that the differential settlement arose from the liquefaction of these layers below the gravel layers. Alternatively, the settlement may be due to the loss of skin friction arising from liquefaction.

- In common pile types, skin friction will develop progressively as construction progresses.

- Pile design in liquefiable ground should not include any allowance for skin friction, and piles should be founded on suitable strata for end bearing. On sites prone to liquefaction, foundation design should allow for the loss of skin friction in the eventuality of liquefaction in significant earthquakes.

- The loss of skin friction due to liquefaction may cause additional displacements on a structure, which would require careful design consideration.

- The structural displacements of a shear wall building were found to be highly sensitive to vertical deformations in the foundations.

- The capacity reduction factor of 0.8 for foundations under earthquake should be reviewed.

3. Concrete Shear Wall Detailing

The building was designed using seismic loads that require limited ductile performance. Our review of the structural drawings indicates that the detailing of the structure is only adequate to achieve nominally ductile performance.

- The previous and current concrete design standards adequately provide classification of structures in terms of ductility, and the detailing requirements corresponding to these levels of ductility.

The current standard NZS 3101:2006 clause 11.4.3 states "*potential plastic hinge regions in walls shall be taken as the length of the wall L_w or one-sixth of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not exceed 2 L_w*".

- For thin walls subject to inelastic deformations, further consideration needs to be given to the resilience of the concrete core of the wall, and in particular the ability to confine an adequate end region to resist compression loads following spalling under repeated inelastic deformations. This requires detailing to maintain bond transfer to horizontal steel as well as buckling of vertical steel.

4. Concrete Material Design Properties

The test results of concrete compressive and tensile strengths were substantially higher than the specified design properties. Where the concrete tension strength exceeds the strength of the reinforcing provided in the end regions of shear walls, the initial seismic resistance is likely to be provided by the concrete without the desired occurrence of ductile yielding, followed by excessive demand on the reinforcement at the first crack forming in the plastic hinge zone.

- It is suggested that the current design requirements for reinforcement in the end regions of shear walls relative to the potential for concrete tension overstrength should be reviewed.

5. Detailing of Other Elements for Seismic Deformations

The building was assessed to have an inter-storey drift of up to 3.4%, exceeding the current design standard's specified maximum drift of 2.5%. Severe damage was found to have occurred in elements that are critical to the support of the structure, but not intended to be part of the seismic load-resisting system.

- It is suggested that current design requirements for detailing the 'deformation capacity' of non-seismic load resisting component parts of a structure under seismic loads should be reviewed.

NZS 3101:2006 clause 2.6.1.1 states *"the structure and its component parts shall be designed to have adequate ductility at the ultimate limit state for load combinations including earthquake actions"*.

- Structural elements required to carry gravity loads that are not intended to form part of the seismic resisting system should be detailed to accept the deformations imposed as a result of the displacements of the seismic resisting system under the ultimate limit state. The provision of deformation capacity must be inclusive of an adequate margin over the expected elastic and inelastic deformations of the primary seismic resisting elements, and the distortions of any other building component attached to the element under consideration.

- There is a need for an adequate margin beyond the provision of seismic movement under the ultimate limit state event around elements subject to brittle failure.

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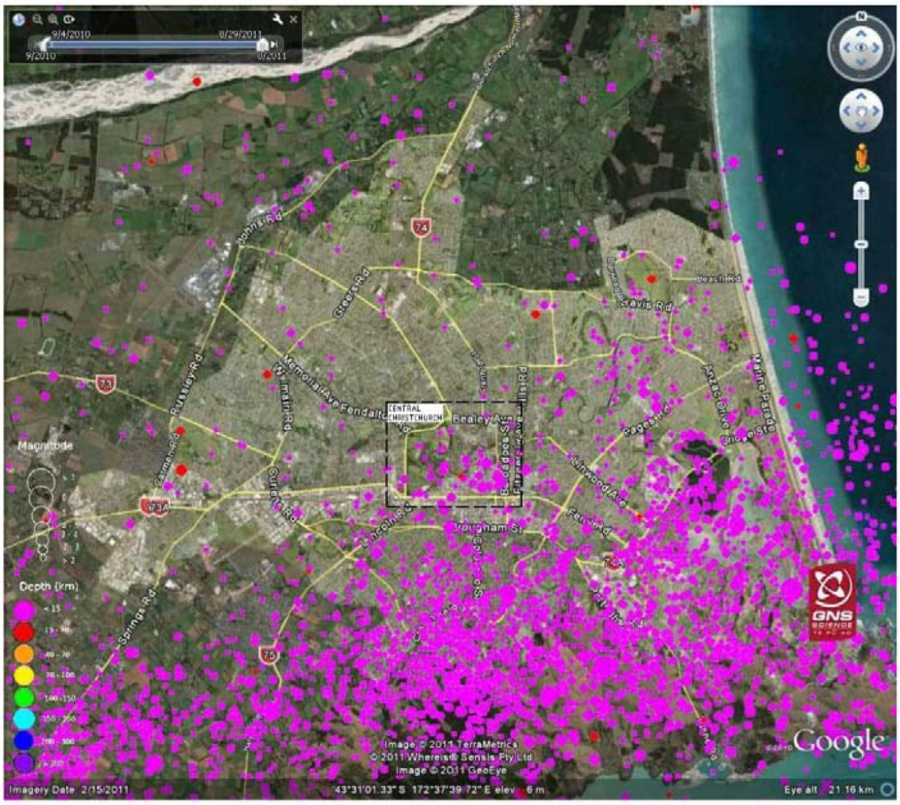
APPENDIX 1

Site Plan




NORTH
Gloucester St
Mw7.1 - 4th SEP 2010
No.62
Mw6.2 - 22nd FEB 2011

APPENDIX 2

Specific photographs of damage following 22nd February and 13th June 2011 earthquakes

Exposed pre-stressing strands from inter-span rib into damaged insitu concrete edge beam

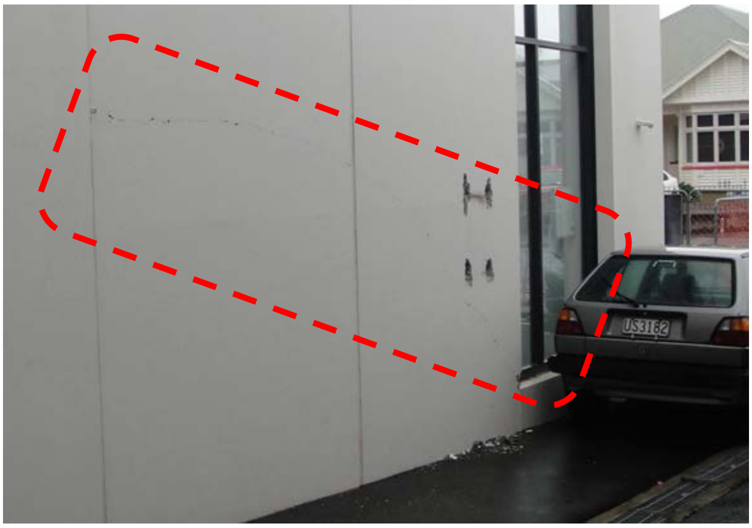
Severe spalling of insitu concrete edge beam along ‘un-bonded’ section adjacent to panels

Severe spalling of insitu concrete edge beam along ‘un-bonded’ section adjacent to panels

Spalling of concrete around vertical reinforcement couplers at end region of panel.

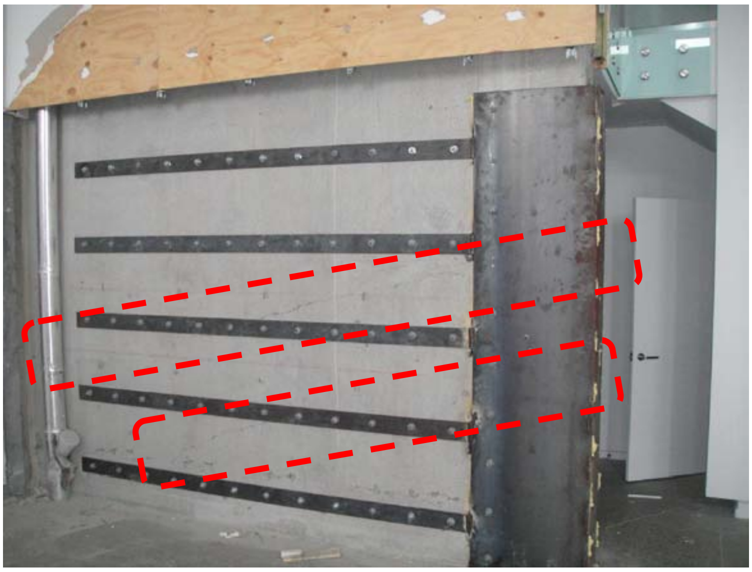
Spalling of concrete around vertical reinforcement couplers at end region of panel.

Inter-span rib parallel to, and directly in line with, east-west concrete panels on grids 3 and 4.



Cracking to grid F external panel on east side.

Cracking to grid F external panel on east side.



Remedial works for damaged panel to provide shear strength and confinement to end region.

APPENDIX 3

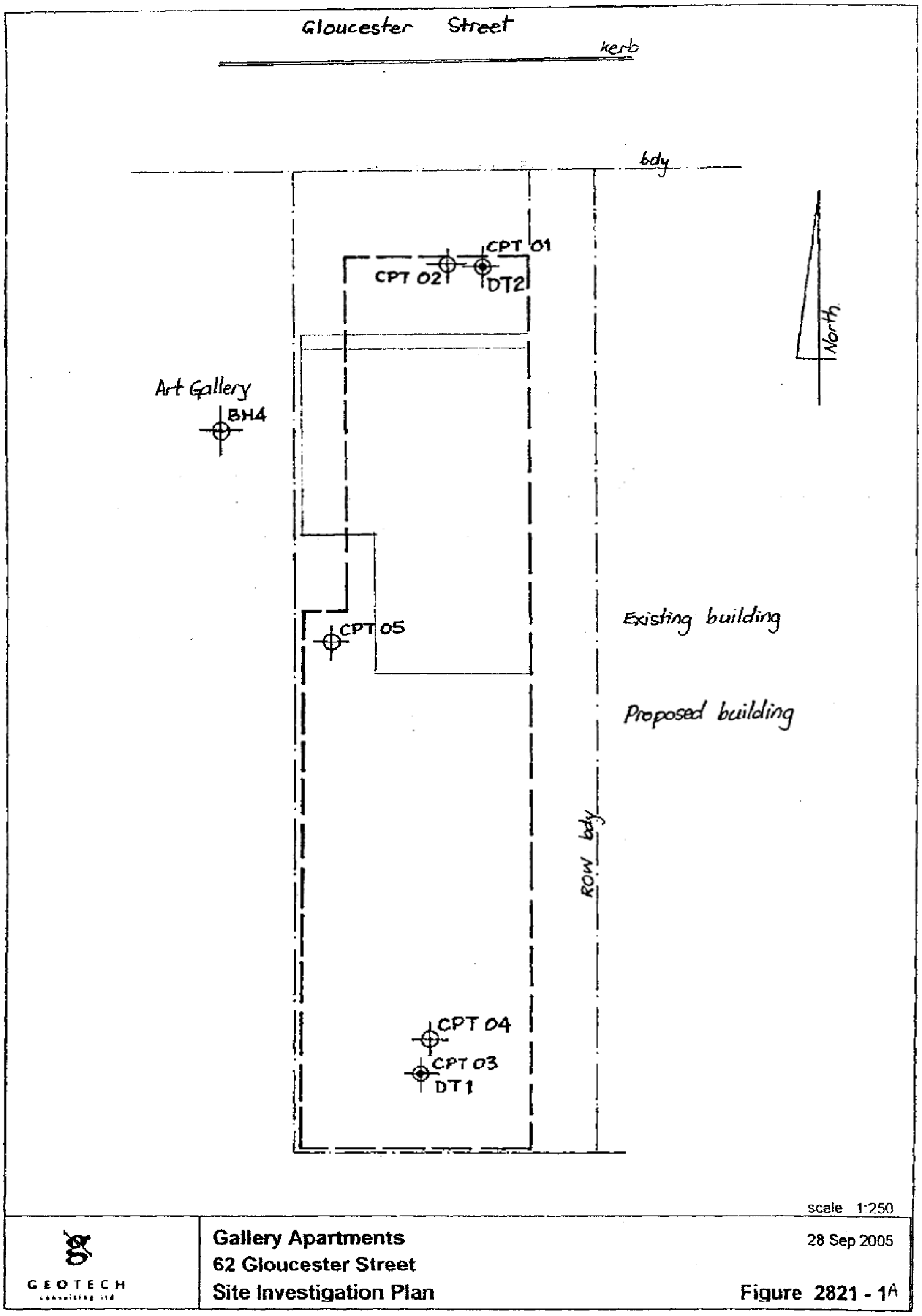
List of documentation for report

The following documents (electronic file names listed) have been reviewed as part of this report:

- ABA10061822 - Stage 1 Foundation-Floor Slab - Consent & Specifications
- ABA10061822 - Stage 1 Foundation-Floor Slab – Plans
- ABA10063897 - South Tower Structure - Consent & Amended Plans
- ABA10063897 - South Tower Structure - Consent & Specifications
- ABA10063897 - South Tower Structure – Plans
- ABA10065788 - North Tower Structure & Envelope – Plans
- ABA10065788 - North Tower Structure & Exterior Envelope - Consent & Specifications
- ABA10073131 - Inspection Photos (during construction)
- EQ SBP 24 - Demolition Appeal Notification - 27-5-2011
- EQ SBP 24 - Updated Engineers Report - 20-5-2011
- Gloucester62.0001 (EQC note 6/05/2011)
- 107261-1(v1.1) – Materials Testing in Buildings of Interest, Holmes Solutions, November 2011

APPENDIX 4

Site Investigation Plan


Gloucester Street
kerb
bdy
CPT 01
CPT 02
DT2
North
Art Gallery
BH4
CPT 05
Existing building
Proposed building
ROW bdy
CPT 04
CPT 03
DT1
scale 1:250
GEOTECH
Gallery Apartments
62 Gloucester Street
Site Investigation Plan
28 Sep 2005
Figure 2821 - 1A

APPENDIX 5

Structural Calculations

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Page C01
By VME
Date NOV 11
Job Ref E110604

Project 62 GLOUCESTER ST

Description SUMMARY

GENERAL

Traverse Period = 2.97s

Longitudinal Period = 3.05s

14 Storeys

PPHZ of Walls 39m/6 = 6.5m

TRANSVERSE – GRID 3/4 WALLS (250mm)

→ Heavily dependent on axial load
→ Vertical – Insufficient bar area 75%
→ Horizontal – Insufficient bar area 82%
→ At $\mu = 1.25$
$\varnothing M_u$ = 2176 kNm < 13190 kNm 16%
At $\mu = 3.0$ < 5963 kNm 36%

→ Max Wall Curvature OK – 0.71% Drift (21mm)
→ Max Storey drift ≈ 4.0% Drift (120mm)
→ At $\mu = 1.25$
$\varnothing V_n$ = 1201 kN > 681 kN 100% +
At O/S (μ=3.0)
$\varnothing V_n$ = 1602 kN > 509 kN 100% +

LONGITUDINAL – GRID F WALL (325mm GFL-2FL)

→ Heavily dependent on axial load
→ Vertical – Insufficient bar area 55%
→ Horizontal – Insufficient bar area 63%
→ At $\mu = 1.25$
$\varnothing M_u$ = 4219 kNm < 25911 kNm 16%
At $\mu = 3.0$ < 11851 kNm 36%

→ Max wall curvature OK – 0.62% Drift (18mm)
→ Max Story drift – 4.0% Drift (120mm)
→ At $\mu = 1.25$
$\varnothing V_n$ = 1687 kN > 1353 kN 100% +
At O/S (μ=3.0)
$\varnothing V_n$ = 2249 kN > 981 kN 100% +

DETAILING

→ NO CONFINEMENT IN END REGIONS OF PANELS FOR LIMITED DUCTILE & DUCTILE PERFORMANCE

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Page	C02
By	VME
Date	15/11/2011
Job Ref	E110604

Project 62 GLOUCESTER ST, CHRISTCHURCH

Description SEISMIC WEIGHTS

LEVEL 14 (ROOF)							
Dead Loads							
Lightweight Roof	0.5 kPa	x	152 m²	=	76 kN		
Lightweight Walls	0.5 kPa	x	79 m²	=	39 kN	ΣG =	115 kN
Live Loads							
Roof	0.25 kPa	x	152 m²	=	38 kN	ΣQ =	38 kN
LEVEL 13							
Dead Loads							
Super-Imposed Dead Load	0.5 kPa	x	152 m²	=	76 kN		
110mm Concrete Topping Slab	2.6 kPa	x	152 m²	=	393 kN		
175 Interspan Units	1.3 kPa	x	152 m²	=	197 kN		
Lightweight Walls	0.5 kPa	x	79 m²	=	39 kN		
175 Precast Panels	4.1 kPa	x	88 m²	=	362 kN	ΣG =	1067 kN
Live Loads							
Floor	1.5 kPa	x	152 m²	=	228 kN	ΣQ =	228 kN
LEVEL 2 to 12							
Dead Loads							
Super-Imposed Dead Load	0.5 kPa	x	152 m²	=	76 kN		
110mm Concrete Topping Slab	2.6 kPa	x	152 m²	=	393 kN		
175 Interspan Units	1.3 kPa	x	152 m²	=	197 kN		
250 Precast Panels	5.9 kPa	x	127 m²	=	747 kN	ΣG =	1413 kN
Live Loads							
Floor	1.5 kPa	x	152 m²	=	228 kN	ΣQ =	228 kN
LEVEL 1							
Dead Loads							
Super-Imposed Dead Load	0.5 kPa	x	47 m²	=	23 kN		
110mm Concrete Topping Slab	2.6 kPa	x	47 m²	=	120 kN		
175 Interspan Units	1.3 kPa	x	47 m²	=	60 kN		
325 Precast Panels	7.6 kPa	x	127 m²	=	971 kN	ΣG =	1175 kN
Live Loads							
Floor	1.5 kPa	x	47 m²	=	70 kN	ΣQ =	70 kN
GRAVITY DEAD LOAD						ΣG =	17902 kN
GRAVITY LIVE LOAD						ΣQ =	2841 kN
LEVEL G (GROUND FLOOR)							
Dead Loads							
325 Precast Panels	7.6 kPa	x	64 m²	=	486 kN	ΣG =	486 kN
Live Loads							
Floor	1.5 kPa	x	0 m²	=	0 kN	ΣQ =	0 kN
TOTAL GRAVITY DEAD LOAD						ΣG =	18388 kN
TOTAL GRAVITY LIVE LOAD						ΣQ =	2841 kN

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Page	C03
By	VME
Date	15/11/2011
Job Ref	E110604

Project 62 GLOUCESTER ST, CHRISTCHURCH

Description EQUIVALENT STATIC ACTIONS - NZS 1170.5:2004

GENERAL STRUCTURE PROPERTIES

			ULS		SLS
Table 0.3.2	Importance Level for Building Types	=	2	=	2
	Design Working Life	=	50 yrs	=	50 yrs
Table 0.3.3	Annual Probability of Exceedance	=	1/500	=	1/25

5.3.1 ELASTIC SITE SPECTRA FOR HORIZONTAL LOADING

			ULS		SLS
	Period	~	1.20 s	~	1.20 s
5.3.1.3	Site Subsoil Class	=	D	=	D
	Table 3.1 - Note 1:	Use Figure:	3.1 (General)		
Table 5.3.1	Spectral Shape Factor - General	$C_h(T)$ =	1.69	=	1.69
Table 5.3.3	Hazard Factor	Z =	0.22	=	0.22
Table 5.3.3	Distance to Fault	D =	20 km	=	20 km
Table 5.3.5	Return Period Factor	R =	1	=	0.25
		ZR =	0.22		
Table 5.3.7	Maximum Near Fault Factor	$N_{max}(T)$ =	1.00	=	1.000
5.3.1.6.2	Near Fault Factor	N (T, D) =	1.00	=	1.000
5.3.1.1	Elastic Site Spectra	$C = C_h(T)\ Z\ R\ N(T,D)$ =	0.37	=	0.09

5.5.2 HORIZONTAL DESIGN ACTION COEFFICIENT

			ULS		SLS
5.4.3	Structural Ductility Factor	μ =	1.25	=	1.0
5.4.4	Structural Performance Factor	S_p =	0.93	=	0.70
5.5.2.1.1		k_μ =	1.25	=	1.00
5.5.2(1)	Design Action Coefficient				
		$\mathbf{C_d(T_1) = C(T_1)\ S_p / k_\mu}$ =	**0.27**	=	**0.06**
		$(Z/20 + 0.02)\ R_u$ =	0.03		
		$0.03\ R_u$ =	0.03		

DIRECTION (X) HORIZONTAL DESIGN ACTION COEFFICIENT (ULS)

		$N_{max}(T)$	N (T, D)	S_p	T_1	$C_h(T)$	C	k_μ	$C_d(T_1)$	100%
μ =	**4.0**	1.000	1.000	0.70	*2.97s*	*0.72*	0.16	4.00	**0.028**	0.028
	3.0	1.000	1.000	0.70	*2.97s*	*0.72*	0.16	3.00	**0.037**	0.037
	2.5	1.000	1.000	0.70	*2.97s*	*0.72*	0.16	2.50	**0.044**	0.044
	2.0	1.000	1.000	0.70	*2.97s*	*0.72*	0.16	2.00	**0.055**	0.055
	1.25	1.000	1.000	0.93	*2.97s*	*0.72*	0.16	1.25	**0.117**	0.117
	1.0	1.000	1.000	1.00	*2.97s*	*0.72*	0.16	1.00	**0.159**	0.159

DIRECTION (X) HORIZONTAL DESIGN ACTION COEFFICIENT (SLS1)

		$N_{max}(T)$	N (T, D)	S_p	T_1	$C_h(T)$	C	k_μ	$C_d(T_1)$
μ =	**1.25**	1.000	1.000	0.70	*2.97s*	*0.72*	0.04	1.25	**0.022**
	1.0	1.000	1.000	0.70	*2.97s*	*0.72*	0.04	1.00	**0.028**

DIRECTION (Y) HORIZONTAL DESIGN ACTION COEFFICIENT (ULS)

		$N_{max}(T)$	N (T, D)	S_p	T_1	$C_h(T)$	C	k_μ	$C_d(T_1)$	100%
μ =	**4.0**	1.000	1.000	0.70	*3.05s*	*0.69*	0.15	4.00	**0.027**	0.027
	3.0	1.000	1.000	0.70	*3.05s*	*0.69*	0.15	3.00	**0.035**	0.035
	2.5	1.000	1.000	0.70	*3.05s*	*0.69*	0.15	2.50	**0.043**	0.043
	2.0	1.000	1.000	0.70	*3.05s*	*0.69*	0.15	2.00	**0.053**	0.053
	1.25	1.000	1.000	0.93	*3.05s*	*0.69*	0.15	1.25	**0.112**	0.112
	1.0	1.000	1.000	1.00	*3.05s*	*0.69*	0.15	1.00	**0.152**	0.152

DIRECTION (Y) HORIZONTAL DESIGN ACTION COEFFICIENT (SLS1)

		$N_{max}(T)$	N (T, D)	S_p	T_1	$C_h(T)$	C	k_μ	$C_d(T_1)$
μ =	**1.25**	1.000	1.000	0.70	*3.05s*	*0.69*	0.04	1.25	**0.021**
	1.0	1.000	1.000	0.70	*3.05s*	*0.69*	0.04	1.00	**0.027**

C004

WALL LOAD DISTRIBUTION

SPENCER HOLMES LIMITED

Input:

Building Dimensions	Lx Total	9.3 m	Ly Total	17.0 m
Equiv. Static Force (at μ = 1.0)	Fx	1000 kN	Fy	1000 kN
Centre of Mass, CoM	x	4.7 m	y	8.5 m

Job Info:

Location:	62 Gloucester St, Christchurch
SHL Job:	E110604
Date:	November 2011

Output:

Centre of Rigidity, CoR	x	7.0 m	y	3.0 m	
Relative ordinates	x'	2.4 m	y'	-5.5 m	
Eccentricity Offset	+0.1b	10.2 m	-0.1b	6.8 m	y ordinate
X-Direction Torsion (kNm)		7234 kNm		3834 kNm	
Eccentricity Offset	+0.1b	5.8 m	-0.1b	3.7 m	x ordinate
Y-Direction Torsion (kNm)		-1464 kNm		-3324 kNm	

Wall	t	Lx	Ly	S_x	S_y	Global ordinates x	y	Relative ordinates about CoM x'	y'	$S_y x'$	$S_x y'$	Relative ordinates about CoR x'	y'	$S_y x'^2$	$S_x y'^2$	Shear Distribution Direct kx	ky	Torsion ktx	kty
#	(m)	(m)	(m)			(m)	(m)	(m)	(m)			(m)	(m)					(1/m)	(1/m)
B1	0.25	0.0	1.5	0.0	0.1	0.0	2.8	-4.7	-5.9	-0.7		-7.0	-0.4	7.0		0.00	0.05	0.00	-0.02
B2	0.25	0.0	2.0	0.0	0.3	0.0	5.2	-4.7	-3.3	-1.2		-7.0	2.2	12.4		0.00	0.09	0.00	-0.04
B3	0.25	0.0	2.0	0.0	0.3	0.0	8.7	-4.7	0.2	-1.2		-7.0	5.7	12.4		0.00	0.09	0.00	-0.04
F1	0.25	0.0	2.6	0.0	0.4	9.3	1.8	4.7	-6.7	2.0		2.3	-1.2	2.1		0.00	0.16	0.00	0.02
F2	0.25	0.0	2.6	0.0	0.4	9.3	5.6	4.7	-2.7	2.0		2.3	2.8	2.1		0.00	0.16	0.00	0.02
F3	0.25	0.0	4.3	0.0	1.2	9.3	9.2	4.7	0.7	5.4		2.3	6.2	5.9		0.00	0.44	0.00	0.06
1A	0.25	1.5	0.0	0.1	0.0	5.0	17.0	0.4	8.5		1.2	-2.0	14.0		27.7	0.04	0.00	0.03	0.00
3A	0.25	3.7	0.0	0.9	0.0	5.6	7.8	0.9	-0.7		-0.6	-1.4	4.8		20.0	0.21	0.00	0.06	0.00
4A	0.25	3.7	0.0	0.9	0.0	5.6	3.3	0.9	-5.2		-4.4	-1.4	0.3		0.1	0.21	0.00	0.00	0.00
5A	0.25	4.3	0.0	1.2	0.0	0.0	0.0	-4.7	-8.5		-9.8	-7.0	-3.0		10.2	0.29	0.00	-0.05	0.00
5D	0.25	4.0	0.0	1.0	0.0	5.3	0.0	0.6	-8.5		-8.5	-1.7	-3.0		6.8	0.25	0.00	-0.04	0.00
				4.01	2.64					6.3	-22.2			42.0	66.7	1.00	1.00	0.00	0.00

Wall	X Force Distribution Shear x Dir	Torsion +0.1b	-0.1b	CHECK TOTAL TORSION		X+0.1b	X-0.1b	MAX Shear
#	kN	kN	kN			kN	kN	kN
B1	0	-171	-90	1233	637	-171	-90	171
B2	0	-304	-161	2138	1133	-304	-161	304
B3	0	-304	-161	2138	1133	-304	-161	304
F1	0	164	87	371	196	164	87	164
F2	0	164	87	371	196	164	87	164
F3	0	449	238	1013	537	449	238	449
1A	35	0	0	0	0	35	35	35
3A	214	0	0	0	0	214	214	214
4A	214	0	0	0	0	214	214	214
5A	288	0	0	0	0	288	288	288
5D	250	0	0	0	0	250	250	250
	1000	0.0	0.0	7234	3834			

Wall	Y Force Distribution Shear y Dir	Torsion +0.1b	-0.1b	CHECK TOTAL TORSION		X+0.1b	X-0.1b	MAX Shear
#	kN	kN	kN			kN	kN	kN
B1	53	0	0	0	0	53	53	53
B2	95	0	0	0	0	95	95	95
B3	95	0	0	0	0	95	95	95
F1	160	0	0	0	0	160	160	160
F2	160	0	0	0	0	160	160	160
F3	438	0	0	0	0	438	438	438
1A	0	-43	-98	-608	-1379	-43	-98	98
3A	0	-91	-206	-439	-995	-91	-206	206
4A	0	-6	-14	-2	-5	-6	-14	14
5A	0	75	171	-223	-505	75	171	171
5D	0	65	148	-193	-438	65	148	148
	1000	0.0	0.0	-1464	-3324			

BUI.GLO62.0003.29

C05

PERIOD CHECK FROM IEP METHOD

Concrete Shear Walls

$$T = 0.09 h_n^{0.75} / A_c^{0.5}$$

Where h_n = height in m from the base of the structure to the uppermost seismic weight or mass.

$A_c = \Sigma A_i(0.2 + L_{wi}/h_n)^2$ Note: A_c shall not be taken as less than 1

A_i = cross-sectional shear area of shear wall i in the first storey of the building, in m^2

l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m with the restriction that l_{wi} / h_n shall not exceed 0.9

Concrete Shear Wall in Longitudinal Direction

T = 2.65 s

A_c = 0.280 m^2

h_n = 39.00 m

	L_{wi}	Thickness	L_{wi}/h_n	A_i	$A_i(0.2 + L_{wi}/h_n)^2$
1	1.5	0.25	0.04	0.38	0.021
2	2	0.25	0.05	0.50	0.032
3	2	0.25	0.05	0.50	0.032
4	2.6	0.25	0.07	0.65	0.046
5	2.6	0.25	0.07	0.65	0.046
6	4.3	0.25	0.11	1.08	0.103
7			0.00	0.00	0.000
8			0.00	0.00	0.000
9			0.00	0.00	0.000
10			0.00	0.00	0.000
11			0.00	0.00	0.000
12			0.00	0.00	0.000
13			0.00	0.00	0.000
14			0.00	0.00	0.000
15			0.00	0.00	0.000
16			0.00	0.00	0.000
17			0.00	0.00	0.000
18			0.00	0.00	0.000
19			0.00	0.00	0.000

Concrete Shear Wall in Transverse Directic

T = 2.29

A_c = 0.38

h_n = 39.00

	L_{wi}	Thickness	L_{wi}/h_n	A_i
1	1.5	0.25	0.04	0.38
2	3.7	0.25	0.09	0.93
3	3.7	0.25	0.09	0.93
4	4.3	0.25	0.11	1.08
5	4	0.25	0.10	1.00
6			0.00	0.00
7			0.00	0.00
8			0.00	0.00
9			0.00	0.00
10			0.00	0.00
11			0.00	0.00
12			0.00	0.00
13			0.00	0.00
14			0.00	0.00
15			0.00	0.00
16			0.00	0.00
17			0.00	0.00
18			0.00	0.00
19			0.00	0.00

EQUIVALENT STATIC FORCES - TRANSVERSE DIRECTION (X)

6.2.1.2 Horizontal Seismic Shear $V = C_d(T_1)\,W_t$ where $W_t = \Sigma(G \,\&\, \psi_a\psi_E Q) =$ 18754 kN

μ	T_1	$C_d(T_1)$	V
4.0	2.97s	0.02	307 kN
3.0	2.97s	0.02	410 kN
2.5	2.97s	0.03	492 kN
2.0	2.97s	0.03	615 kN
1.25	2.97s	0.07	1300 kN
1.0	2.97s	0.09	1757 kN

6.2.1.3 Equivalent Static Horizontal Force at Each Level

$$F_i = F_t + 0.92\,V * \frac{W_i h_i}{\Sigma(W_i h_i)}$$

$F_t = 0.08\,V$ at level 13 (top full concrete level)

Level	h_i	W_i	$W_i h_i$	Equivalent Static Force, F_i μ 4.0	μ 3.0	μ 2.5	μ 2.0	μ 1.25	μ 1.0
Level 13	39.00 m	1262 kN	49223kNm	60 kN	80 kN	96 kN	120 kN	253 kN	342 kN
Level 12	36.00 m	1481 kN	53332kNm	38 kN	51 kN	61 kN	76 kN	161 kN	218 kN
Level 11	33.00 m	1481 kN	48888kNm	35 kN	47 kN	56 kN	70 kN	148 kN	200 kN
Level 10	30.00 m	1481 kN	44443kNm	32 kN	42 kN	51 kN	64 kN	135 kN	182 kN
Level 9	27.00 m	1481 kN	39999kNm	29 kN	38 kN	46 kN	57 kN	121 kN	164 kN
Level 8	24.00 m	1481 kN	35555kNm	25 kN	34 kN	41 kN	51 kN	108 kN	145 kN
Level 7	21.00 m	1481 kN	31110kNm	22 kN	30 kN	36 kN	45 kN	94 kN	127 kN
Level 6	18.00 m	1481 kN	26666kNm	19 kN	25 kN	31 kN	38 kN	81 kN	109 kN
Level 5	15.00 m	1481 kN	22222kNm	16 kN	21 kN	25 kN	32 kN	67 kN	91 kN
Level 4	12.00 m	1481 kN	17777kNm	13 kN	17 kN	20 kN	25 kN	54 kN	73 kN
Level 3	9.00 m	1481 kN	13333kNm	10 kN	13 kN	15 kN	19 kN	40 kN	55 kN
Level 2	6.00 m	1481 kN	8889kNm	6 kN	8 kN	10 kN	13 kN	27 kN	36 kN
Level 1	3.00 m	1196 kN	3589kNm	3 kN	3 kN	4 kN	5 kN	11 kN	15 kN
		$\Sigma W_i h_i =$	396025kNm	Σ = 307 kN	410 kN	492 kN	615 kN	1300 kN	1757 kN

TRANSVERSE PERIOD

4.5.2 Rayleigh Method for Determining Period of Vibration

$I_{eff} = 0.25\,I_g$

$$T_1 = 2\pi\sqrt{\frac{\Sigma(W_i u_i^2)}{g\,\Sigma(F_i u_i)}} \qquad \text{(Eq 4.5.1)}$$

		T_1 1.20 s				T_1 2.97 s			
Level	W_i	F_i	u_i	$\Sigma(W_i u_i^2)$	$\Sigma(F_i u_i)$	F_i	u_i	$\Sigma(W_i u_i^2)$	$\Sigma(F_i u_i)$
Level 13	1262 kN	1355 kN	1787 mm	4030479	2421037	581 kN	767 mm	742503	445789
Level 12	1481 kN	865 kN	1596 mm	3773548	1379637	371 kN	685 mm	695129	254064
Level 11	1481 kN	793 kN	1405 mm	2924400	1113481	340 kN	603 mm	538665	205013
Level 10	1481 kN	720 kN	1217 mm	2194144	876808	309 kN	522 mm	403669	161340
Level 9	1481 kN	648 kN	1032 mm	1577769	669169	278 kN	443 mm	290731	123230
Level 8	1481 kN	576 kN	853 mm	1077909	491646	247 kN	366 mm	198448	90499
Level 7	1481 kN	504 kN	683 mm	691075	344455	216 kN	293 mm	127180	63392
Level 6	1481 kN	432 kN	524 mm	406768	226515	185 kN	225 mm	74998	41726
Level 5	1481 kN	360 kN	380 mm	213920	136889	155 kN	163 mm	39360	25190
Level 4	1481 kN	288 kN	254 mm	95577	73199	124 kN	109 mm	17601	13476
Level 3	1481 kN	216 kN	149 mm	32889	32205	93 kN	65 mm	6259	6027
Level 2	1481 kN	144 kN	69 mm	7053	9942	62 kN	30 mm	1333	1854
Level 1	1196 kN	58 kN	18 mm	388	1047	25 kN	8 mm	77	200
					T_1 2.97 s				T_1 2.97 s

C07

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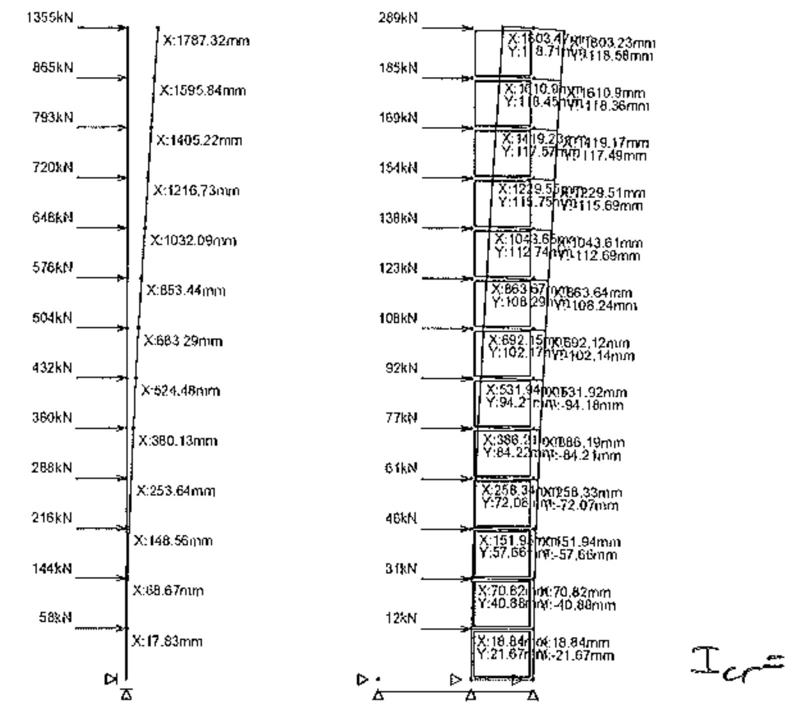
Load cases:
1 Eu (ductility 1.0, T=1.20s)



No general restraint

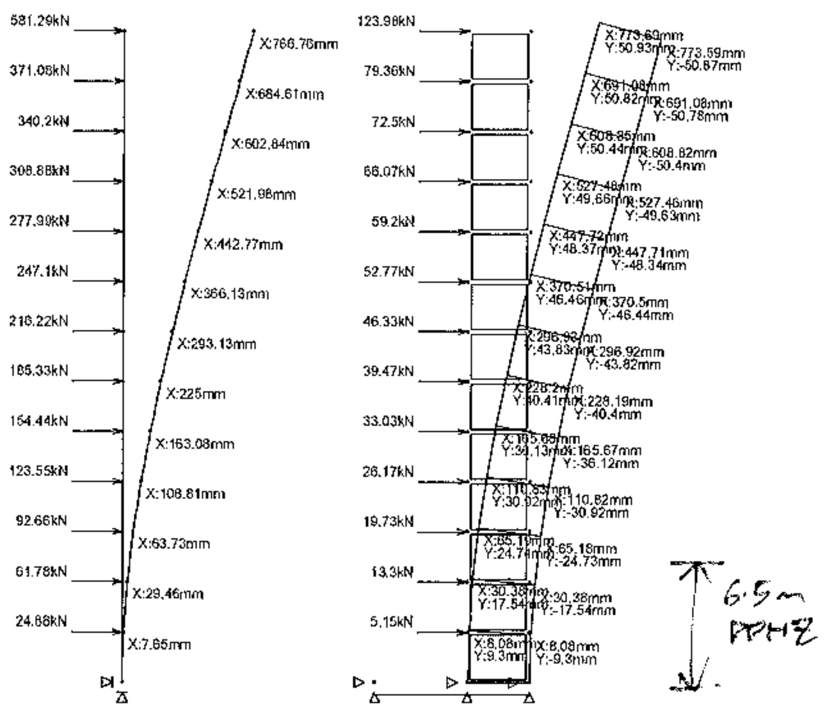
Materials:
1 25CRK30

Sections:
1 FB-1500x1000
2 SW-250 THK

Job: G:\2011jobs\110600-19\...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 1, Disp: 1, Moment: None, Shear: None, Axial: None, Torsion: None

SPACE GASS 10.85 - SPENCER HOLMES LTD 15 Nov 2011, 4:26 pm

Load cases:
2 Eu (ductility 1.0, T=2.97s)



No general restraint

Materials:
1 25CRK30

Sections:
1 FB-1500x1000
2 SW-250 THK

Job: G:\2011jobs\110600-19\...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 1, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

EQUIVALENT STATIC FORCES - LONGITUDINAL DIRECTION (Y)

6.2.1.2 Horizontal Seismic Shear $V = C_d(T_1)\,W_t$ where $W_t = \Sigma\,(G\ \&\ \psi_a\psi_E Q) =$ 18754 kN

μ	T_1	$C_d(T_1)$	V
4.0	3.05s	0.02	294 kN
3.0	3.05s	0.02	393 kN
2.5	3.05s	0.03	471 kN
2.0	3.05s	0.03	589 kN
1.25	3.05s	0.07	1245 kN
1.0	3.05s	0.09	1683 kN

6.2.1.3 Equivalent Static Horizontal Force at Each Level

$$F_i = F_t + 0.92\,V * \frac{W_i h_i}{\Sigma\,(W_i h_i)} \qquad F_t = 0.08\,V$$

at level 13 (top full concrete level)

Level	h_i	W_i	$W_i h_i$	μ 4.0	μ 3.0	μ 2.5	μ 2.0	μ 1.25	μ 1.0
				Equivalent Static Force, F_i					
Level 13	39.00 m	1262 kN	49223kNm	57 kN	76 kN	92 kN	115 kN	242 kN	328 kN
Level 12	36.00 m	1481 kN	53332kNm	37 kN	49 kN	59 kN	73 kN	155 kN	209 kN
Level 11	33.00 m	1481 kN	48888kNm	34 kN	45 kN	54 kN	67 kN	142 kN	192 kN
Level 10	30.00 m	1481 kN	44443kNm	30 kN	41 kN	49 kN	61 kN	129 kN	174 kN
Level 9	27.00 m	1481 kN	39999kNm	27 kN	37 kN	44 kN	55 kN	116 kN	157 kN
Level 8	24.00 m	1481 kN	35555kNm	24 kN	33 kN	39 kN	49 kN	103 kN	139 kN
Level 7	21.00 m	1481 kN	31110kNm	21 kN	28 kN	34 kN	43 kN	90 kN	122 kN
Level 6	18.00 m	1481 kN	26666kNm	18 kN	24 kN	29 kN	37 kN	77 kN	104 kN
Level 5	15.00 m	1481 kN	22222kNm	15 kN	20 kN	24 kN	30 kN	64 kN	87 kN
Level 4	12.00 m	1481 kN	17777kNm	12 kN	16 kN	20 kN	24 kN	52 kN	70 kN
Level 3	9.00 m	1481 kN	13333kNm	9 kN	12 kN	15 kN	18 kN	39 kN	52 kN
Level 2	6.00 m	1481 kN	8889kNm	6 kN	8 kN	10 kN	12 kN	26 kN	35 kN
Level 1	3.00 m	1196 kN	3589kNm	2 kN	3 kN	4 kN	5 kN	10 kN	14 kN
		$\Sigma\,W_i h_i =$	395025kNm	Σ = 294 kN	393 kN	471 kN	589 kN	1245 kN	1683 kN

LONGITUDINAL PERIOD

4.5.2 Rayleigh Method for Determining Period of Vibration

$I_{eff} = 0.32\,I_g$ (blockwork)

$$T_1 = 2\pi \sqrt{\frac{\Sigma(W_i u_i^2)}{g\,\Sigma(F_i u_i)}} \qquad \text{(Eq 4.5.1)}$$

		T_1 1.20 s				T_1 3.05 s			
Level	W_i	F_i	u_i	$\Sigma(W_i u_i^2)$	$\Sigma(F_i u_i)$	F_i	u_i	$\Sigma(W_i u_i^2)$	$\Sigma(F_i u_i)$
Level 13	552 kN	593 kN	1910 mm	2013751	1132630	243 kN	783 mm	338425	190371
Level 12	648 kN	378 kN	1699 mm	1870517	642222	155 kN	696 mm	313902	107866
Level 11	648 kN	347 kN	1488 mm	1434765	516336	142 kN	610 mm	241121	86785
Level 10	648 kN	315 kN	1280 mm	1061683	403200	129 kN	525 mm	178605	67804
Level 9	648 kN	284 kN	1077 mm	751634	305868	116 kN	442 mm	126596	51466
Level 8	648 kN	252 kN	881 mm	502952	222012	103 kN	361 mm	84448	37299
Level 7	648 kN	221 kN	696 mm	313902	153816	91 kN	285 mm	52634	25824
Level 6	648 kN	189 kN	525 mm	178605	99225	77 kN	215 mm	29954	16660
Level 5	648 kN	158 kN	372 mm	89673	58776	65 kN	153 mm	15169	9911
Level 4	648 kN	126 kN	242 mm	37949	30492	52 kN	99 mm	6351	5114
Level 3	648 kN	95 kN	138 mm	12341	13110	39 kN	57 mm	2105	2220
Level 2	648 kN	63 kN	63 mm	2572	3969	26 kN	26 mm	438	672
Level 1	523 kN	25 kN	17 mm	151	425	10 kN	7 mm	26	72
					T_1 3.05 s				T_1 3.05 s

Cog

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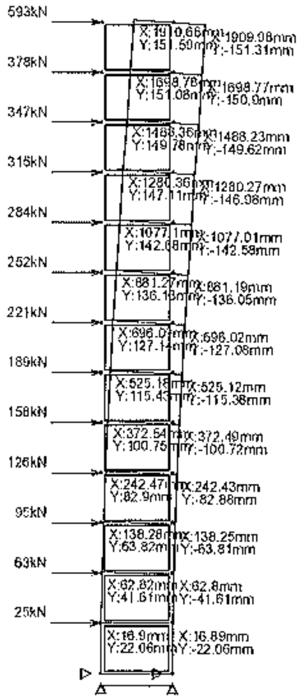
Load cases:
1 Eu (ductility 1.0, T=1.20s)



No general restraint

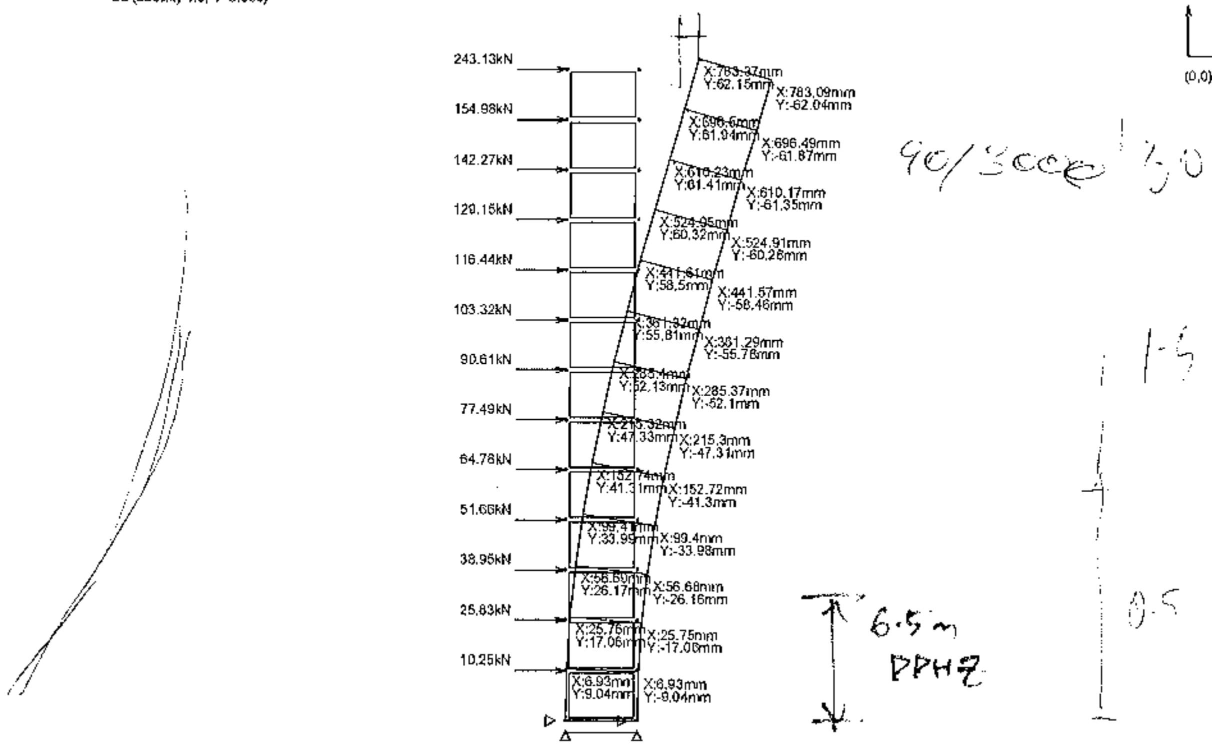
Materials: 1 34CRK30

Sections: 1 FB-1500x1000

Job: G:\2011jobs\110600-19\110604\62 Gloucester St\110604 62 Gloucester Grid F
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 1, Disp: 1, Moment: None, Shear: None, Axial: None, Torsion: None

SPACE GASS 10.85 - SPENCER HOLMES LTD 15 Nov 2011, 4:27 pm

Load cases:
2 Eu (ductility 1.0, T=3.05s)



No general restraint

Materials: 1 34CRK30

Sections: 1 FB-1500x1000

Job: G:\2011jobs\110600-19\110604\62 Gloucester St\110604 62 Gloucester Grid F
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 1, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

C10

P-DELTA ANALYSIS - METHOD A
X-Direction

μ = 3.00
k_p = 0.0165
k_{dm} = 1.50
k_d = 0.85

V_i	δ_{ui}	θ	$\theta > 0.1$	$P\Delta$	$P\Delta k_{dm}\delta_{ui}$	Displace	Drift	$P\Delta F_i$	M_i
136 kN	57 mm	0.18	YES	1.44	106 mm	989 mm	3.52%	42 kN	1629 kNm
222 kN	57 mm	0.24	YES	1.44	106 mm	883 mm	3.52%	27 kN	963 kNm
301 kN	57 mm	0.26	YES	1.44	104 mm	777 mm	3.48%	24 kN	806 kNm
374 kN	55 mm	0.28	YES	1.44	102 mm	673 mm	3.39%	22 kN	668 kNm
439 kN	54 mm	0.29	YES	1.44	99 mm	571 mm	3.31%	20 kN	539 kNm
496 kN	51 mm	0.30	YES	1.44	94 mm	472 mm	3.14%	18 kN	427 kNm
547 kN	48 mm	0.29	YES	1.44	88 mm	378 mm	2.92%	16 kN	328 kNm
590 kN	43 mm	0.29	YES	1.44	80 mm	290 mm	2.66%	13 kN	239 kNm
626 kN	38 mm	0.26	YES	1.44	70 mm	210 mm	2.32%	11 kN	167 kNm
655 kN	31 mm	0.23	YES	1.44	57 mm	140 mm	1.89%	9 kN	106 kNm
677 kN	25 mm	0.19	YES	1.44	45 mm	84 mm	1.50%	7 kN	60 kNm
691 kN	15 mm	0.13	YES	1.44	28 mm	39 mm	0.95%	4 kN	27 kNm
697 kN	6 mm	0.05	YES	1.44	10 mm	10 mm	0.34%	2 kN	5 kNm
						0 mm	0.00%		
								215 kN	5963 kNm

μ = 1.25
k_p = 0.0015
k_{dm} = 1.50
k_d = 0.85

V_i	δ_{ui}	θ	$\theta > 0.1$	$P\Delta$	$P\Delta k_{dm}\delta_{ui}$	Displace	Drift	$P\Delta F_i$	M_i
430 kN	76 mm	0.07	NO	1.00	97 mm	914 mm	3.22%	92	3578
705 kN	76 mm	0.10	NO	1.00	97 mm	817 mm	3.22%	59	2114
956 kN	75 mm	0.11	YES	1.01	97 mm	720 mm	3.22%	54	1793
1185 kN	73 mm	0.12	YES	1.01	94 mm	623 mm	3.15%	50	1485
1391 kN	71 mm	0.12	YES	1.01	92 mm	529 mm	3.07%	44	1198
1574 kN	68 mm	0.12	YES	1.01	87 mm	437 mm	2.91%	40	949
1734 kN	63 mm	0.12	YES	1.01	81 mm	350 mm	2.71%	35	729
1871 kN	57 mm	0.12	YES	1.01	74 mm	269 mm	2.47%	30	532
1985 kN	50 mm	0.11	YES	1.01	64 mm	195 mm	2.15%	25	371
2077 kN	41 mm	0.10	YES	1.01	53 mm	130 mm	1.75%	20	235
2145 kN	32 mm	0.08	YES	1.01	42 mm	78 mm	1.39%	15	133
2191 kN	20 mm	0.05	YES	1.01	26 mm	36 mm	0.88%	10	60
2210 kN	7 mm	0.02	YES	1.01	10 mm	10 mm	0.32%	4	12
						0 mm	0.00%		
								476 kN	13190 kNm

C.11

P-DELTA ANALYSIS - METHOD A
Y-Direction

$\mu =$ 3.00
$k_p =$ 0.0165
$k_{dm} =$ 1.50
$k_d =$ 0.85

V_i	δ_{ui}	θ	$\theta > 0.1$	$P\Delta$	$P\Delta k_{dm}\delta_{ui}$	Displace	Drift	$P\Delta F_i$	M_i
57 kN	61 mm	0.20	YES	1.46	114 mm	1023 mm	3.79%	83 kN	3240 kNm
93 kN	60 mm	0.26	YES	1.46	112 mm	910 mm	3.75%	53 kN	1907 kNm
126 kN	60 mm	0.29	YES	1.46	111 mm	797 mm	3.70%	49 kN	1604 kNm
156 kN	58 mm	0.31	YES	1.46	108 mm	686 mm	3.62%	44 kN	1324 kNm
183 kN	57 mm	0.32	YES	1.46	106 mm	578 mm	3.53%	40 kN	1074 kNm
208 kN	53 mm	0.32	YES	1.46	99 mm	472 mm	3.31%	35 kN	847 kNm
229 kN	49 mm	0.32	YES	1.46	91 mm	373 mm	3.05%	31 kN	650 kNm
247 kN	43 mm	0.30	YES	1.46	81 mm	281 mm	2.70%	26 kN	477 kNm
262 kN	38 mm	0.28	YES	1.46	71 mm	200 mm	2.35%	22 kN	332 kNm
274 kN	29 mm	0.23	YES	1.46	55 mm	129 mm	1.83%	18 kN	212 kNm
283 kN	22 mm	0.18	YES	1.46	41 mm	75 mm	1.35%	13 kN	120 kNm
289 kN	13 mm	0.12	YES	1.46	25 mm	34 mm	0.83%	9 kN	53 kNm
291 kN	5 mm	0.05	YES	1.46	9 mm	9 mm	0.30%	4 kN	11 kNm
						0 mm	0.00%		
								427 kN	11851 kNm

$\mu =$ 1.25
$k_p =$ 0.0015
$k_{dm} =$ 1.50
$k_d =$ 0.85

V_i	δ_{ui}	θ	$\theta > 0.1$	$P\Delta$	$P\Delta k_{dm}\delta_{ui}$	Displace	Drift	$P\Delta F_i$	M_i
180 kN	80 mm	0.08	NO	1.00	103 mm	934 mm	3.42%	180	7017
295 kN	80 mm	0.11	YES	1.01	103 mm	832 mm	3.43%	116	4184
400 kN	79 mm	0.12	YES	1.01	102 mm	729 mm	3.39%	107	3520
495 kN	77 mm	0.13	YES	1.01	99 mm	627 mm	3.31%	97	2005
582 kN	75 mm	0.14	YES	1.01	97 mm	528 mm	3.23%	87	2357
658 kN	70 mm	0.14	YES	1.01	91 mm	431 mm	3.03%	77	1859
725 kN	65 mm	0.13	YES	1.01	84 mm	341 mm	2.79%	68	1427
782 kN	57 mm	0.12	YES	1.01	74 mm	257 mm	2.47%	58	1046
830 kN	50 mm	0.12	YES	1.01	65 mm	183 mm	2.15%	49	729
869 kN	39 mm	0.10	YES	1.01	50 mm	118 mm	1.67%	39	465
897 kN	29 mm	0.07	YES	1.01	37 mm	68 mm	1.23%	29	263
917 kN	18 mm	0.05	YES	1.01	23 mm	31 mm	0.76%	19	116
924 kN	6 mm	0.02	YES	1.01	8 mm	8 mm	0.28%	8	23
						0 mm	0.00%		
								934 kN	25911 kNm

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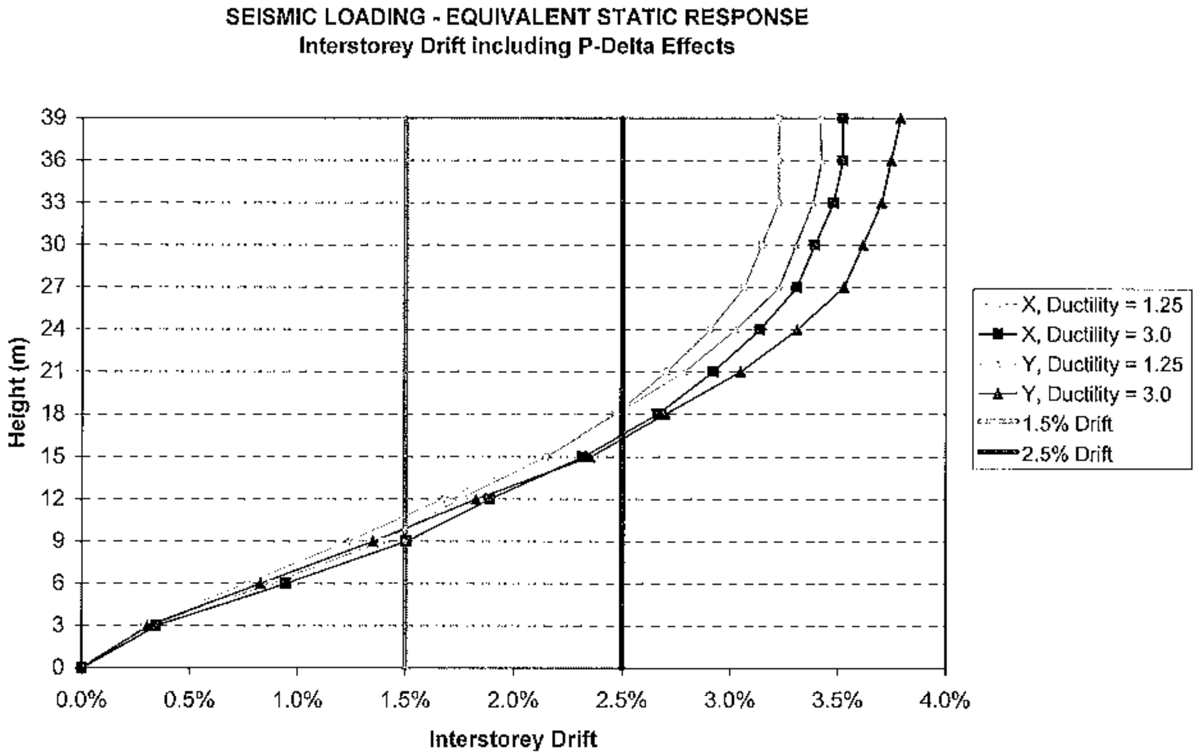
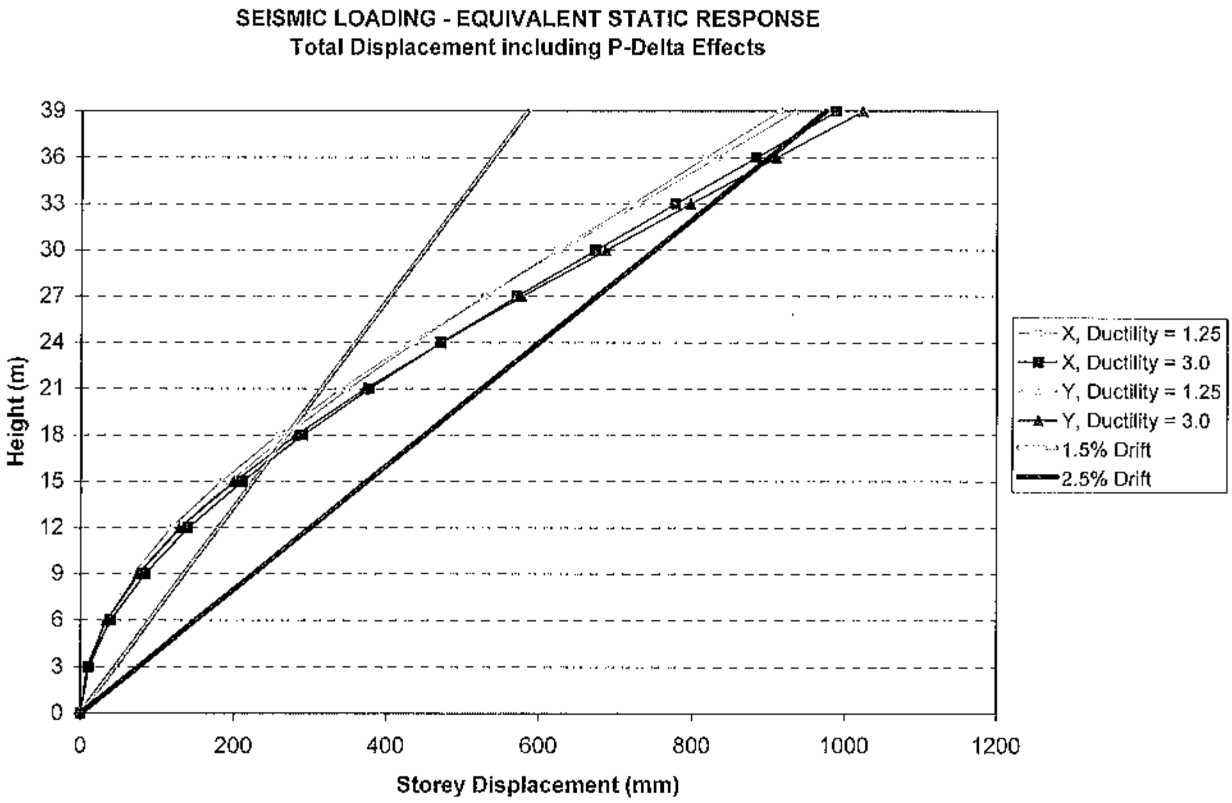
Project	62 GLOUCESTER ST, CHRISTCHURCH	Page	C12
		By	VME
		Date	18/11/2011
Description	DEFLECTIONS	Job Ref	E110604





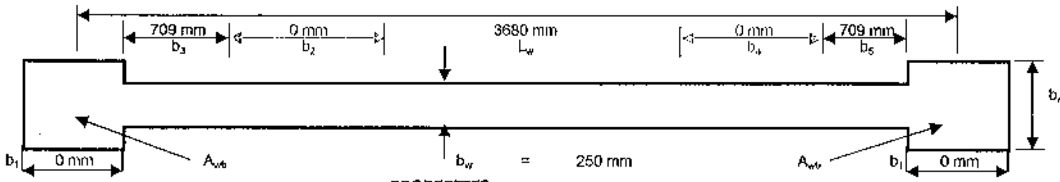
SpencerHolmes engineers · surveyors · planners

Page	C13
By	VME
Date	15/11/2011
Job Ref	E110604

Project 62 GLOUCESTER ST

Description Moment Yield Strength of Reinforced Concrete Wall in accordance with NZS 3101:2006 - GRID 3 & 4 PANEL SHEAR WALLS

DIMENSIONS



b_m = 250 mm < 250 mm OK

b_w = 250 mm

LOADS

Moment	M^*_E	=	5963 kNm	at ductility = 3.00
Shear	$V^*_{1.0}$	=	920 kN	at ductility = 1.00
Shear	V^*_E	=	215 kN	at ductility = 3.00
Web Axial	N_w^*	=	150 kN	(G & Ψ_EQ)
Flange Axial	N_{f1}^*	=	825 kN	(G & Ψ_EQ)
Flange Axial	N_{f2}^*	=	150 kN	(G & Ψ_EQ)

PROPERTIES

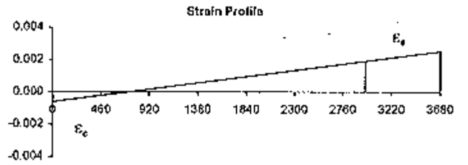
f_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0025	Steel Strain Yield Limit
ρ_{conc}	=	2400 kg/m²	5.2.2 Concrete Density
E_c	=	26738 MPa	5.2.3 Concrete Modulus of Elasticity

11.4.2	Dimensional Limitations of Boundary Elements	h_w	=	39000 mm	(Total height of wall from base to top)			
		L_n	=	3000 mm	(Between Floors)			
	Factor for determining thickness of boundary section	α_f	=	1.00				
	Factor for determining ductility factor	β	=	5				
		k_m	=	L_n / (0.25 + 0.055 A_r) L_w	=	0.98		(Eq 11-21)
		ξ	=	0.3 - (ρ_l f_y / 2.5 f'_c)	=	0.29		(Eq 11-22)
		b_m	>=	α_f k_m β (A_r + 2) L_w / 1700 $\sqrt{\xi}$	=	249 mm	N/A	(Eq 11-20)
11.4.2.3	Dimensions of Enlarged Boundary Element	b_m^2	=	62216 mm²				
			<=	A_{wb}	=	0 mm²	N/A	(Eq 11-23)
			>=	b_m L_w / 10	=	92000 mm²	N/A	(Eq 11-23)
11.4.2.4	Flange Thickness	k L_n / b	=	11.74551387	<	30	N/A	
11.3.10.3.8	(d) Ratio of Vertical Reinforcement to Gross Area	ρ_n	>=	0.7 / f_{yn}	=	0.0014		
11.3.11.3	Min & Max Area of Reinforcement	ρ_{min}	>=	$\sqrt{f'_c}$ / 4 f_y	<=	0.00274		
		ρ_{max}	>=	16 / f_y	<=	0.03200		
		ρ_{lap}	>=	21 / f_y	<=	0.04200		

11.3.10.3.8	(e) Max Spacing of Vertical Reinforcement	s_1	<=	L_w / 3	or	3 t	or	450
			<=	1227 mm		750 mm		450 mm

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²	
Vertical Bar Spacing	s_1	=	440 mm	n_{layers}	=	2	
Vertical Bar Area	$A_{s,prov}$	=	514 mm²/m				
		>=	350 mm²/m		=	A_n	OK
		>=	685 mm²/m		=	$A_{s,min}$	CHECK
		<=	8000 mm²/m		=	$A_{s,max}$	OK
Vertical Bar Area at Lap (assume double normal area)	$A_{s,lap}$	=	1028 mm²/m	<= 10500 mm²/m	=	$A_{s,max,lap}$	OK



Assume Neutral Axis Depth	c	=	709 mm
Concrete Strain	ε_{cc}	=	ε_{sy} { L - [L/(L-c-b_1/2)] }
		=	0.000597
Stress in Concrete	f_c	=	E_c ε_{cc}
(Linear, adequate to fc ~ 0.6 f'c)		=	15.96 MPa

11.3.11.3 Distance from extreme compression fibre to neutral axis 0.75 c_b = 1380 mm > 709 mm OK

11.3.1.3 Effective flange projections for walls with returns

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0	
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²				
		>=	0 mm²/m		=	$A_{s,min,flange}$	OK
		<=	0 mm²/m		=	$A_{s,max,flange}$	OK

Equilibrium Condition - Axial

$C_{c,flange}$	=	fc b_m b_1	=	0 kN	
$C_{c,wall 1}$	=	$0.5f_cb_w(c-b_1/2)^2/c - 0.5f_cb_w(c-b_1/2-b_3)^2/c$	=	1416 kN	
$C_{c,wall 2}$	=	$0.5f_cb_w(c-b_1/2-b_3)^2/c - f_cb_wb_2(c-b_1/2-b_3-b_2)/c - 0.5b_wb_2[fc(c-b_1/2-b_3)/c - f_c(c-b_1/2-b_3-b_2)/c]$	=	0 kN	
$\Sigma C_{s,flange}$	=	$A_{s,flange}$ ε_{cc} E_s	=	0 kN	
$\Sigma C_{s,wall 1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2)^2/c - 0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c$	=	91 kN	
$\Sigma C_{s,wall 2}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c - A_{s,prov}\varepsilon_{sy}E_sb_2(c-b_1/2-b_3-b_2)/c - 0.5A_{s,prov}E_sb_2[\varepsilon_{sy}(c-b_1/2-b_3)/c - \varepsilon_{sy}(c-b_1/2-b_3-b_2)/c]$		0 kN	
$\Sigma T_{s,wall 1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s[(L-c-b_1/2-b_4-b_5)^2/(L-c-b_1/2)]$	=	221 kN	SOLVE FOR EQUILIBRIUM
$\Sigma T_{s,wall 2}$	=	$(A_{s,prov}b_4\varepsilon_{sy}E_s)$ {0.5(L-c-b_1/2-b_4)/(L-c-b_1/2)+[(L-c-b_1)/(L-c-b_1/2)-(L-c-b_1/2-b_4)/(L-c-b_1/2)]}	=	161 kN	
$\Sigma T_{s,flange}$	=	$A_{s,flange}$ ε_s E_s	=	0 kN	
N^*	=	(C_c + ΣC_{si}) - ΣT_{si} (C_c + ΣC_{si}) - ΣT_{si} - N^*	=	0 kN	OK

Equilibrium Condition - Moments about Neutral Axis

$M_{c,flange}$	=	$C_{c,flange}$ (c - b_1/2)	=	0 kNm
$M_{c,wall 1}$	=	$C_{c,wall 1}$ (c - b_1/2 - b_3/2)	=	502 kNm
$M_{c,wall 2}$	=	$C_{c,wall 2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{s,flange}$	=	$C_{s,flange}$ (c - b_1/2)	=	0 kNm
$M_{s,wall 1}$	=	$C_{s,wall 1}$ (c - b_1/2 - b_3/2)	=	32 kNm
$M_{s,wall 2}$	=	$C_{s,wall 2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{t,wall 1}$	=	$T_{s,wall 1}$ 2[(L - c) - b_1/2 - b_5 - b_4]/3	=	333 kNm
$M_{t,wall 2}$	=	$T_{s,wall 2}$ (L - c - b_1/2 - b_4/2)	=	420 kNm
$M_{t,flange}$	=	$T_{s,flange}$ (L - c - b_1/2)	=	0 kNm
M_{axial}	=	N* (L/2 - c)	=	1272 kNm
		$\Sigma M_{s,bout\ c}$	=	**2560 kNm**

36%
at μ=3.0

7.5.1	**General**	(Eq 7-1)	**ΦM_y**	=	**2176 kNm**	>=	**M***	=	**5963 kNm**	**CHECK**

USE:	**A 250 mm**	**SHEAR WALL WITH**	**30 MPa**	**CONCRETE STRENGTH AT 28 DAYS**		
	REINFORCED WITH VERTICAL		**HD 12 BARS IN**	**2 LAYERS AT**	**440 mm**	**CENTRES**
	AND HAVING A FLANGE EACH END WITH		**0 - HD 12 BARS**	**WITH A 250 mm**		**TOTAL WIDTH**

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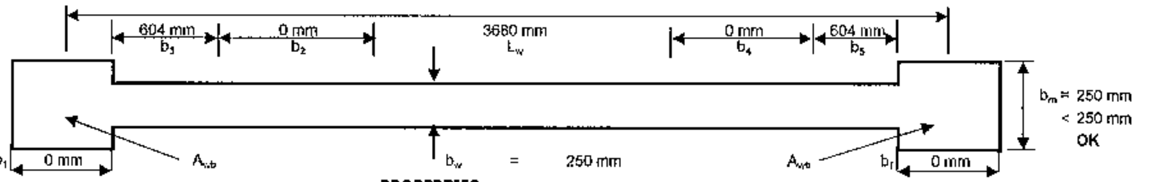
Project	62 GLOUCESTER ST	Page	C14
		By	VME
		Date	15/11/2011
Description	Overstrength Moment Yield of Reinforced Concrete Wall in accordance with NZS 3101:2006 - GRID 3 & 4 PANEL SHEAR WALLS	Job Ref	E110604

DIMENSIONS



b_w = 250 mm

LOADS

Moment	M^*_E	=	5963 kNm	at ductility = 3.00
Shear	$V^*_{1.0}$	=	920 kN	at ductility = 1.00
Shear	V^*_E	=	215 kN	at ductility = 3.00
Web Axial	N_w^*	=	150 kN	(G & Ψ_EQ)
Flange Axial	N_{f1}^*	=	825 kN	(G & Ψ_EQ)
Flange Axial	N_{f2}^*	=	150 kN	(G & Ψ_EQ)

PROPERTIES

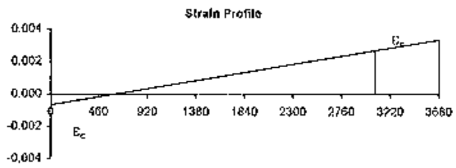
f'_c	=	45 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	675 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0034	Steel Strain Yield Limit
ρ_{conc}	=	2400 kg/m³	5.2.2 Concrete Density
E_c	=	31094 MPa	5.2.3 Concrete Modulus of Elasticity

11.4.2	Dimensional Limitations of Boundary Elements	h_w	=	39000 mm	(Total height of wall from base to top)				
		L_n	=	3000 mm	(Between Floors)				
	Factor for determining thickness of boundary section	α_r	=	1.00					
	Factor for determining ductility factor	β	=	5					
		k_m	=	L_n / (0.25 + 0.055 A_r) L_w		=	0.98		(Eq 11-21)
		ξ	=	0.3 - (ρ_l f_y / 2.5 f'_c)		=	0.29		(Eq 11-22)
		b_m	>=	α_r k_m β (A_r + 2) L_w / 1700 $\sqrt{\xi}$		=	249 mm	N/A	(Eq 11-20)
11.4.2.3	Dimensions of Enlarged Boundary Element	b_m^2	=	61920 mm²					
			<=	A_{wb}		=	0 mm²	N/A	(Eq 11-23)
			>=	b_m L_w / 10		=	92000 mm²	N/A	(Eq 11-23)
11.4.2.4	Flange Thickness	k L_n / b	=	11.74551387		<	30	N/A	
11.3.10.3.8	(d) Ratio of Vertical Reinforcement to Gross Area	ρ_n	>=	0.7 / f_{yn}		=	0.001037037		
11.3.11.3	Min & Max Area of Reinforcement	ρ_{min}	>=	$\sqrt{f'_c}$ / 4 f_y		<=	0.00248		
		ρ_{max}	>=	16 / f_y		<=	0.02370		
		ρ_{lap}	>=	21 / f_y		<=	0.03111		

11.3.10.3.8	(e) Max Spacing of Vertical Reinforcement	s_1	<=	L_w / 3	or	3 t	or	450
			<=	1227 mm		750 mm		450 mm

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²	
Vertical Bar Spacing	s_1	=	440 mm	n_{layers}	=	2	
Vertical Bar Area	$A_{s,prov}$	=	514 mm²/m				
		>=	259 mm²/m		=	A_n	OK
		>=	621 mm²/m		=	$A_{s,min}$	CHECK
		<=	5926 mm²/m		=	$A_{s,max}$	OK
Vertical Bar Area at Lap (assume double normal area)	$A_{s,lap}$	=	1028 mm²/m <= 7778 mm²/m		=	$A_{s,max,lap}$	OK



Assume Neutral Axis Depth	c	=	604 mm
Concrete Strain	ε_{cc}	=	ε_{sy} { L - [L/(L-c-b_1/2)] }
		=	0.000662
Stress in Concrete	f_c	=	E_c ε_{cc}
(Linear, adequate to fc ~ 0.6 f'c)		=	20.59 MPa

11.3.11.3	Distance from extreme compression fibre to neutral axis	0.75 c_b	=	1380 mm	>	604 mm	OK

11.3.1.3 Effective flange projections for walls with returns

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0	
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²				
		>=	0 mm²/m		=	$A_{s,min,flange}$	OK
		<=	0 mm²/m		=	$A_{s,max,flange}$	OK

Equilibrium Condition - Axial

$C_{c,flange}$	=	fc b_m b_1	=	0 kN	
$C_{c,wall\ 1}$	=	$0.5f_cb_w(c-b_1/2)^2/c - 0.5f_cb_w(c-b_1/2-b_3)^2/c$	=	1554 kN	
$C_{c,wall\ 2}$	=	$0.5f_cb_w(c-b_1/2-b_3)^2/c - f_cb_wb_2(c-b_1/2-b_3-b_2)/c - 0.5b_wb_2[fc(c-b_1/2-b_3)/c - f_c(c-b_1/2-b_3-b_2)/c]$	=	0 kN	
$\Sigma C_{s,flange}$	=	$A_{s,flange}\ \varepsilon_{cc}\ E_s$	=	0 kN	
$\Sigma C_{s,wall\ 1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2)^2/c - 0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c$	=	105 kN	
$\Sigma C_{s,wall\ 2}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c - A_{s,prov}\varepsilon_{sy}E_sb_2(c-b_1/2-b_3-b_2)/c - 0.5A_{s,prov}E_sb_2[\varepsilon_{sy}(c-b_1/2-b_3)/c - \varepsilon_{sy}(c-b_1/2-b_3-b_2)/c]$	=	0 kN	
$\Sigma T_{s,wall\ 1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s[(L-c-b_1/2-b_4-b_5)^2/(L-c-b_1/2)]$	=	345 kN	SOLVE FOR EQUILIBRIUM
$\Sigma T_{s,wall\ 2}$	=	$(A_{s,prov}b_4\varepsilon_{sy}E_s)\ \{0.5(L-c-b_1/2-b_4)/(L-c-b_1/2)+[(L-c-b_1)/(L-c-b_1/2)-(L-c-b_1/2-b_4)/(L-c-b_1/2)]\}$	=	189 kN	
$\Sigma T_{s,flange}$	=	$A_{s,flange}\ \varepsilon_s\ E_s$	=	0 kN	
N^*	=	$(C_c + \Sigma C_{si}) - \Sigma T_{si}$ $(C_c + \Sigma C_{si}) - \Sigma T_{si} - N^*$	=	0 kN	OK

Equilibrium Condition - Moments about Neutral Axis

$M_{c,flange}$	=	$C_{c,flange}$ (c - b_1/2)	=	0 kNm
$M_{c,wall\ 1}$	=	$C_{c,wall\ 1}$ (c - b_1/2 - b_3/2)	=	469 kNm
$M_{c,wall\ 2}$	=	$C_{c,wall\ 2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{s,flange}$	=	$C_{s,flange}$ (c - b_1/2)	=	0 kNm
$M_{s,wall\ 1}$	=	$C_{s,wall\ 1}$ (c - b_1/2 - b_3/2)	=	32 kNm
$M_{s,wall\ 2}$	=	$C_{s,wall\ 2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{t,wall\ 1}$	=	$T_{s,wall\ 1}$ 2[(L - c) - b_1/2 - b_5 - b_4]/3	=	568 kNm
$M_{t,wall\ 2}$	=	$T_{s,wall\ 2}$ (L - c - b_1/2 - b_4/2)	=	524 kNm
$M_{t,flange}$	=	$T_{s,flange}$ (L - c - b_1/2)	=	0 kNm
M_{axial}	=	N* (L/2 - c)	=	1391 kNm
		$\Sigma M_{about\ c}$	=	**2984 kNm**

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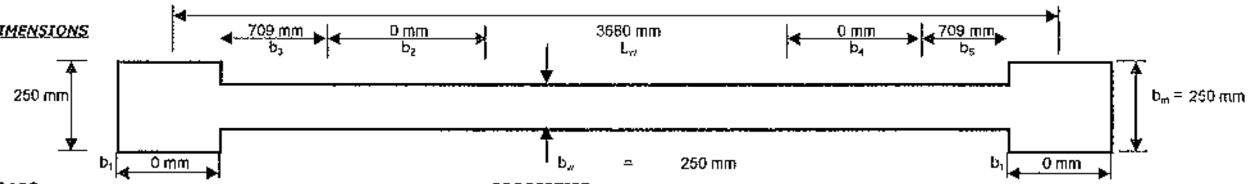
Page	C15
By	VME
Date	15/11/2011
Job Ref	E110604

Project 62 GLOUCESTER ST

Description Moment Ultimate Strength of Reinforced Concrete Wall in accordance with NZS 3101:2006 - GRID 3 & 4 PANEL SHEAR WALLS

DIMENSIONS



LOADS

Moment	M^*_E	=	5963 kNm	at ductility = 3.0
Shear	$V^*_{1.0}$	=	920 kN	at ductility = 1.25
Shear	V^*_E	=	215 kN	at ductility = 3.0
Axial	N^*	=	1125 kN	(G & Ψ_EQ)

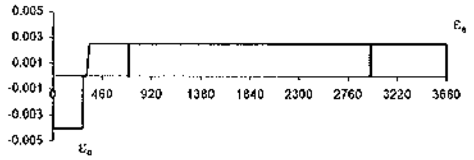
PROPERTIES

f_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0025	Steel Strain Yield Limit

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²
Vertical Bar Spacing	s_1	=	440 mm	n_{layers}	=	2
Vertical Bar Area	$A_{s\ prov}$	=	514 mm²/m			

7.4.2.7	Equivalent Rectangular Concrete Stress Distribution	α_1	=	0.85 - 0.004 (f_c - 55)	=	0.85	(Eq 7-2)
		β_1	=	0.85 - 0.008 (f_c - 30)	=	0.85	(Eq 7-3)



Assume Neutral Axis Depth	c	=	312 mm
Concrete Strain	ε_c	=	0.0040
Steel Strain	ε_s	=	ε_c (L - a) / (c - a)
		=	0.2922

	Distance from Neutral Axis of Steel Yield Strain	x	=	ε_{sy} (L - c) / ε_s	=	29 mm	
	Distance from Compression End of Wall	x'	=	c - x	=	283 mm	
7.4.2.7	(a) Equivalent Compression Zone	a	=	β_1 c	=	265 mm	
11.3.11.3	Distance from extreme compression fibre to neutral axis	0.75 c_b	=	1380 mm	>	312 mm	OK

11.3.1.3 **Effective flange projections for walls with returns**

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²			

Equilibrium Condition - Axial

$C_{c,flange}$	=	$f_c\ b_m\ b_1$		=	0 kN
$C_{c,wall\ 1}$	=	$f_c\ b_w\ b_3$		=	1987 kN
$C_{c,wall\ 2}$	=	$f_c\ b_w$ (a - b_1 - b_3 - b_2)		=	0 kN
$\Sigma\ T_{s,wall\ 1}$	=	$A_{s\ prov}\ f_y$ (L - c - x - b_4 - b_5 - b_1/2) + 0.5 $A_{s\ prov}\ f_y$ x		=	680 kN
$\Sigma\ T_{s,wall\ 2}$	=	$A_{s\ prov}\ f_y\ b_4$		=	182 kN
$\Sigma\ T_{s,flange}$	=	$A_{s\ prov}\ f_y$		=	0 kN
N^*	=	(C_c + $\Sigma\ C_{si}$) - $\Sigma\ T_{si}$	(C_c + $\Sigma\ C_{si}$) - $\Sigma\ T_{si}$ - N^*	=	**0 kN**

SOLVE FOR EQUILIBRIUM

OK

Equilibrium Condition - Moments about Neutral Axis

$M_{c,flange}$	=	$C_{c,flange}$ c	=	0 kNm
$M_{c,wall\ 1}$	=	$C_{c,wall\ 1}$ (c - b_1/2 - b_3/2)	=	-85 kNm
$M_{c,wall\ 2}$	=	$C_{c,wall\ 2}$ (c - b_1/2 - b_3 - b_2/2)	=	0 kNm
$M_{t,wall\ 1}$	=	$T_{s,wall\ 1}$ (L - c - x - b_1/2 - b_5 - b_4) / 2	=	894 kNm
$M_{t,wall\ 2}$	=	$T_{s,wall\ 2}$ (L - c - b_1/2 - b_4/2)	=	550 kNm
$M_{t,flange}$	=	$T_{s,flange}$ (L - c)	=	0 kNm
M_{axial}	=	N^* (L/2 - c)	=	1719 kNm

Nominal Ultimate Moment Capacity M_n = **3077 kNm**

CURVATURE DUCTILITY

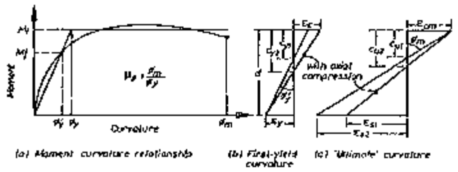
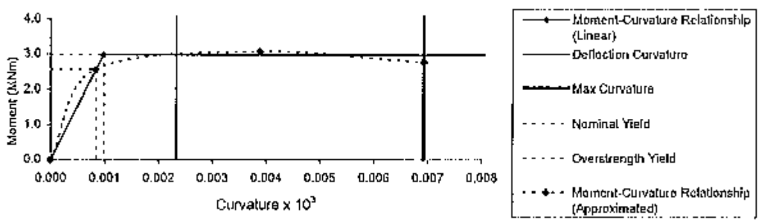


Fig. 3.26 Definition of curvature ductility.

From "Seismic Design of Reinforced Concrete and Masonry Buildings", 1992, T. Paulay & M. J. N. Priestley

Φ'_y	=	ε_{sy} / (L - c_y)	=	8.416E-07
Φ_y	=	$\Phi'_y\ M_y'$ / M_y	=	9.811E-07

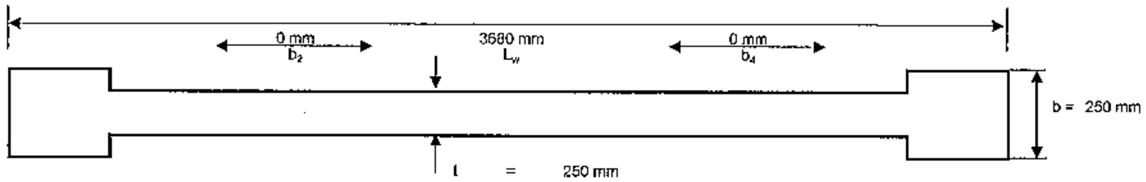
2.6.1.3.4	Material strain limits	Φ_y	=	2 f_y / E_s h	=	1.155E-06
Table 2.4	(b) K_d factor for limiting curvatures in walls	$K_{d,limit}$	=	6		
	Maximum Curvature	Φ_{max}	=	$K_d\ \Phi_y$	=	6.929E-06
	Factored deflection with cracked section properties			Δ_μ	=	21 mm



PPHZ	=	3000 mm
1 < β_a	=	2.5 - 0.5 A_r < 2
	=	1.00

Table 2.5	Maximum available ULS structural ductility factor	(i) Two or more cantilevered	μ <= 5 / β_a	=	5.0
		(ii) Two or more coupled	μ <= (3A + 4) / β_a	=	6.0
		(iii) Single cantilever	μ <= 4 / β_a	=	4.0

DIMENSIONS



LOADS

Moment	M^*_E	=	5963 kNm	at ductility = 3.0
Shear	$V^*_{1.25}$	=	681 kN	at ductility = 1.25
Shear	V^*_e	=	509 kN	at ductility = 3.0
Axial	N^*_G	=	1125 kN	(G & Ψ_EQ)

PROPERTIES

f'_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
f_{yt}	=	500 MPa	Shear Reinforcement Yield Strength

11.4.7.2 In the estimation of the maximum shear demand on a wall of limited ductility, the maximum shear need not exceed that corresponding to the elastic response of the wall element derived using μ = 1.25 and S_p = 0.9.

11.3.10.3.8	(c) Max Spacing of Horizontal Shear Reinforcement	s_2	<=	L_w / 5	or	3 t	or	450		
			<=	736 mm		750 mm		450 mm		
11.3.10.3.8	(b) Min Area of Horizontal Shear Reinforcement	$A_{v,min}$	>=	$0.7\, b_w\, s_2 / f_{yt}$		=	140 mm²		(Eq 11-19)	
11.3.11.3	Min & Max Area of Reinforcement	p_{min}	>=	$\sqrt{f'_c} / 4 f_{yt}$		<=	0.00274			
		p_{max}	>=	$16 / f_{yt}$		<=	0.03200			

CRACK CONTROL

2.4.4.4	Spacing of rebar for crack control on the tension face	f_s	=	180 MPa			
		s_{max}	=	$90000 / f_s - 2.5c_c$	=	389 mm	(Eq 2-5)

SHEAR REINFORCEMENT

Horizontal Bar Size	d_{bt}	=	12 mm	$A_{s,bar}$	=	113 mm²	
Horizontal Bar Spacing	s_2	=	400 mm	n_{layers}	=	2	
Transverse Bar Area	$A_{v,prov}$	=	226 mm²				
		>=	140 mm²		=	$A_{v,min}$	OK
		>=	274 mm²		=	$s_2\, t\, p_{min}$	CHECK
		<=	3200 mm²		=	$s_2\, t\, p_{max}$	OK

11.3.10.3.8	**(a) Shear Reinforcement Contribution**	V_s	=	$A_v\, f_{yt}\, d / s_2$	=	**832 kN**	**(Eq 11-18)**
11.3.10.3.4	Concrete Shear Strength - Simplified	v_c	=	$0.17\sqrt{f'_c}$	=	0.93 MPa	(Eq 11-12)
			=	$0.17\ [\sqrt{f'_c} + N^* / A_g]$	=	1.14 MPa	(Eq 11-13)
11.3.10.3.5	Concrete Shear Strength - Detailed		=	$0.27\sqrt{f'_c} + (N^* / 4\, A_g)$	=	1.78 MPa	(Eq 11-14)
			=	$0.05\sqrt{f'_c} + \dfrac{L_w[0.1f'_c + 0.2(N^*A_g)]}{(M^*/V^*) - (L_w/2)}$	=	1.64 MPa	(Eq 11-15)
9.3.9.3.3(a) & 10.3.10.2.1(a)	Effective Shear Area	A_{cv}	=	$b_w\, d$	=	736000 mm²	
11.4.7.3	Factor for ductility of plastic region	λ	=	0.5			
		$V_{c,max}$	=	$(0.27\lambda\sqrt{f'c} + N^*/4A_g)b_w d$	=	769 kN	(Eq 11-28)
11.3.10.3.8	**(a) Concrete Shear Contribution**	V_c	=	$v_c\, A_{cv}$	=	**769 kN**	**(Eq 11-17)**
7.5.1	Max Nominal Shear Stress	v_{max}	=	Min (0.2 f'_c, 8.0 MPa)	=	6.0 MPa	(7.5.2)
11.4.7.3	Overstrength Factor	$\Phi_{o,w}$	=	$\Phi_{o,m}\, M_y / \Phi M_y$	=	1.37	
	Factor for ductility of plastic region	α	=	3.00			
	For walls subject to tension	V_n	<=	$(\Phi_{ow}/\alpha + 0.15)\sqrt{f'_c}\, A_{cv}$	=	2448 kN	(Eq 11-29)
7.5.3	**Nominal Shear Strength**	V_n	=	$V_c + V_s$	=	**1602 kN**	**(Eq 7-6)**
		v_n	=	V_n / A_{cv}			
			=	1.74 MPa	<=	6.0 MPa	OK
7.5.1	**General**	ΦV_n	>=	V^*			(Eq 7-4)
	Nominal Ductility	ΦV_n	=	**1201 kN**	>=	**681 kN**	OK
	Bending Overstrength at ductility = 3.0	ΦV_n	=	**1602 kN**	>=	**509 kN**	OK

USE: A 250 mm SHEAR WALL WITH 30 MPa CONCRETE STRENGTH AT 28 DAYS
REINFORCED WITH HORIZONTAL HD 12 BARS IN 2 LAYERS AT 400 mm CENTRES

TRANSVERSE REINFORCEMENT REQUIREMENTS FOR DUCTILITY IN EARTHQUAKES

11.4.6.3	Lateral restraint in plastic hinge regions	A_s	=	226 mm²	s_v	=	440 mm	
		p_l	=	$A_s / t\, s_v$		=	0.00206	(Eq 11-24)
	ductile regions		=	$2 / f_y$		<	0.00400	
	limited ductile regions		=	$3 / f_y$		<	0.00600	
11.4.6.5	Confinement requirements in the plastic hinge region	λ	=	1.0				
		c_c	=	$0.1\, \Phi_{ow}\, L_w / \lambda$		=	505 mm	(Eq 11-25)
						<	709 mm	Check 11.4.6.5
	(a) Rectangular hoops used in confined columns	α	=	0.175				
		h''	=	150 mm				
	Area of shear reinforcement to be provided	A_{sh}	=	$\alpha s_h h''(A_g{}^* f'_c / A_c{}^* f_{yt})(c/L_w - 0.07)$		=	322 mm²/m	(Eq 11-26)
	(b) Length of confined region of compressed wall	c'	>=	$c - 0.7\, c_c$ > 0.5 c		=	356 mm	(Eq 11-27)
	(d) Spacing limitation within ductile plastic regions	s_{max}	=	Min ($6d_b$, t/2)		=	72 mm	
	Spacing limitation within limited ductile plastic regions		=	Min ($10d_b$, t)		=	120 mm	

CHECK: SPECIFIC REQUIREMENTS OF DUCTILE & LIMITED DUCTILE REGIONS UNDER 11.4.6.5

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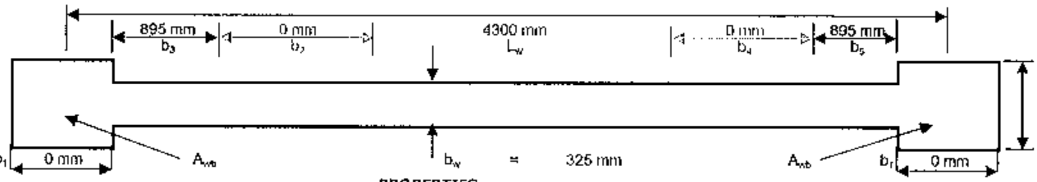
Page	C17
By	VME
Date	15/11/2011
Job Ref	E110604

Project 62 GLOUCESTER ST

Description Moment Yield Strength of Reinforced Concrete Wall in accordance with NZS 3101:2006 - GRID F PANEL SHEAR WALLS

DIMENSIONS



b_m = 325 mm
< 325 mm
OK

b_w = 325 mm

LOADS

Moment	M^*_E	=	11851 kNm	at ductility = 3.00
Shear	$V^*_{1.0}$	=	1829 kN	at ductility = 1.00
Shear	V^*_E	=	427 kN	at ductility = 3.00
Web Axial	N_w^*	=	0 kN	(G & Ψ_EQ)
Flange Axial	N_{f1}^*	=	2250 kN	(G & Ψ_EQ)
Flange Axial	N_{f2}^*	=	0 kN	(G & Ψ_EQ)

PROPERTIES

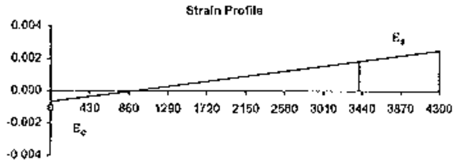
f'_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0025	Steel Strain Yield Limit
ρ_{conc}	=	2400 kg/m³	5.2.2 Concrete Density
E_c	=	26738 MPa	5.2.3 Concrete Modulus of Elasticity

11.4.2	Dimensional Limitations of Boundary Elements	h_w	=	39000 mm	(Total height of wall from base to top)			
		L_n	=	3000 mm	(Between Floors)			
	Factor for determining thickness of boundary section	α_r	=	1.00				
	Factor for determining ductility factor	β	=	5				
		k_m	=	L_n / (0.25 + 0.055 A_r) L_w	=	0.93		(Eq 11-21)
		ξ	=	0.3 - ($p_l f_y$ / 2.5 f'_c)	=	0.29		(Eq 11-22)
		b_m	>=	$\alpha_r k_m \beta$ (A_r + 2) L_w / 1700 $\sqrt{\xi}$	=	242 mm	OK	(Eq 11-20)
11.4.2.3	Dimensions of Enlarged Boundary Element	b_m^2	=	58684 mm²				
			<=	A_{wb}	=	0 mm²	N/A	(Eq 11-23)
			>=	$b_m L_w$ / 10	=	139750 mm²	N/A	(Eq 11-23)
11.4.2.4	Flange Thickness	k L_n / b	=	8.600095557	<	30	OK	
11.3.10.3.8	(d) Ratio of Vertical Reinforcement to Gross Area	p_n	>=	0.7 / f_{yn}	=	0.0014		
11.3.11.3	Min & Max Area of Reinforcement	ρ_{min}	>=	$\sqrt{f'_c}$ / 4 f_y	<=	0.00274		
		ρ_{max}	>=	16 / f_y	<=	0.03200		
		ρ_{lap}	>=	21 / f_y	<=	0.04200		

11.3.10.3.8	(e) Max Spacing of Vertical Reinforcement	s_1	<=	L_w / 3	or	3 t	or	450
			<=	1433 mm		975 mm		450 mm

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²	
Vertical Bar Spacing	s_1	=	460 mm	n_{layers}	=	2	
Vertical Bar Area	$A_{s,prov}$	=	492 mm²/m				
		>=	455 mm²/m		=	A_n	OK
		>=	890 mm²/m		=	$A_{s,min}$	CHECK
		<=	10400 mm²/m		=	$A_{s,max}$	OK
Vertical Bar Area at Lap (assume double normal area)	$A_{s,lap}$	=	983 mm²/m	<= 13650 mm²/m	=	$A_{s,max,lap}$	OK



Assume Neutral Axis Depth	c	=	895 mm
Concrete Strain	ε_{cc}	=	ε_{sy} { L - [L/(L-c-b_1/2)] }
		=	0.000658
Stress in Concrete	f_c	=	$E_c\ \varepsilon_{cc}$
(Linear, adequate to fc ~ 0.6 f'c)		=	17.58 MPa

11.3.11.3 Distance from extreme compression fibre to neutral axis 0.75 c_b = 1612.5 mm > 895 mm OK

11.3.1.3 **Effective flange projections for walls with returns**

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0	
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²				
		>=	0 mm²/m		=	$A_{s,min,flange}$	OK
		<=	0 mm²/m		=	$A_{s,max,flange}$	OK

Equilibrium Condition - Axial

$C_{c,flange}$	=	fc b_m b_1	=	0 kN
$C_{c,wall1}$	=	$0.5f_cb_w(c-b_1/2)^2/c - 0.5f_cb_w(c-b_1/2-b_3)^2/c$	=	2558 kN
$C_{c,wall2}$	=	$0.5f_cb_w(c-b_1/2-b_3)^2/c - f_cb_wb_2(c-b_1/2-b_3-b_2)/c - 0.5b_wb_2[fc(c-b_1/2-b_3)/c - f_c(c-b_1/2-b_3-b_2)/c]$	=	0 kN
$\Sigma C_{s,flange}$	=	$A_{s,flange}\ \varepsilon_{cc}\ E_s$	=	0 kN
$\Sigma C_{s,wall1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2)^2/c - 0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c$	=	110 kN
$\Sigma C_{s,wall2}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s(c-b_1/2-b_3)^2/c - A_{s,prov}\varepsilon_{sy}E_sb_2(c-b_1/2-b_3-b_2)/c - 0.5A_{s,prov}E_sb_2[\varepsilon_{sy}(c-b_1/2-b_3)/c - \varepsilon_{sy}(c-b_1/2-b_3-b_2)/c]$	=	0 kN
$\Sigma T_{s,wall1}$	=	$0.5A_{s,prov}\varepsilon_{sy}E_s[(L-c-b_1/2-b_4-b_5)^2/(L-c-b_1/2)]$	=	227 kN
$\Sigma T_{s,wall2}$	=	$(A_{s,prov}b_4\varepsilon_{sy}E_s)\ \{0.5(L-c-b_1/2-b_4)/(L-c-b_1/2)+[(L-c-b_1)/(L-c-b_1/2)-(L-c-b_1/2-b_4)/(L-c-b_1/2)]\}$	=	191 kN
$\Sigma T_{s,flange}$	=	$A_{s,flange}\ \varepsilon_s\ E_s$	=	0 kN
N*	=	(C_c + Σ C_{si}) - Σ T_{si} (C_c + Σ C_{si}) - Σ T_{si} - N*	=	0 kN

SOLVE FOR EQUILIBRIUM — OK

Equilibrium Condition - Moments about Neutral Axis

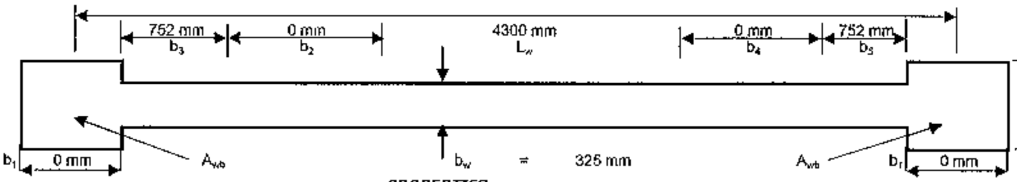
$M_{c,flange}$	=	$C_{c,flange}$ (c - b_1/2)	=	0 kNm
$M_{c,wall1}$	=	$C_{c,wall1}$ (c - b_1/2 - b_3/2)	=	1146 kNm
$M_{c,wall2}$	=	$C_{c,wall2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{s,flange}$	=	$C_{s,flange}$ (c - b_1/2)	=	0 kNm
$M_{s,wall1}$	=	$C_{s,wall1}$ (c - b_1/2 - b_3/2)	=	49 kNm
$M_{s,wall2}$	=	$C_{s,wall2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{t,wall1}$	=	$T_{s,wall1}$ 2[(L - c) - b_1/2 - b_5 - b_4]/3	=	380 kNm
$M_{t,wall2}$	=	$T_{s,wall2}$ (L - c - b_1/2 - b_4/2)	=	565 kNm
$M_{t,flange}$	=	$T_{s,flange}$ (L - c - b_1/2)	=	0 kNm
M_{axial}	=	N* (L/2 - c)	=	2823 kNm
		Σ $M_{about\ c}$	=	4963 kNm

36%, at
μ = 3.0

7.5.1	General	(Eq 7-1)	ΦM_y	=	4219 kNm	>=	M*	= 11851 kNm	CHECK

USE:	A 325 mm	SHEAR WALL WITH	30 MPa	CONCRETE STRENGTH AT 28 DAYS		
	REINFORCED WITH VERTICAL		HD 12 BARS IN	2 LAYERS AT	460 mm	CENTRES
	AND HAVING A FLANGE EACH END WITH		0 - HD 12 BARS	WITH A 325 mm		TOTAL WIDTH

DIMENSIONS



b_w = 325 mm

b_m = 325 mm
< 325 mm
OK

LOADS

Moment	M^*_E	=	11851 kNm	at ductility = 3.00
Shear	$V^*_{1.0}$	=	1829 kN	at ductility = 1.00
Shear	V^*_E	=	427 kN	at ductility = 3.00
Web Axial	N_w^*	=	0 kN	(G & Ψ_EQ)
Flange Axial	N_{f1}^*	=	2250 kN	(G & Ψ_EQ)
Flange Axial	N_{f2}^*	=	0 kN	(G & Ψ_EQ)

PROPERTIES

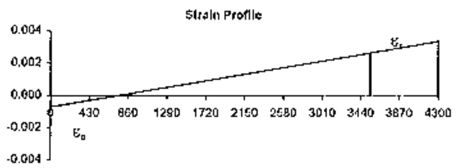
f'_c	=	45 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	675 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0034	Steel Strain Yield Limit
ρ_{conc}	=	2400 kg/m³	5.2.2 Concrete Density
E_c	=	31094 MPa	5.2.3 Concrete Modulus of Elasticity

11.4.2	Dimensional Limitations of Boundary Elements	h_w	=	39000 mm	(Total height of wall from base to top)			
		L_n	=	3000 mm	(Between Floors)			
	Factor for determining thickness of boundary section	α_f	=	1.00				
	Factor for determining ductility factor	β	=	5				
		k_m	=	L_n / (0.25 + 0.055 A_r) L_w	=	0.93		(Eq 11-21)
		ξ	=	0.3 - ($\rho_l f_y$ / 2.5 f'_c)	=	0.29		(Eq 11-22)
		b_m	>=	α_f k_m β (A_r + 2) L_w / 1700 $\sqrt{\xi}$	=	242 mm	N/A	(Eq 11-20)
11.4.2.3	Dimensions of Enlarged Boundary Element	b_m^2	=	58480 mm²				
			<=	A_{wb}	=	0 mm²	N/A	(Eq 11-23)
			>=	b_m L_w / 10	=	139750 mm²	N/A	(Eq 11-23)
11.4.2.4	Flange Thickness	k L_n / b	=	8.600095557	<	30	N/A	
11.3.10.3.8	(d) Ratio of Vertical Reinforcement to Gross Area	ρ_n	>=	0.7 / f_y	=	0.001037037		
11.3.11.3	Min & Max Area of Reinforcement	ρ_{min}	>=	$\sqrt{f'_c}$ / 4 f_y	<=	0.00248		
		ρ_{max}	>=	16 / f_y	<=	0.02370		
		ρ_{lap}	>=	21 / f_y	<=	0.03111		

11.3.10.3.8	(e) Max Spacing of Vertical Reinforcement	s_1	<=	L_w / 3	or	3 t	or	450
			<=	1433 mm		975 mm		450 mm

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²	
Vertical Bar Spacing	s_1	=	460 mm	n_{layers}	=	2	
Vertical Bar Area	$A_{s\ prov}$	=	492 mm²/m				
		>=	337 mm²/m		=	A_n	OK
		>=	807 mm²/m		=	$A_{s,min}$	CHECK
		<=	7704 mm²/m		=	$A_{s,max}$	OK
Vertical Bar Area at Lap (assume double normal area)	$A_{s,lap}$	=	983 mm²/m	<= 10111 mm²/m	=	$A_{s,max,lap}$	OK



Assume Neutral Axis Depth	c	=	752 mm
Concrete Strain	ε_{cc}	=	ε_{sy} { L - [L/(L-c-b_1/2)] }
		=	0.000715
Stress in Concrete	f_c	=	E_c ε_{cc}
(Linear, adequate to fc ~ 0.6 f'c)		=	22.23 MPa

11.3.11.3	Distance from extreme compression fibre to neutral axis	0.75 c_b	=	1612.5 mm	>	752 mm	OK

11.3.1.3 Effective flange projections for walls with returns

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0	
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²				
		>=	0 mm²/m		=	$A_{s,min,flange}$	OK
		<=	0 mm²/m		=	$A_{s,max,flange}$	OK

Equilibrium Condition - Axial

$C_{c,flange}$	=	fc b_m b_1	=	0 kN	
$C_{c,wall\,1}$	=	0.5$f_c$$b_w$(c-$b_1$/2)²/c - 0.5$f_c$$b_w$(c-$b_1$/2-$b_3$)²/c	=	2714 kN	
$C_{c,wall\,2}$	=	0.5$f_c$$b_w$(c-$b_1$/2-$b_3$)²/c - $f_c$$b_w$$b_2$(c-$b_1$/2-$b_3$-$b_2$)/c - 0.5$b_w$$b_2$[fc(c-$b_1$/2-$b_3$)/c - f_c(c-b_1/2-b_3-b_2)/c]	=	0 kN	
Σ $C_{s,flange}$	=	$A_{s,flange}$ ε_{cc} E_s	=	0 kN	
Σ $C_{s,wall\,1}$	=	0.5$A_{s,prov}$$\varepsilon_{sy}$$E_s$(c-$b_1$/2)²/c - 0.5$A_{s,prov}$$\varepsilon_{sy}$$E_s$(c-$b_1$/2-$b_3$)²/c	=	125 kN	
Σ $C_{s,wall\,2}$	=	0.5$A_{s,prov}$$\varepsilon_{sy}$$E_s$(c-$b_1$/2-$b_3$)²/c - $A_{s,prov}$$\varepsilon_{sy}$$E_s$$b_2$(c-$b_1$/2-$b_3$-$b_2$)/c - 0.5$A_{s,prov}$$E_s$$b_2$[$\varepsilon_{sy}$(c-$b_1$/2-$b_3$)/c - ε_{sy}(c-b_1/2-b_3-b_2)/c]	=	0 kN	
Σ $T_{s,wall\,1}$	=	0.5$A_{s,prov}$$\varepsilon_{sy}$$E_s$[(L-c-$b_1$/2-$b_4$-$b_5$)²/(L-c-$b_1$/2)]	=	366 kN	SOLVE FOR EQUILIBRIUM
Σ $T_{s,wall\,2}$	=	($A_{s,prov}$$b_4$$\varepsilon_{sy}$$E_s$) {0.5[(L-c-$b_1$/2-$b_4$)/(L-c-$b_1$/2)+[(L-c-$b_1$)/(L-c-$b_1$/2)-(L-c-$b_1$/2-$b_4$)/(L-c-$b_1$/2)]}	=	223 kN	
Σ $T_{s,flange}$	=	$A_{s,flange}$ ε_s E_s	=	0 kN	
N^*	=	(C_c + Σ C_{si}) - Σ T_{si} (C_c + Σ C_{si}) - Σ T_{si} - N^*	=	0 kN	OK

Equilibrium Condition - Moments about Neutral Axis

$M_{c,flange}$	=	$C_{c,flange}$ (c - b_1/2)	=	0 kNm
$M_{c,wall\,1}$	=	$C_{c,wall\,1}$ (c - b_1/2 - b_3/2)	=	1020 kNm
$M_{c,wall\,2}$	=	$C_{c,wall\,2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{s,flange}$	=	$C_{s,flange}$ (c - b_1/2)	=	0 kNm
$M_{s,wall\,1}$	=	$C_{s,wall\,1}$ (c - b_1/2 - b_3/2)	=	47 kNm
$M_{s,wall\,2}$	=	$C_{s,wall\,2}$ 2(c - b_1/2 - b_3 - b_2)/3	=	0 kNm
$M_{t,wall\,1}$	=	$T_{s,wall\,1}$ 2[(L - c) - b_1/2 - b_5 - b_4]/3	=	682 kNm
$M_{t,wall\,2}$	=	$T_{s,wall\,2}$ (L - c - b_1/2 - b_4/2)	=	708 kNm
$M_{t,flange}$	=	$T_{s,flange}$ (L - c - b_1/2)	=	0 kNm
M_{axial}	=	N^* (L/2 - c)	=	3147 kNm
		Σ $M_{about\,c}$	=	**5603 kNm**

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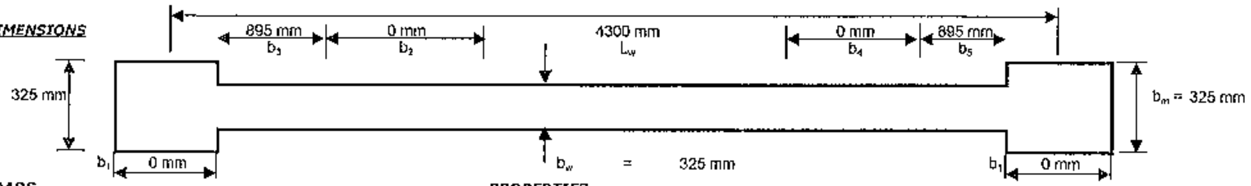
Page	C19	
By	VME	
Date	15/11/2011	
Job Ref	E110804	

Project 62 GLOUCESTER ST

Description Moment Ultimate Strength of Reinforced Concrete Wall in accordance with NZS 3101:2006 - GRID F PANEL SHEAR WALLS

DIMENSIONS



b_w = 325 mm

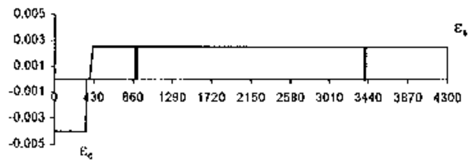
LOADS				
Moment	M^*_E	=	11851 kNm	at ductility = 3.0
Shear	$V^*_{1.0}$	=	1829 kN	at ductility = 1.25
Shear	V^*_E	=	427 kN	at ductility = 3.0
Axial	N^*	=	2250 kN	(G & Ψ_EQ)

PROPERTIES			
f'_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
E_s	=	200000 MPa	Reinforcement Modulus of Elasticity
ε_{sy}	=	0.0025	Steel Strain Yield Limit

FLEXURAL REINFORCEMENT

Vertical Bar Size	d_b	=	12 mm	$A_{s,bar}$	=	113 mm²
Vertical Bar Spacing	s_1	=	460 mm	n_{layers}	=	2
Vertical Bar Area	$A_{s,prov}$	=	492 mm²/m			

7.4.2.7	Equivalent Rectangular Concrete Stress Distribution	α_1	=	$0.85 - 0.004\,(f'_c - 55)$	=	0.85	(Eq 7-2)
		β_1	=	$0.85 - 0.008\,(f'_c - 30)$	=	0.85	(Eq 7-3)



Assume Neutral Axis Depth	c	=	387 mm
Concrete Strain	ε_c	=	0.0040
Steel Strain	ε_s	=	$\varepsilon_c\,(L - a)/(c - a)$
		=	0.2736

	Distance from Neutral Axis of Steel Yield Strain	x	=	$\varepsilon_{sy}\,(L - c)/\varepsilon_s$	=	36 mm	
	Distance from Compression End of Wall	x'	=	c - x	=	351 mm	
7.4.2.7	(a) Equivalent Compression Zone	a	=	β_1 c	=	329 mm	
11.3.11.3	Distance from extreme compression fibre to neutral axis	0.75 c_b	=	1612.5 mm	>	387 mm	OK

11.3.1.3 Effective flange projections for walls with returns

Vertical Bar Size	$d_{b,flange}$	=	12 mm	n_{bars}	=	0
Total Area of Vertical bars within Flange	$A_{s,flange}$	=	0 mm²			

Equilibrium Condition - Axial

$C_{c,flange}$	=	$f'_c\, b_m\, b_1$		=	0 kN
$C_{c,wall\,1}$	=	$f'_c\, b_w\, b_3$		=	3208 kN
$C_{c,wall\,2}$	=	$f'_c\, b_w\,(a - b_1 - b_3 - b_2)$		=	0 kN
$\Sigma\, T_{s,wall\,1}$	=	$A_{s,prov}\, f_y\,(L - c - x - b_4 - b_5 - b_1/2) + 0.5\, A_{s,prov}\, f_y\, x$		=	737 kN
$\Sigma\, T_{s,wall\,2}$	=	$A_{s,prov}\, f_y\, b_4$		=	220 kN
$\Sigma\, T_{s,flange}$	=	$A_{s,prov}\, f_y$		=	0 kN
N^*	=	$(C_c + \Sigma\, C_{si}) - \Sigma\, T_{si}$	$(C_c + \Sigma\, C_{si}) - \Sigma\, T_{si} - N^*$	=	**0 kN**

SOLVE FOR EQUILIBRIUM

OK

Equilibrium Condition - Moments about Neutral Axis

$M_{c,flange}$	=	$C_{c,flange}$ c	=	0 kNm
$M_{c,wall\,1}$	=	$C_{c,wall\,1}\,(c - b_1/2 - b_3/2)$	=	-195 kNm
$M_{c,wall\,2}$	=	$C_{c,wall\,2}\,(c - b_1/2 - b_3 - b_2/2)$	=	0 kNm
$M_{t,wall\,1}$	=	$T_{s,wall\,1}\,(L - c - x - b_1/2 - b_5 - b_4)/2$	=	1099 kNm
$M_{t,wall\,2}$	=	$T_{s,wall\,2}\,(L - c - b_1/2 - b_4/2)$	=	763 kNm
$M_{t,flange}$	=	$T_{s,flange}\,(L - c)$	=	0 kNm
M_{axial}	=	$N^*\,(L/2 - c)$	=	3967 kNm

Nominal Ultimate Moment Capacity M_u = **5634 kNm**

CURVATURE DUCTILITY

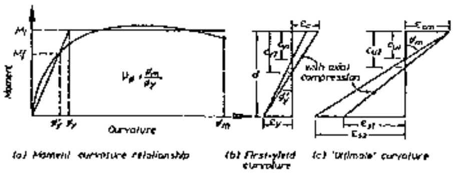


Fig. 3.26 Definition of curvature ductility.

From "Seismic Design of Reinforced Concrete and Masonry Buildings", 1992, T. Paulay & M. J. N. Priestley

Φ'_y	=	$\varepsilon_{sy}/(L - c_y)$	=	7.343E-07
Φ_y	=	$\Phi'_y\, M_y'/M_y$	=	8.290E-07

2.6.1.3.4	Material strain limits	Φ_y	=	$2\, f_y/E_s\, h$	=	9.884E-07
Table 2.4	(b) K_d factor for limiting curvatures in walls	$K_{d,limit}$	=	6		
	Maximum Curvature	Φ_{max}	=	$K_d\, \Phi_y$	=	5.930E-06
	Factored deflection with cracked section properties					

Δ_y	=	18 mm
PPHZ	=	3000 mm
1 < β_a	=	2.5 - 0.5 A_r < 2
	=	1.00
$T_{..}$	··	1000 kN
L'	··	1000 mm
$M_{..}$	·	1000 kNm
1/3 < A	··	$T_{..}\, L'/M_{..}$ < ..
	··	0.67

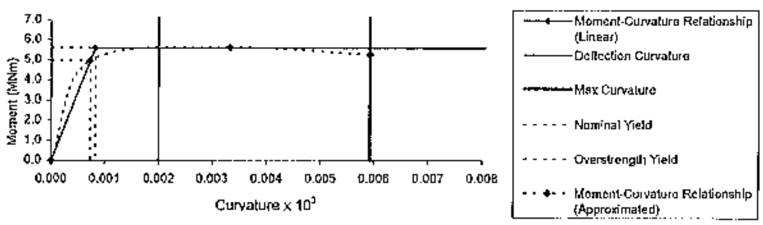
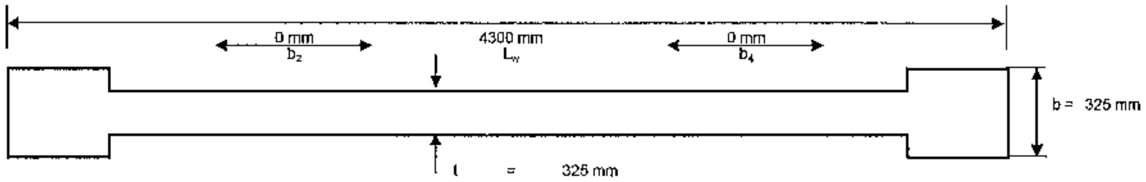


Table 2.5	Maximum available ULS structural ductility factor	(i) Two or more cantilevered	$\mu \le 5/\beta_a$	=	5.0
		(ii) Two or more coupled	$\mu \le (3A + 4)/\beta_a$	=	6.0
		(iii) Single cantilever	$\mu \le 4/\beta_a$	=	4.0

DIMENSIONS



LOADS

Moment	M^*_E	=	11851 kNm	at ductility = 3.0
Shear	$V^*_{1.25}$	=	1353 kN	at ductility = 1.25
Shear	V^*_o	=	981 kN	at ductility = 3.0
Axial	N^*_G	=	2250 kN	(G & Ψ_EQ)

PROPERTIES

f'_c	=	30 MPa	5.2.1 Concrete Strength at 28 Days
f_y	=	500 MPa	Flexural Reinforcement Yield Strength
f_{yt}	=	500 MPa	Shear Reinforcement Yield Strength

11.4.7.2 In the estimation of the maximum shear demand on a wall of limited ductility, the maximum shear need not exceed that corresponding to the elastic response of the wall element derived using μ = 1.25 and S_p = 0.9.

11.3.10.3.8 (c) Max Spacing of Horizontal Shear Reinforcement s_2 <= L_w / 5 or 3 t or 450
<= 860 mm 975 mm 450 mm

11.3.10.3.8 (b) Min Area of Horizontal Shear Reinforcement $A_{v,min}$ >= $0.7\, b_w\, s_2 / f_{yt}$ = 182 mm² (Eq 11-19)
11.3.11.3 Min & Max Area of Reinforcement ρ_{min} >= $\sqrt{f'_c} / 4 f_y$ <= 0.00274
ρ_{max} >= $16 / f_y$ <= 0.03200

CRACK CONTROL

2.4.4.4 Spacing of rebar for crack control on the tension face f_s = 180 MPa
s_{max} = $90000 / f_s - 2.5c_c$ = 389 mm (Eq 2-5)

SHEAR REINFORCEMENT

Horizontal Bar Size	d_{bt}	=	12 mm	$A_{s,bar}$	=	113 mm²	
Horizontal Bar Spacing	s_2	=	400 mm	n_{layers}	=	2	
Transverse Bar Area	$A_{v,prov}$	=	226 mm²				
		>=	182 mm²		=	$A_{v,min}$	OK
		>=	356 mm²		=	$s_2\, t\, \rho_{min}$	CHECK
		<=	4160 mm²		=	$s_2\, t\, \rho_{max}$	OK

11.3.10.3.8 (a) Shear Reinforcement Contribution V_s = $A_v\, f_{yt}\, d / s_2$ = **973 kN** **(Eq 11-18)**

11.3.10.3.4 Concrete Shear Strength - Simplified v_c = $0.17\sqrt{f'_c}$ = 0.93 MPa (Eq 11-12)
= $0.17\ [\sqrt{f'_c} + N^* / A_g\,]$ = 1.20 MPa (Eq 11-13)

11.3.10.3.5 Concrete Shear Strength - Detailed = $0.27\sqrt{f'_c} + (N^* / 4\, A_g)$ = 1.88 MPa (Eq 11-14)
= $0.05\sqrt{f'_c} + \dfrac{L_w[0.1f_c + 0.2(N^*A_g)]}{(M^*/V^*) - (Lw/2)}$ = 1.67 MPa (Eq 11-15)

9.3.9.3.3(a) & 10.3.10.2.1(a) Effective Shear Area A_{cv} = $b_w\, d$ = 1118000 mm²

11.4.7.3 Factor for ductility of plastic region λ = 0.5
$V_{c,max}$ = $(0.27\lambda\sqrt{fc} + N^*/4A_g\,)b_w d$ = 1277 kN (Eq 11-28)

11.3.10.3.8 (a) Concrete Shear Contribution V_c = $v_c\, A_{cv}$ = 1277 kN **(Eq 11-17)**

7.5.1 Max Nominal Shear Stress v_{max} = Min (0.2 f'_c, 8.0 MPa) = 6.0 MPa (7.5.2)
11.4.7.3 Overstrength Factor Φ_{ow} = $\Phi_{o,m}\, M_y / \Phi M_y$ = 1.33
Factor for ductility of plastic region α = 3.00
For walls subject to tension V_n <= $(\Phi_{ow}/\alpha + 0.15)\sqrt{f'_c}\, A_{cv}$ = 3630 kN (Eq 11-29)

7.5.3 Nominal Shear Strength V_n = $V_c + V_s$ = **2249 kN** **(Eq 7-6)**
v_n = V_n / A_{cv}
= 1.61 MPa <= 6.0 MPa OK

7.5.1 General ΦV_n >= V^* (Eq 7-4)
Nominal Ductility ΦV_n = **1687 kN** >= **1353 kN** OK
Bending Overstrength at ductility = 3.0 ΦV_n = **2249 kN** >= **981 kN** OK

USE: A 325 mm SHEAR WALL WITH 30 MPa CONCRETE STRENGTH AT 28 DAYS
REINFORCED WITH HORIZONTAL HD 12 BARS IN 2 LAYERS AT 400 mm CENTRES

TRANSVERSE REINFORCEMENT REQUIREMENTS FOR DUCTILITY IN EARTHQUAKES

11.4.6.3 Lateral restraint in plastic hinge regions A_s = 226 mm² s_v = 460 mm
p_l = $A_s / t\, s_v$ = 0.00151 (Eq 11-24)
ductile regions = $2 / f_y$ < 0.00400
limited ductile regions = $3 / f_y$ < 0.00600

11.4.6.5 Confinement requirements in the plastic hinge region λ = 1.0
c_c = $0.1\, \Phi_{ow}\, L_w / \lambda$ = 571 mm (Eq 11-25)
< 895 mm Check 11.4.6.5

(a) Rectangular hoops used in confined columns α = 0.175
h'' = 225 mm
Area of shear reinforcement to be provided A_{sh} = $\alpha s_h h''(A_g{}^* f'_c/A_c{}^* f_{yt})(c/L_w - 0.07)$ = 472 mm²/m (Eq 11-26)

(b) Length of confined region of compressed wall c' >= $c - 0.7\, c_c$ > $0.5\, c$ = 496 mm (Eq 11-27)

(d) Spacing limitation within ductile plastic regions s_{max} = Min ($6d_b$, t/2) = 72 mm
Spacing limitation within limited ductile plastic regions = Min ($10d_b$, t) = 120 mm

CHECK: SPECIFIC REQUIREMENTS OF DUCTILE & LIMITED DUCTILE REGIONS UNDER 11.4.6.5

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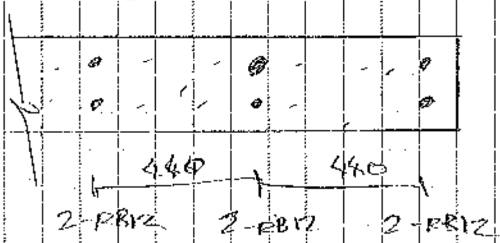
Page	C21
By	VMB
Date	NOV 11
Job Ref	E110604

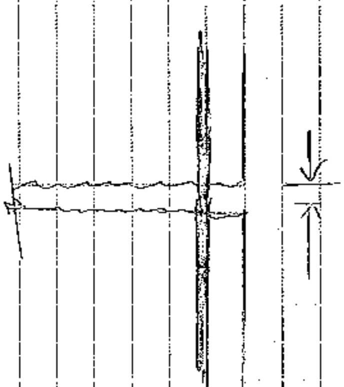
Project 62 GLOUCESTER

Description CONCRETE TENSION

GRID 3 & 4 WALL

VERTICAL REBAR





TENSILE STRENGTHS

Steel : RB12 - 440

$\phi N_{TS} = \phi A_s f_y$
$= 0.9 \times 514 mm^2/m \times 500MPa$
$= 231.3 kN/m$

Concrete : $f_{ct} = 0.36\sqrt{f'_c}$ (5.2.6)
$= 0.36 \times \sqrt{30MPa}$
$= 1.97MPa$

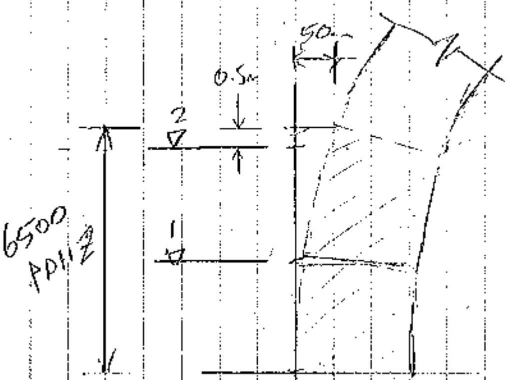
$\phi N_{TC} = 0.85 \times 1000 \times 250 \times 1.97MPa$
$= 419.0 kN/m$ $\boxed{181\% \ \phi N_{TS}}$

Concrete O/S $f_{cto} = 0.36 \times \sqrt{(30+15)} = 2.41MPa$

$N_{TCO} = 1000 \times 250 \times 2.41 = 603.7 kN/m$ $\boxed{261\% \ \phi N_{TS}}$

BAR ELONGATION

* Concentrated Cracking within PPNZ



Single Crack Width (Worst Case)

$(50/6500) \times (3680-710) = 22.8mm$

Force in Rebar

$\delta = PL/AE$

$P = \delta AE/L$

where

$A = 2-RB12 = 226mm^2$
$E = 200000MPa$
$L = 6500mm$

1ST BAR

Crack = (50/6500) × [(3680 − 710) − 50] = 22.5mm

P = 22.5 × 226 × 200000 / 6500
= 156.5 kN
> 101.7 kN = ϕN_{Ty} (2 - RB12)

2ND BARS

Crack = (50/6500) × [3680 − 710) − 490] = 19.1mm

P = 19.1 × 226 × 200000 / 6500
= 132.8 kN
> 101.7 kN

3RD BARS

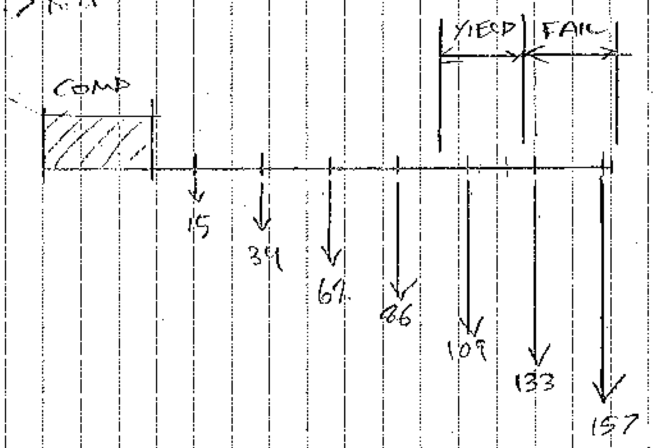
Crack = (50/6500) × [(3680 − 710) − 930] = 15.7mm

P = 15.7 × 226 × 200000/6500
= 109.1 kN
> 101.7 kN

4TH BARS

Crack = (50/6500) × [3680 − 710) − 1370] = 12.3mm

P = 12.3 × 226 × 200000/6500
= 85.6 kN
< 101.7 kN



ULTIMATE STRENGTH

2 - RB12 $N_{u,min}$ = 2 × 65.0 kN = 130.0 kN
$N_{u,max}$ = 2 × 79.0 kN = 158.0 kN

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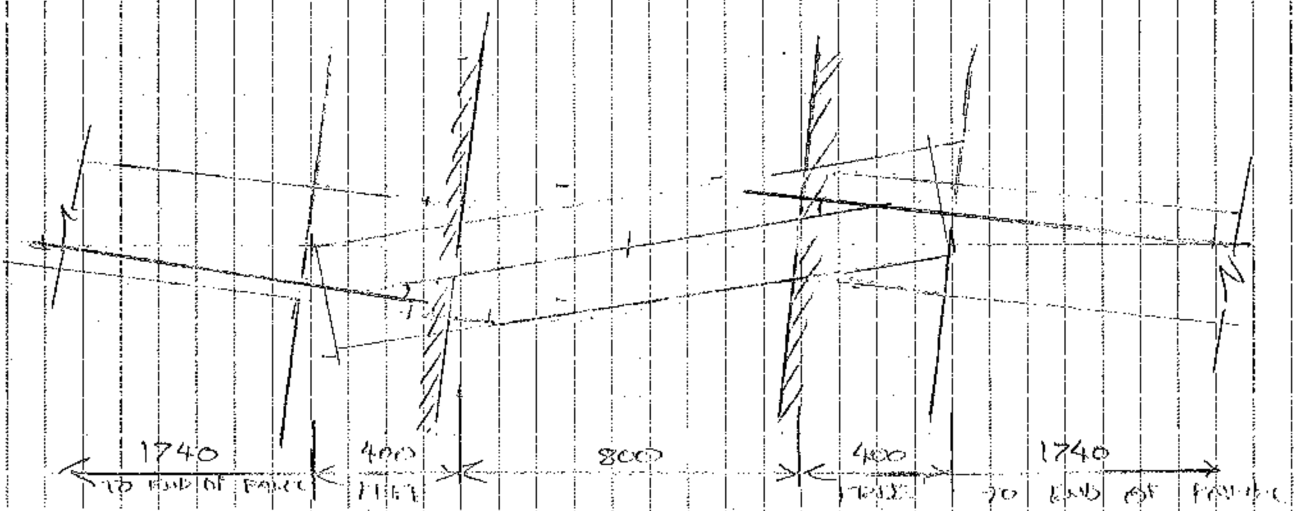
Page	C23
By	VAB
Date	NOV 11
Job Ref	0110604

Project: 62 GLOUCESTER

Description: EDGE BEAM MATERIAL STRAIN

BEAM CURVATURE BETWEEN PANELS



WALL ROTATION

θ_w = 3.4% drift
= 0.0342

⇒ Note the drift occurs and upper levels wall curvature is very low, however interstorey drift is high due to rotation.

BEAM ROTATION

$$\theta_b = \theta_w \Big/ \left[1 - \frac{h_w + 2h_b/4}{L}\right]$$

$$= 0.0342 \Big/ \left[1 - \frac{(2140 - 400) + 2 \times 310/4}{1740 + 1600}\right]$$

$$= 0.0791$$

Effective Plastic Hinge Length

$h_b/2 = 0.31\text{m}/2 = 0.155\text{m}$

Curvature

$\phi_b = \theta_b / L_p$
$= 0.0791 / 0.155$
$= 0.5105 / \text{m}$

Max Curvature

⇒ Refer NZS3101 Table 2.4(a) material strains

$K_d = 11$ (limited ductile plastic region)

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Page	C24
By	VME
Date	NOV 11
Job Ref	E1106.04

Project: 62 GLOUCESTER

Description: EDGE BEAM MATERIAL STRAIN

$\phi_{0/max} = K_d \, 2 f_y / E_s h_b$
$= 11 \times 2 \times 500 / 200000 \times 0.310$
$= 0.1774/m$
$< 0.5103/m = \phi_b$

⇒ Max curvature in the edge beams is exceeded by 188%

Assess again at design ductility of 3.0

$\theta_w = 0.0379$

$\theta_b = 0.0379 / \left[1 - \frac{(2140-400) + 2 \times 310/4}{1740 + 1600} \right] = 0.0876$

$\phi_b = 0.0876 / 0.155 = 0.5652/m$

$\phi_{0/max} = 0.1774/m << 0.5652/m$

⇒ Max curvature in the edge beams is exceeded by 219%

Check at the 1% drift stated in design features report

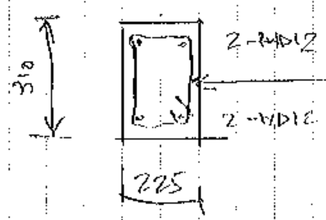
$\theta_w = 0.0100$

$\theta_b = 0.0100 / 0.4326 = 0.0231$

$\phi_b = 0.0231 / 0.155 = 0.1491 < 0.1774 = \phi_{max}$

⇒ Max curvature in the edge beams is within acceptable limit

DETAILING OF BEAM



HR10 STIRRUPS AT 450 CRS
(Dwgs are unclear as to crs, they say refer to beam elevation but there are only panel elevations which seem to show 450 crs?)

⇒ Maximum stirrup spacing for a 310 × 225 concrete beam hinge zone is:

$S_{max} = d/4, \; 6 d_b = 260/4, \; 6 \times 12$
$= 65mm$

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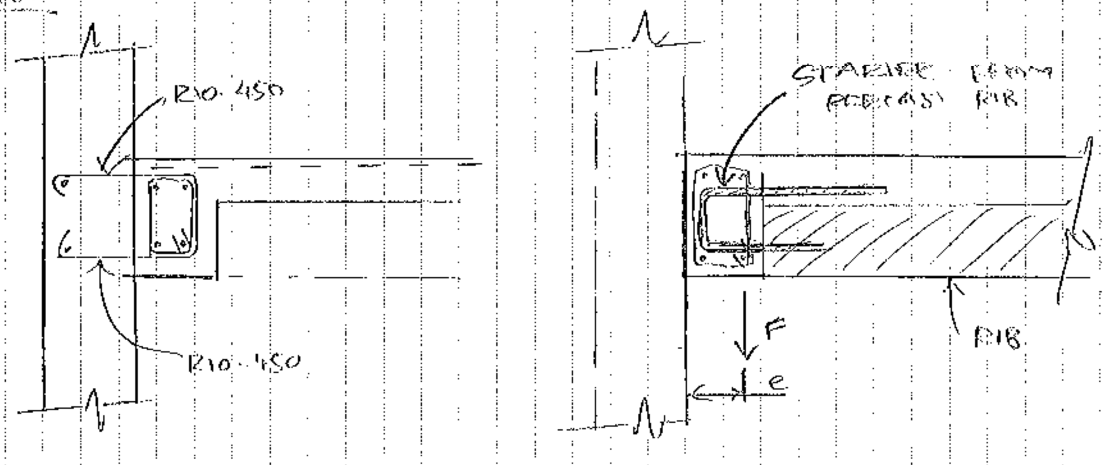
Page	
By	VME
Date	NOV 11
Job Ref	E110604

Project 62 GLOUCESTER

Description EDGE BEAM RIB SUPPORT

FLOOR CONNECTION

Perpendicular



GRAVITY LOAD

$G = 4.4\,kPa \times 0.90\,m \times 9.3\,m / 2 = 18.4\,kN$

$Q = 1.5 \times \times = 6.3\,kN$

$G \,\&\, 0.3Q = 18.4 + 0.3 \times 6.3 = 20.3\,kN/rib$

$= 22.6\,kN/m$

SEISMIC SHEAR

$V_E = 180\,kN$ Max FR wall of $\mu = 1.25$

$= 41.9\,kN/m$

INTERSPAN SUPPORT

$M_c = Fe$

$= 22.6\,kN/m \times (0.225 - 0.05)$

$= 3.96\,kNm/m$

Check: R10-450 hooked into panel

$T^* = M_c / d$

$= 3.96\,kNm/m \;/\; (0.310 - 0.05 - 0.05)$

$= 18.9\,kN/m$

$\phi N_T = \phi A_s f_y$

$= 0.9 \times (78.5 \times 1000/450) \times 300$

$= 47.1\,kN/m$

$> 18.9\,kN/m$ ✓

>> Assume concrete friction does not contribute

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Page	C25
By	VAJE
Date	Nov 11
Job Ref	E 110604

Project 62 GLOUCESTER

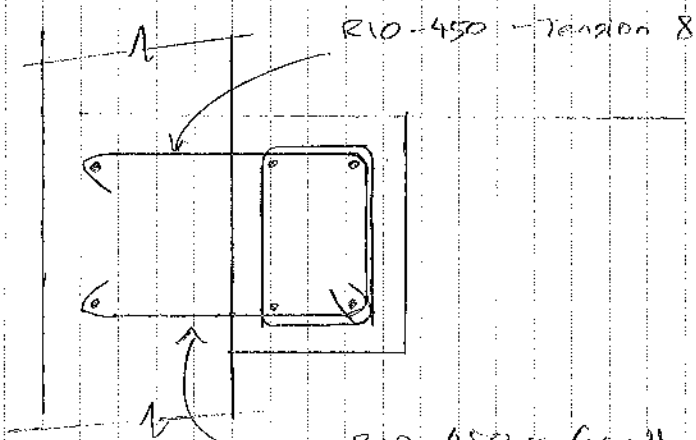
Description EDGE BEAM RIB SUPPORT

$\phi N_F = \phi\, 0.6 A_s f_y$
$= 0.8 \times 0.6 \times (78.5 \times 1000/450) \times 300$
$= 25.1$ kN
> 22.6 kN

Seismic Shear

* Much of the connection strength is used up by the gravity support of the floor. The top bars hooking into the panels have some capacity remaining to resist seismic shear as they are not at full capacity in tension.

$\phi V_F = (47.1 \text{ kN/m} - 18.9 \text{ kN/m}) \times (1.1/0.9) \times 0.8 \times 0.6$
$= 16.5$ kN/m
< 41.9 kN/m $= V_E$ 39%



Combined Action on Top Bars

$1.0 \geq (V_E^s / \phi V_F)^2 + (N_T^g / \phi N_T)^2$
$\geq (41.9 / 25.1)^2 + (18.9 / 47.1)^2$
≥ 2.95 FAILS

Check: ULS Gravity

$1.2G$ & $1.5Q$ $= 1.2 \times 18.4 + 1.5 \times 6.3 = 31.5$ kN

$M_e = 31.5 \times 0.175\text{m} = 5.51$ kNm/m

$N_T^g = 5.51$ kNm $/ 0.21$m $= 26.3$ kN

Top $\phi N_T = 47.1$ kN/m > 26.3 kN/m OK

Bot $\phi V_F = 25.1$ kN/m < 31.5 kN/m Top Bars Take Shear

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Page	C26
By	VME
Date	Nov 11
Job Ref	E110604

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Description [illegible] BEAM RIB SUPPORT

Check: Combined actn in top bars

$1.0 \geq (26.3/47.1)^2 + [(31.5 - 25.1)/25.1]^2$

≥ 0.38 OK

INTERSPAN TO INSITU EDGE BEAM

⇒ Drawings do not specify what the starter bar is that connects to the edge beam.

TOPPING SLAB TO INSITU EDGE BEAM

⇒ There are not starters from the beam to the slab

⇒ 665 mesh runs out to the panel.

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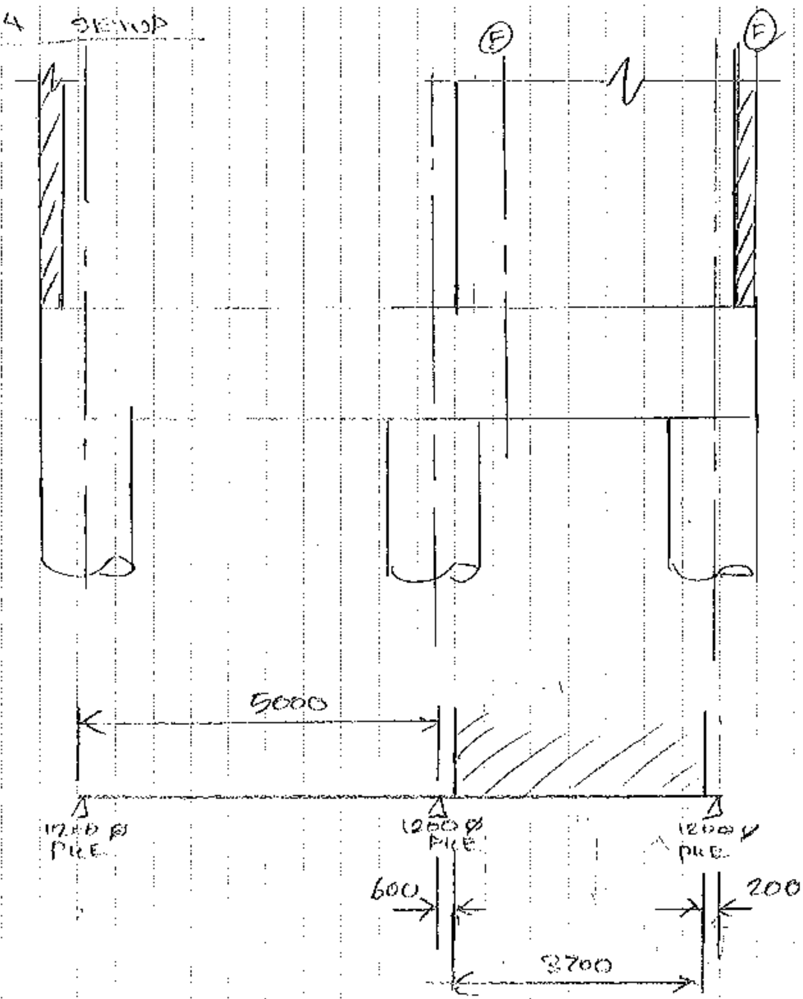
Page	C27
By	VME
Date	NOV 11
Job Ref	E110604

Project 62 GLOUCESTER

Description FOUNDATION BEAM

GRID 3 & 4 SECTION



Conc Shear

$v_b = (0.07 + 10\rho_w)\sqrt{f'_c}$
$= (0.07 + 10 \times 4825/1000 \times 1380)\sqrt{30}$
$= 0.57 MPa$

$V_c = 0.57 MPa \times 1000 \times 1380 = 793 kN$

Steel Shear

$V_s = A_v f_{sy} d / s$

300 $= 804 \times 500 \times 1380 / 300 = 1850 kN$
600 $= \quad / 600 = 925 kN$

Total Shear Overstrength

$\phi = 1.0$

$V = V_s + V_c = 793 + 1850 = 2643 kN$ (300)
$= 793 + 925 = 1718 kN$ (600)

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Page	C28
By	VME
Date	18/11/2011
Job Ref	E110604

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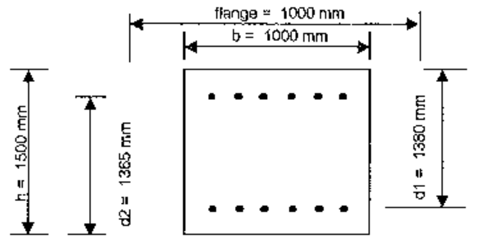
Description GRID 3 & 4 FOUNDATION BEAM

GRID 2-3 & 7-8



No Mesh
6 - HD32
0 - D20
0 - D20

0 - D20
0 - D20
6 - HD32

OVERSTRENGTH LOADS

M^* = 2746 kNm (Space-Gass Output)
M^* = 2016 kNm (at Nominal Ductility)

1500 mm	deep by	1000 mm wide
500 MPa	Bars	
16 mm	Stirrups	
485 MPa	Mesh	($A_{s,mesh}$ = 0 mm²/m)
30 MPa	Concrete	
103 mm	Top Cover	
88 mm	Bottom Cover	

Longitudinal Bar Requirements of NZS 3101:2006

$$A_{s,min} = \frac{\sqrt{f'_c}\, b_w d}{4 f_y} = \frac{\sqrt{30\ \text{MPa}} \times 1000\ \text{mm} \times 1380\ \text{mm}}{4 \times 500\ \text{MPa}} = 3779\ \text{mm}^2 \qquad \text{(Eq 9-1)}$$

Moment Capacity:

$A_{s,top}$ = 4825 mm² >= 3779 mm² = $A_{s,min}$ ($A_{s,mesh}$ = 0 mm²)
$A_{s,bot}$ = 4825 mm² >= 3779 mm² = $A_{s,min}$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4825\ \text{mm}^2 \times 500\ \text{MPa}}{0.85 \times 30\ \text{MPa} \times 1000\ \text{mm}} = 94.6\ \text{mm} \quad \text{(top)}$$

$$= \frac{4825\ \text{mm}^2 \times 500\ \text{MPa}}{0.85 \times 30\ \text{MPa} \times 1000\ \text{mm}} = 94.6\ \text{mm} \quad \text{(bottom)}$$

$\phi M_n = \phi A_s f_y (d - a/2)$

$\phi M_{n,top}$ =	0.85	x	4825 mm²	x	500 MPa	x	(d2 - a/2)	=	2702 kNm		OK
$\phi M_{n,bot}$ =	0.85	x	4825 mm²	x	500 MPa	x	(d1 - a/2)	=	2733 kNm		OK

SHEAR RESISTED BY CONCRETE

$v_c = k_d k_a v_b$ (Eq 9-5)
$v_b = (0.07 + 10\rho_w) \sqrt{f'_c}$ = (0.07 + 10 x (4825 mm² / 1000 mm x 1380 mm) x $\sqrt{30\ \text{MPa}}$
= 0.57 MPa

$V_c = v_c A_{cv}$ = 0.57 MPa x 1000 mm x 1380 mm = 793 kN (Eq 9-4)

NOMINAL SHEAR STRENGTH TO BE PROVIDED BY STIRRUPS

At 'd': $V^*_{b,GOE}$ = 2145 kN
$V_s = V^*/\phi - V_c$ = 2145 kN / 0.75 - 793 kN = 2067 kN (Eq 9-6)

At '2h': $V^*_{b,GOE}$ = 2000 kN
$V_s = V^*/\phi - V_c$ = 2000 kN / 0.75 - 793 kN = 1873 kN (Eq 9-6)

AREA OF STIRRUPS REQUIRED FOR NOMINAL STRENGTH

At 'd': $A_v >= s V_s / d f_{yt}$ = 1000 x 2067 kN / 1380 mm x 500 MPa = 2995 mm²/m
At '2h': $A_v >= s V_s / d f_{yt}$ = 1000 x 1873 kN / 1380 mm x 500 MPa = 2715 mm²/m

MINIMUM AREA OF STIRRUPS TO BE PROVIDED

At 'd': $A_v^{min} >= \sqrt{f'_c}\, b_w s / 12 f_{yt}$ = $\frac{\sqrt{30\ \text{MPa}} \times 1000\ \text{mm} \times 1000\ \text{mm}}{12 \times 500\ \text{MPa}}$ = 913 mm²/m (Eq 9-10)

At '2h': $A_v^{min} >= \sqrt{f'_c}\, b_w s / 16 f_{yt}$ = $\frac{\sqrt{30\ \text{MPa}} \times 1000\ \text{mm} \times 1000\ \text{mm}}{16 \times 500\ \text{MPa}}$ = 685 mm²/m (Eq 9-27)

STIRRUP SPACING REQUIREMENTS

At 'd': s_{max} = Min (d/3, 10d_b) = 320 mm

$A_{te} >= \Sigma A_b f_y s / 96 f_{yt} d_b$ = $\frac{2413\ \text{mm}^2 \times 500\ \text{MPa} \times 300\ \text{mm}}{96 \times 500\ \text{MPa} \times 32\ \text{mm}}$ = 236 mm²/m (Eq 9-28)

At '2h': s_{max} = Min (0.5d, 600) = 600 mm

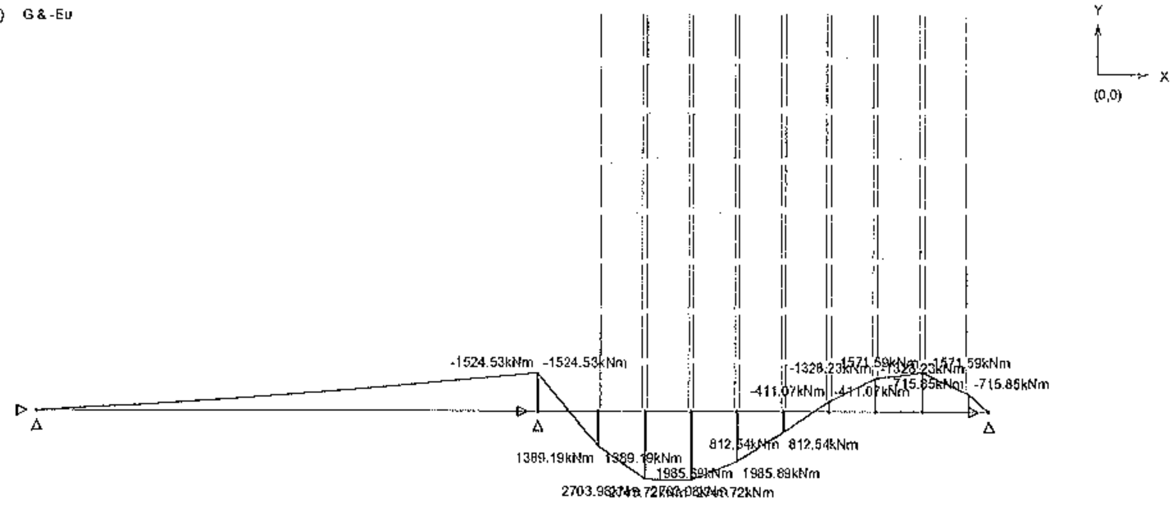
At 'd' ***CHECK: 4-HR16 STIRRUP LEGS AT 300 CRS***
A_v^{prov} = 804 mm² x 1000 mm / 300 mm = 2681 mm² /m >= 2995 mm² /m 90% CHECK

At '2h' ***CHECK: 4-HR16 STIRRUP LEGS AT 600 CRS***
A_v^{prov} = 804 mm² x 1000 mm / 600 mm = 1340 mm² /m >= 2715 mm² /m 49% CHECK

C30

SPACE GASS 10.85 - SPENCER HOLMES LTD 18 Nov 2011, 9.47 am

Load cases:
32 (SW) G & -Eu



$\mu = 1.0$

$M^* = 2746 \times 0.069/0.094 = 2016$ at $\mu = 1.25$

No general restraint

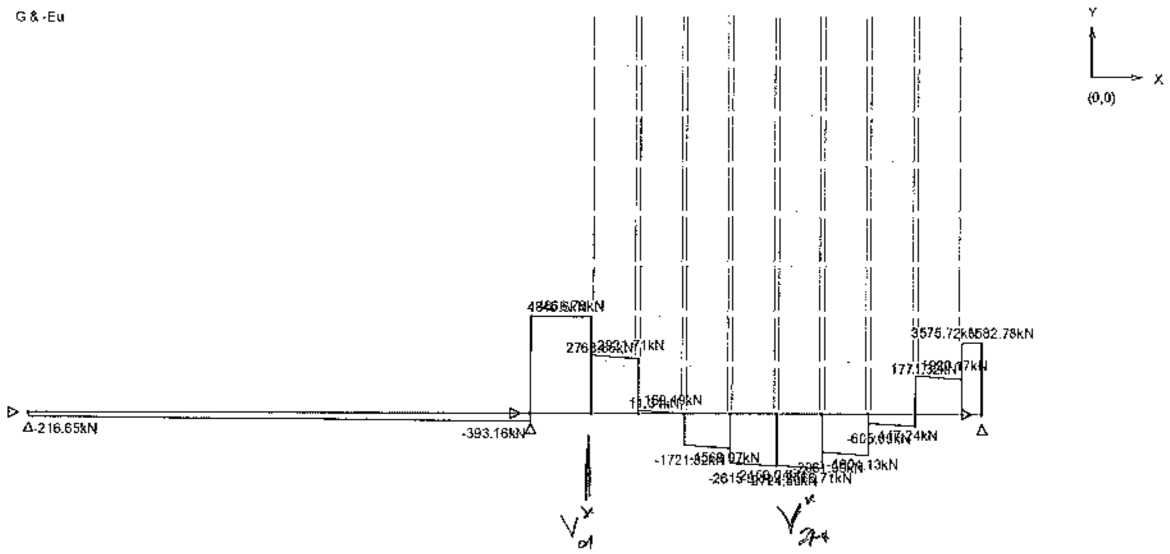
Materials:
1 25CRK30
2 CONCRETE-25

Sections:
1 FB-1500x1000

Job: G:\2011jobs\110600...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4 Fdn
Filter: Fdn
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:50, Load: None, Disp: None, Moment: 200, Shear: None, Axial: None, Torsion: None

SPACE GASS 10.85 - SPENCER HOLMES LTD 18 Nov 2011, 9.47 am

Load cases:
32 (SW) G & -Eu



$V^*_d = 2922 \times 0.069/0.094 = 2145$ kN at $\mu = 1.25$

$V^*_{2d} = 2725 \times$ " $= 2000$ kN at $\mu = 1.25$

No general restraint

Materials:
1 25CRK30
2 CONCRETE-25

Sections:
1 FB-1500x1000

Job: G:\2011jobs\110600...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4 Fdn
Filter: Fdn
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:50, Load: None, Disp: None, Moment: None, Shear: 250, Axial: None, Torsion: None

BUI.GLO62.0003.55

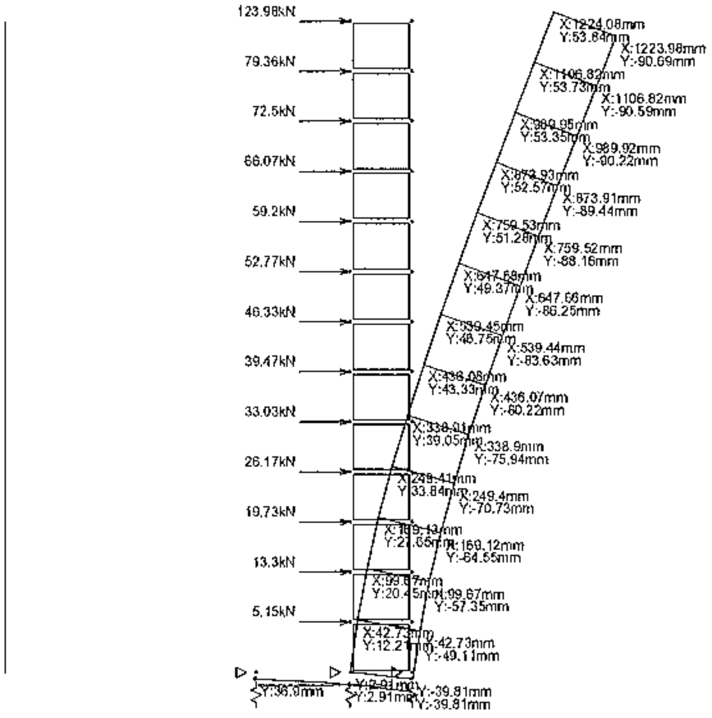
C3(

SPACE GASS 10.85 - SPENCER HOLMES LTD

16 Nov 2011, 1:47 pm

Load cases:
2 Eu (ductility 1.0, T=2.97s)

Y
X
(0,0)



SPRING FDNS

No general restraint

Materials:
1 25CRK30

Sections:
1 FB-1500x1000

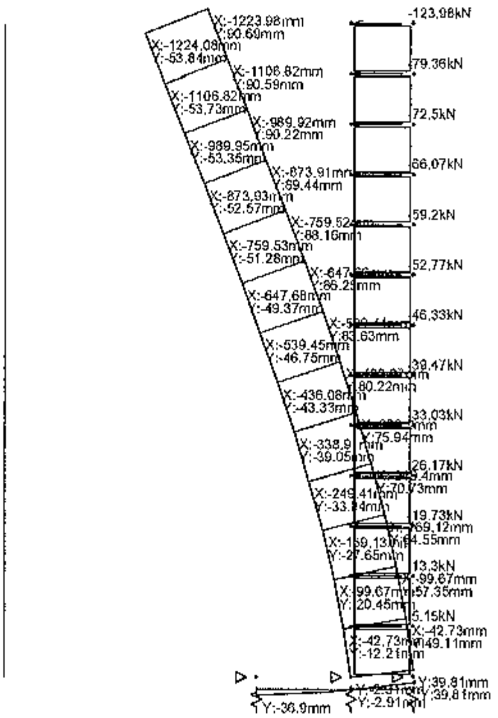
Job: G:\2011jobs\110...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4 Spring
Filter: Wall
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 5.960464, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

SPACE GASS 10.85 - SPENCER HOLMES LTD

16 Nov 2011, 1:47 pm

Load cases:
12 -Eu (ductility 1.0, T=2.97s)

Y
X
(0,0)



No general restraint

Materials:
1 25CRK30

Sections:
1 FB-1500x1000

Job: G:\2011jobs\110...\62 Gloucester St\110604 62 Gloucester Grid 3 & 4 Spring
Filter: Wall
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:300, Load: 5.960464, Disp: 10, Moment: None, Shear: None, Axial: None, Torsion: None

APPENDIX 6

Holmes Solutions Report "107267-1 v1.1, Materials Testing in Buildings of Interest, November 2011"

HOLMESSOLUTIONS

REPORT 107267-1 (v1.1)

PREPARED FOR ROYAL COMMISSION

NOVEMBER 2011

MATERIALS TESTING IN BUILDINGS OF INTEREST

GALLERY APARTMENTS
WESTPAC CENTRE
IRD BUILDING

HOLMESSOLUTIONS

DISCLAIMER

This document was prepared by Holmes Solutions Ltd (HSL) under contract. The information presented in this document relates to non-destructive structural load testing and does not address any other related or un-related issues, including but not limited to environmental durability of the product, nor applications for the tested product. It is the responsibility of the user to assess relevant performance of the product and determine suitable applications.

This document does not constitute a standard, specification, or regulation. In undertaking the testing described in this report, Holmes Solutions have exercised the degree of skill, care, and diligence normally expected of a competent testing agency. The name of specific products or manufacturers listed herein does not imply endorsement of those products or manufacturers.

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Rev No.	Date	Revision
V1.0	15/11/11	Issued for client review

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1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes significant structural damage was noted in a large number of buildings in the Christchurch CBD. In particular, a number of buildings appear to have undergone greater damage than previously expected. The Royal Commission appointed an engineering team to review the damage in a number of building in the CBD in an effort to gain a greater understanding of the buildings behaviour under the induced seismic loads. From this investigation, a series of three buildings were identified as requiring materials testing to be completed, namely the Gallery Apartments on Glouster St, the Westpac Centre on Cashel St, and the IRD building on Cashel St. Holmes Solutions was commissioned to undertake the required materials testing.

All three buildings requiring investigation are reinforced concrete, with a mixture of precast concrete and in-situ cast concrete elements. The Royal Commission requested a series of destructive and non-destructive testing to be completed on the concrete and reinforcing steel used in the buildings. Furthermore, Holmes Solutions was independently engaged by external third parties working for the owners of the building to undertake additional testing on the reinforcing steel in the Westpac Centre and IRD building.

Testing of the concrete elements included the removal of concrete cores for destructive testing to determine the tensile and compressive properties of the concrete. Additional non-destructive testing was completed using Schmidt Hammer testing in the buildings.

The material properties of the reinforcing steel were investigated in zones of damage in the building, to determine the likely damage the earthquake has induced in the steel, and control samples in areas away from any noted damage. The use of Leeb Hardness testing has been shown to provide a strong correlation with the peak strain the steel has been subjected to during in-elastic loading cycles and is become increasingly adopted as a tool for assessing structural damage.

The results from the testing indicated that the reinforcing steel in the Westpac Centre had undergone previous inelastic strain cycles of between 2% and 8%. The reinforcing steel testing in the IRD building showed significant reduction in strain capacity with only 2% strain capacity remaining.

Concrete strength results for the Gallery Apartments indicated that the walls had compressive strengths of 46 MPa to 56 MPa, with associated tensile strengths ranges from 3.4 MPa to 2.6 MPa respectively.

No significant variations in concrete strengths were noted between the precast and in-situ concrete items in the Westpac Centre.

Concrete results from the IRD indicated that the precast concrete was stronger than the in-situ concrete elements by approximately 10 MPa.

2.0 TEST METHODOLOGY

2.1. CONCRETE CORE TESTING

A series of concrete core samples, approximately 100 mm in diameter, were removed from elements in the Gallery Apartment and the Westpac Centre. The cores were removed using a diamond tipped drilling head. Wherever possible, samples were taken from areas showing no physical damage and remote from reinforcing steel embedded in the concrete. If a reinforcing bar was impacted by the drilling head, the sample was discarded and an alternative sample taken from a nearby position. Prior to removing the core, the orientation of the sample was clearly identified to allow the subsequent testing to be undertaken in the correct orientation.

The concrete cores were subjected to either tensile splitting tests or compression testing. All tensile splitting tests were performed to the specific requirements of NZS 3112: 1986, Pt 2, Clause 8. Care was taken to ensure the samples were oriented as per location in the building. All samples were prepared in accordance with the standard prior to completion of the testing.

All concrete cores subjected to compression testing were firstly capped, in accordance with the requirement of NZS 3112: Part 2: 1986, clause 4. Once the capping material had achieved the required hardness the samples were tested in accordance to NZS 3112: Part 2: 1986, Clause 6.

2.2. TENSILE STEEL TESTS

A series of steel samples, approximately 500 mm long were removed from the Westpac Centre and the IRD building. Steel samples from the Westpac centre were obtained from zones of noted damage in the building and additional samples collected from areas that appeared to be free of visual damage to act as control samples and provide a true measure of the stress-strain properties of the parent steel. Prior to their removal from the Westpac Centre, all steel bars were subjected to Leeb Hardness testing in-situ.

2.3. LEEB HARDNESS

Leeb hardness is a direct measure of a materials dynamic hardness and is considered to be accurately measuring the materials elastic and plastic hardness characteristics. Leeb hardness is obtained by firing an impact body containing a permanent magnet and a very hard indenter sphere towards the surface of the test material and measuring the velocity of the impact body. The velocity is measured in three main test phases;

Pre-impact phase~~, where the impact body is accelerated by spring force towards~~ the surface of the test piece.

Impact phase, wh~~ere the impact body and the test piece are in contact. The hard~~ indenter tip deforms the test material elastically and plastically and is deformed itself elastically. After the impact body is fully stopped, elastic recovery of the test material and the impact body takes place and causes the rebound of the impact body.

Rebound phase, ~~where the impact body leaves the test piece with residual energy,~~ not consumed during the impact phase.

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The Leeb hardness is determined by calculation, relating the three recorded velocities. The velocities are measured in a contact-free means via the induction voltage generated by the moving magnet through a defined induction coil mounted on the guide tube of the device. The induced voltage is directly proportional to the velocity of the magnet and therefore used to determine the hardness of the steel sample.

Recent research has shown that hardness can be used as an indicator of the current strain state of steel samples [G1, L1, M2, N2, N3]. Relating the hardness of steel samples to the stress-strain properties of the base material allows an understanding of likely damage (or loss of strain capacity) that the steel sample has sustained and therefore to determine how much residual strain capacity the sample retains. This form of direct comparison can only be achieved if suitable correlations are developed between the measured hardness and the strain state of the specific steel sample.

Holmes Solutions has completed extensive research into the correlation between Leeb hardness and the steel samples strain state for a range of different reinforcing steels. The results from the research have been developed into a series of multi-dimensional correlation factors. When combined with a series of normalisation techniques we can use the measured Leeb hardness results to provide an indication as to the current strain state of the tested steel sample. The degree of uncertainty in the recorded measurements is decreased through the physical testing of a control section of the steel to a uniaxial tension test and undertaking hardness measurements at a series of predefined stress and strains. The resulting correlation is used, in conjunction with the normalisation techniques derived from obtaining numerous hardness readings in the area surrounding the expected zone of damage, to determine the value of strain in the steel from the recorded Leeb measurements. These results are then directly compared to the properties of the parent material to estimate the potential reduction in strain capacity that has been sustained by the steel sample.

Leeb readings are collected from in-situ reinforcing bars. The surface of the bars is carefully prepared to specific requirements prior to testing. Readings are obtained at critical locations along the length of the reinforcing bar to allow the strain profile of the steel to be determined and to assist in the normalisation procedures.

The overall estimation of strain degradation for the tested steel samples is achieved by using the derived strain damage from the Leeb testing in conjunction with engineering knowledge of the particular application.

All in situ hardness testing is completed in accordance with ASTM A959-06 Standard Test Methods for Leeb Hardness Testing of Steel Products [A2]. For all locations, a minimum of 6 individual hardness tests were completed with the results averaged to obtain the recorded Leeb value [A1]. All recorded values were then normalised using the derived multi-dimensional correlation factors.

2.4. CONCRETE REBOUND HARDNESS

Concrete hardness is often used as a non-destructive means of determining the compressive strength of concrete. The most common method employed is the rebound hardness, obtained from a portable Schmidt Hammer. The Schmidt hammer works using a similar principle to the Leeb Hardness measurements, whereby a weight is impacted on the surface of the material and the change in velocity between the impact speed and rebound speed is determined. Correlations are then applied to convert the change in speed to hardness and compressive strength.

As with the Leeb Hardness measurements, increased accuracy in the obtained results is achieved if the hardness measurements can be directly correlated against

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the specific material being tested, by completing destructive materials testing on samples of the material. This s typically achieved by removing core samples from the structure and subjecting them to compressive testing. However, if no materials testing is completed, standard conversion tables can be used to form the correlations, with an associated reduction in accuracy.

The correlations for the Gallery Apartments and Westpac Centre were completed using the results from the physical testing of concrete core samples removed the buildings. No cores could be removed from the IRD building and as such the standard lower 10 percentile strength curves specifically developed for the instrument used in the testing. The curves were derived from testing of over 2,300 discrete locations. Use of the lower 10 percentile curve is recommended by the leading Standards, EN 13791 and ASTM C805/ACI 228.1.

In each tested location, a grid of readings were recorded. The results from the grid of readings were then averaged to provide the concrete hardness and associated concrete strength of that location. This testing method is endorsed by most International Testing Standards, and the manufacturers of the test equipment.

Steel samples from the IRD building were supplied to HSL by the engineers who designed the building. The steel samples were taken from a damaged zone in the central core of the building. Leeb Hardness testing was completed on the steel samples prior to the completion of the physical tensile testing.

All tensile testing was completed to the requirements of ASTM E8/ E8M:08.

3.0 TEST EQUIPMENT

3.1. LEEB HARDNESS TESTER

A Proceq Equotip 3 portable hardness tester was used to collect all material hardness values. The device is generally acknowledged as the industry standard for the determination of Leeb hardness. The hardness tester was installed with a DL impact device, allowing measurements on smaller diameter steel samples than the conventional D device.

The Equotip 3 has a reported accuracy of ±4 HL and is traceably calibrated to NIST standards.

3.2. SILVERSCHMIDT HAMMER

A Proceq Silverschmidt Rebound Hammer was used to undertake all field based concrete hardness testing for concretes of compressive strength ranging from 10 to 100 MPa. This device and methodology generally accepted as the industry leading device for determining the compressive strength of concrete in-situ.

The Proceq Silverschmidt was fitted with the N-Type rebound hammer providing test impact energy of 2.207 Nm.

3.3. UNIVERSAL TEST MACHINE

A UH600 Shimazu servo-controlled Universal Test Machine (UTM) with a 600 kN capacity was used to undertake all laboratory based materials testing. The UTM has a maximum stroke of 250 mm and a peak table velocity of 150 mm/min.

Steel Elongation was recorded using a strain gauge based digital extensometer with a gauge length of 50 mm. Applied loads were recorded directly using the internal pressure transducer of the Shimazu control system.

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4.0 GALLERY APARTMENT RESULTS

4.1. CONCRETE RESULTS

A series of four concrete cores were removed from the concrete shear wall elements towards the front of the Gallery apartments. Two cores were subjected to uniaxial compression testing whilst the remaining two cores were subjected to split cylinder testing in order to determine the tensile properties of the concrete. The results from the physical testing on the cores are presented below.

Table 1 Compressive Cylinder results for the Gallery Apartment

Specimen Name		RWRC	FWRC
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	93.9
Average core length (upon receipt)	(mm)	255.6	254.8
Average core length (after docking)	(mm)	190.0	187.6
Mass of core prior to capping	(g)	3191	3098
Density	(kg/m³)	2421	2387
Height diameter ratio		2.02	2.0
Conditioning		Air dried	Air dried
Load at Failure	(kN)	388.8	322.1
Compressive Strength	**(MPa)**	**56.0**	**46.5**
Type of fracture		column	Shear

Table 2 Split Cylinder results for the Gallery Apartment

Specimen Name		RWLC	FWLC
Date Tested		11 Nov 2011	11 Nov 2011
Age	(days)	Unknown	Unknown
Defects in cylinder		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.6	94.0
Average length	(mm)	189.5	167.5
Mass of cylinder in air	(g)	3133	2742
Density	(kg/m³)	2400	2380
Height diameter ratio		2.02	1.78
Conditioning		Air dried	Air dried
Tensile Strength	**(MPa)**	**2.4**	**3.4**

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In addition to physical testing, a series of Schmidt hammer tests were completed in additional locations surrounding the noted zones of damage in the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 3 Schmidt Hammer test results for Gallery Apartments

location: Front Wall - Left Side

	1	2	3	4
A	73	71.5	72	
B	67	73.5	77	72
C	72.5	72.5	71.5	70
D		60	70.5	72

Correct Average:	71.8	
Cube Strength:	87.1	MPa
Cylinder Strength, f'c:	**70.0**	**MPa**

location: Front Wall - Right Side

	1	2	3	4
A	68.5	70.5	67.5	
B	73.5	71	71	71.5
C	72	75.5	66.5	70.5
D		72	70.5	73.5

Correct Average:	70.8	
Cube Strength:	82.8	MPa
Cylinder Strength, f'c:	**66.0**	**MPa**

location: Rear Wall - Left Side

	1	2	3	4
A	75	73	62.5	
B	65	70.5	75	68.5
C	67.5	67.5	69.5	62.5
D		68.5	70.5	68.5

Correct Average:	70.0	
Cube Strength:	80.2	MPa
Cylinder Strength, f'c:	**63.0**	**MPa**

location: Rear Wall - Right Side

	1	2	3	4
A	73	65	72	
B	69.5	65	65	61
C	69.5	64	65	64.5
D		58.5	74	63

Correct Average:	66.2	
Cube Strength:	67.4	MPa
Cylinder Strength, f'c:	**54.0**	**MPa**

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Figure 1 *Drilling concrete core from Gallery Apartments*

Figure 2 Core removed from Gallery Apartment Wall

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Figure 3 *Test locations on Front Wall of Gallery Appartments*

Figure 4 Schmidt Hammer test location GAFLS

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5.0 WESTPAC CENTRE RESULTS

5.1. CONCRETE RESULTS

A series of 6 concrete cores were removed from the concrete elements, all of which were subjected to compression testing. Two of the cores were removed from precast beams, two from column elements, and the remaining two were extracted from the in-situ walls. The results from the physical testing on the cores are presented below.

Table 4 Compressive Cylinder results for the Precast beams in Westpac Centre

Specimen Name		**Precast Beam 2**	**Precast Beam 3**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.8	93.9
Average core length (upon receipt)	(mm)	227.3	211.0
Average core length (after docking)	(mm)	192.0	188.1
Mass of core prior to capping	(g)	3032	2920
Density	(kg/m³)	2311	2253
Height diameter ratio		2.05	2.00
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	149.5
Compressive Strength	**(MPa)**	**23.0**	**21.5**
Type of fracture		shear	shear

Table 5 Compressive Cylinder results for the In-situ walls in Westpac Centre

Specimen Name		**In-situ wall - Bottom**	**In-situ wall - Top**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.7	94.1
Average core length (upon receipt)	(mm)	234.5	218.5
Average core length (after docking)	(mm)	191.1	193.1
Mass of core prior to capping	(g)	3028	3068
Density	(kg/m³)	2315	2305
Height diameter ratio		2.04	2.05
Conditioning		Air dried	Air dried
Load at Failure	(kN)	134.5	119.2
Compressive Strength	**(MPa)**	**19.5**	**17.0**
Type of fracture		column	shear

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Table 6 Compressive Cylinder results for the Circular columns in Westpac Centre

Specimen Name		**Column 1**	**Column 2**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	94.2
Average core length (upon receipt)	(mm)	223.1	154.8
Average core length (after docking)	(mm)	185	123
Mass of core prior to capping	(g)	3074	1992
Density	(kg/m³)	2394	2344
Height diameter ratio		1.97	1.31
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	224.2
Compressive Strength	**(MPa)**	**23.0**	**32.0**
Type of fracture		column	shear

Schmidt hammer tests were also completed on the various concrete elements in the building. All tests were completed in zones remote from where the concrete cylinders were extracted from the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 7 Schmidt Hammer results for the Precast beams in Westpac Centre

location: Precast Beam

	1	**2**	**3**	**4**
A	65.5	53	56	56.5
B	57	63	56.5	54.5
C	54	62	58.5	58
D	67	60	45.5	52

Correct Average:	56.8	
Cube Strength:	42.4	MPa
Cylinder Strength, f'c:	**34.0**	**MPa**

Table 8 Schmidt Hammer results for the Columns in Westpac Centre

location: Column Level 3

	1	**2**	**3**	**4**
A	57	64.5	58.5	56
B	56.5	63.5	63	54.5
C	61	58	56.5	60
D	58.5	56	64	57.5

Correct Average:	58.6	
Cube Strength:	46.2	MPa
Cylinder Strength, f'c:	**37.0**	**MPa**

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Figure 5 *Core Drilling in concrete column*

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Figure 6 *Core and Schmidt hammer location on Wall element*

Figure 7 *Core location on Wall element*

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Table 9 Schmidt Hammer results for the In-situ Wall elements of Westpac Centre

location: Basement Wall

	1	2	3	4
A	68.5	64	60.5	60.5
B	67	620	65.5	64
C	58	57	68	64
D	61.5	58	65	62.5

Correct Average:	62.9	
Cube Strength:	56.9	MPa
Cylinder Strength, f'c:	**46.0**	**MPa**

location: Level 3 wall - RHS

	1	2	3	4
A	67	71	68	57
B	56	56	54	62
C	66	57.5	59.5	70.5
D	61	63.5	54	69.5

Correct Average:	62.1	
Cube Strength:	55.7	MPa
Cylinder Strength, f'c:	**45.0**	**MPa**

location: Level 3 wall - LHS

	1	2	3	4
A	57.5	61.5	66	66.5
B	59	52	66	73
C	55.5	61	52	55.5
D	53	60.5	58	57

Correct Average:	59.8	
Cube Strength:	49.6	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

5.2. STEEL RESULTS

Four 16 mm diameter reinforcing bars were removed from the insitu concrete walls of the structure and subjected to uniaxial tensile testing in the laboratory. Two of the bars were retrieved from areas in the building considered to have sustained little or no damage during the recent earthquakes. As such the material properties obtained from these sample can be assumed to have been unmodified from previous inelastic strain cycles. One of the bars was from the horizontal reinforcing and the other formed an element of vertical reinforcing in the wall

The obtained stress-strain responses of the two undamaged steel samples are shown in *Figure 10* below. The steel samples were subjected to unidirectional cyclic tensile testing rather than cycles of reverse cyclic loading to near equal values of tensile and compressive strain. In the structural element, under imposed lateral loads the neutral axis is likely to have been located near the location of the reinforcing steel during the compression load cycle, and as such the steel would have been subjected to very small induced compressive strains. During the reverse loading cycle the steel located at or near a crack in the concrete section is likely to have been subjected to disproportionately larger tensile strains, thereby significantly skewing the strain profile experienced by the reinforcing steel into the tension domain. Due to the skewed strain profile, it is believed that the unidirectional cyclic tensile test provides an adequate representation of the strains induced in the steel during a seismic event.

Leeb Hardness testing was also completed on the steel samples at various levels of applied strain, both with the load applied and with the load removed from the steel. The points of inspection can be observed in the recorded stress-strain response as areas of load cycling.

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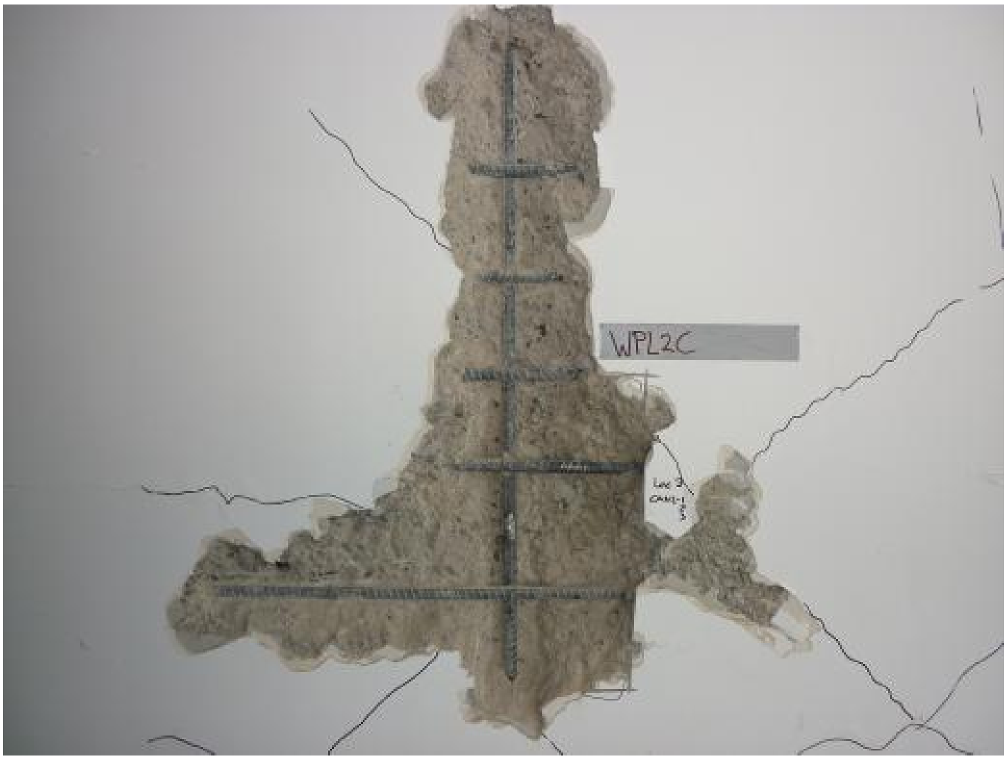
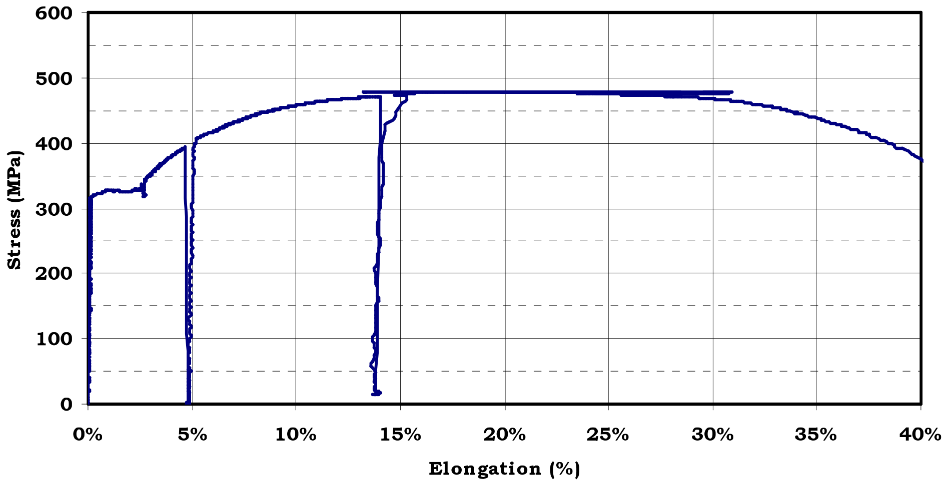
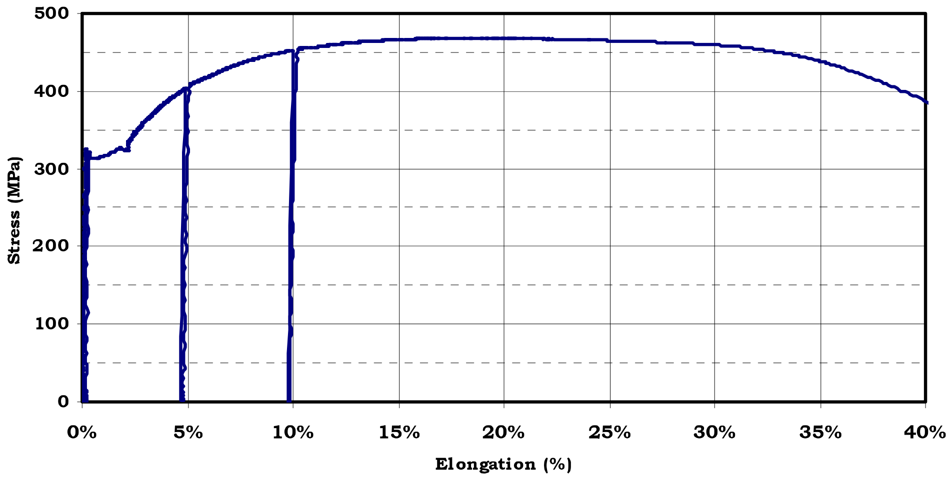


Figure 8 *Exposed reinforcing steel in zone of damage in wall element*

Figure 9 *Exposed reinforcing steel in zone of damage in wall element*



a) Horizontal reinforcing bar sample



b) Vertical reinforcing steel sample

Figure 10 Materials Test Result for the Steel test coupons obtained from undamaged area in the Westpac Centre

The steel samples had an average recorded yield stress (f_y) of 320 MPa and an average maximum recorded stress (f_u) of 472 MPa. The strain hardening ratio (f_u/f_y) of the tested steel sample was defined as 1.475. This value of strain hardening ratio indicates that the steel has a good likelihood of spreading the zone of yield along the bar, a beneficial property for limiting the potential damage at a localised zone of damage in a reinforced concrete member. It also indicates that the steel has a high plastic hardness and therefore it likely to provide suitable variation in Leeb hardness values for various levels of imposed strain.

The recorded Leeb hardness for the steel samples, and the associated stress and strain at the point of testing are reported below. A series of 6 individual Leeb hardness test results were taken and averaged to produce the reported value of Recorded Average Leeb. The recorded Leeb values for the steel show a good variation across the stress range. This is a result of the relatively high plastic stiffness of the material, defined by the extent of strain hardening observed in the recorded stress-strain plot of the tested samples.

The reported values of Leeb hardness were derived for the steel sample supported in the universal testing machine. Additional hardness tests were also completed on the tested steel sample with the bar fully supported in a mortar matrix. Based on the Leeb Hardness results obtain, the reinforcing steel used in the building appears to have a base Leeb Hardness of 610 DLHL.

Table 10 Baseline Material Strength Results for Test Sample 1

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
61.0	0.5	303	610
80.0	5.0	398	650
95.0	14.0	472	680

Table 11 Baseline Material Strength Results for Test Sample 2

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
63.0	0.5	313	612
80.0	4.5	398	650
91.0	11.0	453	670

Leeb Hardness testing was completed on a further 2 horizontal bar and two vertical bar located in zones of heavy damage in the in-situ wall of the building. The results from the Leeb Hardness are presented below.

The Leeb hardness results for the Vertical Bar 2 shows a peak elevated hardness value of 660 DLHL approximately mid way along the length of tested steel. This zone of elevated hardness coincides with the location where the reinforcing bar crosses a significant crack in the wall element. The zone of elevated hardness occurs over a length of approximately 35-40 mm, equivalent to 2 times the diameter of the reinforcing bar. Based on the derived correlations obtained from the undamaged reinforcing bars, this level of Leeb Hardness indicates that the steel has previously been strained to approximately 10% strain. This level of induced strain indicates that the steel has lost approximately 75% of the available strain capacity, and can only undergo an additional 5% strain before fracturing. Based on the short zone observed to have an elevated hardness, this would equate to approximately 2 mm of elongation over a 40 mm length prior to fracture.

The Leeb hardness for the Horizontal Bar 2 shows signs of moderately increased strain hardening over lengths of approximately 75-100 mm. Based on the correlations between Leeb Hardness and strain obtained previously, it is suggested that this steel sample has been previously strained to 2%.

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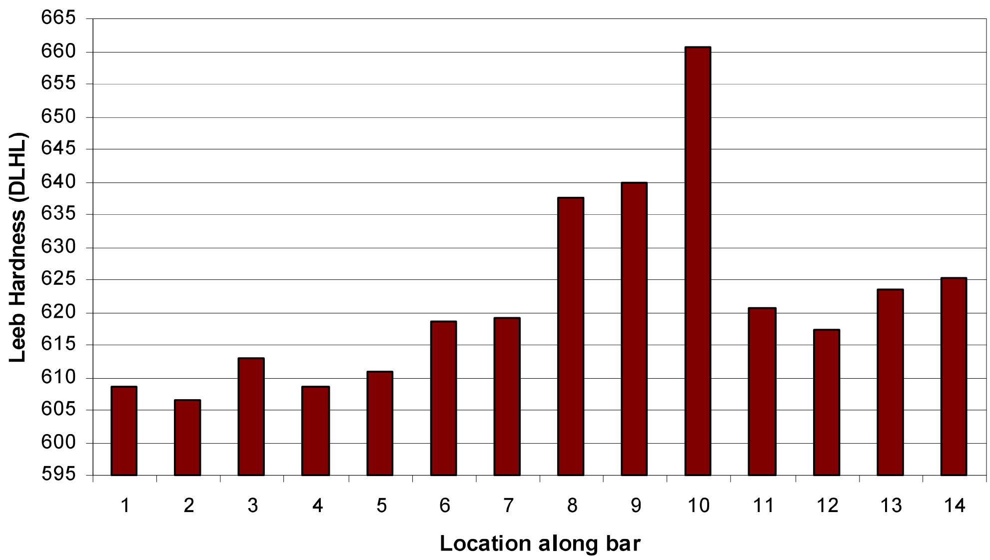

Figure 11 Leeb Hardness result for Vertical Bar 2 in zone of damage

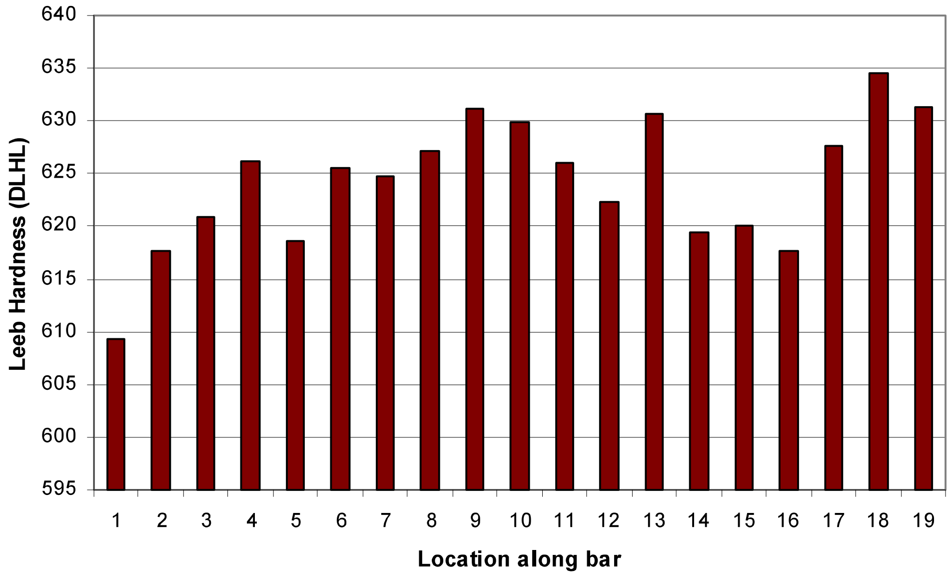

Figure 12 Leeb Hardness result for Horizontal Bar 2 in zone of damage

Vertical reinforcing Bar 3 shows two zones of increased hardness, corresponding to two cracks observed to cross the steel in the wall element. The first zone of elevated hardness is relatively wide, indicating that any yielding of the steel occurred across a relatively long length on the bar. The second zone of elevated hardness has a maximum recorded Leeb value of 640 DLHL and appears to occur over a relatively short distance. This level of hardness indicates that the steel was previously strained to approximately 5%. The results for Horizontal Bar 3 are similar to the previous horizontal bar with Leeb hardness values suggesting the steel was subjected to inelastic strains of approximately 2% over a relatively long length of the steel.

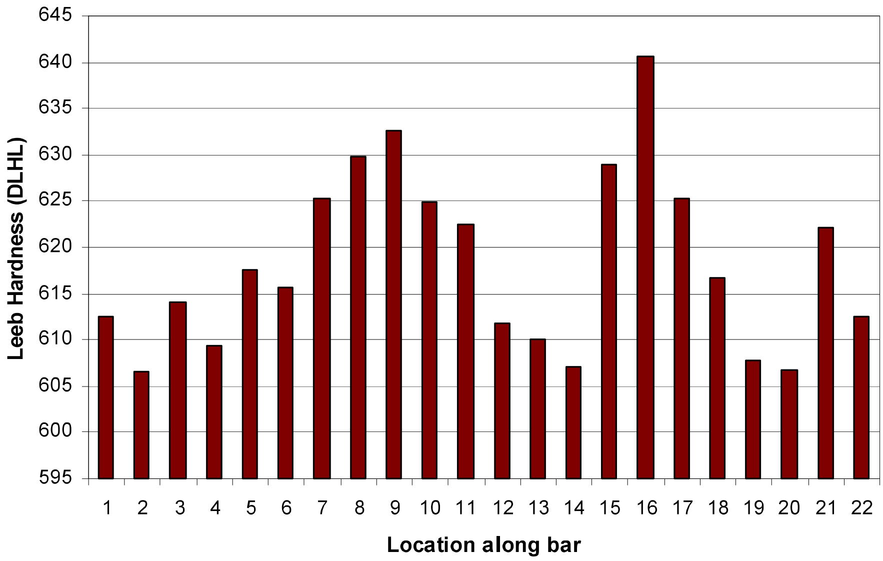

Figure 13 Leeb Hardness result for Vertical Bar 3 in zone of damage

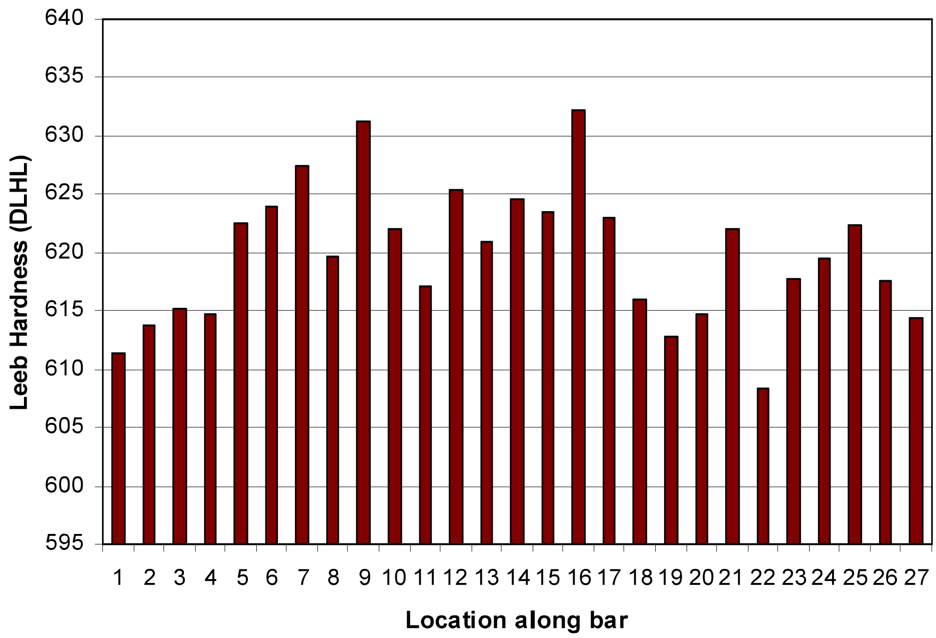

Figure 14 Leeb Hardness result for Horizontal Bar 3 in zone of damage

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A further 4 reinforcing bar samples were removed from the building and subjected to destructive tensile testing. The results from the testing are shown below. The results indicate that the horizontal steel remained undamaged during the earthquake, with recorded uniform strain capacities in excess of 33%. The yield strength of the tested horizontal steel samples was found to be 314 MPa and 315 MPa respectively.

The vertical steel sections were found to have considerable lower uniform elongation capacity when compared to the horizontal steel section, with actual elongation capacities between 11% and 13%. This result indicates that the steel has lost strain capacity due to being exposed to previous cycles of inelastic loading. The yield strength of the vertical steel sections was found to be 319 MPa and 330 MPa respectively.

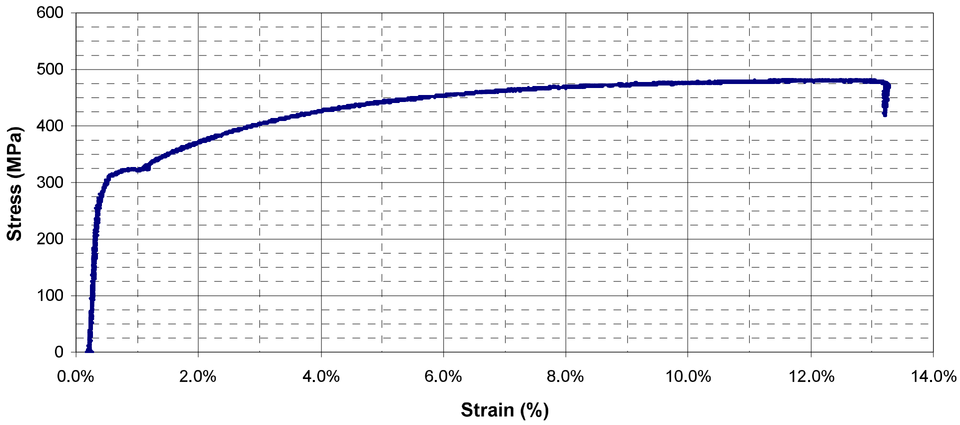

Figure 15   Stress-strain response for vertical steel section located in damaged zone

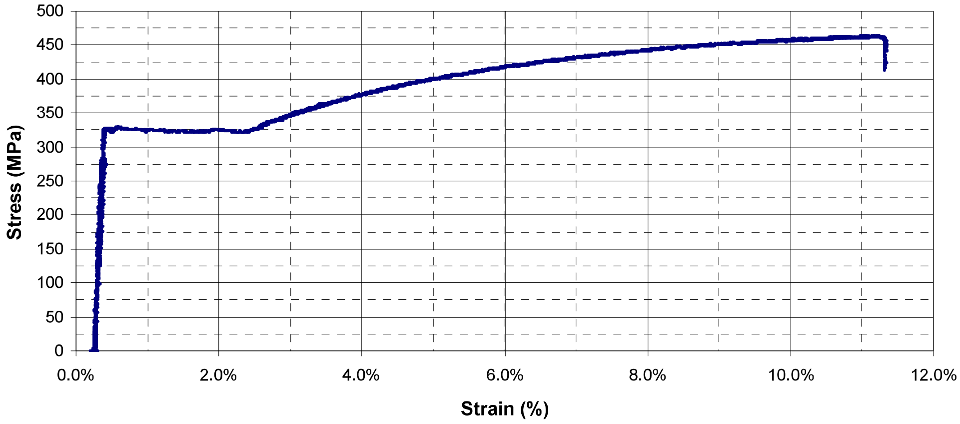

Figure 16   Stress-strain response for vertical steel section located in damaged zone

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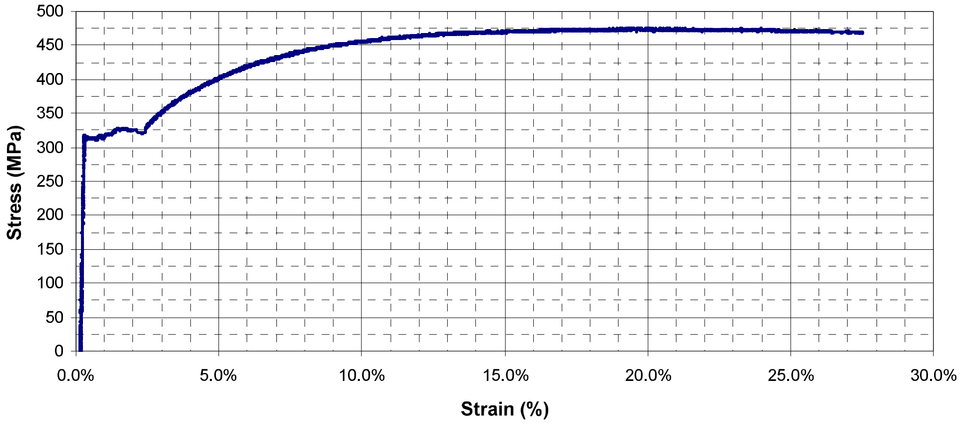

Figure 17 Stress-strain response for horizontal steel section located in damaged zone

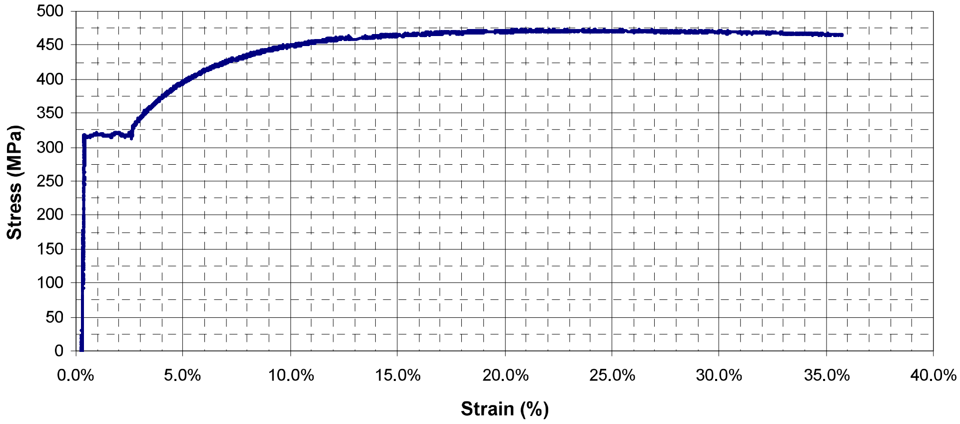

Figure 18 Stress-strain response for horizontal steel section located in damaged zone

6.0 IRD BUILDING RESULTS

6.1. CONCRETE RESULTS

No concrete cores were extracted from the IRD building. As a result, all concrete material information was obtained from Schmidt hammer tests. All tests were completed near the zones of damage in the in-situ and precast concrete shear walls. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe.

Table 12 Schmidt Hammer results for the Precast Walls in the IRD Building

location: Precast Wall section -1

	1	2	3	4
A	69.5	63.5	61.5	60.5
B	66	56.6	65.6	62.1
C	57	68.5	68	62
D	67	68	63	61.5

Correct Average:	64.1	
Cube Strength:	60.1	MPa
Cylinder Strength, f'c:	**48.0**	**MPa**

location: Precast Wall section -2

	1	2	3	4
A	72	63.5	68	69
B	68.5	63	58.5	61,5
C	56.5	63.5	59.5	71
D	63	65.5	58.5	72

Correct Average:	64.3	
Cube Strength:	61.2	MPa
Cylinder Strength, f'c:	**49.0**	**MPa**

location: Insitu Wall section -1

	1	2	3	4
A	58	59.5	63	63.5
B	65	63.5	67	71
C	53.5	55	52.5	53.5
D	55.5	65	57	60

Correct Average:	59.8	
Cube Strength:	50.1	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

location: Insitu Wall section -2

	1	2	3	4
A	65	61	59.5	55.5
B	55.5	65	56	63.5
C	65.5	66	55.5	59.5
D	58.5	59	62	55

Correct Average:	60.3	
Cube Strength:	50.2	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

6.2. STEEL RESULTS

HSL was commissioned independently to undertake materials testing on two steel samples extracted from the concrete walls of the IRD building. Two deformed reinforcing bars, 10 mm in diameter, were supplied for testing. The location of the steel in the building nor the origins of the steel were provided.

Prior to undertaking uniaxial tension testing on the steel, the samples were subjected to Leeb Hardness testing. The obtained results are presented below.

Both steel samples showed significant reduction in Leeb hardness readings at the location marked on the bars as corresponding with the crack in the concrete member. Reduction in Leeb hardness typically only occurs in steel bars immediately prior to the onset of necking, where micro alloy steel has been found to strain soften.

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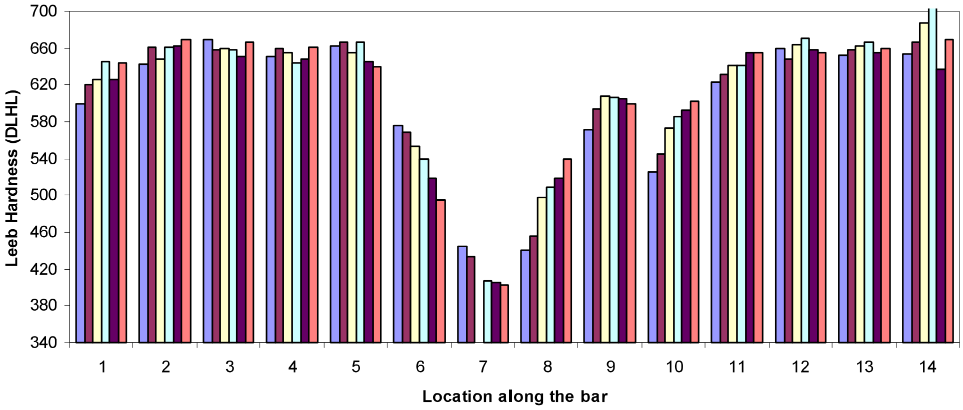

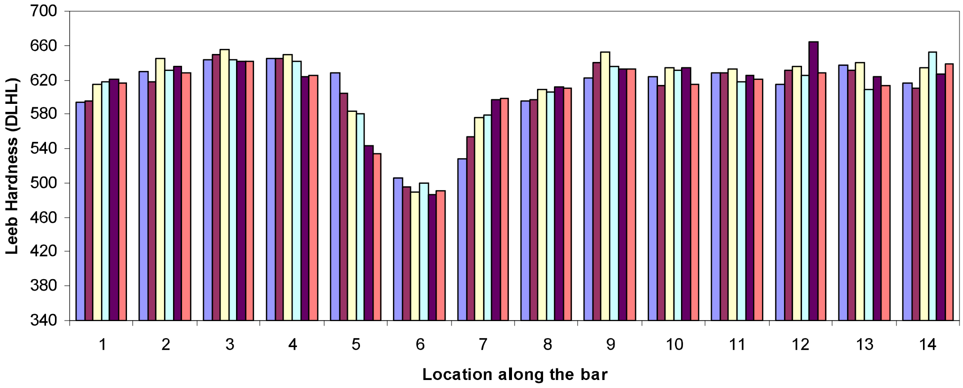

Figure 19 Leeb Hardness result for steel samples provided from IRD Building

The steel samples were then subjected to uniaxial tensile testing, with the obtained stress-strain responses shown in *Figure 20*. From the obtained stress-strain responses it would appear that the parent material was Grade 300E reinforcing steel. Grade 300E reinforcing steel has a lower characteristic yield strength of 300 MPa and is required in the New Zealand manufacturing Standard (AS/NZS 4671) to have a minimum uniform elongation capacity in excess of 15%. The results obtained for the two samples show they have an elongation capacity of 2% and 0.9% indicating that they have undergone significant inelastic deformation and are close to fracturing. This correlates with the observed Leeb Hardness results, showing significant strain softening at the cracked region.

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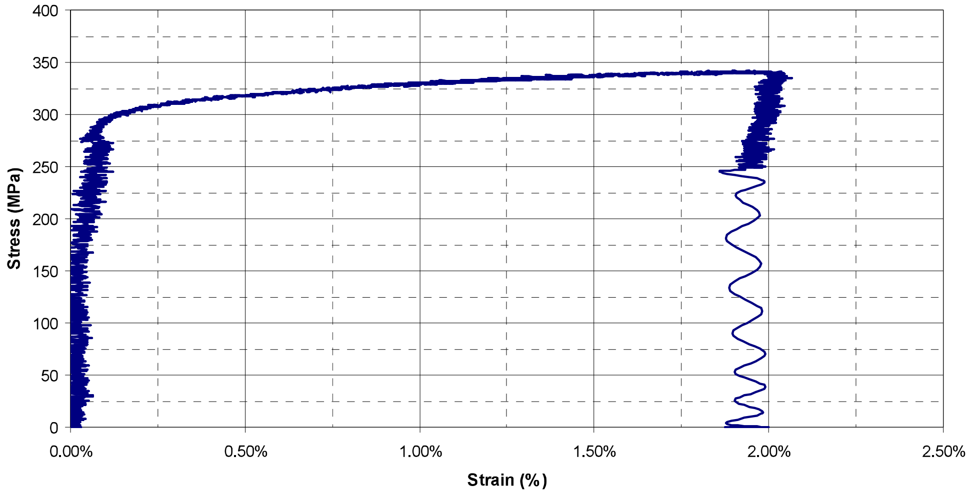

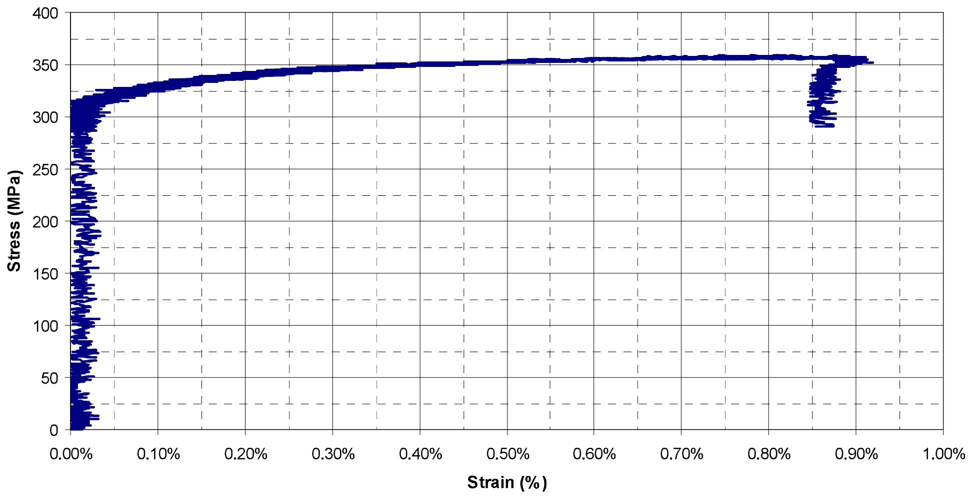

Figure 20 Stress-Strain responses for steel samples provided from IRD Building

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Figure 21 Tensile testing of steel sample

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