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ASPECTS OF THE SEISMIC PERFORMANCE OF CONCRETE BUILDINGS

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SUMMARY

The terms of reference for the Royal Commission into Building Failure caused by the Canterbury Earthquakes were wide ranging covering requirements for design, construction, legal controls and education of structural and geotechnical engineers. Included in the terms of reference was the requirement for the Royal Commission to study a representative sample of buildings to ascertain why some buildings performed satisfactorily and others suffered severe damage, which in some cases caused a loss of life. The representative sample contained a number of multi-storey reinforced concrete buildings. Some of the findings from the study of these buildings are described in the paper.

INTRODUCTION

The representative sample of buildings consisted of 18 reinforced concrete multi-storey buildings, one multi-storey structural steel building and 24 unreinforced masonry buildings. In the paper aspects of the performance of the reinforced concrete buildings are considered with emphasis on those that potentially have important implications for the design of new buildings, or the seismic assessment of existing structures. The performance of foundations is not considered.

The reasons why the seismic performance of some buildings was unsatisfactory can be put into four categories, namely;

1. certain aspects of the design Standards (codes of practice) at the time the building was designed were inadequate, but these short comings have been rectified in more recent editions of the design Standards;
2. fundamental engineering principles were neglected in the design of the buildings;
3. structural details that resulted in unsatisfactory performance that are not adequately considered in our current design Standards and practice. It is this group of problems which receives most attention in this paper as they have implications for revision of design Standards and education of structural engineers;
4 construction errors.

Only a few of the Royal Commission's recommendations for changes in structural design practice are discussed in this paper. Further details are given in volumes 2 and 6 of the Royal Commission report.

In order to interpret the significance of the observed performance of buildings in the Canterbury earthquakes to other centres in New Zealand, it is important to have some

knowledge of the characteristics of these earthquakes. For this reason the earthquake ground motions are briefly described in the following section.

CHARACTERISTICS OF THE CANTERBURY EARTHQUAKE SEQUENCE

While the earthquakes in the sequence were not large in terms of their magnitudes there were a number of ground conditions which resulted in unusually violent ground motion. More detailed information together with technical references on the earthquakes can be found in Volume 1 of the Royal Commission report. Much of the information described below comes from a GNS report to the Royal Commission [Webb et al 2011].

1. The earthquakes were shallow and the seismic energy was released close to the ground surface resulting in intense ground shaking over a relatively small area.
2. The return period of the earthquakes on the faults is in excess of 8,000 years and it is probably considerably in excess of this. In faults which have long return periods there is less damage to the rock adjacent to the fault plane than in faults which have more frequent earthquakes. Consequently in the former case failure occurs at higher stress levels and there is a greater intensity of strain energy released when the earthquake occurs.
3. The faults were initiated 25 or more million years ago when the zone was subjected to spreading. Under this condition the faults developed as normal faults which are steeply inclined at an angle of about 70^{o} to the horizontal, see Figure 1. Subsequently the tectonic situation changed with the zone being subjected to shortening, resulting in reverse faults in which a component of compression, in addition to that resulting from gravity, acted across the plane of the steeply inclined fault plane. This situation combined with the relatively high strength of the underlying rock is one of the possible contributing factors for the high strain energy release in the earthquakes and the unusually violent ground motion.

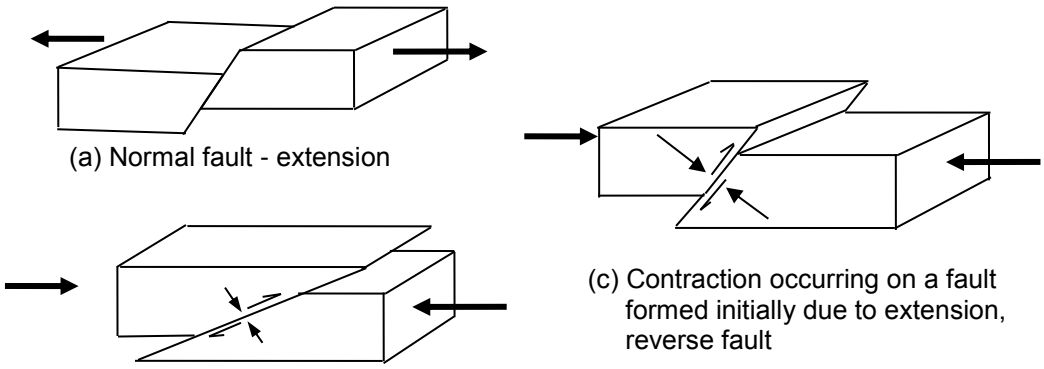


Figure 1: Normal, reverse and thrust faults

Table 1 lists the magnitudes of the four major earthquakes in the Canterbury earthquake sequence. M_w is the moment magnitude which is based on average slip and area of the fault, and M_e is the magnitude calculated on the basis of the energy that is released in the earthquake. The figures indicate that the energy release was high for the magnitude, M_w, of the earthquake.

Table 1: Details of the major Canterbury earthquakes (Source; Webb et al 2011)

Earthquake	Sept. 4th 2010	Feb. 22nd 2011	June 13th 2011	Dec. 23rd 2011
Magnitude M_w M_e	7.1 8.0	6.2 6.75	6.0 6.7	5.9 6.0
Rupture type	Complex	Oblique reverse	Oblique reverse	Oblique reverse
Max. PGA Horiz.(g)	0.3	0.7	0.4	0.4
In CBD Vert. (g)	0.2	0.8	0.2	0.2
Duration (s) accelerations <0.1g	8-15	8-10	6-7.5	3-4

Response Spectra for the and February Earthquake

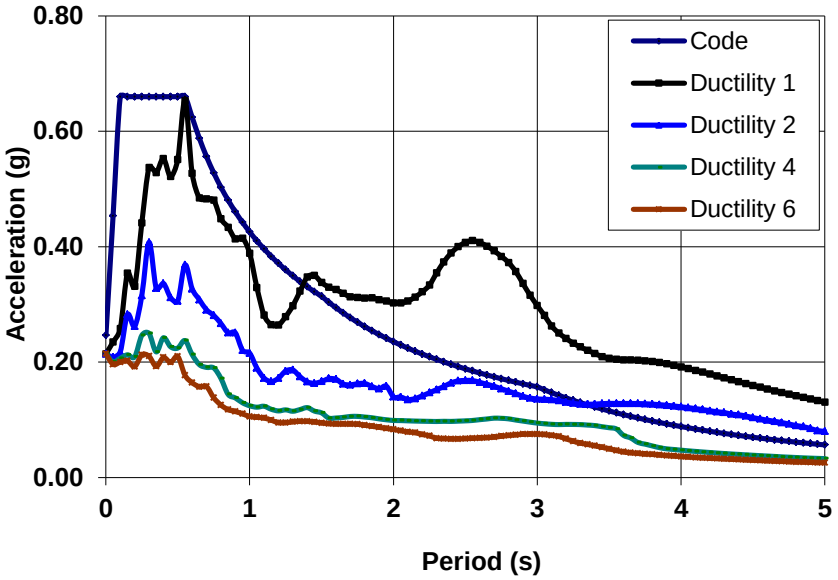
Figure 2 shows the acceleration and displacement response spectra for the September 4th 2010 earthquake in the north-south direction, while the corresponding spectra for the February 22nd 2011 earthquake in the east-west direction are given in Figure 3. These spectra are for the predominate directions of shaking in the longer period range, which is important for multi-storey buildings. Figure 4 shows the spectra for vertical acceleration for the February earthquake. Ground motions were measured at four locations in the CBD and the values in these figures are averaged from these four ground motions. In each of the figures spectra are shown for elastic response and displacement ductility levels of 2, 4 and 6. In calculating these values elastic perfectly plastic behaviour was assumed with 5 percent equivalent viscous damping. In addition the spectral values in the Earthquake Actions Standard (NZS1170.5: 2004) for a 500 year design return period have been added to the figures. The code values are for category 2 buildings with a structural performance factor (S_p) equal to 1.0 and for type D soils, which are appropriate for most of the Christchurch CBD.

The equal displacement concept, in which the peak displacement of a ductile single degree of freedom structure is taken as equal to the peak displacement of an elastic single degree of freedom structure with the same initial stiffness, is incorporated into our design Standards. A comparison of the ductile and elastic displacement spectra for the two earthquakes shows the order of approximation that is involved with this fundamental concept. It should be noted the spectral design displacements in NZS1170.5 shown in the figures are for peak displacements calculated with S_p of 1.0, and not the design displacements, which are found using the S_p factor given in the appropriate materials design Standard.

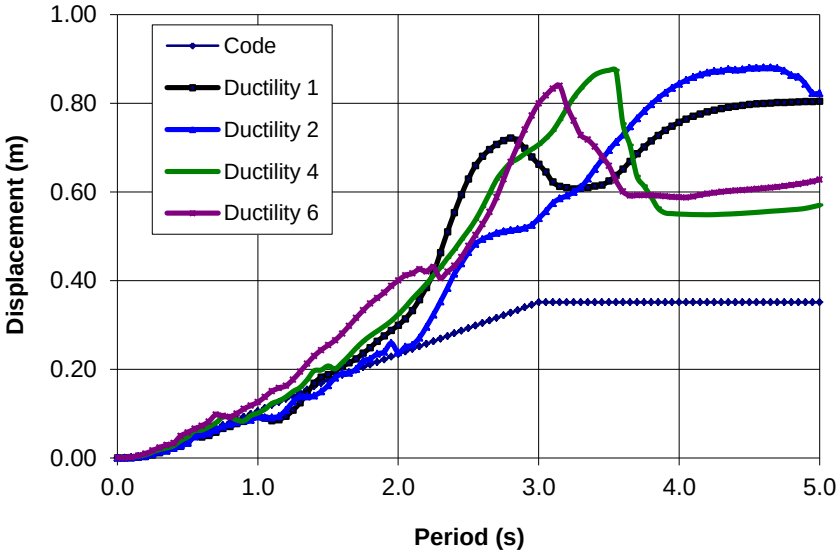
For the September earthquake the response spectral values are less than the design values in the period range of 0 to 1.4 seconds, see Figure 2. There is a marked increase in spectral values above the normal design level (for a return period of 500 years) in the period range above 2 seconds. This is particularly evident in the peak displacement spectra for periods in excess of two and a half seconds. The values in this period range are of the order of 80 percent in excess of the peak design displacement values. These high displacements are particularly significant for high rise multi-storey buildings.

For the February earthquake the response spectral values are greater than the 500 year return design values. When averaged, without allowing for magnitude weighting (described in the next sub-section), the spectra are reasonably consistent with the 2,500 year return design spectra, see Figure 3. A particular feature to notice is the high accelerations and corresponding displacements in the period range of 2.5 to 4 seconds. However, it is important to note that while the elastic response spectral values in this period range are very

high the corresponding ductile response spectra are appreciably smaller. In this period range the equal displacement concept gives a poor prediction of the peak displacement. This is probably due to the short duration of strong ground motion (8-10s) and the relatively few fundamental cycles of vibration that could take place for structures with fundamental periods of vibration in this period range.

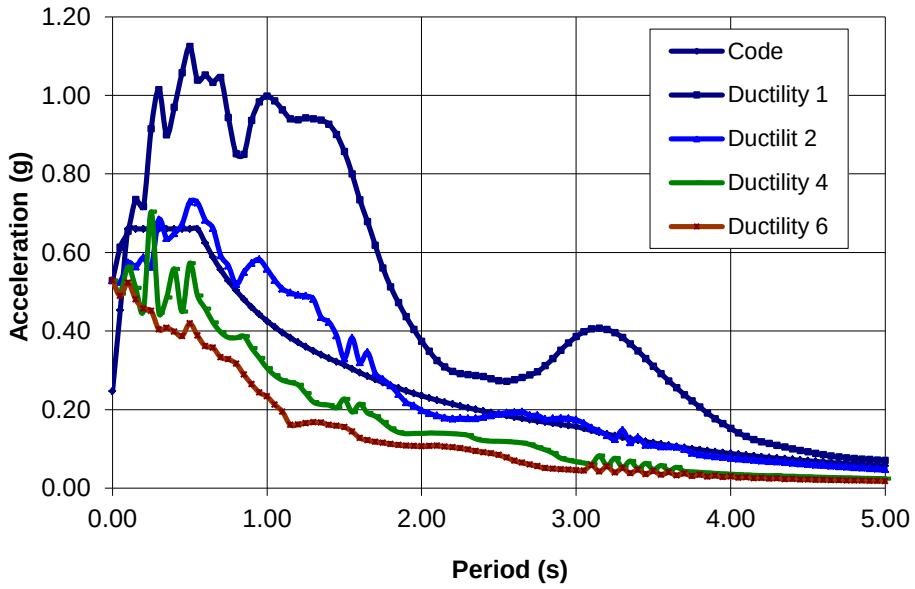


(a) Acceleration response spectra

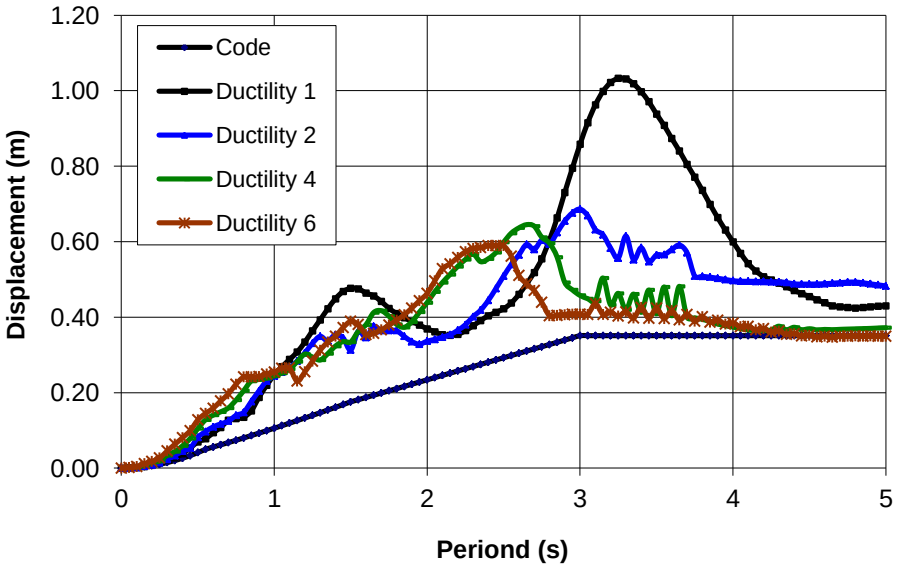


(b) Displacement response spectra

Figure 2: Acceleration and displacement for September earthquake in north-south direction



(a) Acceleration spectra

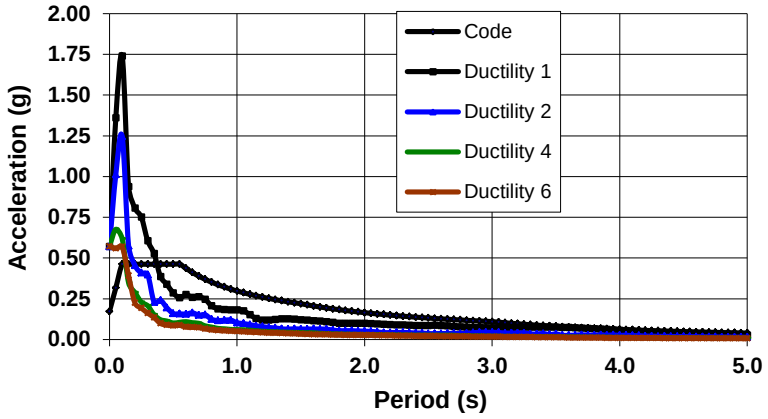


(b) Displacement spectra

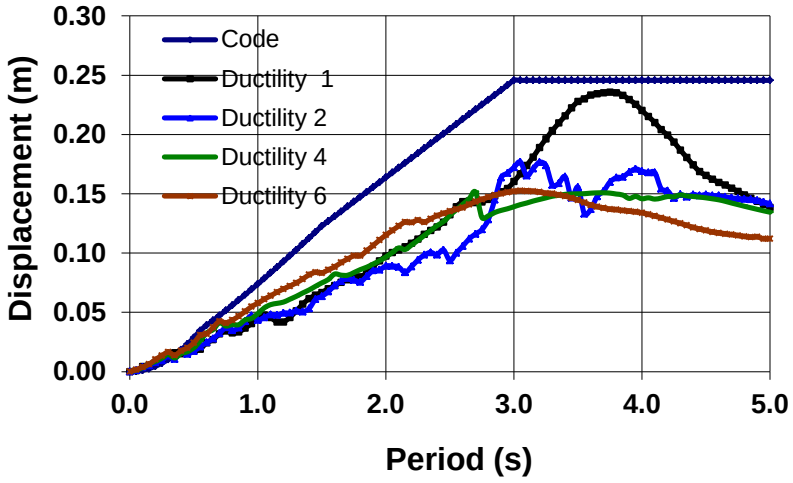
Figure 3: Acceleration and displacement for February earthquake in east-west direction

Figure 4 shows the acceleration and displacement spectra for vertical ground motion for the February earthquake. In this case the shapes of the design spectra are very different from the spectra calculated from the measured ground motions and a review of the design spectrum is justified. However, the high peak in vertical ground accelerations is associated with the shallowness of the earthquake fault and its closeness to the CBD, and this would not be typical of most other situations. While the vertical accelerations were very high for periods close to 0.1 seconds the associated displacements are only of the order of 4mm.

A revision of the design spectrum for vertical ground motion in NZS1170.5 is desirable.



(a) Acceleration response spectra



(b) Displacement response spectra

Figure 4: Response spectra for vertical earthquake motion

Magnitude weighting

Observations made in many structural tests show that, in all but the most brittle structural elements, failure occurs as a result of a combination of the damage accumulated in load cycles involving inelastic deformation and the magnitudes of displacement applied in these cycles. Damage accumulates with the duration of strong shaking and logically this effect should be recognised in seismic design rather than considering just the peak displacement.

In developing design response spectra for seismic actions some allowance has been made for the effect of duration of shaking. Both the duration of shaking and the magnitude of earthquakes increase with the length of the faults. Consequently earthquake magnitude can be used to indicate the likely duration of shaking. A magnitude weighting factor has been used for this purpose. Blind faults, which have no surface features, have short lengths and earthquakes on these faults have short durations of strong ground motion. The February, June and December 2011 earthquakes in the Canterbury sequence were all on blind faults, with magnitudes (M_w) of about 6 and all had short durations of strong ground motion in the range of 4 to 10 seconds (see Table 1).

Magnitude weighting is currently used in the Earthquake Actions Standard, [NZS1170.5: 2004] for the period range of 0 to 0.5s. The weighting factor is given by $\left(\frac{M}{7.5}\right)^{1.284}$ where M is the magnitude of an earthquake (M_w). The weighting factor comes from liquefaction studies with a further limited study that indicated it was appropriate for structural design. Recently, different weighting factors have been developed for the prediction of liquefaction in earthquakes and for rock fall analyses. The background and references on the magnitude weighting factors are given in a report by McVerry et al [2011].

In developing appropriate design spectra for the increased seismicity in Canterbury associated with aftershocks, allowance was made for the likely duration of shaking from known and predicted faults by the application of a magnitude weighting factor [McVerry et al 2011]. However, in this case the weighting factor was applied over the full period range and not just the 0 to 0.5 second range as in NZS1170.5. This approach appears to be logical. The concept of magnitude weighting can be used in reverse to compare the potential damage of an earthquake calculated from recorded ground motion to the corresponding design spectrum. Figure 5 shows an acceleration spectrum from the February earthquake with and without the magnitude weighting (M_w) factor compared to the design spectrum. In this case the earthquake magnitude (M_w) was 6.2 and the corresponding magnitude weighting factor is 0.78.

When the magnitude weighting factor is applied the response spectrum lies between the 2,500 year and 500 year return design spectra, while without the magnitude weighting factor the spectrum is generally in excess of the 2,500 year design values. Further study of the influence of duration of shaking on structural response is desirable given that the quoted magnitude weighting factor comes from liquefaction studies, and not studies on modern ductile structures.

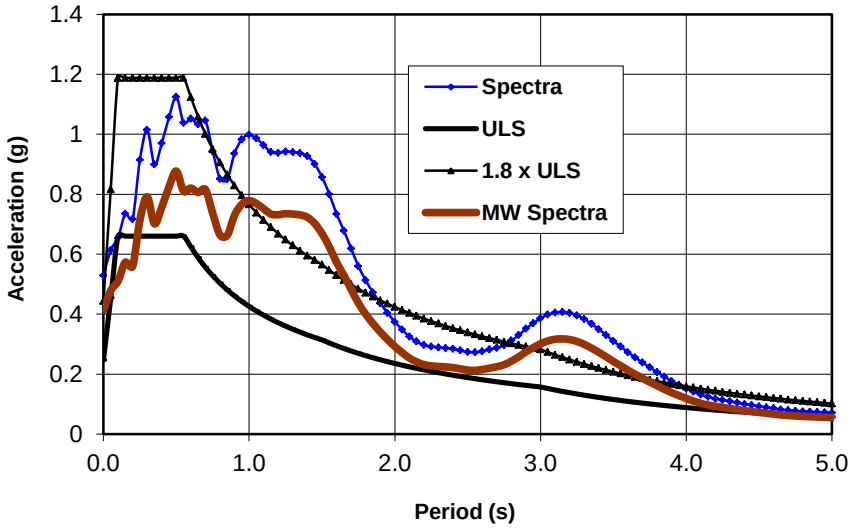


Figure 5: Acceleration response spectrum for February earthquake with and without magnitude weighting compared to design values for 500 and 2,500 year return periods

THE REPRESENTATIVE SAMPLE OF BUILDINGS

The representative sample of buildings consisted of 4 multi-storey buildings which were specified in the terms of reference, 15 multi-storey buildings constructed after 1965 which the Royal Commission selected, and 24 unreinforced masonry buildings that were selected by

the Commission on the basis that failures in these structures caused fatalities. Of the 15 multi-storey buildings 14 were constructed in reinforced concrete and one in structural steel.

The four buildings specified in the terms of reference were, the Pyne Gould Corporation building (PGC), the Hotel Grand Chancellor (HGC), the Forsyth Barr building (FB) and the Canterbury Television building (CTV). The Department of Building and Housing (DBH) arranged for consultants to analyse each of these structures under the guidance of an "Expert Panel". The reports on each of these buildings were forwarded to the Royal Commission, who obtained peer reviews by Mr Holmes, of Rutherford and Chekene, Consulting Engineers of San Francisco. Mr Holmes is a well know international expert in the seismic performance of buildings. The building plans were further studied in the Royal Commission and hearings were held for each of the four structures.

Consulting engineers from Spencer Holmes Ltd, Compusoft Engineering and Rutherford and Chekene were engaged to assess 12 of the group of 15 multi-storey buildings and the remaining three were assessed by the Royal Commission staff. The structural details of all of the 12 buildings were further examined by the Royal Commission. There were no hearings for this group of structures.

The performance of all of the unreinforced masonry buildings was assessed by Spencer Holmes Ltd and hearings were held on each of these buildings.

STRUCTURAL ASPECTS RELATED TO DESIGN

Ratcheting of buildings and building components

The Hotel Grand Chancellor, HGC, was the highest building in Christchurch. Analysis of this structure provides an example of ratcheting under seismic actions. In the February earthquake a relatively short wall that was orientated in the north–south direction failed in the first storey of the building. Due to this failure the building swayed towards the east. It is not known how much the ratcheting action described in the subsequent paragraphs contributed to this failure, or for that matter what was the actual cause of failure of the wall other than that it was subjected to a high axial load. To assess the cause of failure of the wall it is essential to know the axial load level together with the in-plane and out of plane displacements that were imposed on the wall. These factors can not be determined in this highly indeterminate structure with any confidence by an analysis based on elastic response. However, due to the shortage of time and resources, and the very considerable complexity of carrying out an inelastic time history analysis, the DBH accepted a report on the causes of failure based on the elastic response spectrum method of analysis.

Lateral force resistance for the Hotel Grand Chancellor was provided by structural walls up to level 12. The height of this level was equivalent to 6 storeys due to split levels with a parking building next door that was tied into the hotel. Between levels 12 and 28 a ductile reinforced concrete moment resisting frame provided the lateral force and gravity load resistance.

A lane on the east side of the building resulted in the eastern most bay of the building being cantilevered off the remainder of the structure. The lane can be seen on the right hand side of the photo in Figure 6 and in the structural arrangement shown in Figures 7 and 9.

Figure 6: Photo of Hotel Grand Chancellor looking north

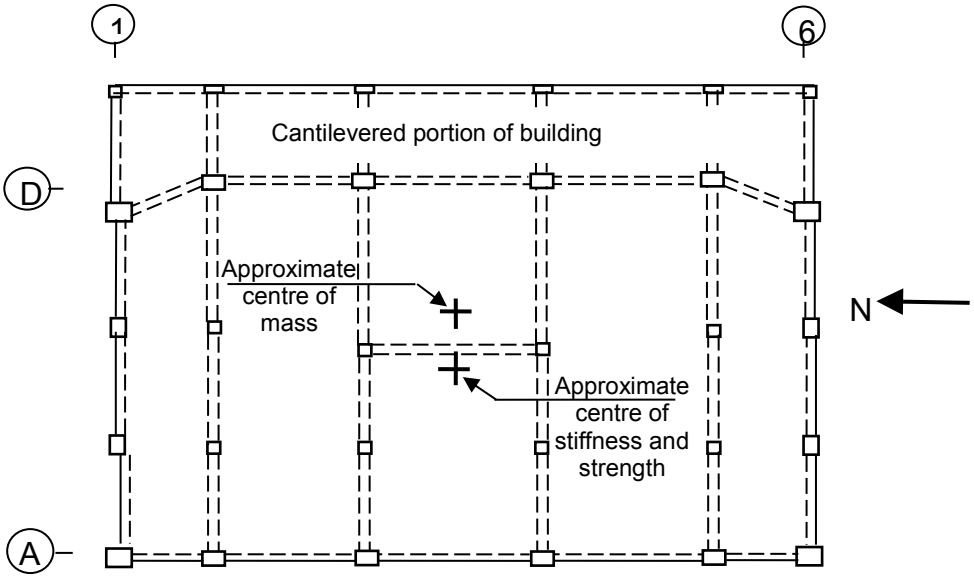


Figure 7: Plan on typical floor between levels 14 and 24

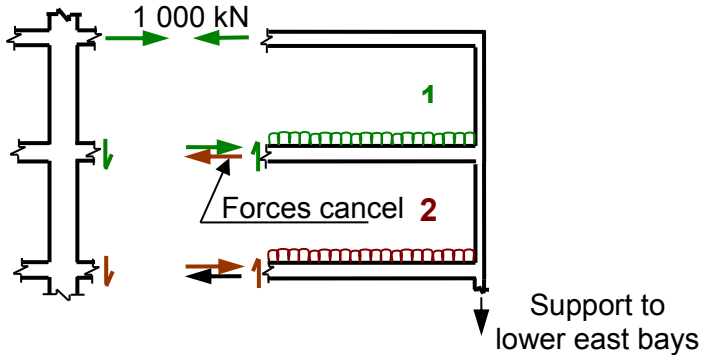


Figure 8: Cantilever action of east bay on the moment resisting frame

Figure 8 shows how the eastern most bays above level 12 were cantilevered off the moment resisting frame. Below level 12 transfer beams were used to carry the eccentric loading to the structural walls, see Figure 9. Structural analysis of the bending moments induced by the gravity loading on the cantilevered spans shows action can be represented by a lateral force of 1,000kN acting at level 25, as shown in Figures 8 and 9. The eccentric gravity loading would have bent the building towards the east, though the lateral deflection would have been small. However, more significantly, it reduced the lateral seismic force that would cause inelastic deformation in the eastward direction and it increased the lateral seismic force which would cause inelastic deformation in the westward direction.

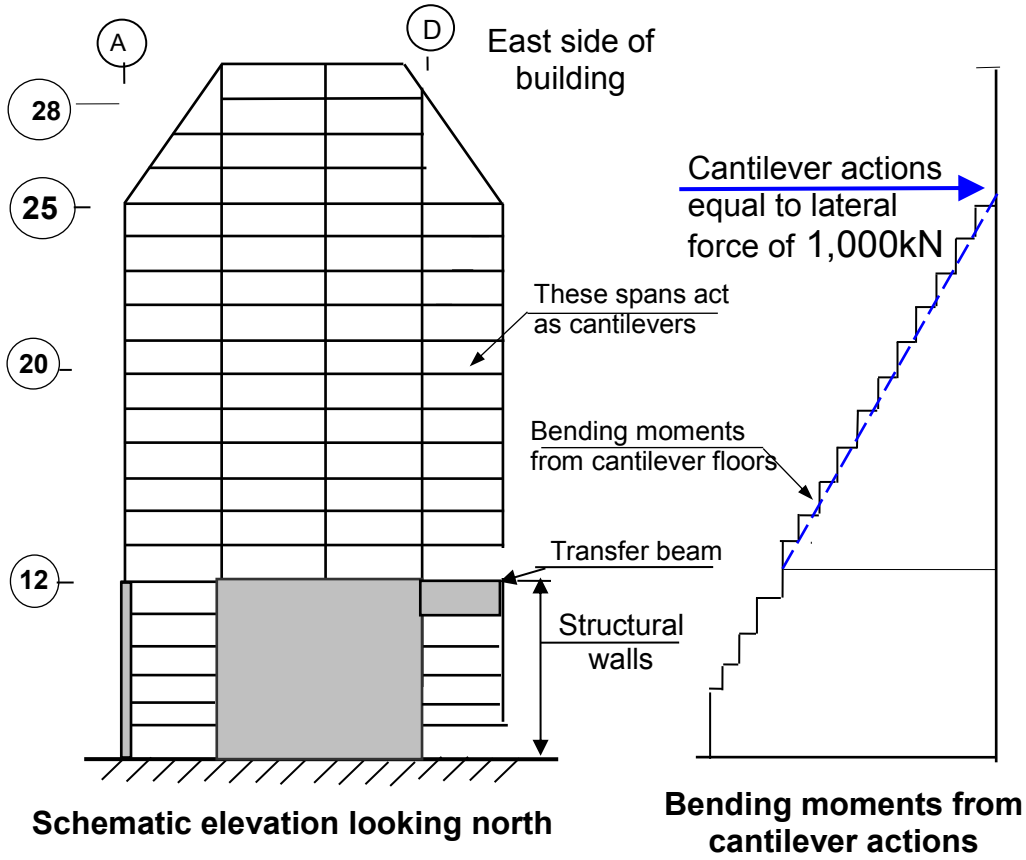


Figure 9: Schematic elevation of HGC and actions due to eccentric gravity loading

To investigate the likely effect of the eccentric gravity loading the Royal Commission arranged for Professor Carr to assist in carrying out a series of time history analyses for seismic actions in the east–west direction. A simplified analytical model of the building was developed. As there was no apparent damage in the main structural walls they were

modelled as elastic elements. The short wall that failed near ground level would not have contributed significantly to seismic actions in the east-west direction. The ductile frame was modelled by a shear resisting element and a column. These two elements were given with the appropriate strength, stiffness and hysteretic properties to match those of the moment resisting frame as designed. Deformation in the foundation soils was neglected.

Analyses were carried out for;

- the four ground motions recorded in the CBD for the February earthquake,
- a compound record built up from the ground motions recorded during the September, Boxing Day and February earthquakes at the REHS site in the CBD, and
- an artificial ground motion representing an Alpine fault earthquake.

In all cases only the east-west excitation was applied to the model.

Figure 10 shows the predicted lateral displacement envelopes found from the analyses. The lateral displacements found from the four recorded ground motions in the CBD of the February earthquake were similar in magnitude to each other with the drifts ranging between 0.72 to 0.95 metres at a height of 70 metres. The corresponding inter-storey drifts are in the range of 3.5 to 4.5%. The compound record of the three earthquakes made little difference to the lateral drifts. The predicted displacements with the simulated Alpine Fault earthquake are about half of the corresponding values found using the February earthquake ground motions.

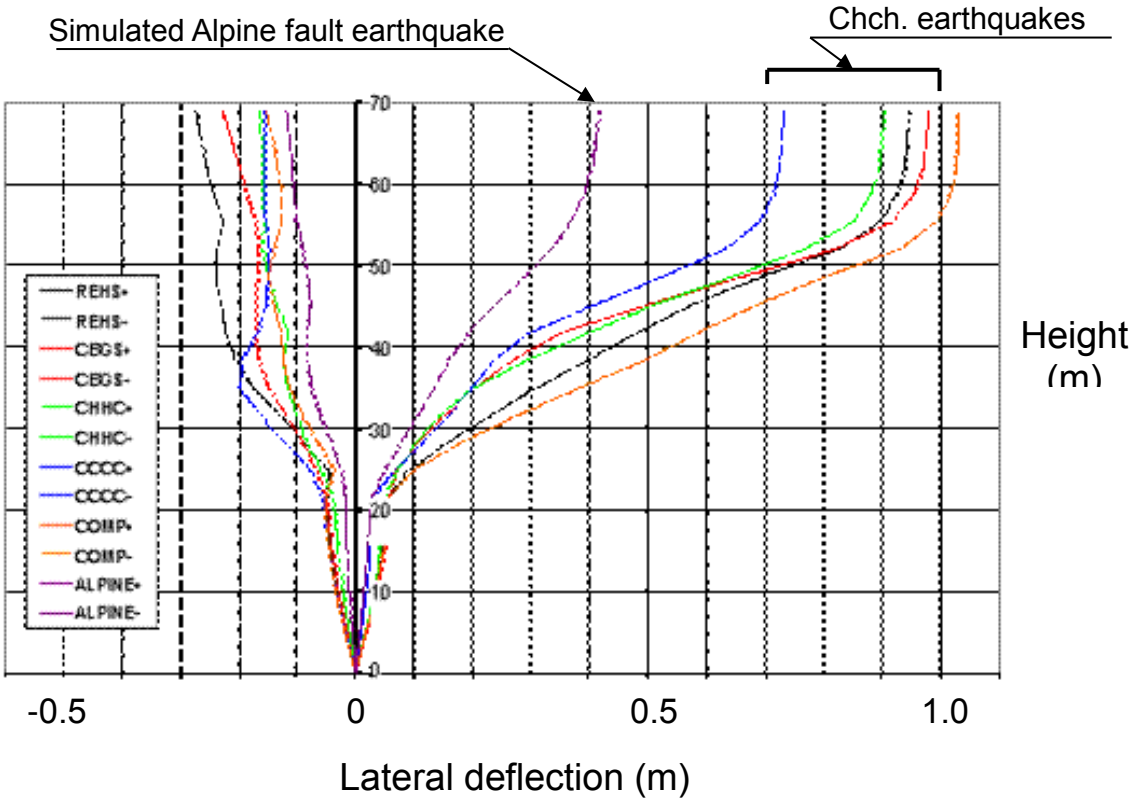


Figure 10: Lateral deflection envelope found from different earthquake ground motions

Figures 11 and 12 both show the predicted displacement versus time for two points, one at the top of the walls and the other at a height of 70m above the base. In Figure 11 concentric gravity loading is assumed while Figure 12 allows for the eccentric gravity loading. These analyses were made using the ground motion recorded at the REHS site in the CBD. A comparison of the two figures shows that the eccentric gravity loading causes the structural

displacement to progressively increase during the earthquake. During the period of strong ground motion four major cycles of displacement occurred. In each of these cycles the structure swayed further towards the east, that is ratcheting occurred. Additional results from the analyses are given in Volume 2 of the Royal Commission Report together with more information on the inelastic model of the building that was used in the analyses.

An inherent assumption in the equivalent static and response spectrum methods of analysis is that the structure has equal strengths and stiffness for forwards and backwards displacements. This assumption is violated in the HGC building due to the cantilevered gravity loading. However, more common structural forms, such as Tee shaped walls and columns with non-symmetrical reinforcement arrangements, also violate this basic assumption. Where this occurs rational allowance should be made for the under prediction of deformation in the direction which has a lesser strength. As far as the HGC was concerned it would have been possible to change the distribution of reinforcement in the beams to give the structure equal strength for seismic actions in the east and west directions. Such a step would have reduced but not entirely eliminated the tendency of the structure to deform progressively towards the east in a major earthquake. In this situation the displacement to the east that results in inelastic deformation is smaller than the corresponding displacement to the west.

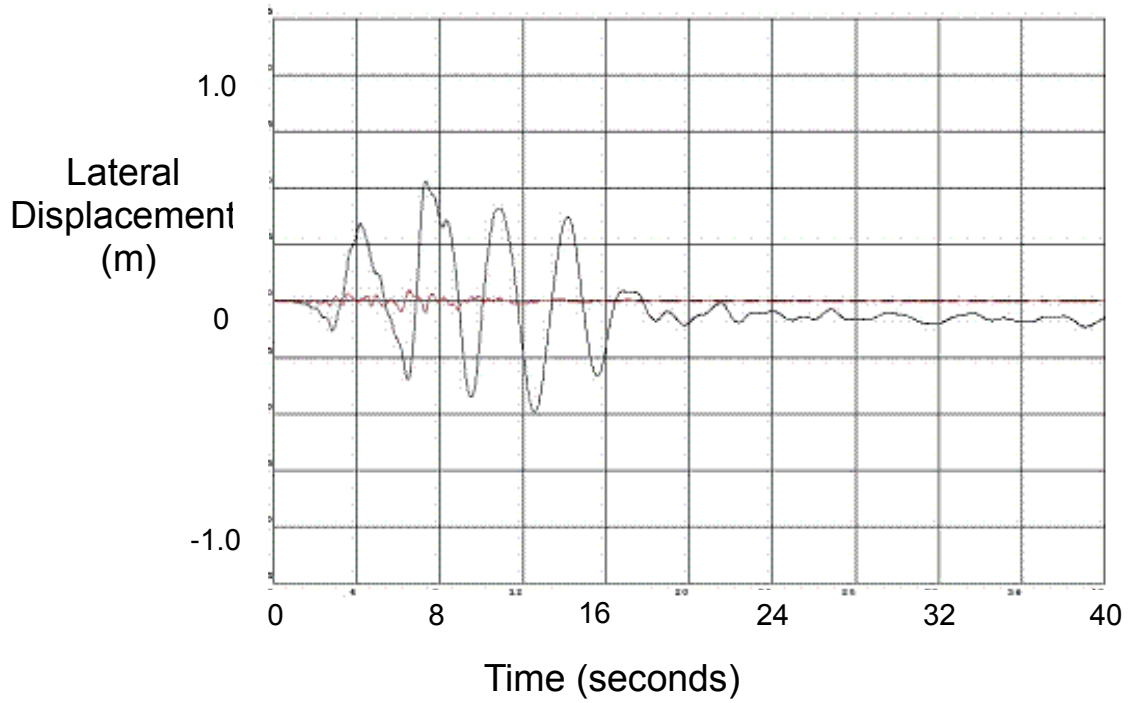


Figure 11: Lateral deflection at top of building versus time for case with no eccentric gravity load

Structural designers need to be aware of the fundamental assumptions inherent in the methods of structural analysis. Ratcheting is one example where erroneous results can be produced when the structure being analysed does not comply with fundamental assumptions inherent in the method of analysis. Guidance on ratcheting should be added to the Earthquake Actions Standard.

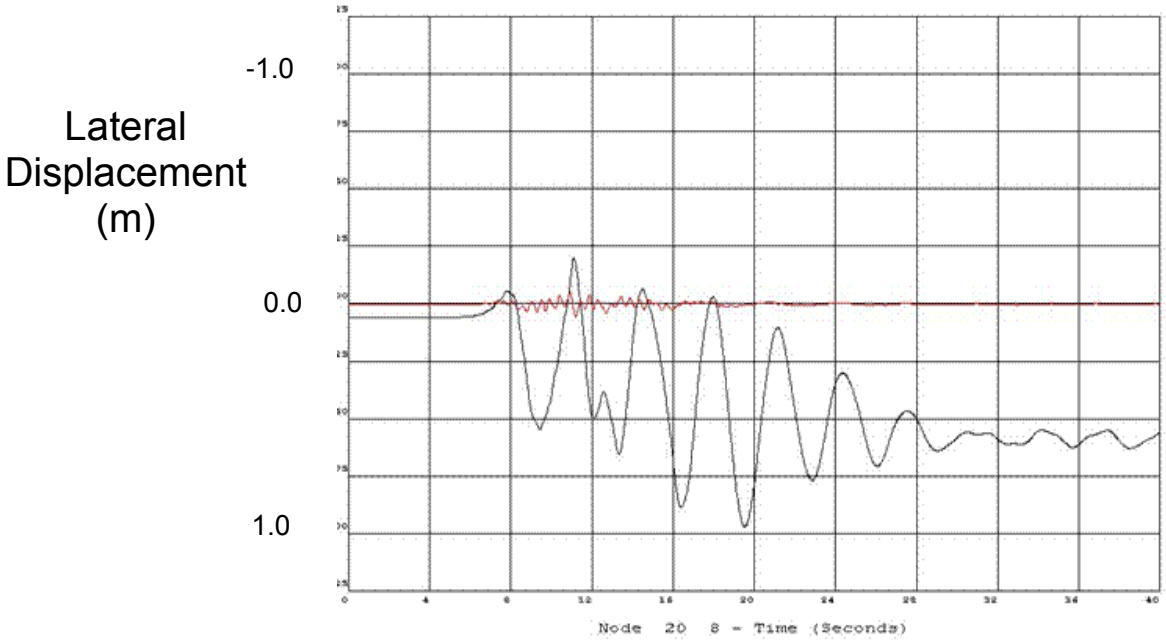
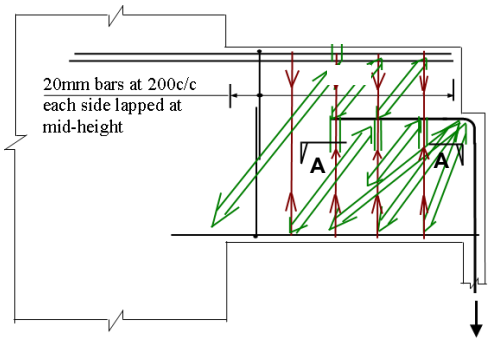


Figure 12: Lateral displacement versus time with eccentric gravity load

Spalling of concrete due to high bond stresses of reinforcement in cover concrete

Figure 13 shows part of one of the transfer beams which supported the floors in the east bay of the building below level 12. In the February earthquake extensive spalling occurred in the beam close to the location where the longitudinal reinforcement from the member supporting the eastern bays of the building was anchored into the beam. The spalling occurred near the mid-depth of the beam where U shaped stirrups were lapped in cover concrete. The figure also shows a strut and tie model of the beam. Section A-A is shown on Figure 14.



Strut and tie model

Spalled concrete in lap zone of stirrups - 2nd

Figure 13: Spalling in a transfer beam in the HGC in the region where stirrups were lapped in cover concrete (Source of photo: Adam Thornton)

It is very likely that the spalling occurred due to the shock loading associated with the failure of the short wall in the first storey. High bond stresses would have been induced in the stirrups in their lap zone. As illustrated in Figure 14 bond forces induce radial tensile stresses in the concrete surrounding the reinforcement which generates hoop tensile stresses in the concrete. Cracking associated with these stresses may develop between the bar and the free surface of the concrete. However, where the cover concrete is thicker that half the clear spacing between the bars, or where the tensile stresses in the cover concrete

are suppressed by compression, cracks may form between the bars leading to spalling of the concrete as shown in Figure 14.

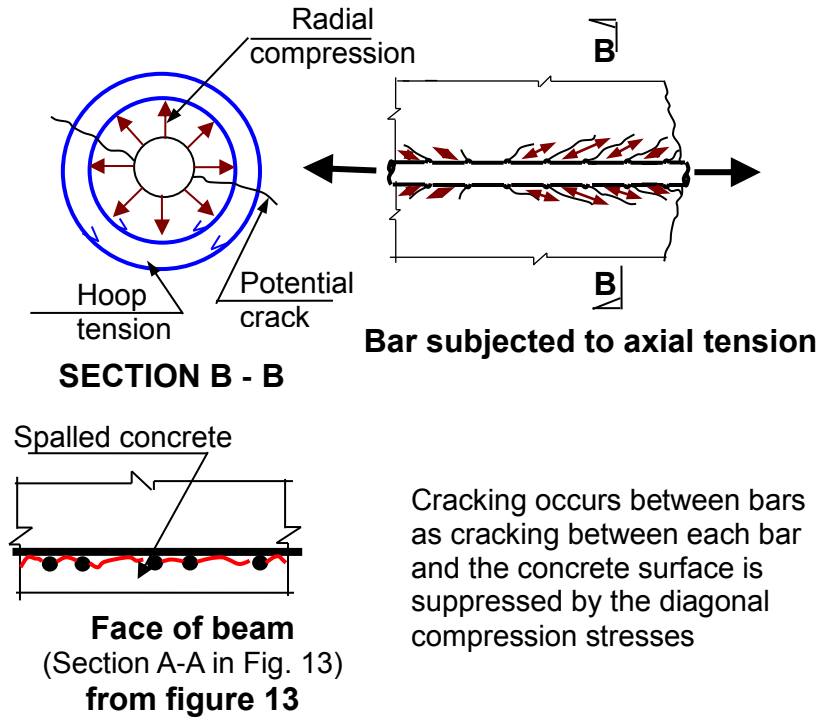


Figure 14: Tensile cracking of concrete due to bond stresses in the reinforcement

The problem of potential spalling caused by bond stresses associated by the lap between U shaped stirrups can be reduced by placing a high proportion of the stirrups so that they are lapped in the confined concrete in the central regions of the beam.

High bond stresses between concrete and reinforcement can result in spalling in a number of different situations. Where yielding of reinforcement occurs at a crack high bond stresses are induced in the concrete on each side of the crack. This action can initiate spalling if either longitudinal or transverse reinforcement is closely spaced and highly stressed. High bond stress is an important factor which contributes to the spalling that occurs in plastic hinge zones.

Load tracking though a structural detail

One of the structural details used in the Clarendon building illustrates the importance of detailed load tracking. Figure 15 shows a photo of the building and Figure 16 gives a simplified sketch of the layout of the floors in the tower of the building.

Figure 15: Photo of Clarendon Building

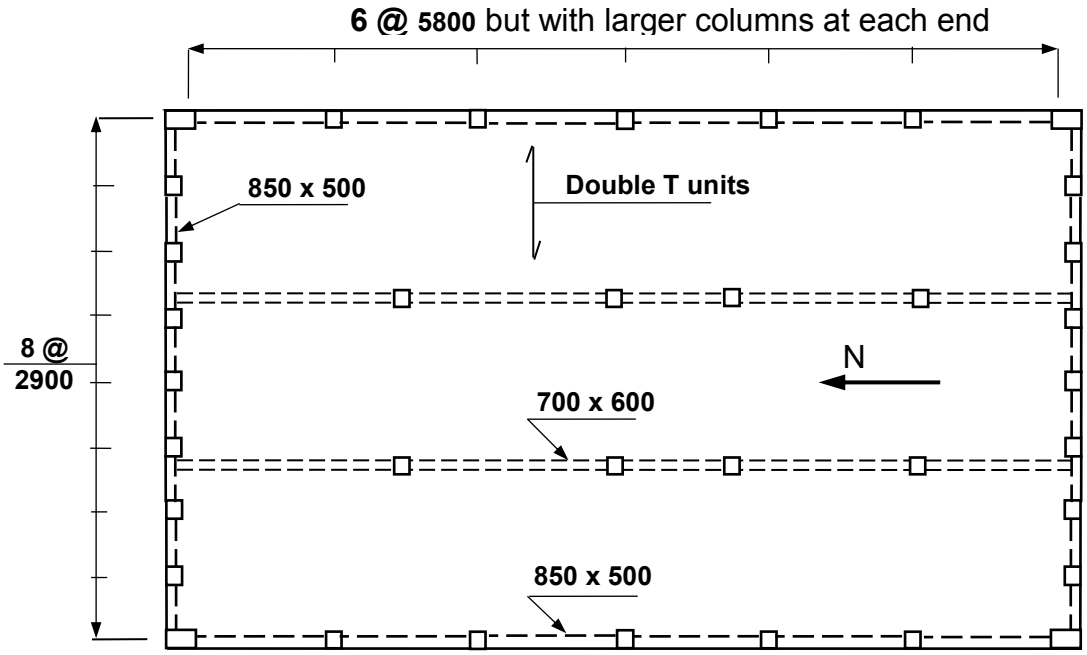
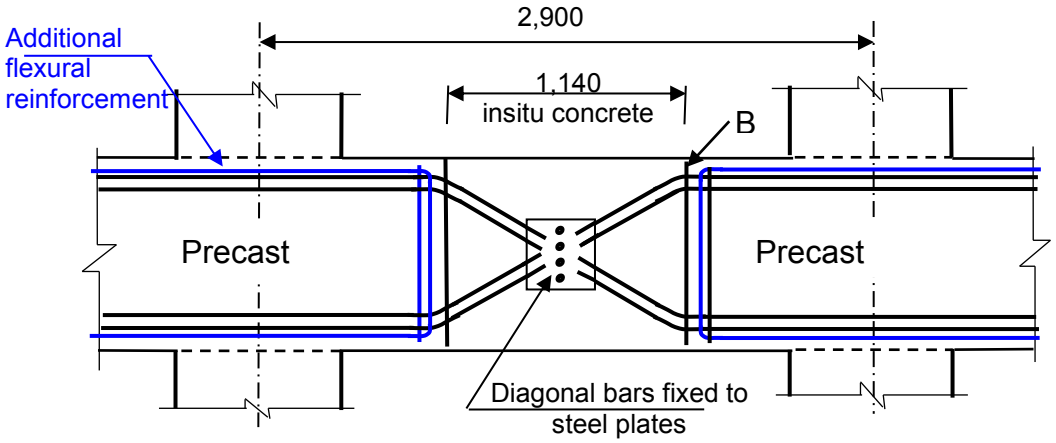


Figure 16: Structural layout of floor

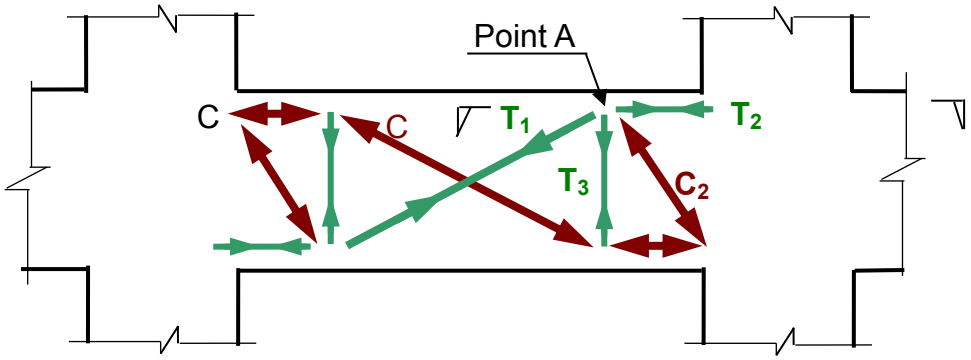
Moment resisting frames were located on all sides of the structure. The beams in these frames were 850mm deep. On the north and south sides of the building the clear span of the beams was 2,100mm. The corresponding beams in the east and west walls had clear spans of 5,000mm. With the longer length the beams could be detailed for ductility with potential plastic hinges adjacent to the columns. However, for the beams with a clear span of

2,100mm this arrangement was not practicable due to the low moment to shear ratio. To obtain ductile performance of these short members they were designed with diagonal reinforcement in their mid regions, which was intended to yield, giving a shear type deformation mode in a major earthquake. This method of obtaining ductile behaviour was based on the observed good performance of diagonally reinforced coupling beams in coupled structural walls. The use of this technique for obtaining ductile performance in beams is described in the book written by Paulay and Priestley [1992]. At the time when the building was designed it was considered an advanced technique. The plastic deformation in diagonally reinforced coupling beams occurs by yielding of the diagonal bars in compression and tension as the structure sways backwards and forwards. The basic outline of a typical beam is shown in Figure 17. To limit yielding to the diagonal bars additional flexural reinforcement was added to the outer regions of the beam. A strut and tie model of the beam is illustrated in parts (b) and (c) of Figure 17.

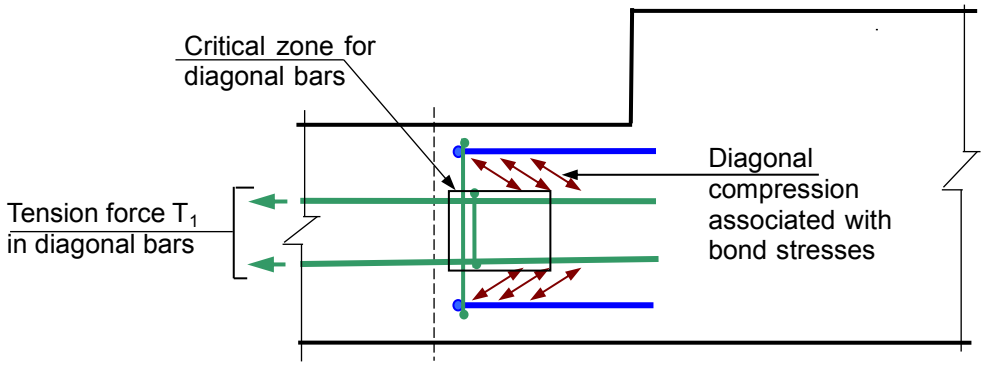
A close examination of the forces in the strut and tie model at point A, see Figure 17 (b), shows that the tension force T_2 in the horizontal continuation of the diagonal bars balances the longitudinal component of the tension force T_1 in the diagonal bars and a proportion of the longitudinal component of the of the compression force C_2. The diagonal compression force C_2 provides the shear resistance in the region between the column face and the start of the diagonal bars. For equilibrium to be satisfied the proportion of C_2 acting against the bend in the diagonal bars must be equal to or greater than the vertical component of the tension force T_1. From this analysis it can be shown that tension force in the horizontal extension of the diagonal bars in the region of the bend and for a short distance from this location towards the column, is greater than the force in the diagonal bars. Between the bend in the diagonal bars and the face of the column, bond stresses distribute some of the tension force in horizontal extension of the diagonal bars to the bars located closer to the sides of the beam, see Figure 17 (c). The outer beam bars were added to prevent yielding at the face of the column, see Figure 17 (a). The load tracking shown in the figure indicates that the highest stress in the longitudinal bars is located at the bends in the diagonal bars and for a short distance from this bend towards the adjacent column. Yielding of reinforcement is confined to this short length and it could not spread over the length of the diagonals as originally intended. This reduced the ductility of the beams. The defect was found in a structural test carried out by a post graduate student at the University of Canterbury after the building had been constructed [Restrepo-Posada 1993].



(a) Elevation on beam, only principle reinforcement is shown



(b) Strut and tie model for beam



(c) Critical zone for diagonal bars

Figure 17: Details of beams in north and south moment resisting frames

Stability of Building in Torsion

There was little eccentricity of the centre of mass to the centre of stiffness in the Clarendon. In terms of the Earthquake actions Standard, the building would be classified as a regular structure. However, in the February earthquake the moment resisting frame on the North face of the tower suffered much more damage than the other moment resisting frames.

As described in the previous section, the moment resisting frames on the north and south sides of the tower had 8 bays of 2.9m while the corresponding moment resisting frames on

the east and west sides had 4 bays with spans of 5.8m and two bays with spans slightly larger bays due to larger columns being used in the corners of the building. The relative stiffness of the internal frames was low due to the use of smaller beams and longer spans, as shown in Figure 16. With this arrangement the lateral seismic and building torsion actions are predominately resisted by the perimeter moment resisting frames. The frames on the north and south sides of the building had a much higher stiffness than the corresponding frames on the east and west sides. On the basis of an approximate elastic analysis about 70 percent of the building torsion would be resisted by the frames on the north and south sides. In addition to the differences in stiffness it should be noted that initial yielding in the beams in the frames on the north and south sides would occur at a drift which is appreciably smaller than the corresponding drift in the frames on the east and west sides.

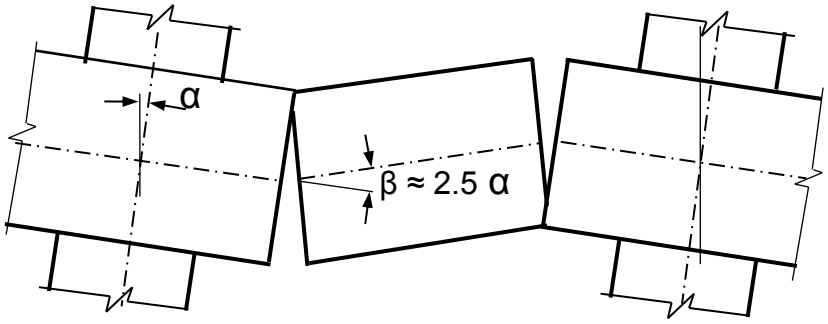
There appear to be two potential explanations for the observed concentration of damage in the north moment resisting frame.

1. Both Mr Hare and Mr Smith[1] pointed out that under seismic actions lateral forces could cause either the moment resisting frame on the south or north sides to yield. If the north frame yields it loses its ability to resist forces arising from building torsion. The centre of stiffness moves towards the south frame and the building torsion is resisted by the torsional inertia of the floors and forces in the frames on the east and west sides of the building. However, to develop forces in these frames the building must twist and this action results in amplified inter-storey drifts on the north frame. For example if the perimeter frames in the east and west walls are to reach their yield point in resisting torsion the displacement imposed on the north wall is of the order of 6 times the initial displacement which would cause this frame to yield. These amplified drifts would accentuate damage on the north frame. It should be noted that the shift in centre of stiffness to the south wall and the enhanced deformation on the north wall would occur predominately while the north wall was yielding. From this analysis it can be deduced that the building was torsionally unstable when one of the stiffer perimeter frames was yielding.

2. Displacing the plastic hinges away from the column face increases the inelastic rotation sustained by the plastic hinges. Figure 18 shows that the inelastic rotation of the plastic hinges (β) in the beams in the north and south frames is close to two and a half times the inelastic drift in the frame (α). This amplified rotation has two main effects. First, geometric elongation is increased and second, the increased inelastic rotation increases the potential for stiffness degradation. Tests on many beams have shown the resultant elongation on beams subjected to inelastic load cycles is generally of the order of two to three times the geometric elongation [Fenwick et al 2010]. This mechanism causes damage to floors, increases stiffness degradation and it can damage columns. For the frames on the east and west sides the plastic hinge rotations are close to one and a quarter times the inelastic drift on the frames. Figure 19 shows the stiffness degradation measured in tests of three moment resisting frames. The beams had different span to depth ratios (L/h). Two of these tests were made by Restrepo-Posada [1993] with one being a scale model of the beams and columns used in north and south sides of the Clarendon, and the second was identical except that the beams contained standard longitudinal reinforcement with plastic hinges forming at the face of the columns. The third test shown in the figure is taken from the test of a multi bay frame in which the beams had considerably longer span to depth ratios (L/h) [Wuu 1996]. It can be seen that the form of beam used in the Clarendon in the north and south frames degraded in stiffness more rapidly than the conventional beams. If the north frame initially sustained inelastic deformation before the south frame, the stiffness degradation

[1] Personal communications with both John Hare and Peter Smith

would have resulted in the centre of stiffness moving towards the south, which would have increased the drifts on the north wall regardless of whether the beams were yielding or not.



Geometric elongation equal to β (d – d')

Figure 18: Diagrammatic illustration of inelastic deformation of a beam with diagonal reinforcement in central region

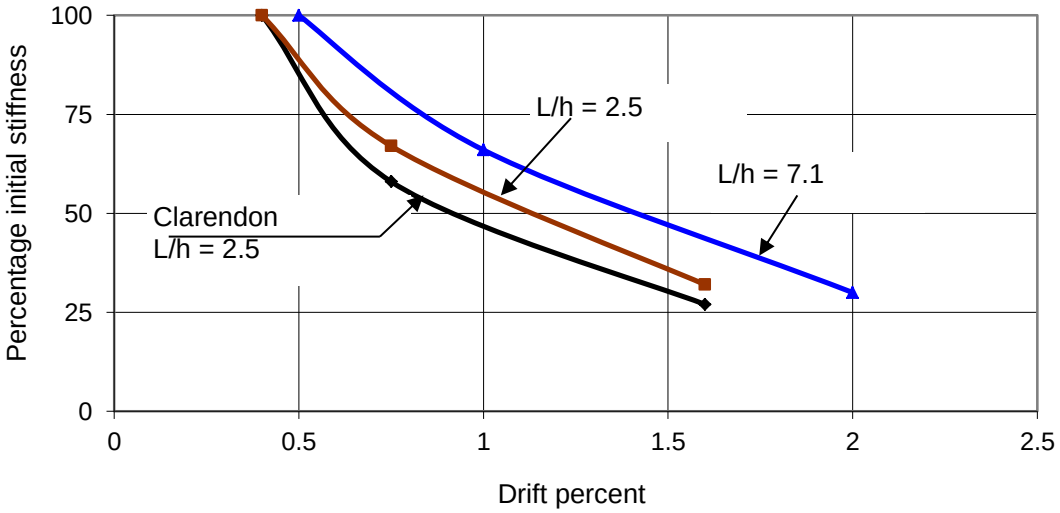


Figure 19: Stiffness loss with cyclic loading

Research is required on the torsional stability of buildings with the object of developing design criteria to guard against this form of instability.

Elongation of Beams

Elongation of structural concrete members has been observed in numerous tests from the late 1970s onwards [Fenwick et al 2010]. However, it has received little attention in design standards, or in most text books, though there are brief descriptions in the books by Booth [1994] and by Paulay and Priestley [1992]. Consequently elongation has been largely neglected in design. Most standard structural analysis packages incorporate a number of assumptions to simplify the analytical process. One of these assumptions is that elongation associated with the formation of flexural and diagonal tension cracking in concrete members is ignored.

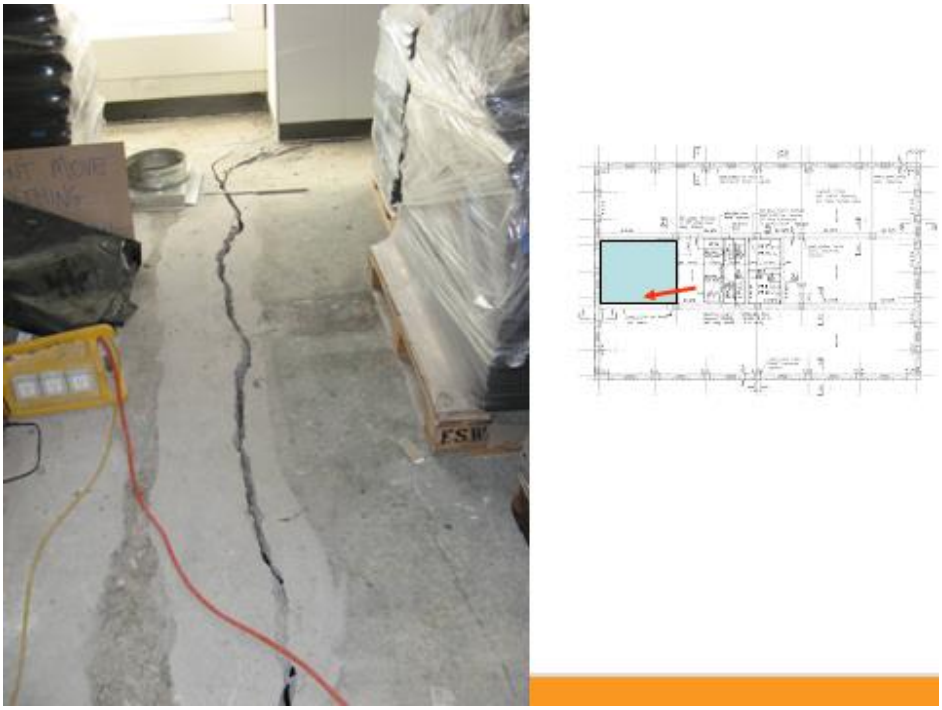


Figure 20: Photo of crack in floor due to elongation in the Clarendon building (Source Holmes Consulting Group)

The Royal Commission examined four buildings in which a significant proportion of the lateral force resistance was provided by ductile moment resisting frames. In all of these structures there was damage in the floors caused by elongation of beams that formed plastic hinges. This demonstrates that elongation is not just an academic problem. The photograph in Figure 20 shows cracking of a floor slab in the Clarendon due to elongation. The contribution of floor slabs to the strength of beams, which is associated with elongation, is considered in the Concrete Structures Standard [NZS3101: 2006], but the wider implications of this action are not covered in any depth.

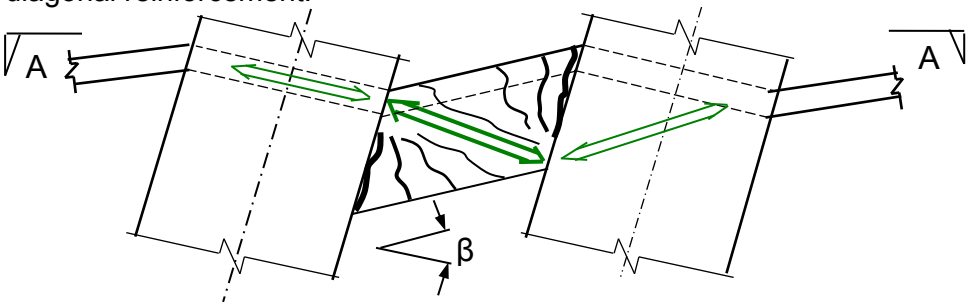
Elongation in structural walls

Most research on elongation has concentrated on beams and beam slab systems. However, elongation also occurs in columns and in walls. The effect in walls can be particularly significant, as elongation increases with the vertical length of the wall. Any increase in height of a wall due to plastic hinge formation, rocking or even flexural cracking, will be partially restrained by the surrounding structure unless the wall is detailed to allow relative vertical movement to develop between it and the surrounding structure. The extent of the restraint to the elongation in a wall depends on its location and its orientation to other vertical structural elements and on the stiffness and strength of the floors that connect the wall to the surrounding structural elements. The structural actions associated with elongation of plastic hinges in walls are generally highly indeterminate, which makes it difficult to assess the magnitude of axial forces that may be induced by elongation in an earthquake.

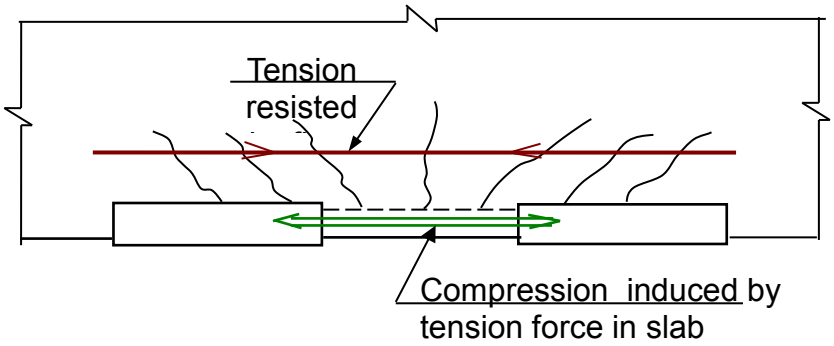
Research on elongation in walls is desirable so that guidance on how to allow for adverse effects in design can be added to design standards.

Interaction of floors with coupled shear walls

Coupling beams with span to depth ratios of less than two are usually designed with diagonal reinforcement. With this arrangement of reinforcement the beams deform in the inelastic range in a shear type mode. Each set of diagonal bars is restrained against buckling by closely spaced ties. The deformation mode is illustrated in Figure 21. The conventional assumption in the design of coupling beams is that the diagonal compression and tension forces in the reinforcement are equal to each other. This assumption implies that there is no axial load acting on the coupling beams, and the shear that can be transferred between the coupled walls by each beam is limited to the vertical component of the strain hardened diagonal reinforcement.



Elevation on coupling beam



Sectional plan A – A showing tension floor slab

Figure 21: Interaction of floors with coupled shear walls

The diagonal compression strut in a coupling beam is stronger and stiffer that the corresponding diagonal tension force owing to the contribution that the concrete can make to the compression strength. As the sway on the walls increases the diagonal compression strut pushes the coupled walls apart as the coupling beam elongates. Restraint against this elongation is provided by the foundation beam and by floor slabs over the height of the building. As shown in Figure 21 the elongation of the coupling beam induces tension forces in the floor slabs with corresponding axial compression forces in the coupling beams. These axial forces increase the shear that can be transferred between the coupled walls.

To achieve ductile performance coupled shear walls are designed to form plastic hinges at the base of the walls and in the coupling beams. If the coupling beams are too strong the inelastic deformation is forced into plastic hinges at the base of the walls. The loss of energy dissipation in the coupling beams reduces the ductility of the structure. There is some evidence that the contribution of the floor slabs to the strength of the coupling beams prevented the coupled shear wall in the CTV building from performing in its intended ductile mode (see Volume 6 of the Royal Commission Report, section 6).

Further research is required to identify how the effect of the restraint to elongation of coupling beams provided by foundation beams and floor slabs can be quantified for design purposes.

Interaction of floors with structural walls

Floor slabs which are tied into structural walls act in a similar manner to stirrups in resisting shear. However, floors are much stiffer than stirrups and strain energy considerations indicate that floors will attract a high portion of the resistance which would otherwise be resisted by stirrups spread over a region of the wall.

The high stiffness of a floor when compared to stirrups can be expected to result in;

- diagonal compression struts that are more concentrated in the wall, and
- compression struts developing at a smaller angle to the horizontal in walls that are longer than the inter-storey height than in a wall without floors that effectively clamp the wall against lateral expansion.

Research on the likely implications of the interaction of floors with walls is desirable.

Flexural cracking and ductility of lightly reinforced structural walls

The first primary flexural crack forms in a structural member when the tensile stress at the extreme tension fibre exceeds the tensile strength. The crack reduces the tensile stresses in the concrete for a distance L_{pc} along the member, as shown in Figure 22 for the case of a beam. When the reinforcement content of the member in its web is low a second primary crack may form at a distance between L_{pc} and $2L_{pc}$ from the first primary crack. The initial spacing of the primary cracks is principally a function of geometry of the member and to a lesser extent the proportion of longitudinal reinforcement in the web of the member. Primary cracks would form at a similar spacing even if the reinforcement was un-bonded in the constant moment region of a beam, provided it was bonded in the shear spans.

If the zone between the two primary flexural cracks is considered it is apparent that the curvature in the compression zone is greater that the corresponding curvature in the tension zone. To prevent incompatible displacements from developing, tensile stresses develop across the neutral axis in the vicinity of the flexural cracks, while compression stresses are sustained in the mid region between the cracks, as illustrated in Figure 22. With an increase in bending moment, the tension and compression forces acting across the neutral axis increase. This may result in additional cracks developing either between existing primary flexural cracks, or in the extreme as occurs in some prestressed concrete members, as cracks due to the tension force “T”. Such cracks form approximately parallel to the neutral axis. Generally when additional load is applied, the spacing of primary cracks in a beam reduces until it is approximately equal to the distance between the flexural tension reinforcement and the neutral axis.

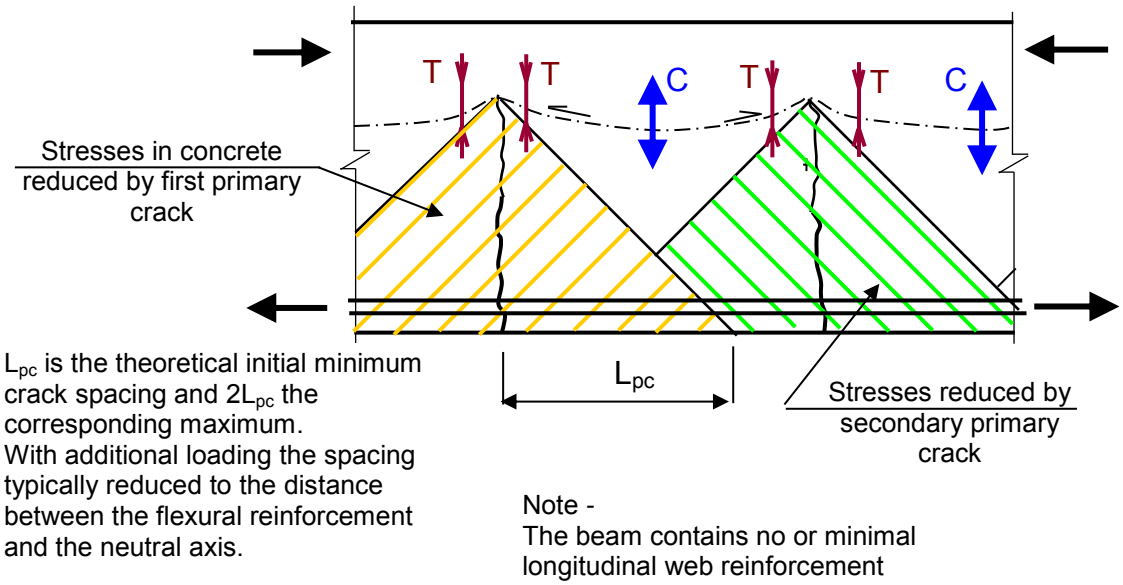


Figure 22: Spacing of primary cracks in a beam

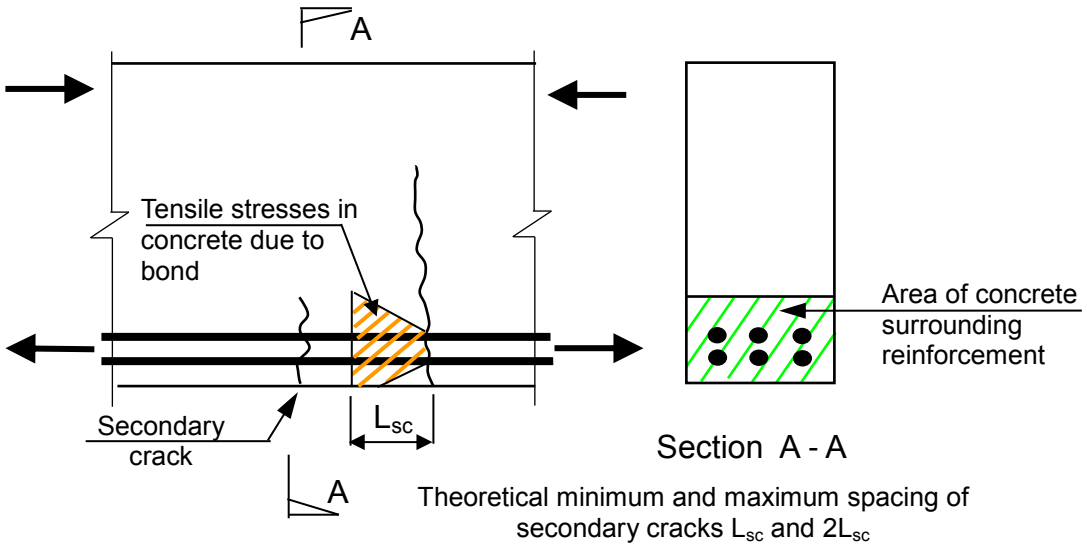


Figure 23: Spacing of secondary cracks in a beam

Secondary crack formation is a function of the quantity and disposition of the reinforcement in the concrete and its bond characteristics. Bond stresses near a crack transfer tension from the reinforcement to the concrete surrounding the reinforcement, see Figure 23. Different proposals have been advanced for the effective area of concrete surrounding the reinforcement. For beams, the ACI code expressions are based on the concrete area that has the same centroid as the flexural tension reinforcement [ACI-224 1992], while the corresponding value in the CEB-FIP Model Code [1992] is equal to the width of the beam multiplied by two and a half times the distance from the centroid of the flexural tension reinforcement to the extreme tension fibre. These effective areas of concrete are used to predict tension stiffening and crack widths and they are not necessarily appropriate for assessing when a secondary crack will form. The maximum crack widths and critical tension stiffening occur at higher stress levels in the reinforcement than those at which secondary cracks are initiated. In beams, the average secondary crack spacing is generally of the order of 3 times the cover distance measured from the centre of the bars closest to the extreme tension fibre. However, where the stirrups are relatively closely spaced the cracks tend to form at the stirrups.

The formation of flexural cracks in a wall is similar to that in a beam. The main difference comes from axial loading and the greater spread of flexural tension reinforcement along the wall, which makes it more difficult to assess the effective concrete area surrounding the flexural tension reinforcement associated with secondary crack formation. The greater length of walls compared to the depths of most of beams can result in primary flexural cracks being spaced several metres apart. Unless secondary cracks form, yielding of the longitudinal reinforcement is confined to a short distance on each side of the primary cracks. This can result in very high strains being induced and limited ductility being sustained before the reinforcement fails. The formation of secondary cracks allows yielding to spread. However, for the secondary cracks to extend to the neutral axis, which is necessary if reinforcement in the mid regions of the wall is to yield, there must be sufficient tension transfer across the primary cracks to generate a critical level of stress in the web at the ends of the secondary cracks to cause them to extend. As there is a natural stress concentration at the head of each crack, the necessary level of tension force transfer across a primary crack is less than that corresponding to the tensile stress in the concrete required to initiate secondary cracks.

Added complications come from the high variability of tensile strength of concrete, the effect of the speed of loading on tensile strength and pre-existing tensile stresses in the concrete due to restraint against shrinkage, creep and thermal movements.

Research is required on cracking and ductility of lightly reinforced walls to allow realistic design criteria to be developed to ensure the required level of ductility can be reliably obtained.

There is some doubt on the effective stiffness of structural walls that should be used in structural analysis. A number of structural walls in Christchurch were observed to sustain very limited cracking in the earthquakes. This implies that their stiffness would not have been reduced by flexural cracking to the extent recommended by the current Concrete Structures Standard. The Standard recommends that to allow for flexural cracking the gross section properties should be multiplied by a factor of 0.25 when the axial load ratio $\left(N/A_g\ f_c'\right)$ is zero and 0.33 when the axial load ratio is 0.1. Given the wide spacing of cracks in lightly reinforced walls, a factor closer to 1.0 may be more appropriate than the lower values currently recommended in the Standard. Where the ultimate flexural strength of a wall is less than the flexural cracking strength only one primary crack can be expected to form. In this respect the current minimum reinforcement in NZS3101: 2006, which is given as $\frac{\sqrt{f_c'}}{4f_y}$ times the concrete area, is marginal, particularly when allowance is made for the decrease in bending moment at the next potential location of a primary crack which can be several metres distant.

Research on control of cracking in walls is necessary to establish design criteria and the influence of cracking on stiffness values appropriate for seismic design.

Flexural torsional interaction

Flexural torsional interaction is not considered in the Concrete Structures Standard, [NZS3101:2006], although it is mentioned in the commentary. If torsional cracking occurs the flexural tension zone of a member beam requires both transverse and longitudinal reinforcement to resist torsion. This applies equally to beams, columns and to a series of walls forming a shear core. Ultimate flexural actions at a section, fully utilise the longitudinal flexural reinforcement in the flexural tension zone leaving no tension capacity to support

torsional actions. Consequently there is a flexural torsional interaction in which torsional capacity trends to zero as the ultimate flexural capacity is approached. Flexural torsional interaction in the shear core of the PGC building was advanced as one of the possible explanations for the collapse of this building.

The Structural Concrete Standard should be amended to include flexural torsional interaction.

COLLAPSE OF CTV BUILDING

Introduction and Structural Arrangement

The collapse of the CTV building resulted in the Department of Buildings and Housing establishing a programme to identify other buildings which may have the same critical weaknesses as those identified in the report that they commissioned [Hyland & Smith 2011]. The initial emphasis was on identifying buildings where the columns were designed as axial load members for gravity loading, but were not confined to the level now considered necessary to resist seismic actions. As a result of the peer review of the DBH report, and additional work carried out for and by the Royal Commission together with information obtained in the hearings, a wider range of defects likely to have contributed to the collapse was identified. A few of these defects are briefly described in this section.

The CTV building had six storeys with lift housing above the sixth storey. Figure 24 shows the basic structural arrangement of the elevated floors. The lateral seismic forces in the north-south direction were resisted by a group of structural walls located on the north side of the building, which is referred to as the north wall complex. Seismic actions in the east-west direction were resisted by the north wall complex and by the coupled shear walls on the south side of the building. The gravity loading system consisted of Hibond metal trays with 200mm of insitu concrete supported on beams and concrete columns. The columns were cast-in-situ up to the level of the beam soffits. The beams were made up of precast units, the majority of which were 350mm deep. This depth was increased to 500mm when insitu concrete was cast on the metal tray floor, the beams and into the beam-column joints. Initially, the precast units were simply supported in the columns. With the addition of the insitu concrete the beams were tied into the columns by bottom bars which extended from the precast units to be anchored by 90^{o} hooks into the mid-regions of the beam column joints. The top flexural reinforcement for the beams, which was located in the insitu concrete above the beam stems, passed through the internal joint zones. The gravity load system was not designed for seismic actions as its lateral stiffness was low compared to that provided by the structural walls.

CTV Columns

In the analysis of the CTV building for the Department of Building and Housing, it was found that the columns did not meet the design requirements for confinement at the time the building was designed. Time history analyses using ground motions recorded in the Christchurch CBD indicated that significant inelastic deformation would have been imposed on the columns to an extent where they could have been expected to lose some of their lateral strength. However, it is uncertain if this would have resulted in a significant loss of axial load capacity. All the columns, except those on the west side of the building, were circular with a diameter of 500mm and they were confined with a 6mm diameter spiral that had a pitch of 250mm. The axial load level on some of the internal columns approached the limit given in the then current design Standard [NZS 3101: 1982]. The columns on the west side of the building were rectangular and they were confined by 10mm stirrups at a spacing

of 250mm. In the DBH report, the vertical excitation of the ground was considered as a possibly contributing factor to the failure of the columns.

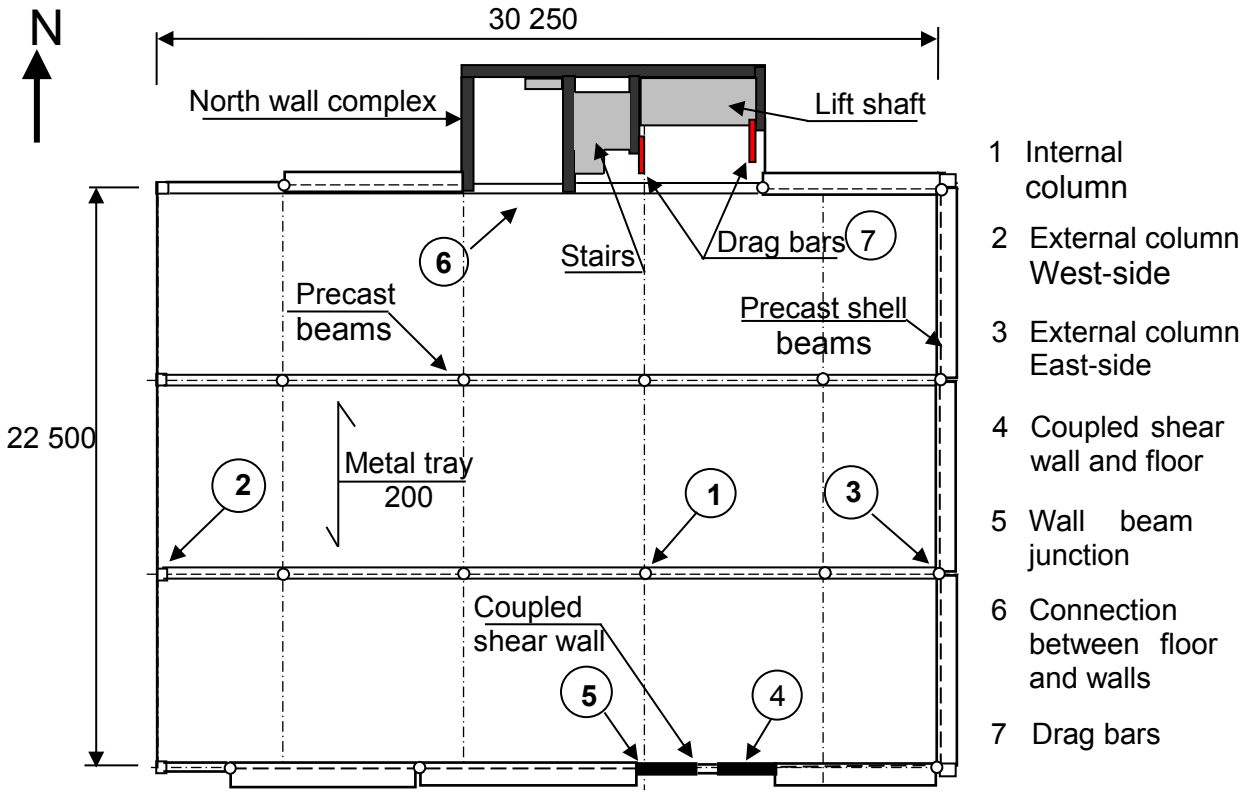
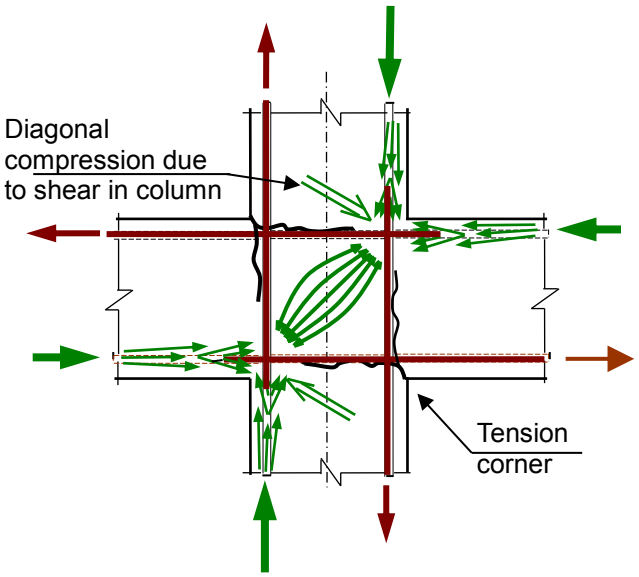


Figure 24: Plan of elevated floor in CTV Building

Beam column joints

The columns were designed purely as props for gravity loading and no consideration was given in the design for bending moment that could have been induced in the columns due to either gravity loading or seismic actions. As the columns were detailed as continuous with the joint zones and the beams, significant moments would have been induced in the gravity load structure under seismic conditions. Unfortunately, this action was not considered in the design.

The mode of action of a lightly confined reinforced beam-column joint zone with a conventional reinforcement arrangement is illustrated in Figure 25. Flexural tension forces in the beam bars are anchored on the far side of the joint zone from where the bars enter the beam-column joint. Flexural cracking, which develops along the beam bars due to bending moments in the columns, reduces the bond stress that can be sustained in the tension corners of the joint. The anchorage of these bars can extend into the beam on the far side of the joint zone, as shown in the figure. This involves some slippage of the reinforcing bars through the joint. Where this occurs, the anchored tension forces combined with the flexural compression forces sustain a diagonal compression force across the joint zone. The slippage of the beam bars reduces the flexural strength of the beams by a small proportion and this leads to any inelastic deformation that subsequently develops being pinched into the joint zone. With this form of detailing, the joint zones can sustain a few cycles of inelastic deformation before axial load carrying capacity is significantly reduced.



Structural actions in conventional beam-column joint with little or no shear reinforcement

Figure 25: Conventional Internal beam-column joint

An examination of the structural drawings shows that there were significant weaknesses in all the beam-column joints in the structure. A major source of weakness was associated with the anchorage of the bottom reinforcement in the precast beams. This reinforcement was extended into the mid-region of the beam-column joints and anchored by 90^{o} hooks, as shown in Figures 26, 27 and 28. This detail did not provide a continuous load path for the flexural tension reinforcement through the joint zones. The lack of load path is illustrated in the figures for three of the different forms of beam-column joint that were used in the building.

In the beam-column joints located in the central regions of the building flexural tension forces in the bottom beam bars, due to seismic actions, would have been partially suppressed by gravity load bending moments in the beams. However, this suppression was insufficient to eliminate tension in the bottom beam bars for design level seismic actions. As shown Figure 26, any significant tension in the bottom beam bars induces tension in the concrete in the mid-region of the beam-column joint. The shear force in the columns is small and it acts in the compression zone of the column at the beam column joint. For the internal columns, which are illustrated in Figure 26, the lap length of the hooked bottom bars would be ineffective due to its short length. The situation was not helped by hooked tension reinforcement from the beam on one side of the joint zone lapping beam bars from the other side of the joint zone, which were subjected to compression. This left an inclined component of the diagonal compression force in the joint zone and a component of the axial force in the column to balance the tension force in the beam bars. Given the small angle between these two compression forces, it is evident that this is not a mechanism that can resist the flexural tension force in the bottom bars. The only viable load path for the tension force in the bottom beam bars is through the tension in the concrete in the mid-region of the joint zone. Once this concrete cracks there is little to stop the tension bars being pulled out of the joint zone, which would destroy the axial load capacity of the column.

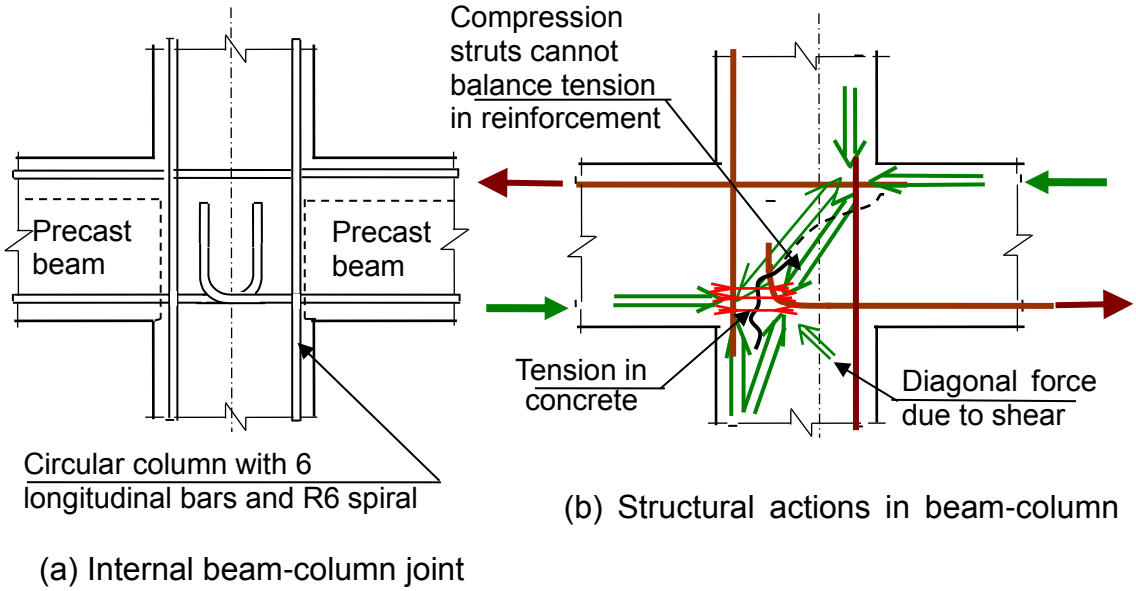


Figure 26: Internal Beam-column joint (see 1 in Figure 24)

Figure 27 shows the beam-column joint zone detail found in the columns on the west side of the building. The anchorage of both the top and bottom beam bars is by 90° hooks located in the mid-region of the beam-column joint zones. With this arrangement, the only viable load path for bending moment transfer from the beams to the columns depends on the tensile strength of the concrete. Once this concrete cracks, the axial load capacity is reduced. For these joint zones, gravity loading on the beams cannot suppress the tension force in the concrete as both the top and bottom beam bars are anchored in the mid-region of the joint zone. Critical cracking of these joint zones could be expected to develop in the early stages of strong ground motion. As cracking is confined to one side of the joint zone, collapse might not occur immediately as axial load in these columns was low and it could possibly be sustained by the remaining half of the joint zone.

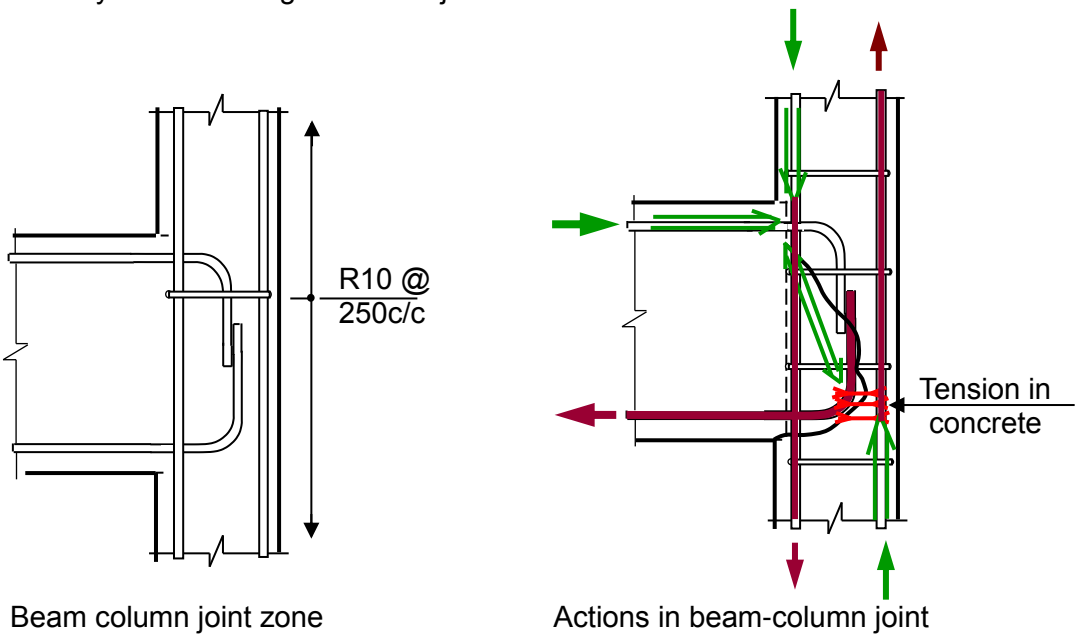


Figure 27: Beam column joint zone in the West wall of building (see 2 on Figure 23)

Figure 28 shows the reinforcement arrangement in the beam-column joints on the west side of the building. In this case there was no over-lap of the bottom beam reinforcement in the joint zones. The beams on the west side of the buildings were shell beams with the bottom beam bars embedded in the shells. The width of the shell beams was of just under a metre and once it was in place it was filled with 450 deep insitu concrete. The only dead loading on the beams was due to their self weight and hence, unlike the internal beams, gravity loading would not have had any appreciable effect in suppressing flexural tension forces due to seismic actions in the bottom beam bars. Positive bending moments associated with seismic actions, if sufficiently large to overcome the flexural strength of the unreinforced concrete in the beams in the joint zones, would initiate cracks that would severely reduce the axial load capacity of the columns. It would probably have taken a reversal of the seismic actions to cause collapse.

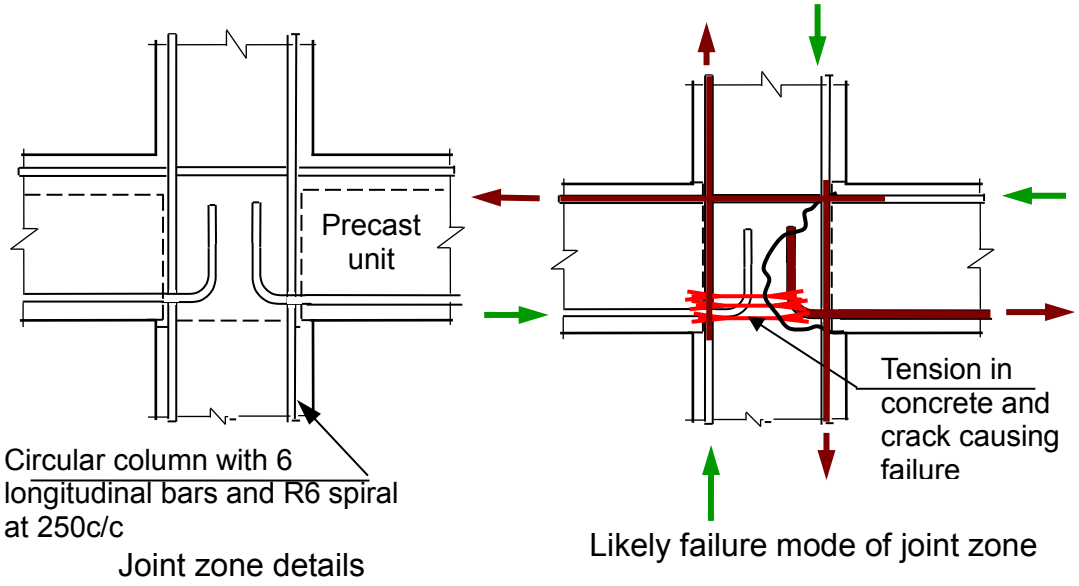


Figure 28: Beam column joints West wall of building (see 3 on Figure 23)

Coupled shear wall (see points 4 and 5 on Figure 24)

The coupled shear wall located on the south side of the building collapsed in the earthquake. Photographs and descriptions of the wall [Hyland 2011] indicate only limited cracking occurred in the coupling beams. Analysis of this wall indicates that extensive yielding in the coupling beams would not have occurred, partly because the coupling beams were too strong as detailed, and partly due to the increased strength associated with interaction of the floor slab with the coupling beam, see Figure 21 and associated text.

It was apparent from the site investigation that many of the precast beams cast into the coupled walls failed at their junction to the walls. Separation of the beams from the walls may have occurred during the collapse. However, a study indicates that the reinforcement joining the beams to the walls was not extended sufficiently to ensure the structural actions in the wall were adequately anchored into the wall. Details are given in the Royal Commission report Volume 6.

Junction of floors to north wall complex (see point 6 and 7 on Figure 24)

The seismic forces in the north-south direction were resisted by the structural walls on the north side of the building. After the construction of the building had been completed it was discovered that, due to openings in the floors for the lift shaft and stair well, there was

inadequate strength in the connection between the floors and the north wall complex for seismic actions in the north-south direction. To rectify this deficiency drag bars, which consisted of steel angles that were bolted to the floors and walls, were fitted between each of the top three elevated floors and the walls on each side of the lift shaft, see Figures 24 and 29. With the addition of these drag bars, the connection strength specified in the then current design Standard (NZS4203: 1984) was met. However, the connections were brittle and they failed either in the September or February earthquakes.

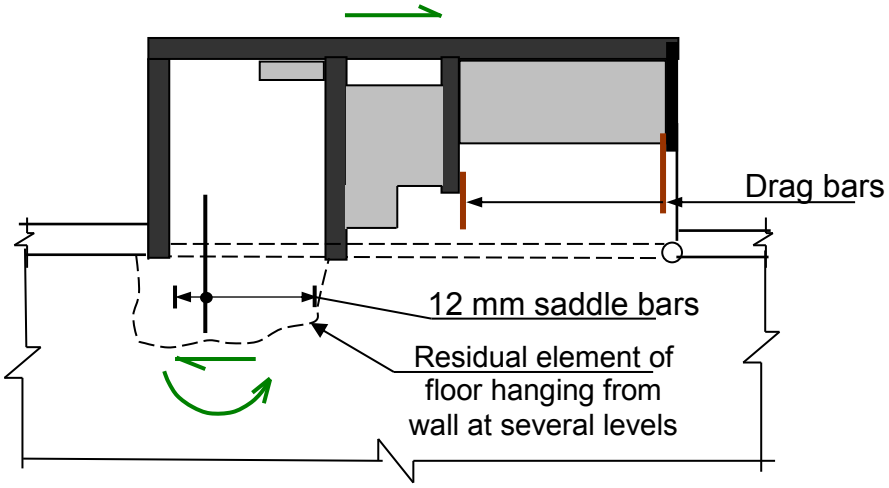


Figure 29: Bending moment and shear in floors due to lateral force resistance by north wall

There were no design calculations for the connection forces between floors and the north wall complex for seismic actions in the east-west direction. For these actions lateral resistance by the north wall complex induces both in-plane shear and flexural actions into the floors. The holes in the floors for the lift shaft and stairs left only the floor in the left hand bay of the north wall complex, see Figure 29, to transfer the east-west inertial forces from each floor to the north wall.

The strength of the connections between the floors and the north wall complex for the east-west seismic forces was less than the design actions derived from the then current design Standard [NZS4203: 1984], and the strength that was provided largely relied on mesh reinforcement which meant it did not have appreciable ductility. Photographs [Hyland 2011] taken of the building following the collapse show that the floors tore away from the walls in a manner that was consistent with a failure associated with in-plane flexure and shear in the floors. If the tear in these floors was initiated during the earthquake, it would have reduced the lateral stiffness of the structure and increased the lateral displacements acting on the columns. This is a possible cause of the collapse. Alternatively, the tear in the floors may have been caused when the vertical load carrying capacity of the columns was exceeded and the floors dropped away from the north wall complex. The time history analyses carried out by Compusoft [2012] indicated lateral forces between the floors and the north wall were considerably greater than the design actions specified in the Loading Standard [NZS4203; 1984] which was current at the time the design was made.

The design actions for connection forces between the floors and lateral force resisting elements in NZS4203:1984 did not meet the requirements of capacity design. However, more worrying is the lack of any guidance in our current Earthquake Action Standard [NZS1170.5: 2004], for design forces between floors and lateral force resisting elements. This is a problem that is likely to concern us either after the next major earthquake in an urban centre, or in coming years while we struggle to check out and upgrade our existing buildings.

Research to establish rational design criteria for connection forces between floors and lateral force resisting elements is urgently required.

SEISMIC ASSESSMENT OF BUILDINGS

A key step in the seismic assessment of a building is the examination of the structural details. This involves establishing the load path through elements such as beam-column joints, the connections between lateral force resisting elements and floors, beams and walls, walls and foundations etc. The previous section illustrates the process for a number of structural elements. In addition to assessing the strength of the details, it is important to identify where inelastic deformation may occur in a major earthquake, and the extent of this inelastic deformation that can safely develop in the critical zones. This process identifies the critical structural weaknesses. It is only after this step has been carried out that detailed structural analysis should be undertaken.

The seismic assessments of the PGC and the CTV buildings included inelastic time history analyses. However, detailed examinations of many of the individual components in these buildings were not described in the reports and a number of critical structural weaknesses were not identified. While inelastic time history analyses are considered the most advanced method of analysis, this approach will not necessarily identify critical structural weaknesses unless the parameters representing the critical structural elements have been identified and appropriately set prior to running the analyses.

DISCUSSION AND CONCLUSIONS

1. The background of a few of the approximately 70 changes to design and/or assessment of buildings, made by the Royal Commission for the Canterbury earthquakes, are described in the paper.

2. The earthquakes in the Canterbury earthquake sequence had very high ground accelerations, with short durations and long return periods. Due to the infrequency of these events and the limited area that sustained significant damage, these earthquakes are unlikely to be typical of seismic events in other towns and cities in New Zealand.

3. The spectral values for the September and February earthquakes are very much higher than the corresponding design values for periods in excess of 2.5 seconds. Research is required to identify the reasons for these high values. If appropriate design spectra values should be revised for Canterbury.

4. The vertical ground motion design spectra need to be revised to recognize the very different frequency content of horizontal and vertical ground motions.

5. Research is required to identify when it is important to allow for vertical ground motion in the design or assessment of existing structures.

6. The proposed approach of allowing for duration of strong shaking on seismic damage by the use of magnitude weighting should be assessed. Research should be undertaken to find magnitude weighting factors that are appropriate to the typical range of modern structures.

7. The current approach for the analysis for seismic actions using the Equivalent Static, Response Spectrum, Elastic Time History and Displacement Based Design methods is based on the fundamental assumption that structures have equal stiffness and strength

in the forward and backward directions of displacement. Where this is not the case and inelastic deformation is induced in an earthquake, ratcheting occurs in one direction. This results in higher drifts than those predicted in the analysis. Research is required to identify how allowance can be made for ratcheting. Designers need to be aware of this problem so that any tendency for this action can be minimised and allowance made for any adverse effects.

8. High bond stresses between reinforcement and concrete can cause premature spalling of concrete. To avoid potentially brittle failures where the clear spacing between bars is small, the portion of transverse or longitudinal reinforcement lapped in cover concrete at any section should be limited.

9. Much of our current design approach for structural elements has been developed from structural tests of individual elements together with associated analytical work. Relatively little consideration has been given to the inter-action between different structural elements. This aspect needs to be researched so that practical design methods can be developed to over-come adverse effects. Typical examples of interactions between different structural elements include the elongation of beams on floor slabs, the increase in strength of coupled shear walls due to restraint from floor slabs, increased axial forces in walls due to restrain against elongation provided by other structural elements and the potential change in performance of structural walls due to floors providing horizontal restraint to diagonal cracking.

10. In the design of a building, it is important to ensure there is a valid load path for inertial forces from the point where they are applied to the foundations. This includes identifying the load path in connections between structural elements, such as beam-column joints and the connections between floors to walls or other lateral force resisting elements. The load path must satisfy the requirements of both equilibrium and compatibility.

11. Research is required on structural walls to ensure that yielding of longitudinal reinforcement can spread over a sufficient length of wall to provide the required level of ductility.

12. Designers need to ensure that the level of cracking assumed in structural elements, for the purposes determining their effective stiffness levels in the structural analysis, are consistent with the results of the structural actions found in the analysis.

13. In assessing the potential seismic performance of a building, the structural components making up the building should be examined to establish their strengths and displacement capacities ***before*** detailed structural analysis is undertaken.

14. The current Earthquake Actions Standard does not have guidance on determining design forces between floors and lateral force resisting elements. Research is urgently required to establish appropriate design actions.

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