5 Earthquake Effects on Site and Building

5.1 Response Spectra

Earthquake ground motions were recorded at locations around the Christchurch CBD during the 4 September earthquake and subsequent aftershocks. These records have been translated into both acceleration spectra and displacement spectra. Acceleration spectra show the response accelerations of a building structure compared to its natural period (of vibration). Displacement spectra relate the displacement of the centre-of-mass to period.

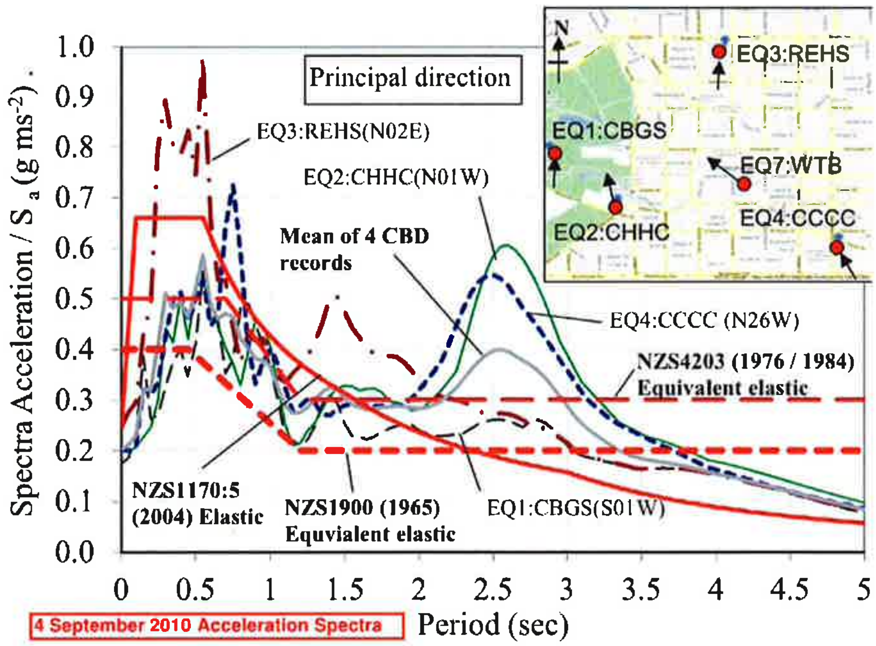


Fig. 6(a)

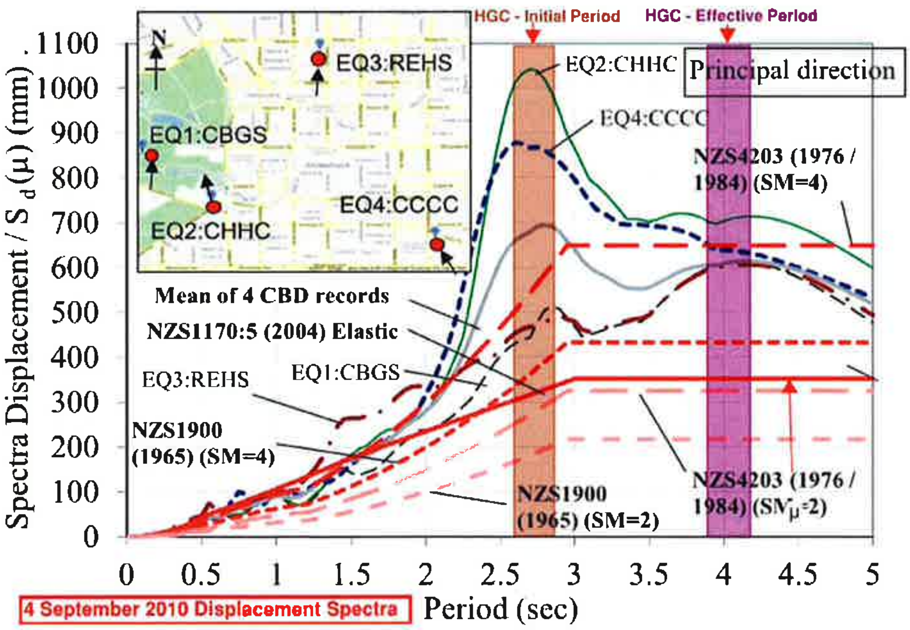


Fig. 6(b)

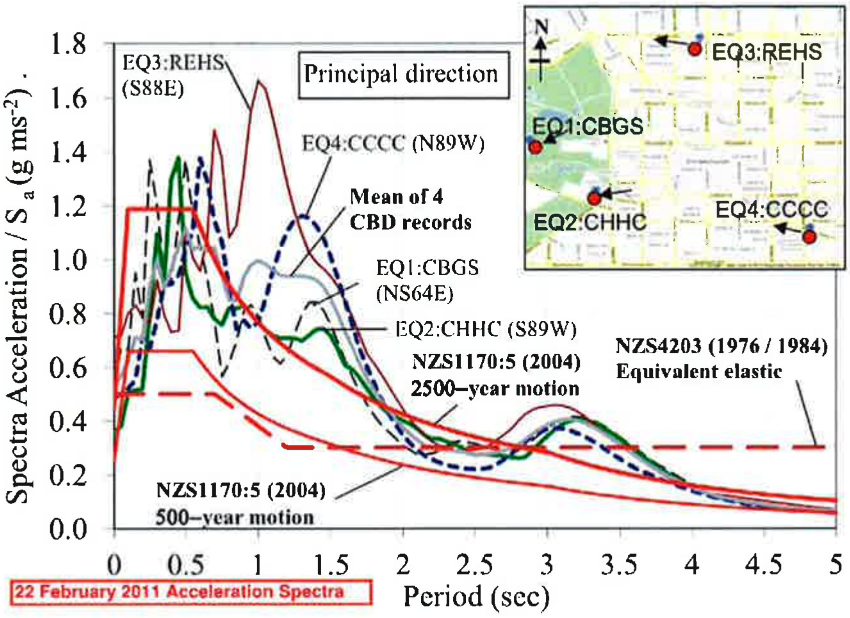


Fig.6 (c)

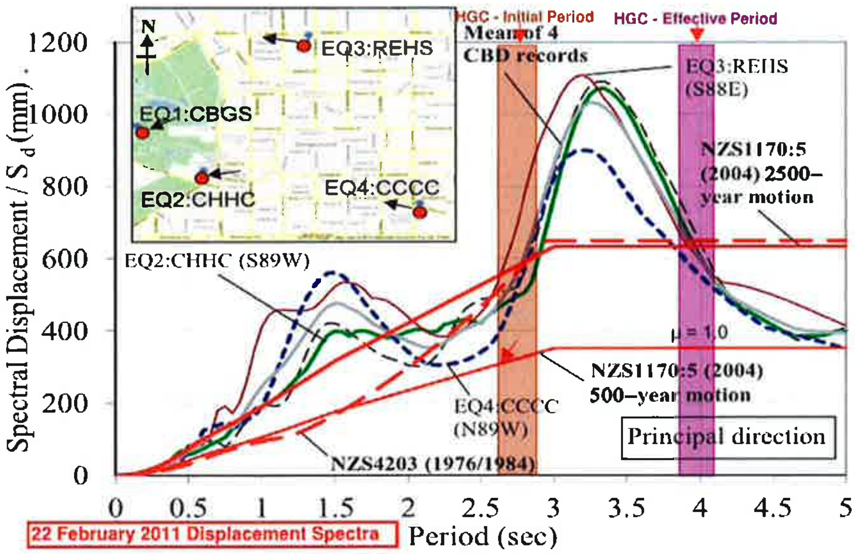


Fig. 6(d)

Spectra source
Kam, W.Y., Pampanin, S., 2011 " General Performance of Buildings in Christchurch CDB after the 22 Feb 2011 Earthquake: a Contextual Report", Department of Civil and Natural Resources Engineering, University of Canterbury

Figures 6(a), (b), (c) and (d) show acceleration and displacement spectra as recorded at 4 locations around the central business district for the 4 September event and the 22 February events. Only the principal direction of motion at each location is shown (the ground motion is normally recorded in two orthogonal directions, and one vertical). For analysis of the Hotel Grand Chancellor average values have been used to determine the response of the structure.

In the September event the north-south ground motions were stronger than the east-west motions at the Hotel Grand Chancellor site. In the 22 February event the motions were stronger in the east-west direction. Actions in this direction, in particular, accentuated vertical loads on the critical wall D5-6.

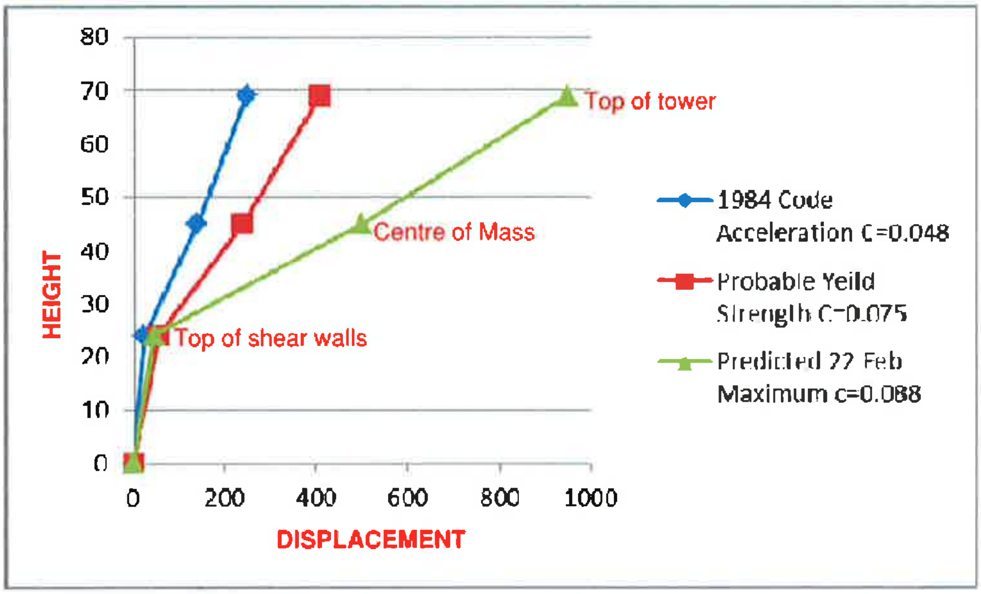


Fig.7

This response (ignoring the failure) is similar to what is implied by the 1984 loadings code (NZS4203:1984). While the initial accelerations and displacements for a 3 second period structure were higher in the February event than implied by the code, as the structure yielded and softened the demands were similar.

If the damage resulting from the wall collapse is disregarded then the observed damage is generally consistent with the ductility demand. While the extent of cracking is perhaps less than expected in the upper tower beams, this may be explained by the limited number of strong motion cycles, maybe only 2 or 3.

5.2 Liquefaction and Foundation Issues

Visual observation and ground floor survey level data suggests that neither liquefaction nor foundation failure have had significant effects on the performance of the Grand Chancellor structure. There have been no significant surface signs of liquefaction in the vicinity and geotechnical advice is that the area has not been subject to slumping or localized displacement. There are also no signs of significant local level changes around the building.

5.3 Damage Prior to the February Event

Information from the Christchurch City Council relating to assessment of the building following the September event is not extensive. However investigations have established that it was given a G1 building safety assessment which implies that little or no structural damage was observed.

This is consistent with private engineering and maintenance inspections that reported no significant structural damage. There was some 'non-structural' damage that included:

- broken windows and frames
- damaged sealant between precast panels
- movement in stair-floor joints

In the February event, strong vertical ground accelerations were also recorded. While the strongest vertical motions were not necessarily concurrent with the strongest horizontal motions, vertical accelerations had the potential to significantly increase the vertical loading on wall D5-6. This loading can be accentuated by the dynamic response of cantilever elements which in this case formed a major load component on wall D5-6.

The Hotel Grand Chancellor has a calculated initial period (at yield of the tower frames) of around 2.8 seconds. As a structure yields it also softens and as a consequence the period lengthens. In a post-elastic scenario the effective period is calculated to be around 4 second.

Initial review of the spectra suggests that the structure would have been subject to high accelerations and displacements both in September and in February. While this is demonstrably true for the February 22 aftershock, the response of the structure to the original September earthquake did not match what is indicated from the spectra. This apparent disparity can be explained as follows:

- The period shift as the structure softened increased displacement demand (We note that the extreme peak around a 3 second period is unusual and is related to the geological conditions beneath the Christchurch CBD). In September the maximum possible displacement was 700mm (average) while in February the maximum possible displacement was 1050mm (average). Note that the displacement of any particular structure will be less than the maximum and is influenced by damping.
- The variability between the records was greater in September (+/- 40%) than in February (+/- 15%). This means that there was more uncertainty about the magnitude of displacement in September
- There is uncertainty about the influence of hysteretic damping on the response. In September the shaking was of longer duration and hysteretic damping is likely to have been more effective. In February the event was short and it contained some violent pulses. In that situation hysteretic damping is less effective and so the displacement was likely to be relatively greater.

In addition, academic research has suggested that the September earthquake did not have the effect on medium-high frequency structures as may be inferred from the spectra. Refer to: *"Considerations on the Seismic Performance of Pre-1970s RC Buildings in the Christchurch CBD During the 4th Sept 2010 Canterbury Earthquake: Was that Really a Big One?"* - *S. Pampanin and others : 9th Pacific Conference of Earthquake Engineering*

This helps to explain the relative lack of structural damage observed following the September earthquake. Minor cracking was recorded in some of the upper tower frames and this suggests that at least some frame elements reached yield.

It is clear that during the February 22 aftershock, the response generated in the Hotel Grand Chancellor was much more dramatic. The lower tower shear walls had been designed to have a lesser ductility than the upper tower moment resisting frames. As a consequence (and as intended) the frames yielded before the shear walls. Beam hinge cracking patterns in the east-west tower seismic frames suggests that one or two cycles of horizontal yielding occurred in the upper tower frames before the wall failure occurred.

Using an average of the displacement response spectra, from the strong motion recording sites around the Christchurch CBD on 22 February, the following is derived from modeling and analysis:

a) At initial yield of the upper tower frames, *assuming a fixed base* (rigid foundations) which calculations suggest may have been the basis of the original analysis:

Displacement of effective centre of mass	140mm
Displacement at top of shear walls	25mm
Ductility demand on shear wall structure	<1
Displacement at top of tower	250mm
Ductility demand on upper tower	1

b) At initial yield of the upper tower frames, *assuming some pile flexibility* based on the driving records only (no flexibility of the soil bulb below):

Displacement of effective centre of mass	170mm
Displacement at top of shear walls	40mm
Ductility demand on shear wall structure	<1
Displacement at top of tower	295mm
Ductility demand on upper tower	1

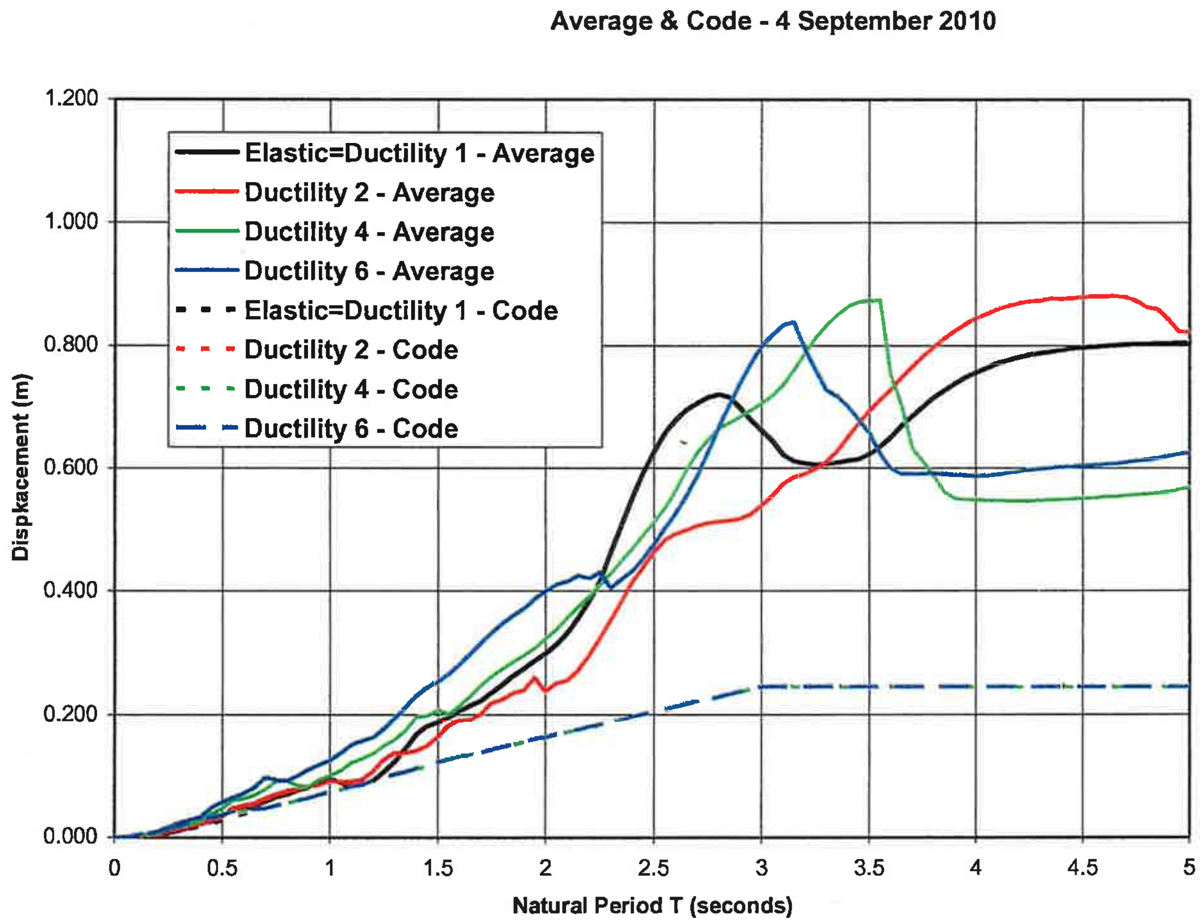
c) If probable strengths of the materials are used for initial tower yield:

Yield Displacement of effective centre of mass	240mm
Yield Displacement at top of shear walls	55mm
Yield Displacement at top of tower	370mm
Ductility demand on upper tower	2.3
Effective ductility demand on overall structure	2

d) At maximum displacement predicted from the 22 February records *allowing for pile flexibility*

Displacement of effective centre of mass	500mm
Displacement at top of shear walls	70mm
Ductility demand on shear wall structure	1 – 1.5 depending on wall length and axial load
Displacement at top of tower	950mm
Ductility demand on upper tower	3.3
Effective ductility demand on overall structure	2.9
Average drift in upper tower	1.9% (65mm/floor)

This can be summarised in the following graph:



N-S

Figure 46. Average Displacement Response Spectra vs NZS1170.5 Christchurch 4 September 2010
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

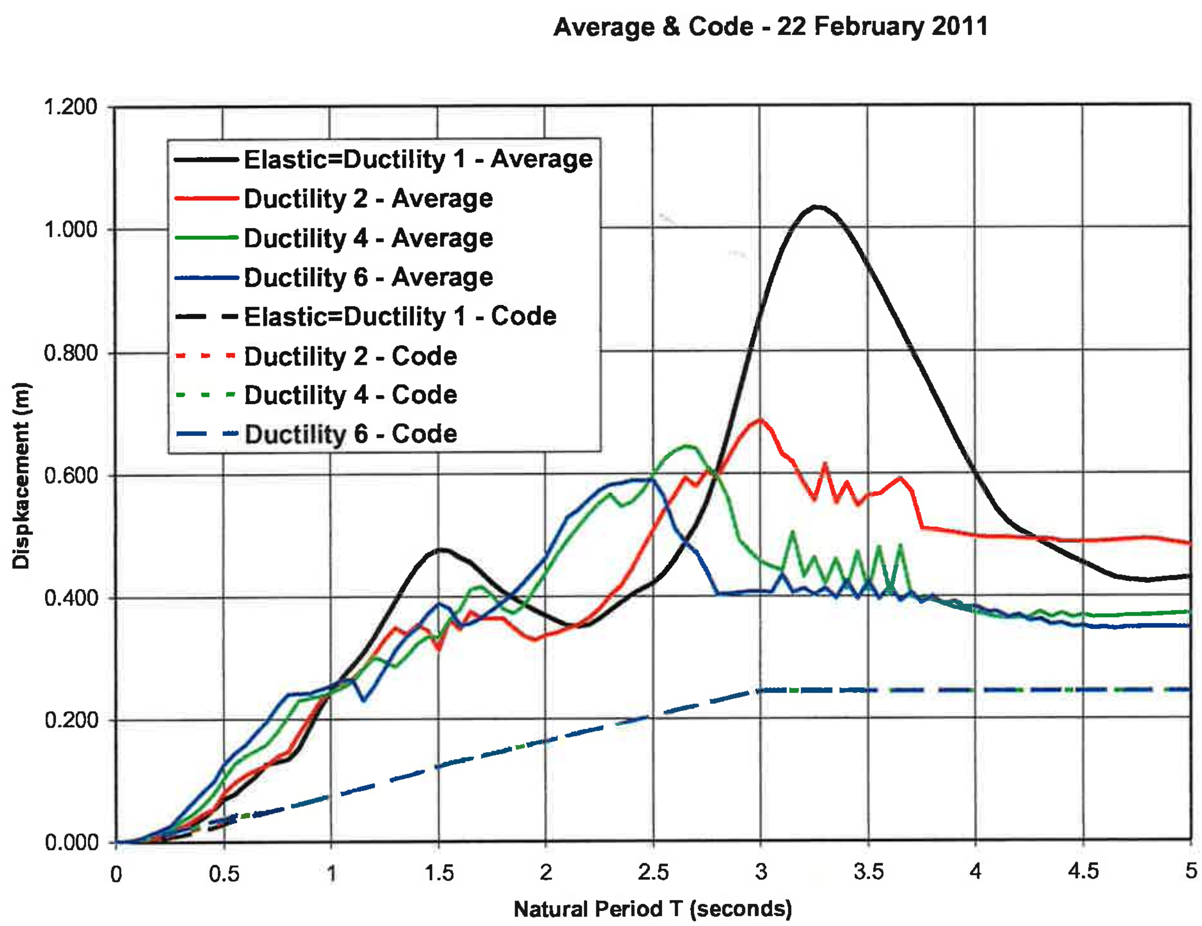
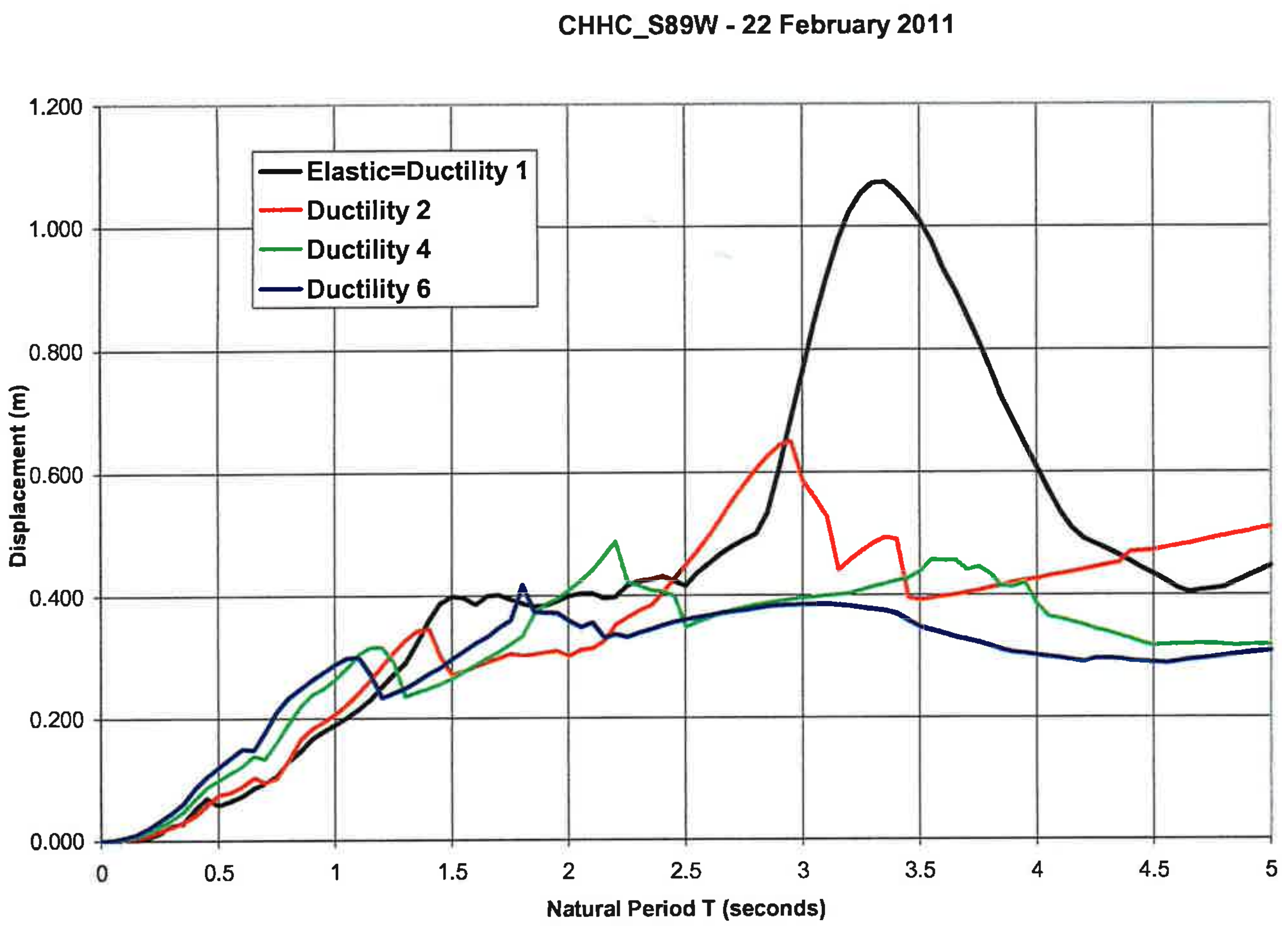


Figure 49. Average Displacement Response Spectra vs NZS1170.5
Christchurch 22 February 2011
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

E-W

BUI.CAS161.0038.8

Appendix E
22 February 2011 Earthquake Horizontal Components – Modified Takeda Hysteresis



E -W

Figure E-1. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Modified Takeda Hysteresis

- The wall central (web), vertical reinforcing had a lap just above ground floor level although the primary (at the ends of the wall) reinforcing bars were not lapped until the first floor. In a situation where the compression block/neutral axis extends beyond the confined area the web reinforcing effectively becomes primary reinforcing. NZS 3101:1982 did not permit lapping of primary reinforcing within the end/hinge zone (lower portion) of a shear wall. Part of the reason for this is that within a zone of ultimate concrete strain the end of the reinforcing bars can cause stress raisers within the concrete.
- It is likely that the diagonal failure plane initiated immediately behind the small confined zone, at the top of the lapped bars, possibly encouraged by a tension yield crack and/or by stress raisers at the top of the web laps. This is consistent with photographic evidence that shows the top of the failure plane coincident with the top of the lap bars. *[Ref App C Photo 10]*
- The compressive actions exerted on the wall are likely to have been considerably higher than the loads used in the original calculations (possibly by more than a factor of 2), due to bullet points 2, 4 and 5 above. Analysis and calculations suggest that induced axial loads could have reached 28MN during the 22 February event, without the influence of vertical acceleration. With vertical acceleration included an axial load of between 33MN and 45MN was possible. These values result in very high axial load ratios – between 0.4f'c and 0.65f'c. By comparison the maximum permitted axial stress on a highly confined concrete column is currently around 0.72f'c.
- Even without the addition of vertical acceleration loads, it is highly probable that the conditions for wall failure existed, when subject to severe shaking,

Calculations supporting these assessments are summarised in Appendix F.

Of the factors listed above that contributed to the brittle failure of the wall, it is the lack of effective confinement that is considered to be the critical factor. Adequately detailed confinement provides concrete (an inherently brittle material) with ductility , which is an ability to withstand post-elastic strains. In many respects, and in retrospect, the actions on wall D5-6 can be likened to those on a highly loaded concrete column.

For a concrete column, confining hoops and ties give strength to the concrete in a way that may be likened to the steel hoops around a barrel. In a barrel the hydrostatic pressure from the liquid contents attempts to force open the gaps between the vertical timber slats but the confining pressure from the hoops prevents the gaps from opening.

A concrete column loaded in compression will naturally shorten and as a consequence, expand its girth. This redistribution of volume can result in internal tensile stresses, particularly if one end of the column is constrained from expansion. Confining links and hoops within a column or wall effectively restrains the expansion and forces the concrete into transverse compression

10.5 What was the %NBS on 21 February?

Based on a simple force-to-cause-yield comparison the Hotel Grand Chancellor could be considered to have a strength in excess of 100%NBS (New Building Standard). However, when issues of displacement and available ductility are considered the structure clearly did not meet 100%NBS.

10.6 Would the Stairs have Collapsed without the Critical Wall Failure?

Evidence and analysis suggest that catastrophic stair collapse would not have occurred without the critical wall failure. Although there was no provision to accommodate shortening of the distance between the stair supports, the shortening which did occur did not significantly damage the stair flights themselves. Rather, the shortening resulting from the inter-storey drifts caused the steel supporting stubs to break out of the seating pockets which supported the stubs. This action did not lead to collapse, as is apparent from the surviving flights. It was the excessive lengthening between the support points that only occurred as a consequence of the Wall D5-6 failure that led to the collapse of the stairs.

11 Recommendations

This section contains some recommendations arising from observations made during the preparation of this report and the meetings of the investigative panel. Some are quite specific to structural features that are contained within the Hotel Grand Chancellor and some are more generic, relating to design codes and practice generally.

- ***Design Rigour for Irregularity.***
  While current codes do penalise structures for irregularity, greater emphasis should be placed on detailed modelling, analysis and detailing. – DBH should require an increase in design rigour for irregularity
- ***Design Rigour for Flexural Shear Walls.***
  The behaviour of walls subject to flexural yielding, particularly those with variable and /or high axial loads has perhaps not been well understood by design practitioners. – DBH should require an increase in design rigour for wall design generally and in particularly for confinement of walls that are subject to high axial loads
- ***Stair Separation*** – DBH should promote the review and retrofit of existing stairs, particularly precast scissor stairs. DBH should consider introducing larger empirical stair seating requirements (potentially 4%) for both shortening and lengthening. The review of this aspect should be included within earthquake-prone building policies.
- ***Floor-Depth Walls***
  The consequence of connecting floor diaphragms with walls that are not intended to be shear walls require particular consideration. – DBH should consider a design advisory relating to walls/beams that are

11. Relationship between the Observed Response Spectra and the Design Response Spectra (NZS1170.5:2004)

Figures 45 to 56 present the averaged spectra of the records listed with each figure plotted together with the design response spectra given in NZS1170.5.

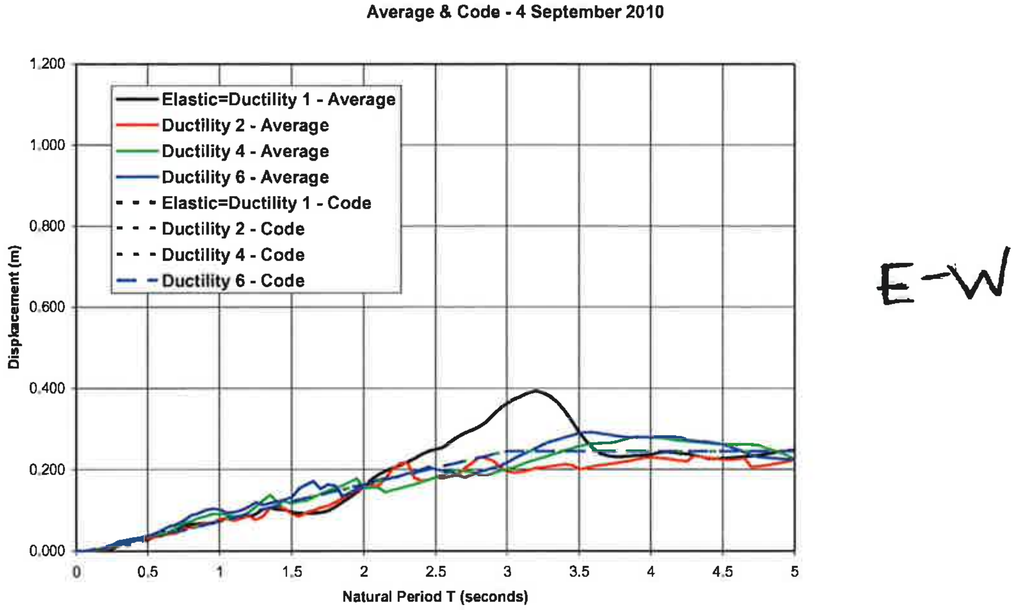


Figure 45. Average Displacement Response Spectra vs NZS1170.5
Christchurch 4 September 2010
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

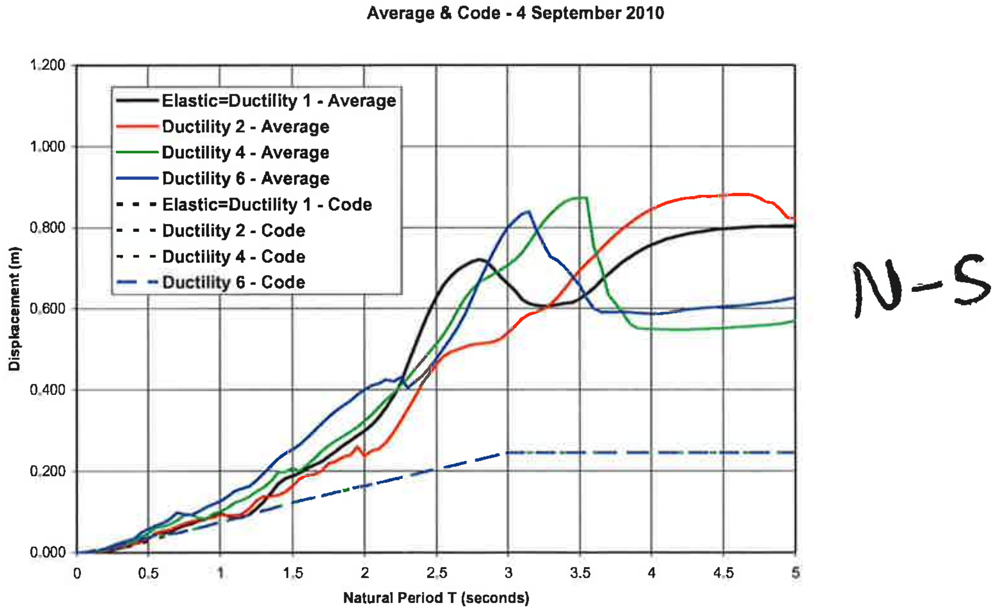


Figure 46. Average Displacement Response Spectra vs NZS1170.5
Christchurch 4 September 2010
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

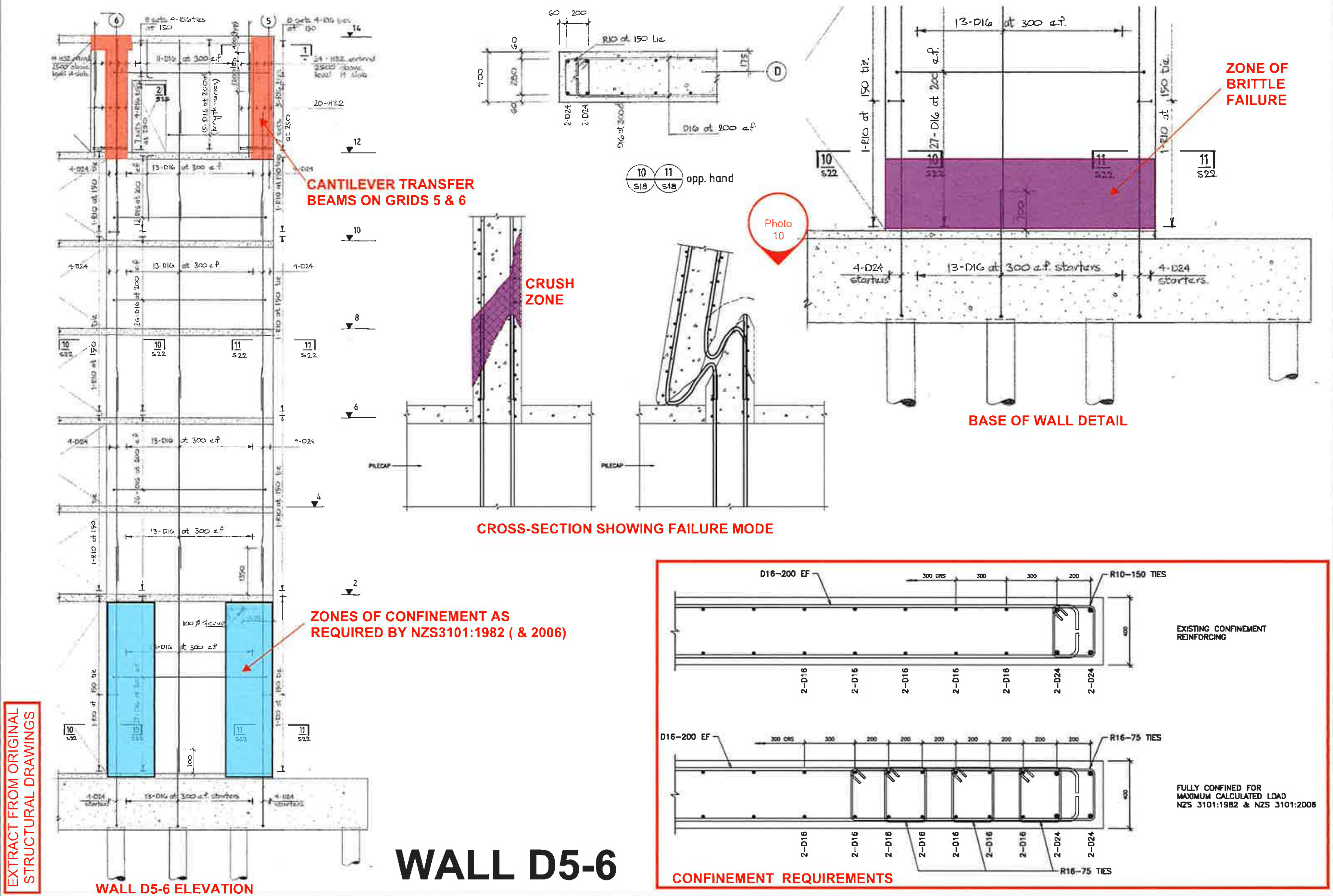
Photo 9 - Shearwall D5-6 Failure - End View

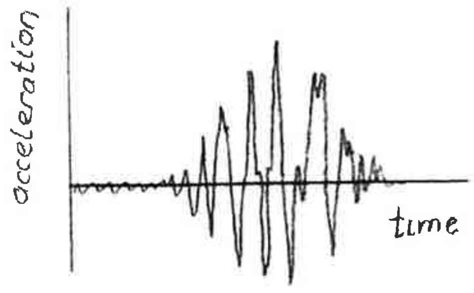
Brittle, transverse failure,
absence of confinement

Lapped reinforcing

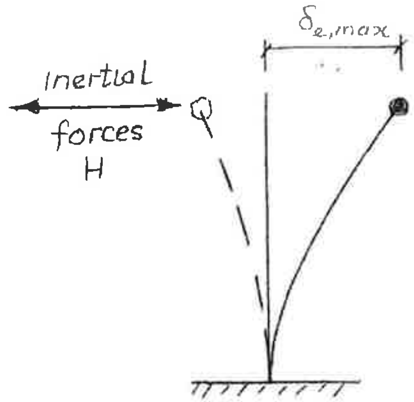
Photo 10 - Shearwall D5-6 Failure - Close-up

WALL D5-6

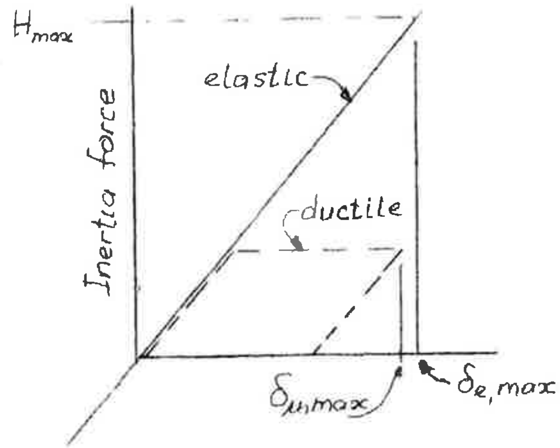

EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS
CANTILEVER TRANSFER BEAMS ON GRIDS 5 & 6
ZONES OF CONFINEMENT AS REQUIRED BY NZS3101:1982 (& 2006)
WALL D5-6 ELEVATION
CRUSH ZONE
CROSS-SECTION SHOWING FAILURE MODE
PILECAP
Photo 10
ZONE OF BRITTLE FAILURE
BASE OF WALL DETAIL
13-D16 at 300 c.f.
27-D16 at 200 c.f.
1-R10 at 150 tie
4-D24 starters
13-D16 at 300 c.f. starters
R10 at 150 tie
D16 at 200 c.f.
opp. hand
D16-200 EF
R10-150 TIES
EXISTING CONFINEMENT REINFORCING
R16-75 TIES
FULLY CONFINED FOR MAXIMUM CALCULATED LOAD NZS 3101:1982 & NZS 3101:2006
2-D16
2-D24
CONFINEMENT REQUIREMENTS



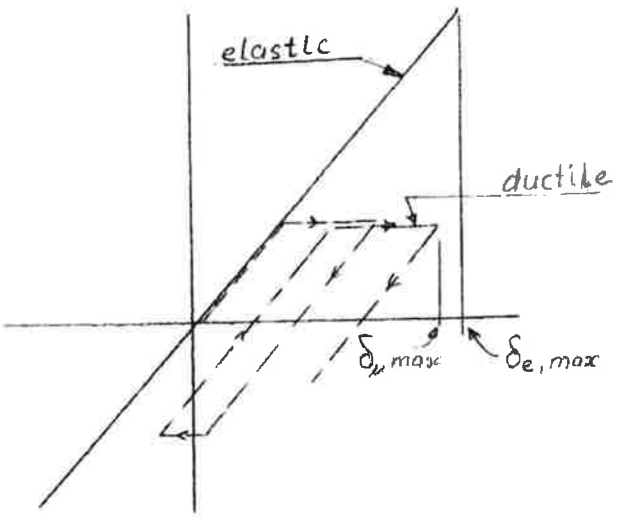
(a) Ground acceleration



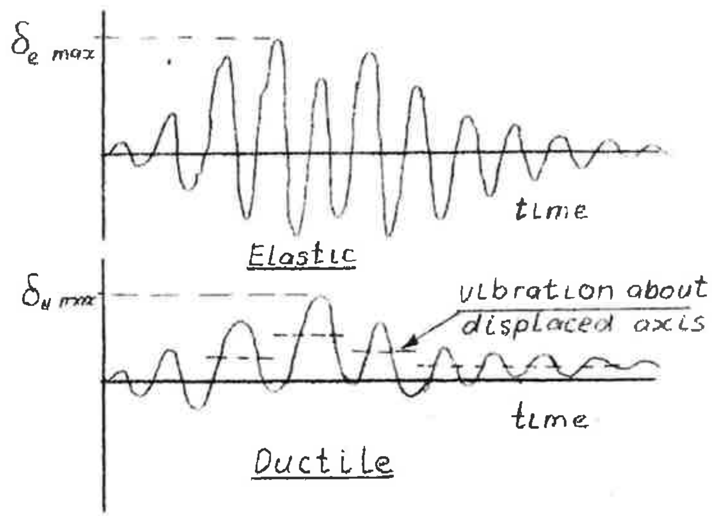
(b) Single degree of freedom structure



(c) Elastic & ductile structures at maximum displacement

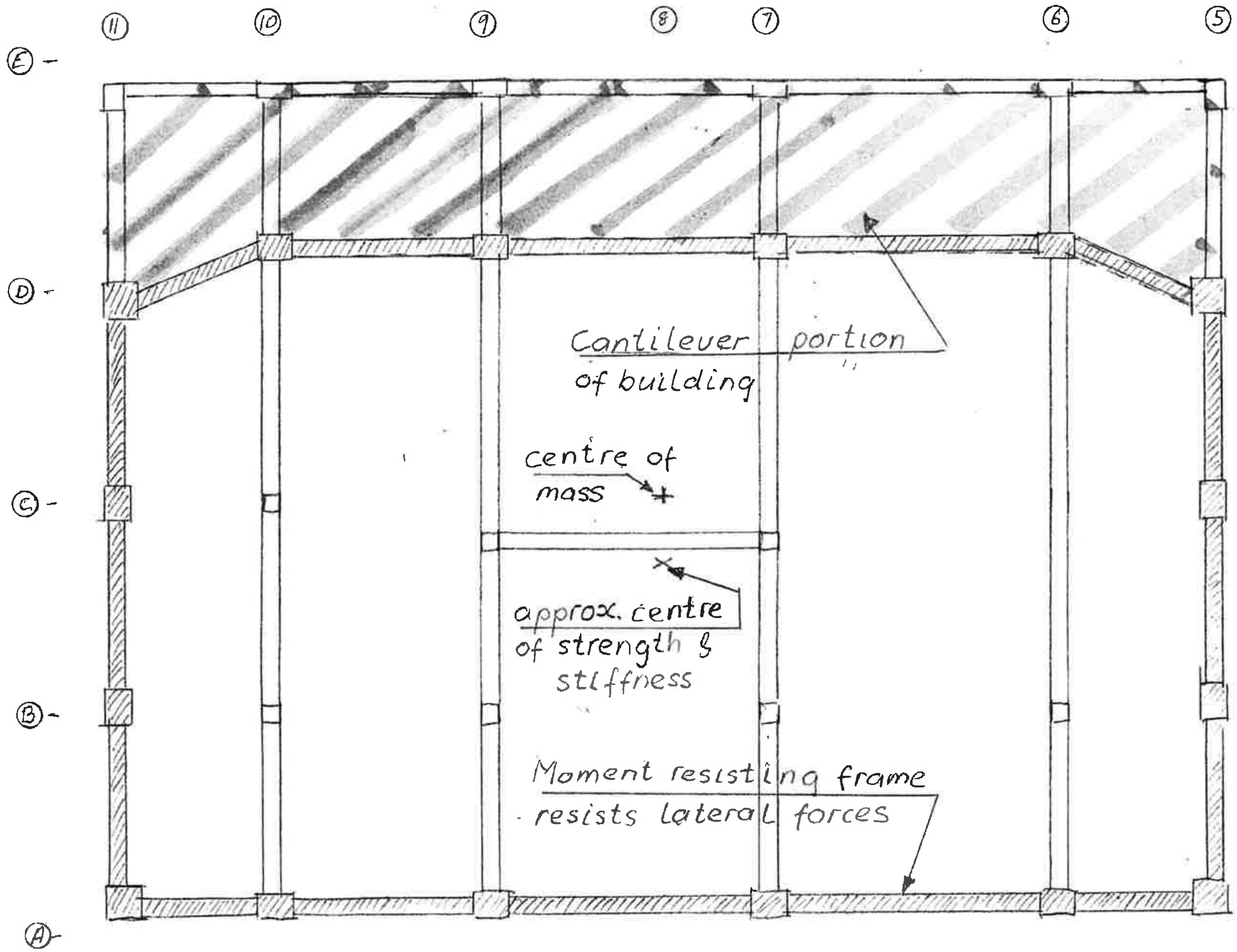


(d) Elastic and ductile response



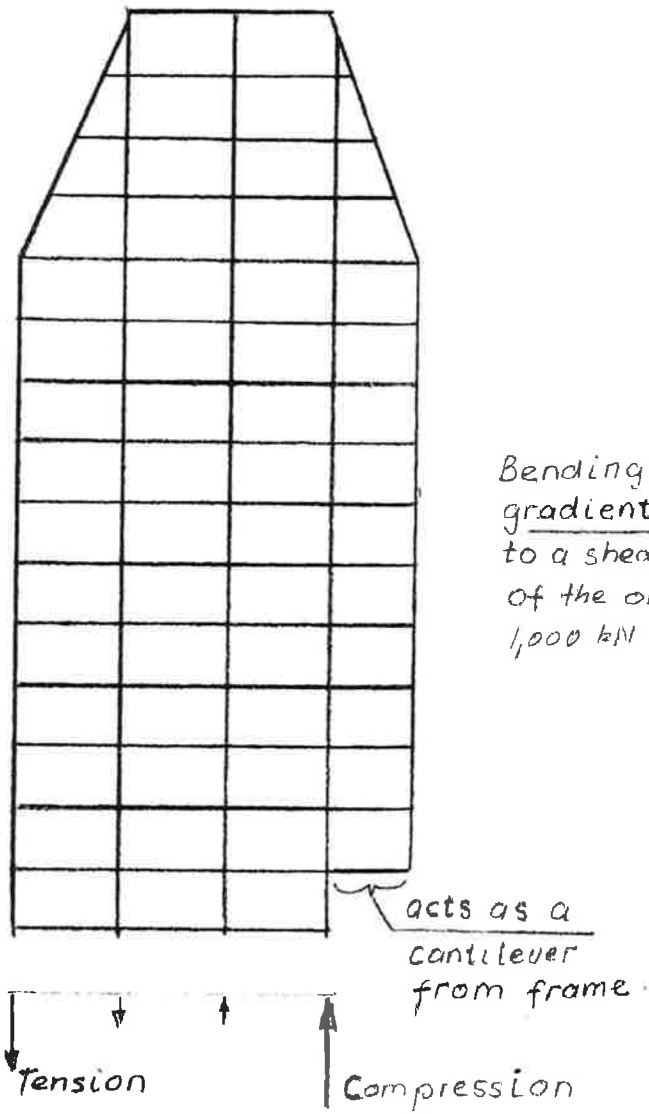
(e) Displacement versus time for elastic and ductile structures

Sketch 1

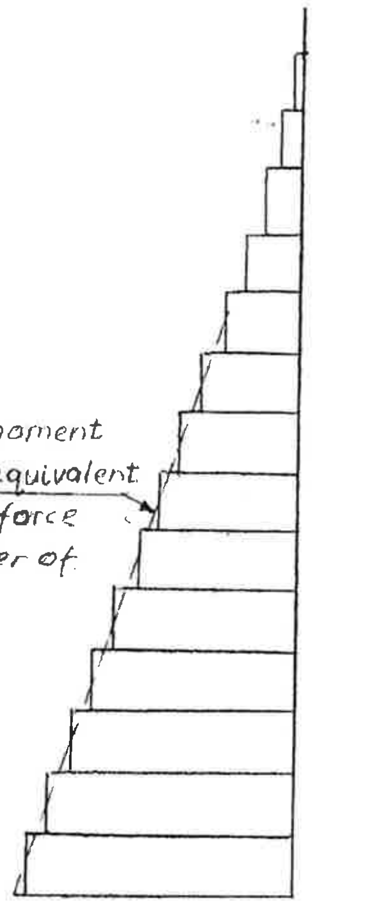


11
10
9
8
7
6
5
E
D
C
B
A
Cantilever portion of building
centre of mass
approx. centre of strength & stiffness
Moment resisting frame resists lateral forces

Sketch 2



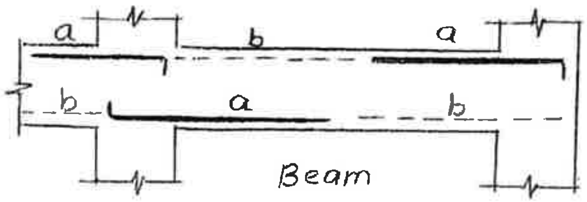
Elevation and column forces due to cantilevered loads



Bending moment due to cantilevered dead and live loads

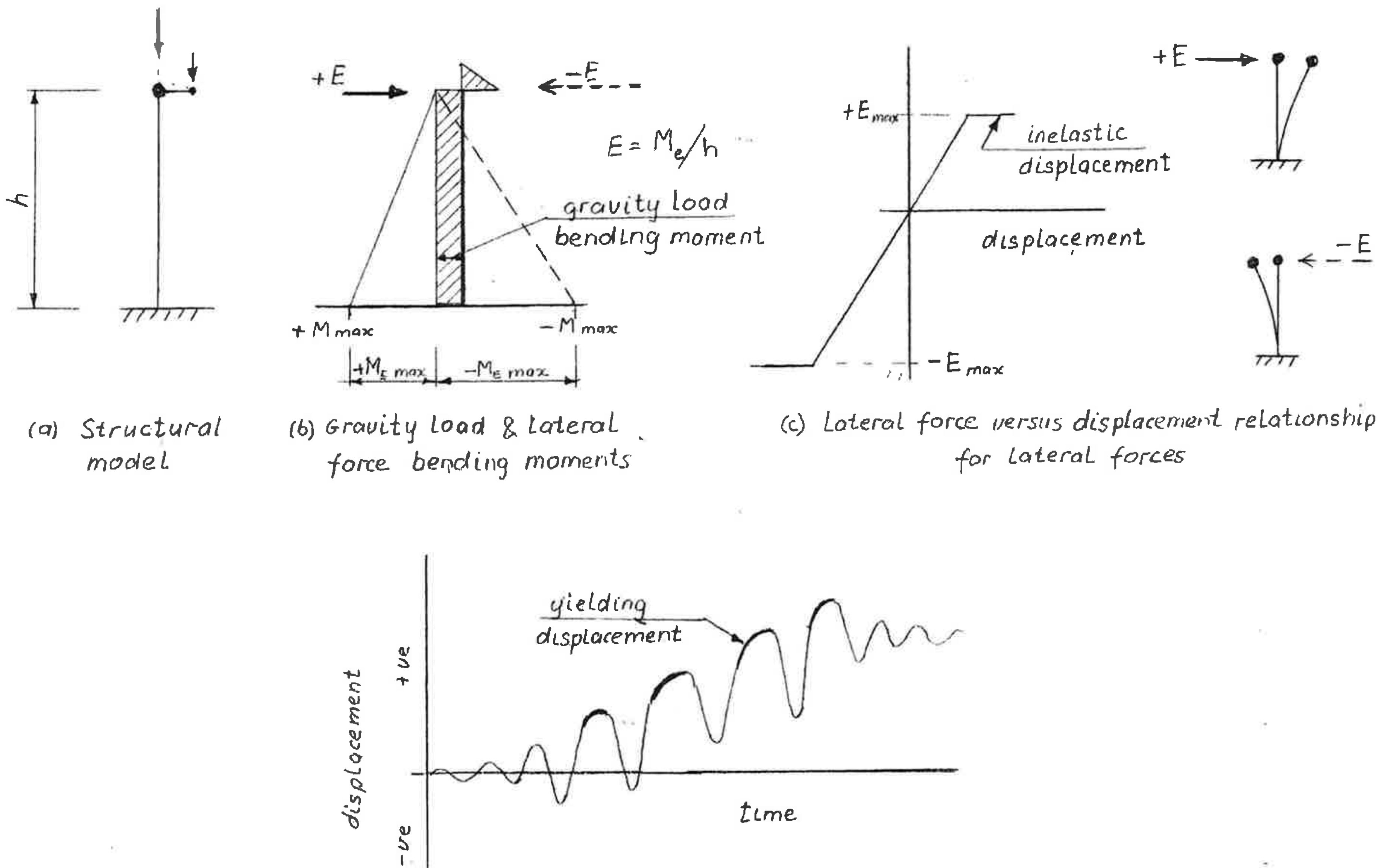
Bending moment due to cantilever portion of floor at one level ≏ 3,500 kNm, This is equivalent to a shear of the order of 1,000 kN.
Hence the strength to deflect to the East is reduced by 1,300 kN, and the strength to deflect to the West is increased by 1,300 kN.

This could be compensated by increasing strength for Eastward deflections, but this has not been done.



To balance strengths would require more reinforcement in zones "a" than in zones "b", but this has not been done.

Sketch 3



(a) Structural model

(b) Gravity load & lateral force bending moments

(c) Lateral force versus displacement relationship for lateral forces

(d) Displacement versus time

Sketch 4

Questions on "Report on Structural Performance of Hotel Grand Chancellor"

12/12/11

It is important that the findings from the analysis of the performance of the Hotel Grand Chancellor in the Christchurch are understood so that any lessons can be applied both to assist in identifying;

- other buildings with similar characteristics which may as a result be earthquake risk buildings;
- changes that should be made in the design of new structures.

There are some outstanding features of the report. In particular I note the diagrams outlining the form of the structure made it very easy to understand how this structure worked.

There is no doubt why the building failed. This was caused by the collapse of a wall close to ground level. It is also apparent that this wall did not fully comply with the design provisions of the time or with current design requirements. However, the question remains if either the analysis undertaken when the structure was designed could have identified the inherent weakness in the wall for a design level earthquake.

The questions below are aimed at trying to identify additional information to clarify some aspects of the report and to gain additional information on the behaviour of the building in the Christchurch earthquakes.

1. On pages 13 and 14 in Figure 6 parts (a) and (c) of the report the spectral accelerations are shown for the September and February earthquakes together with the design spectral accelerations given in the design standards, NZS1900: 1965, NZS4203: 1976 and 1984 and NZS1170.5: 2004. In the design standards of 1965, 1976 and 1984 the spectral accelerations are shown as constant for periods between 1.4 and 5 seconds. In Figure 6, parts (b) and (d) the corresponding spectral displacements are shown as constant over the range of 3 to 5 seconds. Is this logical and is it consistent with the clauses in the relevant standards?

2. On page 17 Figure 7 there is reference to the "Centre of mass". Can you please define what is meant by this term?

3. On page 15 of the report in the first bullet point it is indicated that; in September the maximum possible displacement was 700mm (average) while in February it was 1050mm (average) displacement.
   - By (average) are you referring to a response spectra found by taking the mean values at each period of the 4 CBD records in the same direction?
   - How do you know that the response of the structure to the September earthquake did not match what is indicated by the spectra?
   - How do you relate the spectral displacements values from the response spectra to the predicted displacements of 700mm and 1050mm and what is the position on the structure where these displacements predicted are expected to occur?

- Do these predicted displacements include allowance from; higher mode effects, bi-axial actions (North-South and East – West excitation), and in particular what allowance has been made for torsional rotation of the building?

- In the assessment of the displacements of 700mm and 1050mm was any allowance was made for deformation associated with P-delta actions?

- What allowance was made for the expected change in deflected shape between an elastic structure and a ductile structure?

4. On page 16 it is indicated that the ductility demand (which I assume refers to displacement ductility) is in the range of 2-4 for the February earthquake (values of 2.3 and 3.3 are quoted in the report).
   - Did you consider assessing the predicted displacements demands for the February and September earthquakes from the response spectra for ductile structures (these are given in the Carr report, available on the Royal Commission web site)?

   - For the September earthquake see page 40, Figure 46 in the Carr report, the spectral displacement for a ductile structure in the period range of interest is of the order of 700mm in the North South direction, while for the February earthquake, see pages 42 and 108 of the Carr report, the spectral displacements in the East West direction are of the order of 500mm for the bilinear hysteretic model and 450mm for the Takeda hysteretic models.

   - Given these figures what would you conclude about relative severities of the two earthquakes for the Hotel Grand Chancellor?

5 On page 25 in second to last bullet point, it is indicated that the calculated axial load acting on wall 5-6 in line D was possibly more than twice the value assumed in the original design.
   - The original design calculations are available so why is it ***possibly*** twice?

   - How would the axial force found using current design standards compare with the value in the original design calculations following the then current design standards?

6 On page 25 the axial load on the wall 5-6 on line D has been assessed as 28MN if vertical acceleration is ignored and up to 45MN if vertical acceleration is included.
   - Do current design standards require allowance to be made for vertical ground motion in determining design actions in buildings?

   - How did you assess the increase in seismic force due to vertical acceleration to be 17MN?

   - Would speed of loading associated with vertical ground motion have had a significantly influence on the strength of wall 5-6 on line D?

   - Does this indicate that current design practice should be changed to include allowance for actions induced by vertical ground motion?

7 In the assessment of the building axial loads of the order of 28MN and 45MN have been predicted for the wall 5-6 on line D.

- How does this compare with the calculated crush load using the measured concrete strength? *(about 68MN by my estimate assuming f_c' is 40MPa)*

- What other factors would you require before it is possible to predict that the wall would fail or survive the assessed axial load?

8 On page 29 in the last bullet point it is indicated that a factor contributing to vulnerability of the wall was "Code defined actions exceeded by the February Earthquake". The axial load was determined by capacity design and the displacement response spectra for structures with a displacement ductility of between 2 and 4 is of the order or 400-500mm (see Carr report and question 4) and the design spectral displacement is a little under 600mm (see figure 6 (b) in the report). What code actions are you referring to and in which code?

9 Page 31, section 10.5, comments are made about %NBS. It is indicated that the structure clearly did not meet 100%NBS. How would you assess what %NBS the building would have had before the February 22nd earthquake? (Note the design strength for seismic actions was based on a base shear coefficient of 0.048 with a structural ductility factor of 4 while the corresponding coefficient in NZS1170:2006 is 0.0405 for a structural ductility factor of 4)

10 What methods of analysis were used in the original design of the Hotel Grand Chancellor? *(Equivalent static, Modal response spectrum or time history?)*

11 Do you agree that elastic response spectra for both accelerations and displacements are based on the assumption that a single degree of freedom structure has equal strengths and stiffness in for both backwards and forwards displacements for ground shaking? (see sketch 1)

12 Do you agree that the equal displacement concept implies that a single degree of freedom ductile structure subject to earthquake shaking sustains approximately the same magnitude of peak displacement as an elastically responding structure with the same initial stiffness? (see sketch 1)

13 The top ¾ of the building (HGC) consists of a tower in which the lateral force resistance is provided by moment resisting frames. A portion of the floor on the East side of the building is cantilevered out from the moment resisting frames, see Sketch 2. This cantilever action induces bending moments into the structure as a whole as illustrated in sketch 3.

- How would this cantilever action influence the behaviour of the building in a major earthquake?

- What would be the likely significance of the cantilever actions for the behaviour of the building in the 22nd February earthquake of 2011?

- Do you agree this would cause the building to ratchet towards the East?

- Does your analysis make any allowance for this ratcheting action and if so how much?

- What would be the significance of the cantilever actions in the 4th September 2010 earthquake?

- Are there any other features of the design of the building which could give rise to ratcheting?

14 Do you have any idea of why the wall 5-6 on line D failed on a diagonal near the base with top part of the wall moving to the West relative to the base of the wall?