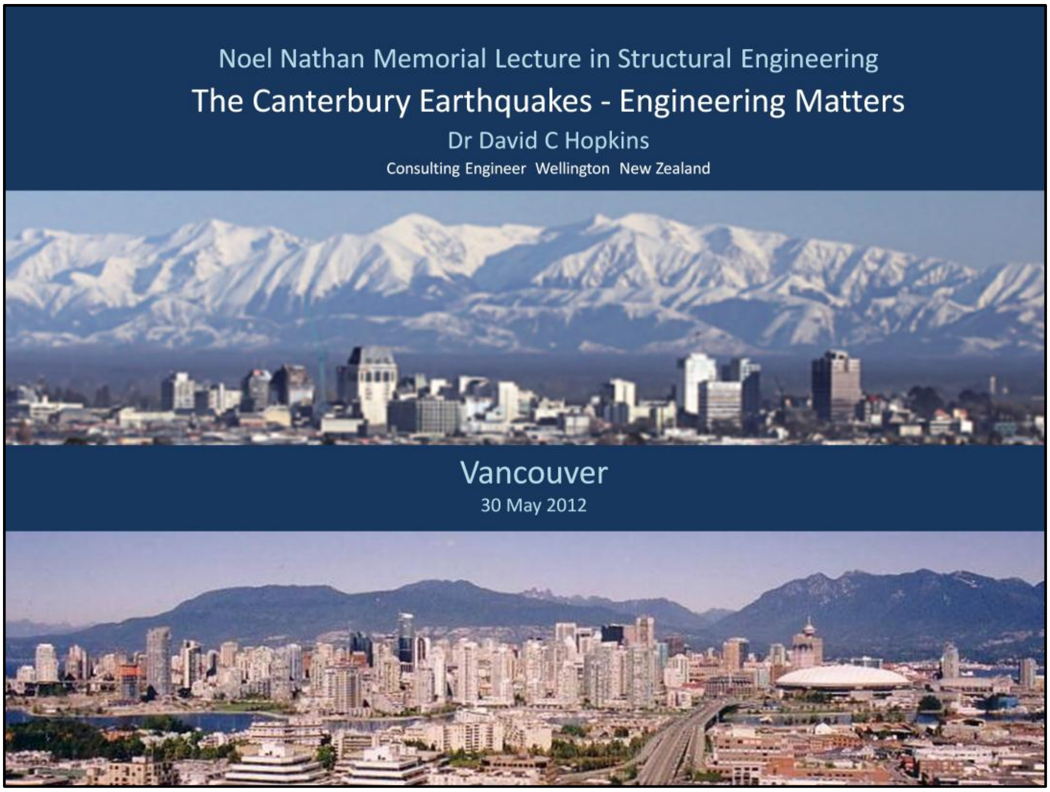


Title slide

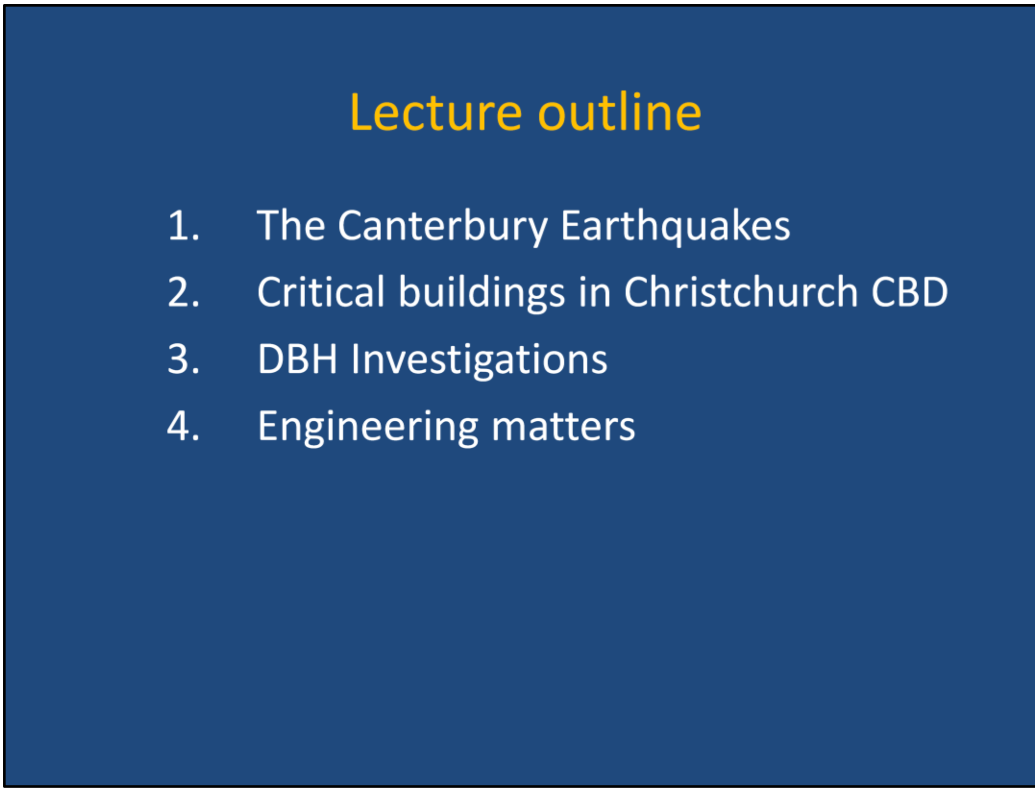
It is a great pleasure and privilege to present the 2012 Noel Nathan Lecture in Structural Engineering.

I know that many of you are well informed on the Canterbury earthquakes through Ken Elwood's presentations.

You may not know that Ken provided invaluable assistance in the immediate aftermath of the 22 February earthquake. He showed a consistent ability to identify key issues and to quietly indicate that they should be considered.

His knowledge, insights and support at that time were much appreciated and I take this opportunity to thank him again. It makes it a special privilege to share some of my experiences with you in this Lecture.

Noel Nathan was a highly respected member of the faculty at UBC, and I note particularly his emphasis on the importance of fundamentals of structural engineering. The reasons for noting this particular emphasis will become evident towards the end of my presentation.



Lecture outline

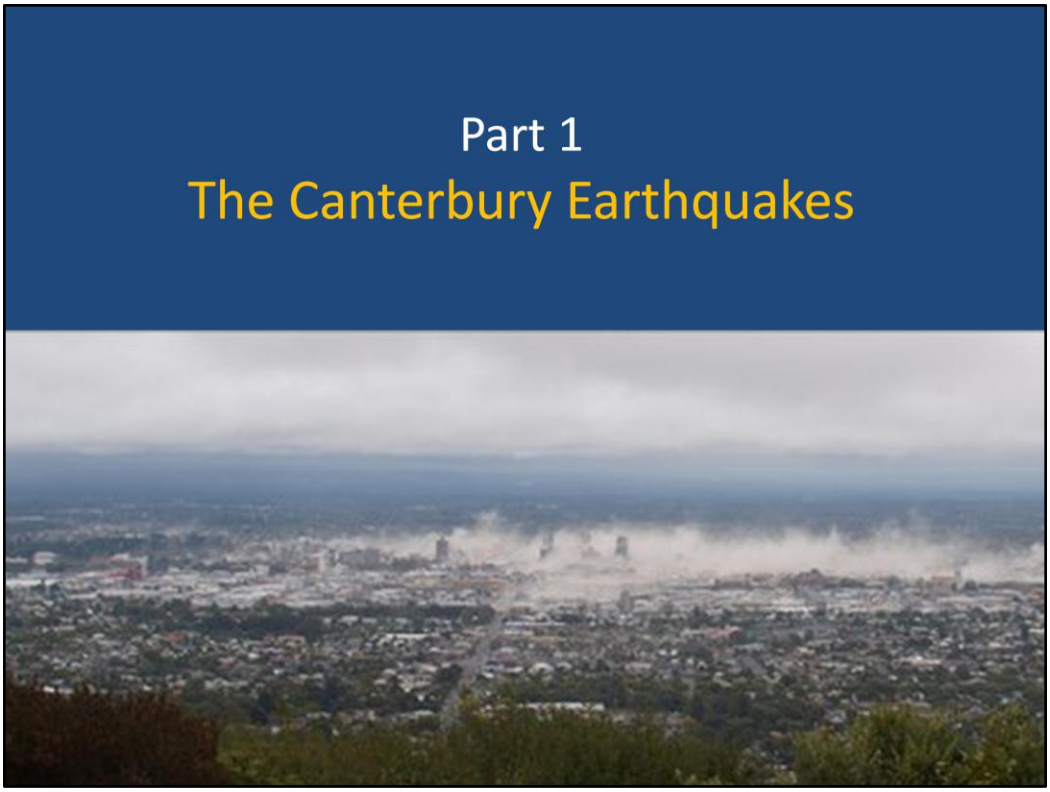
To provide some context, I will make some brief comments about the Canterbury earthquakes and their impact.

Then I will describe some of my experiences in the immediate aftermath of the 22 February earthquake - helping to assess and stabilise some of the tallest buildings.

After six weeks in this critical buildings role, I was asked by the Department of Building and Housing to manage investigations into the collapse of four buildings in the 22 February Earthquake. I will outline that work, with some detail about the collapse of the CTV Building in which 115 people died.

I will then share some personal observations relating to structural engineering that result from the critical buildings assessment and the DBH investigation.

I will conclude with some remarks that are at the heart of the title of this Lecture – to explain what I mean by "engineering matters".

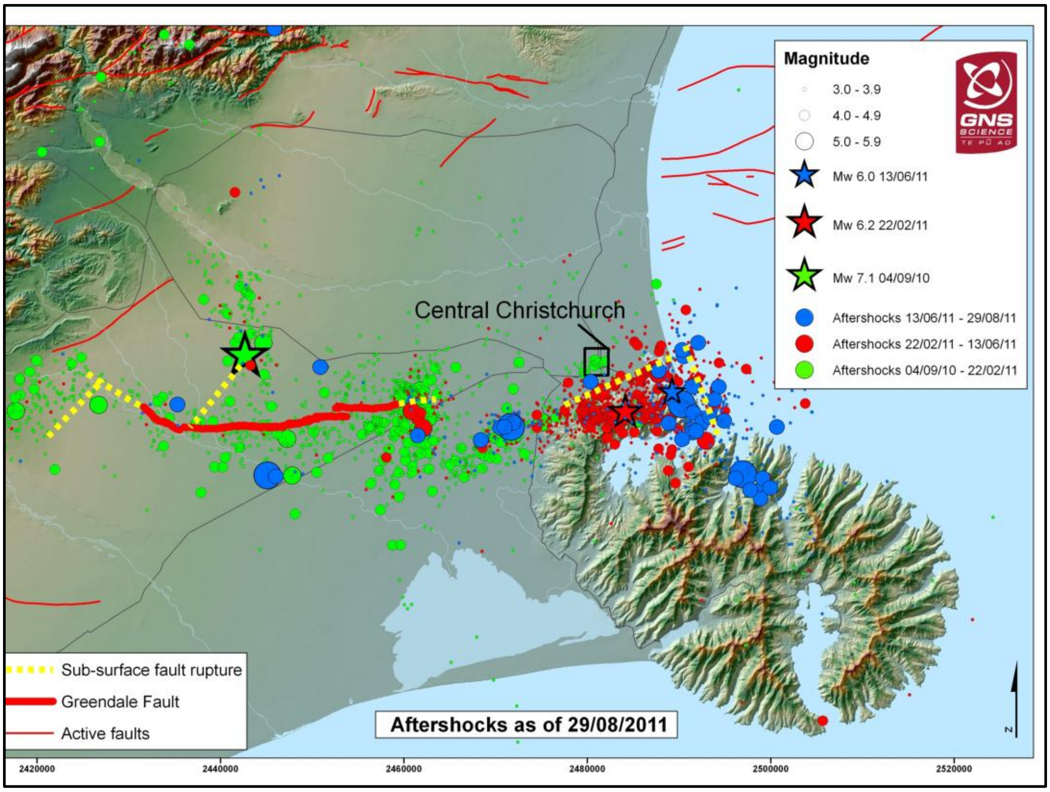


Part 1 The Canterbury Earthquakes

The earthquake of 22 February 2011 has devastated the CBD of Christchurch.

Even 15 months on we are still learning the true extent.

But this dust cloud, pictured soon after the main shock, was a signal that damage downtown was extensive.



Canterbury Map – Earthquakes and Faults

This map shows the Canterbury Plains and the volcanic origin of Banks Peninsula.

The M 7.1 earthquake of 4 September 2010 was caused by movement of the Greendale Fault near Darfield – the green star.

The M6.2 earthquake of 22 February 2011 is classified as an aftershock. It was 10km deep and 10km distant from the CBD.

In spite of its smaller size, the 22 February event did much more damage in the CBD than the 4 September event.

Summary of effects

- 4 Sep 10; 26 Sep 10; 22 Feb 11; 13 Jun 11
- Short duration events
- Thousands of aftershocks
- Extensive liquefaction damage
  - Residential
  - Underground services
- Severe shaking damage in CBD (22 Feb)
- Rock falls and landslides
- NZ$30 billion losses and rising. (> 15% GDP)

Summary of effects

The main shock on 4 September 2010 and the several thousand aftershocks since have made it particularly difficult for the citizens of Christchurch.

Even last week there was a M5.2 aftershock 10 km from the CBD.

The extensive liquefaction, particularly in the 4 September and 22 February events has taken a heavy toll on dwellings over a wide area.

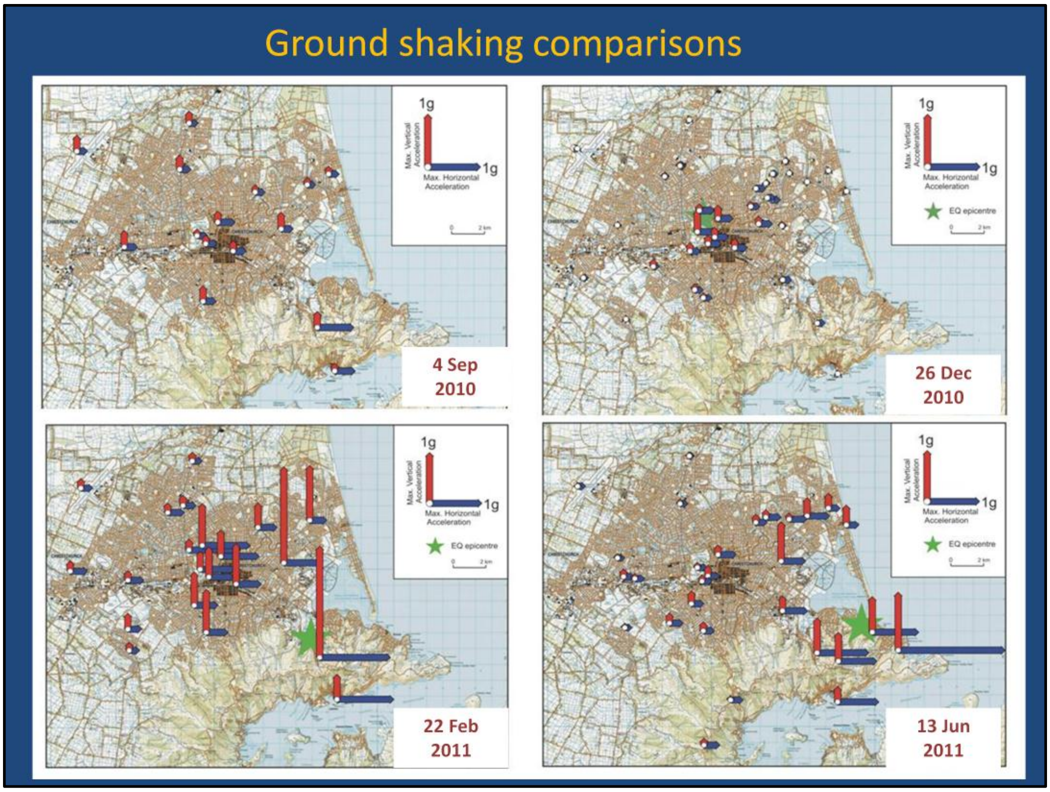
Considerable geotechnical work has been required to determine the viability of site remediation and rebuilding on the same sites. Many people have been forced to move to more stable land.

Damage to underground services, particularly sewers has been severe.

As if that was not enough, landslides and rock falls destroyed homes and many prime building sites. The challenge of dealing with these risks is ongoing.

To some extent the effects of the Canterbury Earthquakes have been cumulative.

Overall, the losses have been reported at NZ$30 billion. This is more than 15% of GDP for New Zealand, 70% of the Canterbury GDP and probably more than 200% of the GDP of the city – where most of the damage occurred.



Ground shaking comparisons

Here are four maps showing the peak ground accelerations for the four largest events, 4 September 2010, 26 December 2010, 22 February 2011 and 13 June 2011.

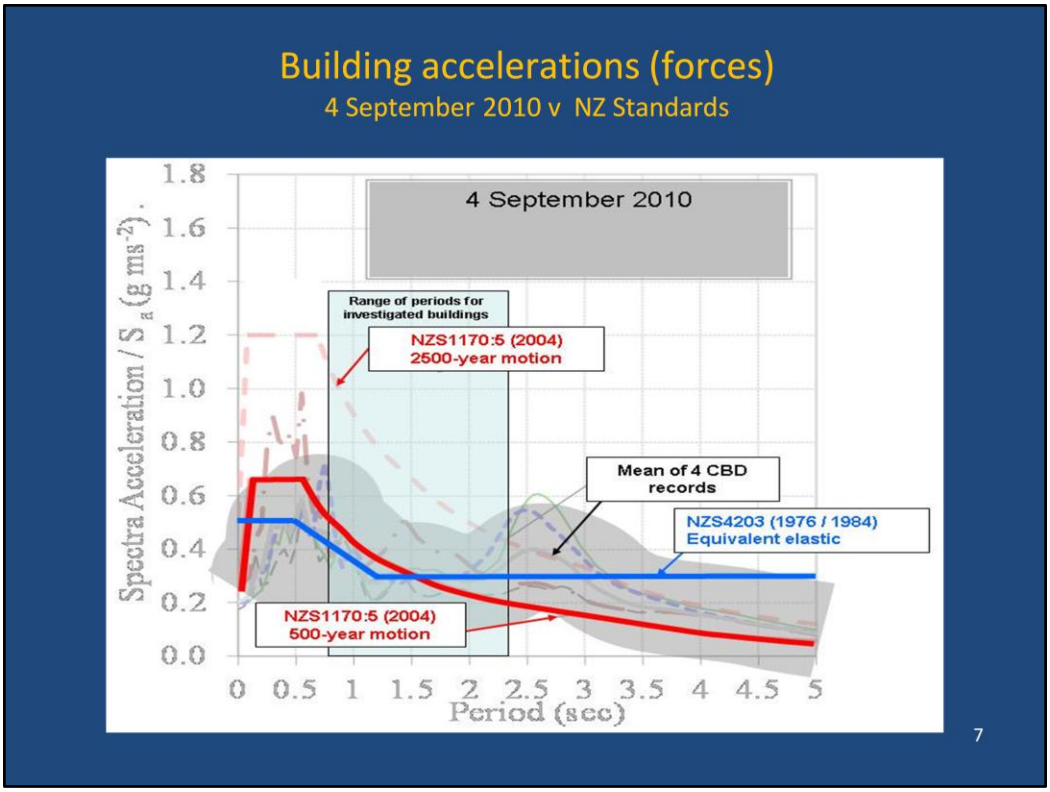
The scales are the same in each diagram. The red vertical arrows measure vertical acceleration and the blue horizontal arrows measure horizontal acceleration.

It is not so much the absolute but the comparative values that are of most interest.

It was thought that the 4 September event was somewhere near design level intensity and engineers were very pleased to see the CBD little affected – though shocked at the extent and severity of liquefaction.

The 22 February event was much more severe in the CBD and even the 13 June event registered some high values of ground acceleration.

Overall, one can sense the cumulative effect on the built environment and on the community with significant ground shaking from earthquakes spread over many months.



Building accelerations – 4 September 2010

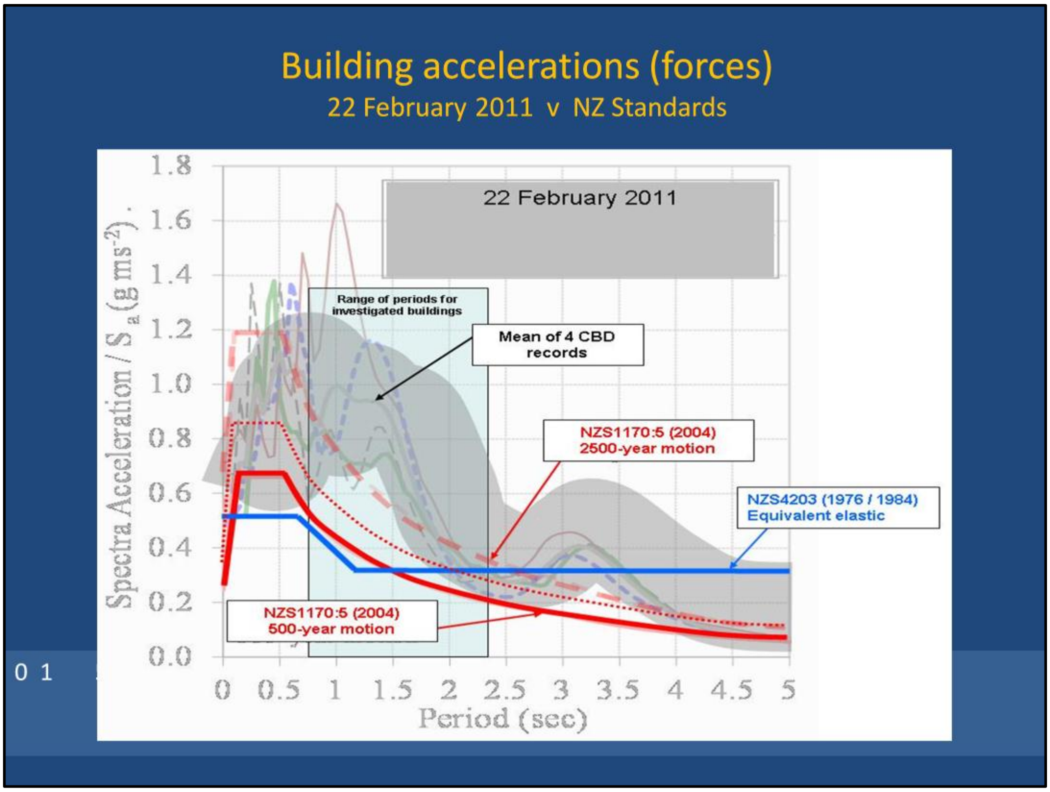
Response spectra are familiar to structural engineers. Here we see the estimated response of buildings of varying heights (periods) to various measured ground motions.

The wide grey band represents the estimated response to the 4 September event – averaged over four ground motion records. Behind the grey band are widely varying responses estimated using the individual ground motion records.

The grey band attempts to convey this variability. This is helpful when considering the wide variation in building performance that has been observed.

The red line shows the 2004 design spectrum for normal buildings, and the blue line the 1976/84 design spectrum.

Overall the 4 September responses are broadly similar to the design spectra.



Building accelerations – 22 February 2011

Similar responses for the 22 February event show a different story.

Although it may be argued that this was a short duration event, there is no doubting that the intense ground shaking placed extraordinary demands on buildings in the CBD

Part 2 – Critical buildings in the Christchurch CBD

In this part of the Lecture I will relate some of my experiences following the 22 February event.

These spectra and other summaries were not available on 22 February 2011 when we first heard the news of another aftershock affecting Christchurch city.

The first impression was that this was just another of the many aftershocks that had been felt since 4 September 2010 but this quickly turned into realisation that this had been a seriously damaging event.

David Hopkins response and tasks

- Flew from Wellington 23 Feb 8 a.m.
- Certify Art Gallery as EM Centre
- Assess buildings for safe access
- Hotel Grand Chancellor
- Critical Buildings Team
- 40+ Critical buildings
- 6 weeks

David Hopkins response and tasks

I was one of hundreds of engineers, building officials and others who travelled as soon as possible to Christchurch to help with building assessment. Thousands of buildings needed assessment right across the city.

None of those involved had the benefit of a summary of the impacts of this event.

We just knew that a special response was needed.

After checking a couple of buildings, I was asked to certify that the Art Gallery was safe to use as an Emergency Operations Centre. This challenge was made easier by the timely arrival of the designer of the building – we did the task together.

My involvement with what was to become the Critical Buildings Team began with appraisal of the Hotel Grand Chancellor.

The Team went on to appraise over 40 buildings over the next few weeks.

Christchurch City Art Gallery

Emergency Operations Centre

On the fringe of the CBD is the Christchurch City Art Gallery.

Most people were amazed that the structure and the glazing suffered little damage in any of the earthquakes.

They were even more amazed when the Art Gallery became the Emergency Operations Centre after the 22 February event.

The engineering response

Many engineers came to Christchurch immediately – an incredible response that was truly amazing to experience first hand.

The individual and collective will to co-operate and help is hard to describe. But hundreds turned up early each morning and left late at night.

Day after day engineers and building officials kept arriving to help and were drafted into teams according to their skills and sent to the field.

The 4 September event had provided good "practice" (we thought *that* was the real thing!) for the inspection of buildings, especially residential. Damage to CBD buildings in the 4 September earthquake was insignificant in comparison to that in the 22 February event.

It was clear from the start that the 22 February event was different – but the true extent of the impact has been revealed only with time.

Reporting lines, key personnel and team locations changed frequently as the demands of the situation overwhelmed the resources available, as the extent of the damage became evident, and as the most critical demands changed.

Hotel Grand Chancellor

- Grave concern of imminent total collapse
- “Moving on its foundations”.
- USAR access needed
- Need to make roads usable

Hotel Grand Chancellor

This hotel was one of the tallest buildings in Christchurch and was reported to be moving on its foundations. It had a very noticeable lean and it gave the appearance of being on the point of collapse.

The danger was preventing a full USAR search of the hotel and almost a city block of adjoining buildings. At that stage it seemed important to get the roads open.

The challenge was to find a way of stabilising the building.

HGC The Challenge 23 February 2011

Standing “downstream” of the lean was not a comfortable feeling on 23 February.

USAR Engineers, including Bruce Galloway, made initial safety evaluations but evidence available was external only.

No-one knew at that stage that one quarter of the building was cantilevered over Tattersall’s Lane (see right photo.)

The appearance of this south wall offered few clues and .....

South Wall Close-up

.....looking closer did not help much., but ....

HGC The Challenge 23 February 2011

... from the east, it was clear that there had been a substantial drop in the south-east corner.

Was this structural or foundation failure?

Or both?

How stable was the building?

Was it really moving?

Wall failure in foyer

The entry foyer offered a fundamental clue. A wall column had failed and the structure had dropped 800mm at that location.
(Until drawings were found, it was not known that this was supporting a cantilevered bay.)

But the remainder of the building held together

An initial proposal, developed by Bruce Galloway, in a hand written evaluation and proposal, was to pump concrete to surround failed wall. This would be without any formwork to avoid risk to personnel.

Surveys over the next two days showed the building was not moving, even after some strong aftershocks.

This gave enough confidence to refine Bruce's proposal to a solution involving formwork – prefabricated and bolted to existing columns etc – minimising the time of personnel in the building.

Closer inspection showed that the whole wall including the top of the foyer section would need to be stabilised

Building an understanding

Drawings were obtained.

Personnel involved in design and construction were contacted.

Meetings were held of all key players to discuss approach to stabilisation.

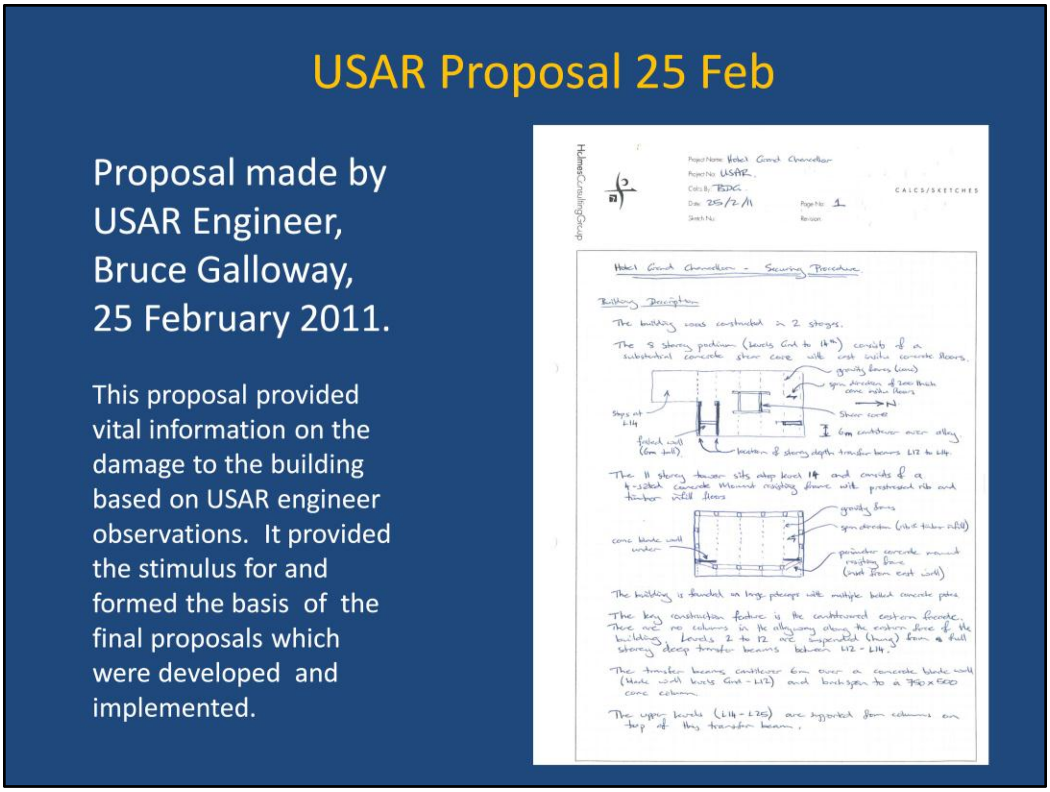
An approximate structural computer analysis was made by Alistair Cattanach overnight to explain why building was standing and assess the strain it was under.

The final meeting to design stabilising works was a wonderful example of co-operative brainstorming.

This included many engineers and the contractors who had been asked or who had volunteered to help.

In the discussions each idea was seen for its merits, and spawned better ideas from others and so on.

The aim was to stabilise as quickly and effectively as possible with minimum exposure of workers to danger.



USAR Proposal 25 February

This proposal provided vital information on the damage to the building based on USAR engineer observations.

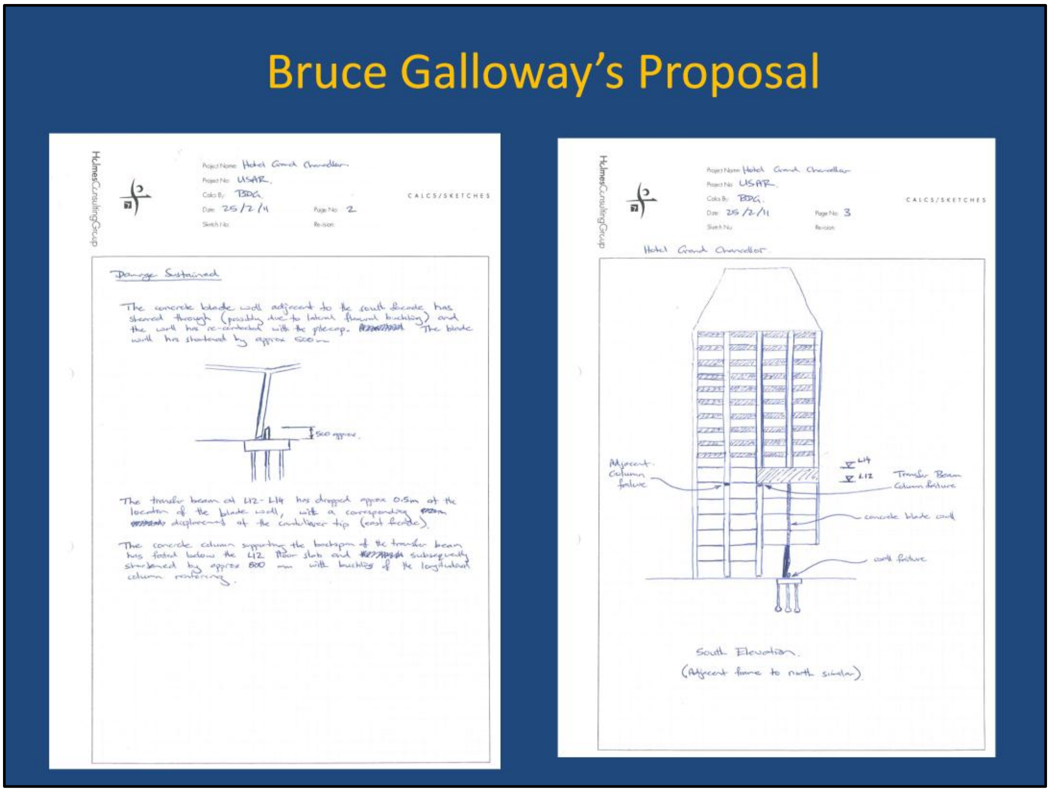
It provided the stimulus for and formed the basis of the final proposals which were developed and implemented.

The first page summarised the basic structural form of the Grand Chancellor Hotel, noting its two-stage construction and unusual cantilever feature.

Sketches and a brief specification were done by 26 February

Work commenced 28 February and the lower section of wall was stabilised by 1 March.

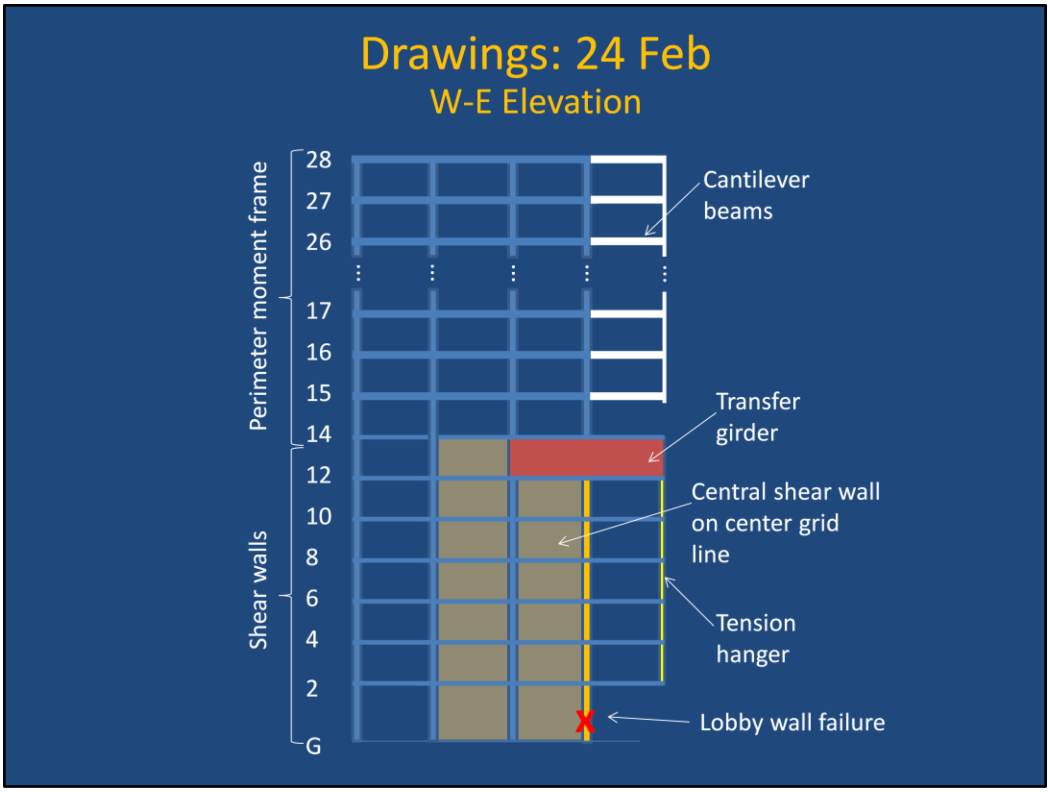
The USAR room-by -room search commenced on 1 March



Bruce Galloway's Proposal

Bruce Galloway's notes showed clearly the fundamental nature of the failure and its significance in the context of the structural form.

It seemed likely that a strong aftershock would collapse the building unless the failed wall could be strengthened or a means found to prop the cantilevers.



Drawings 24 February

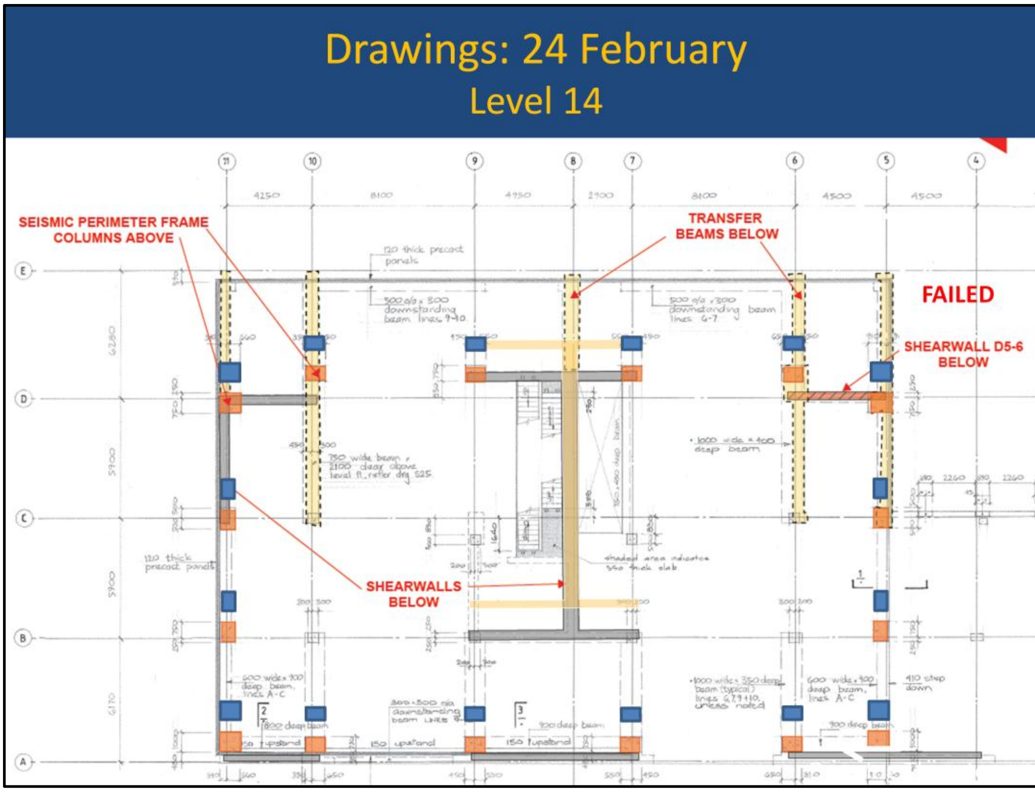
The first 12 floors were in fact half floors for car-parking. This accounted for reports of a 28-storey building when it was equivalent in height to 22 normal storeys.

The cantilever construction over Tattersall's lane is clear, but takes two main forms.

In the lower levels up to Level 14, a single storey-height cantilever supports the floors below using tension columns.

Above that level, each floor is cantilevered and no additional load is placed on the storey-height cantilever below.

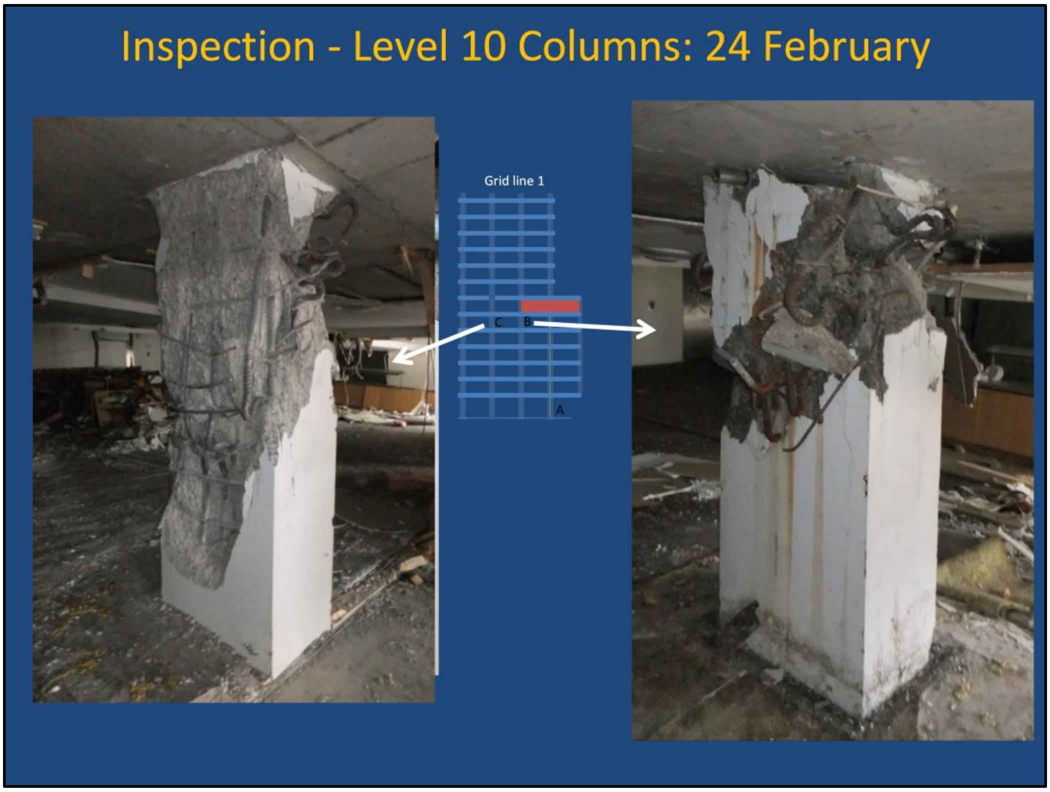
The critical position of the failed element is again apparent.



Drawings 24 February

In plan the situation looks a little less perilous. There are other storey-height cantilevers and stabilising walls.

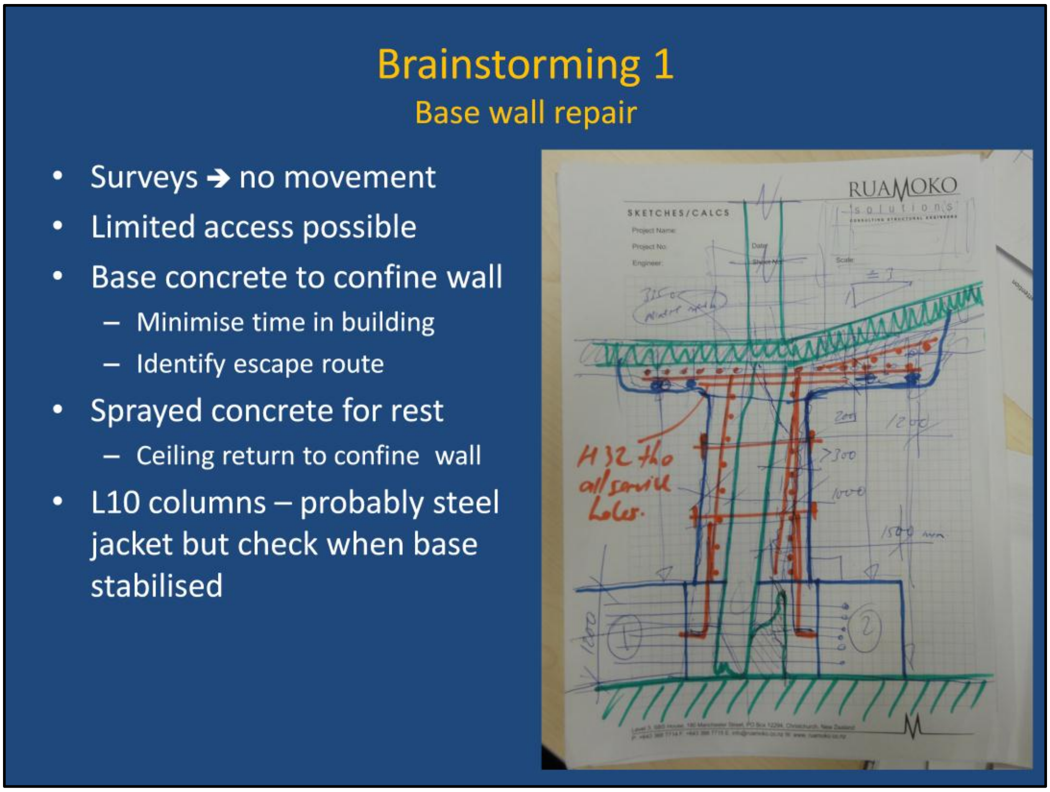
But there is still the sense that the failed wall is under high strain and any further movement / failure of it could precipitate progressive collapse.



Inspection Level 10 Columns

The 800mm drop in the shear wall in the foyer caused the storey-height cantilever to rotate and drop. Something else had to give. It was columns on Level 10.

It was as well that the failed wall was still carrying load – and stopped further collapse.



Brainstorming 1

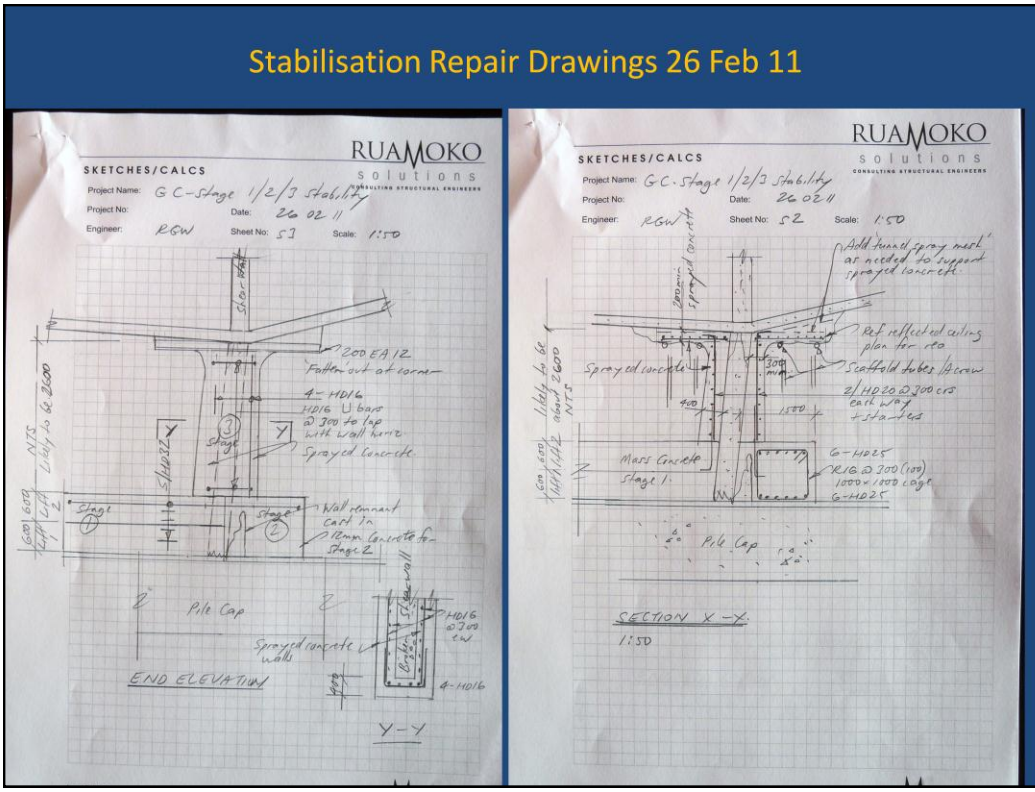
The result of the brainstorming involving engineers and contractors was this sprayed concrete solution.

Because surveys had shown no movement, even in some very strong aftershocks, there was confidence enough to allow formwork to be made offsite and brought in. Likewise reinforcement.

But the plan for the critical bottom section was still to pump concrete all around the failed length.

That would bring some relief so that the remainder could be sprayed concrete. The return at the top was to prevent bursting at a section that was showing signs of cracking.

Once this work was completed inspections of the Level 10 Columns could be more thorough and solutions devised to stabilise these.

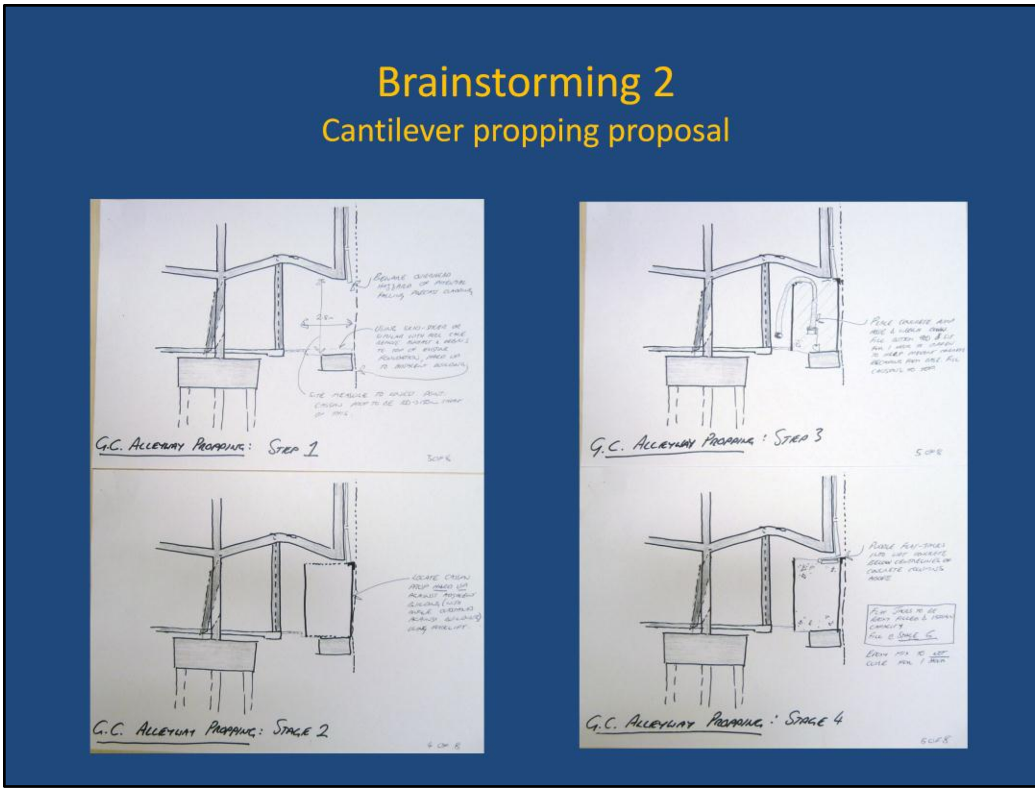


Stabilising Repair Drawings

Consulting engineer, Grant Wilkinson, produced sketches and specifications in line with the brainstorming decisions. This enabled the contractor, Lunds, (led by Andrew McGregor) to plan the construction carefully to complete in the shortest possible time and have workers exposed for the least time.

The basic plan was to:

- Install formwork and reinforcing for the first 1.2m
- Pump concrete to a depth of 1.2m to contain the failed area. (Stage 1)
- Install reinforcing both sides and on the soffit
- Gunite (spray concrete) to walls and soffit.
- Pump concrete from a supply truck at a safe distance



Brainstorming 2

Solutions were all about speed and putting a minimum people at risk.

An obvious way of stabilising the Grand Chancellor Hotel was to prop the cantilever and thus take load off (or prevent further load on) the damaged wall.

This prospect was examined in some detail and a methodology determined, complete with sketches of a step by step procedure.

Essential elements of the proposal were:

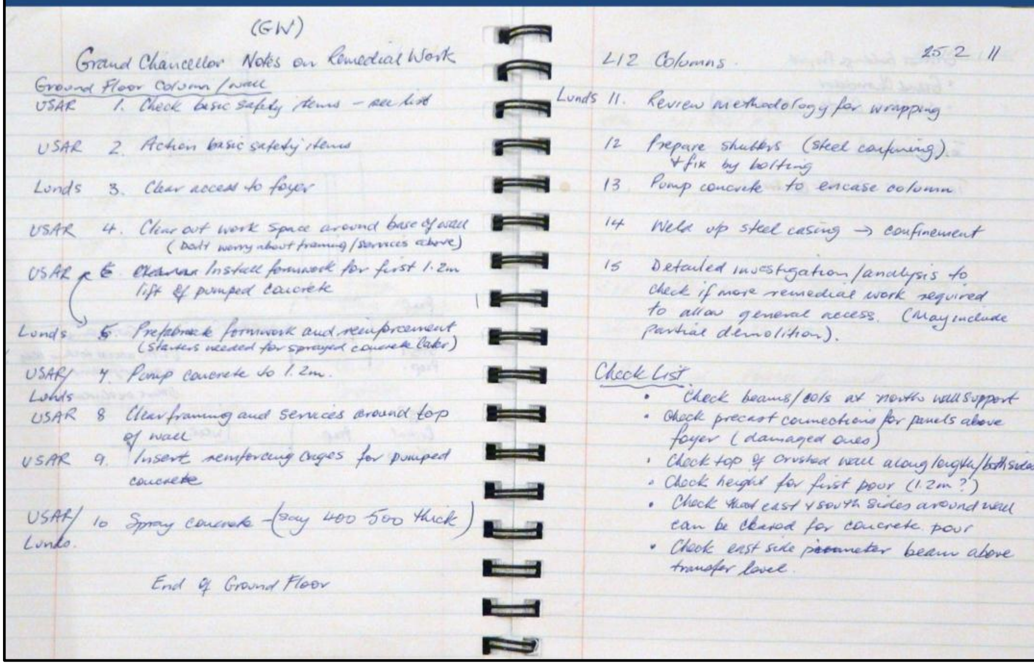
- Large diameter steel cylinders filled with concrete almost to the underside of the lowest part of the cantilever (over the lane).
- Insertion of flat jacks on top of the wet concrete
- Pressurising the flat jacks to induce load into the tension columns
- Use of the existing temporary propping piles left in place in the lane

(Piles existed at these locations which had been used during construction of the first six levels)

Inspections of the work area revealed unacceptable risks from falling of dislodged precast panels.

A decision not to proceed was made after ideas to make the area safe with steel containers proved impractical . Even placing the containers was seen as an unacceptable risk – especially in light of the wall and upper column repair.

Grand Chancellor Work Plan 25 Feb 11


(GW)

Grand Chancellor Notes on Remedial Work

Ground Floor Column / wall

USAR 1. Check basic safety items – see list

USAR 2. Action basic safety items

Lunds 3. Clear access to foyer

USAR 4. Clear out work space around base of wall (Don't worry about framing / services above)

USAR 6. ~~Clear~~ Install formwork for first 1.2m lift of pumped concrete

Lunds 5. Prefabrick formwork and reinforcement (Starters needed for sprayed concrete later)

USAR/Lunds 7. Pump concrete to 1.2m.

USAR 8 Clear framing and services around top of wall

USAR 9. Insert reinforcing cages for pumped concrete

USAR/Lunds. 10 Spray concrete – (say 400 500 thick)

End of Ground Floor

L12 Columns. 25 2 11

Lunds 11. Review methodology for wrapping

12 Prepare shutters (steel confining) + fix by bolting

13 Pump concrete to encase column

14 Weld up steel casing → confinement

15 Detailed investigation / analysis to check if more remedial work required to allow general access. (May include partial demolition).

Check List

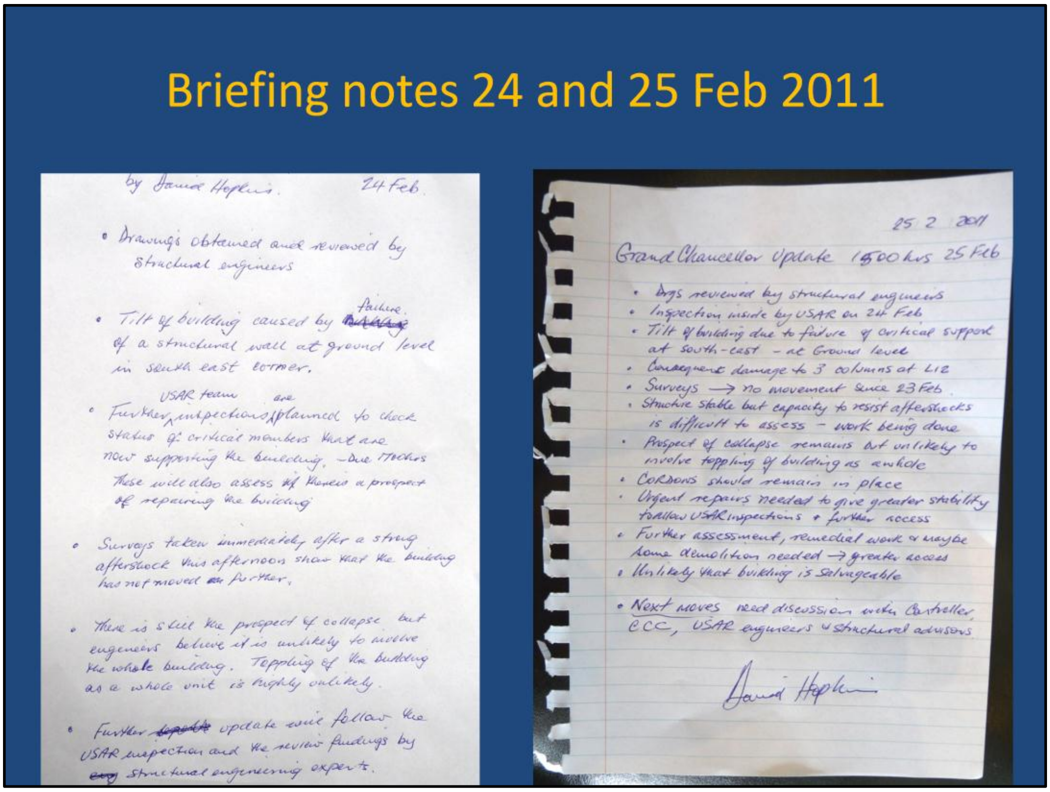
- Check beams / cols at north wall support
- Check precast connections for panels above foyer (damaged ones)
- Check top of crushed wall along length / both sides
- Check height for first pour (1.2m ?)
- Check that east + south sides around wall can be cleared for concrete pour
- Check east side ~~perimeter~~ beam above transfer level

Grand Chancellor Work Plan

This is the basic work plan written out to record and confirm decisions at the brainstorm meeting.

Note that this is Day 3 and the use of pen and paper.

The plan was further refined in later meetings and in the course of implementation.



by David Hopkins. 24 Feb

- Drawings obtained and reviewed by structural engineers
- Tilt of building caused by ~~buckling~~ failure of a structural wall at ground level in south east corner.
- Further USAR team inspections are planned to check status of critical members that are now supporting the building. – Due 1700hrs These will also assess ~~if~~ if there is a prospect of repairing the building
- Surveys taken immediately after a strong aftershock this afternoon show that the building has not moved ~~an~~ further.
- There is still the prospect of collapse but engineers believe it is unlikely to involve the whole building. Toppling of the building as a whole unit is highly unlikely.
- Further ~~update~~ update will follow the USAR inspection and the review findings by ~~eng~~ structural engineering experts.

25 2 2011

Grand Chancellor Update 1500 hrs 25 Feb

- Dwgs reviewed by structural engineers
- Inspection inside by USAR on 24 Feb
- Tilt of building due to failure of critical support at south-east – at Ground level
- Consequent damage to 3 columns at L12
- Surveys → no movement since 23 Feb
- Structure stable but capacity to resist aftershocks is difficult to assess – work being done
- Prospect of collapse remains but unlikely to involve toppling of building as a whole
- CORDONS should remain in place
- Urgent repairs needed to give greater stability to allow USAR inspections + further access
- Further assessment, remedial work & maybe some demolition needed → greater access
- Unlikely that building is salvageable
- Next moves need discussion with Controller, CCC, USAR engineers & structural advisors

David Hopkins

Briefing notes 24 and 25 February

These handwritten notes are an indication of the technology used for communication in these frenetic and trying times

Media reports persisted that the Hotel Grand Chancellor was “ moving on its foundations” days after the above advice that the surveys had established that the building was not moving, even in some big aftershocks.

These reports were most upsetting at the time. Surveys of this (and many other) buildings had been commissioned immediately and provided consistent evidence that all the major damaged buildings were not moving.

There was enough real danger without exaggerating it in the media.

Clearing the work area

USAR engineers clear the collapsed wall to make way for formwork and reinforcing.

Time inside the building was minimised but the stabilisation work could not be done without some risk to those who did the work.

Careful planning by the USAR engineers minimised the time inside and identified possible escape paths in the event of further collapse.

There was some comfort in the lack of movement in the aftershocks thus far, but there was always the possibility of a larger aftershock.

This was dangerous and nerve-wracking work.

Preparing the formwork

Formwork was prepared at a safe distance and designed in such a way as to minimise the time needed inside the building to put it in place.

Concrete for the base was pumped to its final position from a safe distance.

Completed Ground Floor Repair

There were no reprisals or requests for re-work when it was found that the dimensions of the real thing differed from those on the drawings.

The general consensus was that this part of the structure would not fail.

The result was not pretty – but it was effective.

Tidying up was not part of the work description.

The base concrete work was considered enough (together with survey results confirming no movement) to allow USAR operations to continue.

The sprayed concrete work followed.

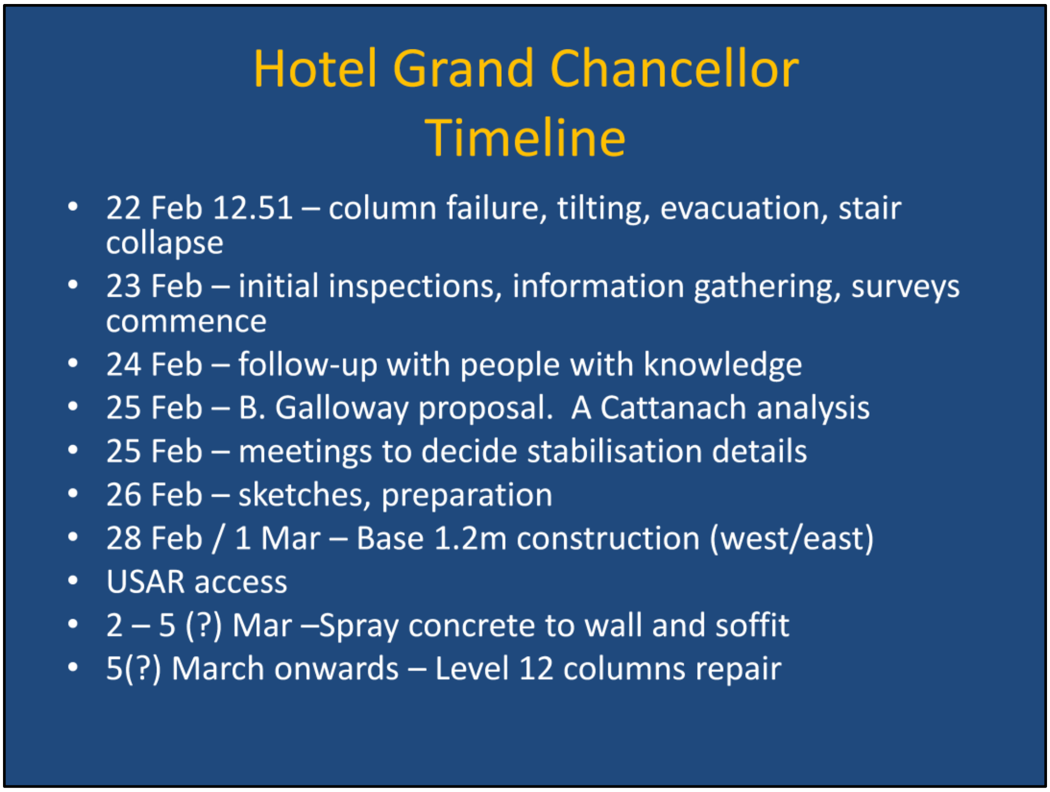
Once the wall was stabilised, the damaged columns at Level 10 were more closely examined and techniques for repair refined with this new information.

Level 10 columns were subsequently jacketed with steel cylinders filled with concrete.

Columns Level 10 and Repair

These pictures show a damaged column at Level 10 (right) and a jacketed column (left) (not the same one)

Once again, the solution was not pretty but very effective.



Hotel Grand Chancellor Timeline

This shows the timeline of key stages in the stabilisation of the Hotel Grand Chancellor

The brainstorming to understand the structure and then to devise remedial work was an amazing experience for all involved. The nearest description would be in sporting terms for athletes to all be "in the zone" simultaneously.

I took no part in the implementation, but the professionalism, skills, commitment and courage of those that did the work to stabilise the Hotel Grand Chancellor was nothing short of outstanding.

Hotel Grand Chancellor Acknowledgement

- The constructive approach of all involved was outstanding
- People contacted included:
  - the original designer in Sydney
  - the drafter who did the structural drawings
- Special thanks are due to Steve McCarthy of Christchurch City Council who liaised with and directed the Critical Buildings Group (Amongst many other tasks and responsibilities)

Key People

- Alastair Cattanach, consulting engineer, performed an overnight analysis of the damaged GCH providing valuable insights into its stability. He later prepared schemes and sketches for the proposed cantilever propping.
- Ken Elwood, professor of structural engineering at UBC, Vancouver contributed to key discussions on solutions to stabilise the GCH (and other buildings)
- Bruce Galloway was the USAR engineer who initiated the moves to stabilise the Grand Chancellor
- David Hopkins led the group responsible for the stabilisation of the Grand Chancellor and became one of the leaders of the Critical Buildings Group established by the Christchurch City Council. This group went on to evaluate over 40 buildings of 6 storeys and more.
- Dr Weng Yuen Kam, post-graduate student at the University of Canterbury, had special knowledge of Christchurch buildings and contributed with remarkable energy and skill.
- Andrew McGregor, construction contractor, advised on practical aspects in the GCH discussions and led the team at Lunds in carrying out the agreed work.
- Chris Van den Bosch, Christchurch City Council design engineer, provided engineering and logistical support to the Critical Buildings Group.
- Grant Wilkinson, consulting engineer, was a key contributor to the GCH stabilisation effort, particularly in providing specification and sketches of work to be done and liaising with the contractor on practical aspects.
- Other engineers, both local and overseas contributed.

Copthorne Hotel Durham St

This hotel had critical damage to basement columns.

The danger required major road closures and denial of access to many important buildings nearby which were not so badly damaged.

There were strong pressures to stabilise the building

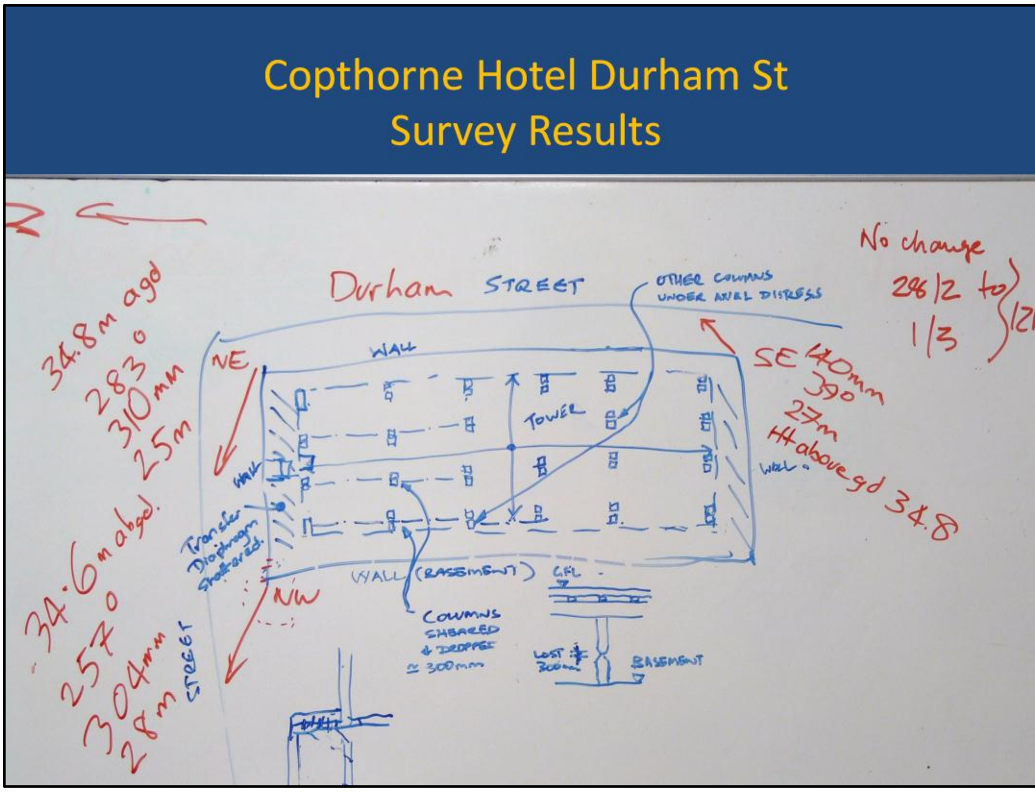
The approach to stabilisation was similar to that for the Grand Chancellor:

- Survey for movement (no movement in aftershocks)
- Obtain drawings/interview designers
- Inspect damage (if possible)
- Brainstorm solutions (in conjunction with USAR engineers and nominated contractor)
- Prepare sketches and specifications
- Obtain approval for the work to proceed.

Access to the flooded basement was very difficult and dangerous. Protective frames were installed to allow a safe place for workers in the event of collapse in an aftershock.

The main method was jacketing of columns.

This work was proceeding in parallel with other work, including the Grand Chancellor.



Copthorne Hotel Survey Results

An important part of developing the approach was this whiteboard plan.

Whiteboards are a great way of summarising information, sharing it with others and building on one another's ideas towards the best possible solution *in the time available.*

In this case the whiteboard was used to record survey results and then discuss the implications of those. This building had twisted and dropped in places and in some respects was more dangerous to remedy than the Hotel Grand Chancellor.

A feature of the Critical Buildings Group was the focused way in which information and ideas were used to drive towards practical and effective solutions.

There were time and resources to deal only with the essentials. "Nice to haves" were recognised as such.

The question was simply: "What is *needed now?"*

Park Terrace Apartments

A major shear wall failed in this block of apartments – one of five similar ones in the development

Repairs were devised that enclosed the damaged portions of shear wall with reinforced concrete either side.

The concrete was poured from above.

(This and similar blocks in this development have been demolished after further damage to them in the aftershock of 23 December 2011.)

Critical Buildings Team

Set up by: Christchurch City Council

Role: Advise on buildings 6+ storeys

Objective: Assess buildings. Recommend stabilisation / cordoning

Operation: Leaders + engineer groups

Resources: Ad hoc each day. Experience necessary. Whoever available.

Output: Status spreadsheet + recommendations

Duration: 4-6 weeks “full-on”. 3 months overall

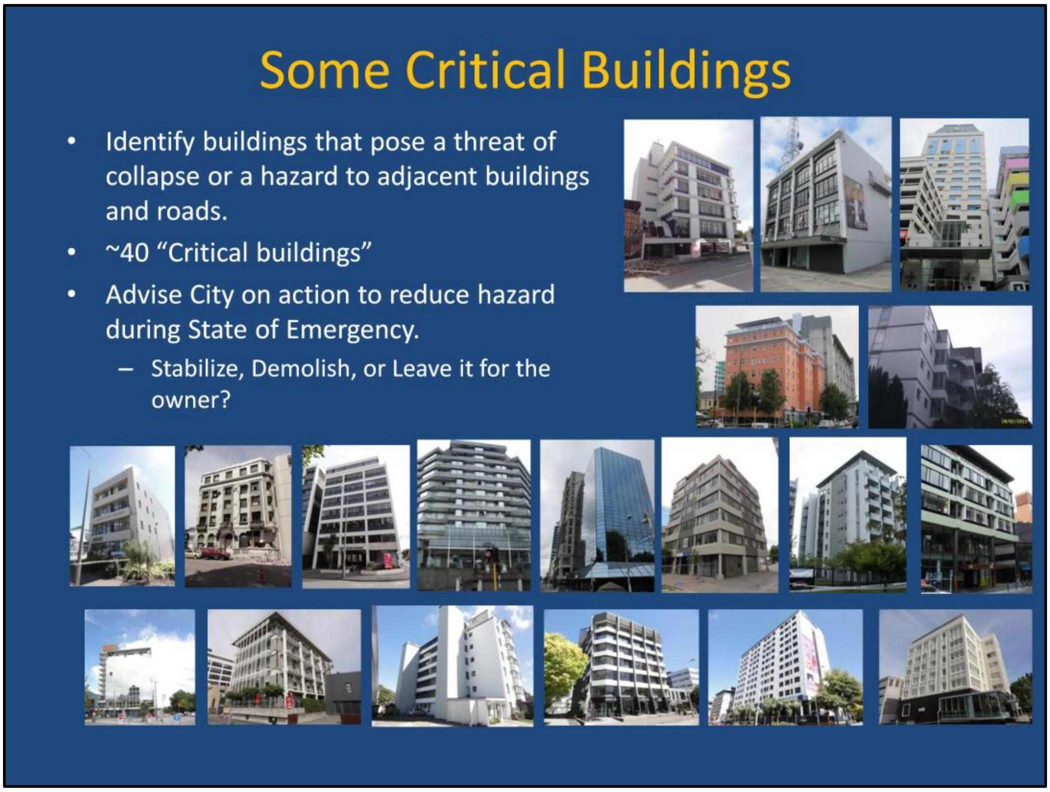
Critical Buildings Team

Steve McCarthy of Christchurch City Council liaised with and directed the Critical Buildings Group (amongst many other tasks and responsibilities)

Noel Evans, consulting engineer, shared the role of leading the Critical Buildings Group.

Mike Stannard, Chief Engineer, Department of Building and Housing, assisted with the leadership of the Critical Buildings Group

Many other engineers formed field teams to inspect buildings and bring the results back to the leadership group.



Some Critical Buildings

This gives an indication of the nature of some of the buildings that were reviewed by the Critical Buildings Team.

The list of buildings grew in waves.

The first wave was the obviously critical buildings identified in the first two or three days – as part of brief emergency inspections.

There was then a surge of buildings included as the survey teams covered larger areas and looked in more detail at key features

The third wave came when owners began to get access and engage engineers to examine the buildings more extensively. Buildings that looked to have survived well were found to have subtle but critical damage.

A visit made to the CBD on 18 May 2012 showed that the majority of the buildings we had dealt with had been or would be demolished.

Such is the extent of the devastation. But most of the buildings did their job of saving lives.

Critical Buildings in Christchurch CBD
Concluding remarks

- Unprecedented context
- Urgent need to stabilise major buildings
  - USAR access
  - Road clearance
- Immense challenge
  - Building data
  - Surveys to check movement
  - Stabilisation solutions
  - Innovative implementation
- Amazing sustained co-operative effort of all involved
- Most buildings now demolished but stabilisation was critical at the time
- Role of engineers – "under the radar"

Critical Buildings Concluding remarks

No-one would want to find themselves in the situation of the Critical Buildings Group (or any other group) in the aftermath of the 22 February Lyttelton aftershock.

We would all prefer for the earthquake of 22 February not to have happened.

We were responding to immediate demands in what was clearly a serious situation.

None of us knew the extent of the damage. This became evident as field teams reported back. On several days there was a steady stream of inspecting groups with descriptions of seriously damaged major buildings.

All those who participated felt highly privileged to be part of an extraordinarily co-operative, technically demanding and focused human endeavour in the most trying of circumstances.

It was humbling to witness and experience the commitment, motivation, skills and positive approach of all others who participated so openly and willingly.

Overall the work of the Critical Buildings Team was regarded as a very successful and important response to an unprecedented situation.

Footnote: Many of the buildings that were stabilised or identified as critical in the immediate aftermath have now been or are being demolished. The change to the Christchurch CBD skyline is brutal testimony to the scale of the impact on the city. Full recovery is decades away.

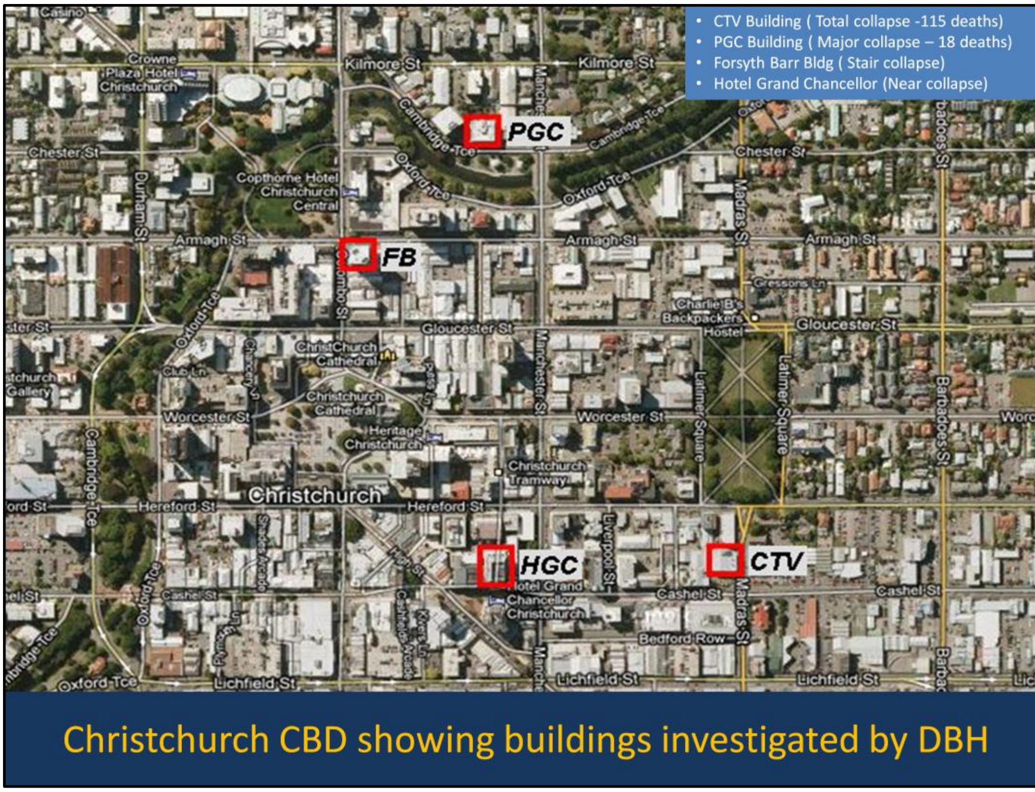
Part 3.

Department of Building and Housing Investigations

Part 3 DBH Investigations

The Department of Building and Housing is responsible for building quality in New Zealand. This includes structural safety.

The Department was asked to investigate the collapses of (or in) four buildings to determine why the collapses occurred and, most importantly, what lessons were to be learnt for setting future standards and requirements for design and construction.



Christchurch CBD showing investigated buildings

The four buildings investigated by the Department of Building and Houseing were:

- The Canterbury Television (CTV) Building which collapsed completely with the loss of 115 lives

- The PGC Building which collapsed almost completely with 18 lives lost

- The Forsyth Barr Building where 15 storeys of stairs collapsed to the bottom of the stair well

- The Grand Chancellor Hotel which almost collapsed and in which some stairs collapsed

This Google Satellite View shows their location in the Christchurch CBD.

DBH Building Collapse Investigations

- Investigating consultant for each building
- Expert Panel to review and guide
- Project Manager role on behalf of DBH
- April 2011 to February 2012
- Focus:
  - What was state of building on 22 February?
  - How did the collapse occur?
  - Why did the collapse occur?
  - What issues for design and construction standards?

DBH Building Collapse Investigations

An engineering consultant was appointed for each building.

An Expert Panel was appointed to review the investigations and to provide direction and oversight. A range of experts with different specialities included construction law, construction, seismology, architecture, structural engineering (practitioners and researchers) and local authority consenting processes

As project manager I was responsible for co-ordinating the work of the consultants and the Expert Panel. This involved considerable technical input, especially to elicit the Expert Panel views and resolve overall conclusions and recommendations.

The investigations were scheduled to be completed by 31 July 2011, but took until September for the PGC, Forsyth Barr and Hotel Grand Chancellor, and until February 2012 for the CTV Building.

These investigations have been done on behalf of the Government and are provided to the Minister in charge of Building and Construction.

They are separate from the investigations being carried out by the Canterbury Earthquakes Royal Commission (CERC) which reports to the Governor General of New Zealand.

The DBH investigation reports and much other detail from the investigations have been provided to the CERC to assist in their deliberations. CERC hearings are due to commence on 25 June 2012 and can be viewed on line at http://canterbury.royalcommission.govt.nz/

Hotel Grand Chancellor – 1988

The investigation showed that the failed wall in the foyer was under particularly heavy stress and that this may have been compounded by torsional movement and secondary effects of the storey-height cantilevers.

The strong vertical components of acceleration in this earthquake may have caused higher compression forces than would otherwise have been the case.

Improved detailing of the wall may have prevented its failure, even in the intense shaking.

The stair failure was determined to be due to the collapse of the wall.

The fact that the building did not collapse totally in spite of losing a very critical element says a lot for its overall integrity.

PGC Building-1963

PGC Building 1963

This building had a core of singly reinforced walls around the lift and stairs. This core extended from the north face of the building into the centre. It was the sole means of lateral support.

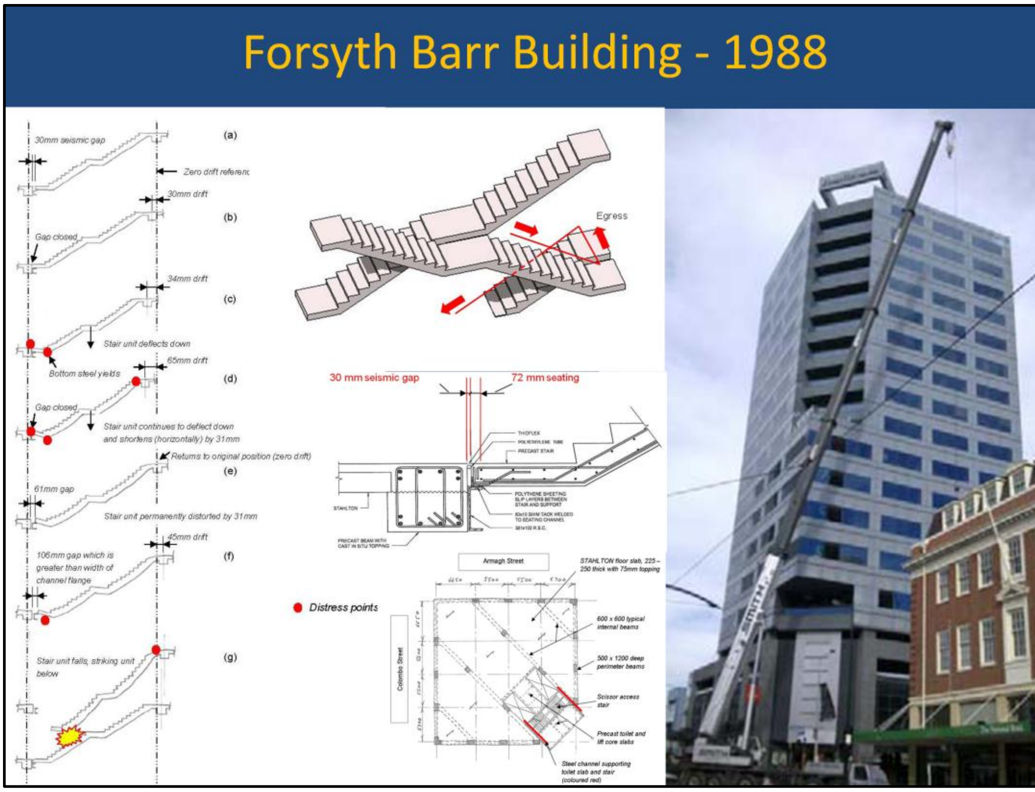
The investigation found that the intense shaking caused failure of the east core wall. Excessive displacements were then caused on the gravity columns which can be seen in the photograph.

Above the ground floor, these columns were 10” x 10” square and many had downpipes. They had no ductility. Some additional steel props had been installed but no additional lateral resistance was provided at the perimeter on the west, south and east faces.

The columns on the ground floor were much larger and set back from the perimeter. Collapse was confined to the upper floors and was limited adjacent to the core when the floor slabs remained attached.

The collapse confirmed the value of regular layout of lateral load-resisting elements, ductility and overall structural integrity.

There were opportunities in more recent years to identify the vulnerability of this building.



Forsyth Barr Building - 1988

This building survived well overall with comparatively little structural or non-structural damage.

The concern was that 15 levels of "scissor" stair collapsed all the way down to bottom of the stair well.

The location of the stairs at the rear of the building is shown as is the "scissors" configuration.

The stairs were precast concrete elements. There was a single unit spanning between floors. The lower support detail was as shown with an ability to slide horizontally. The upper end of the stair unit was tied to the floor slab.

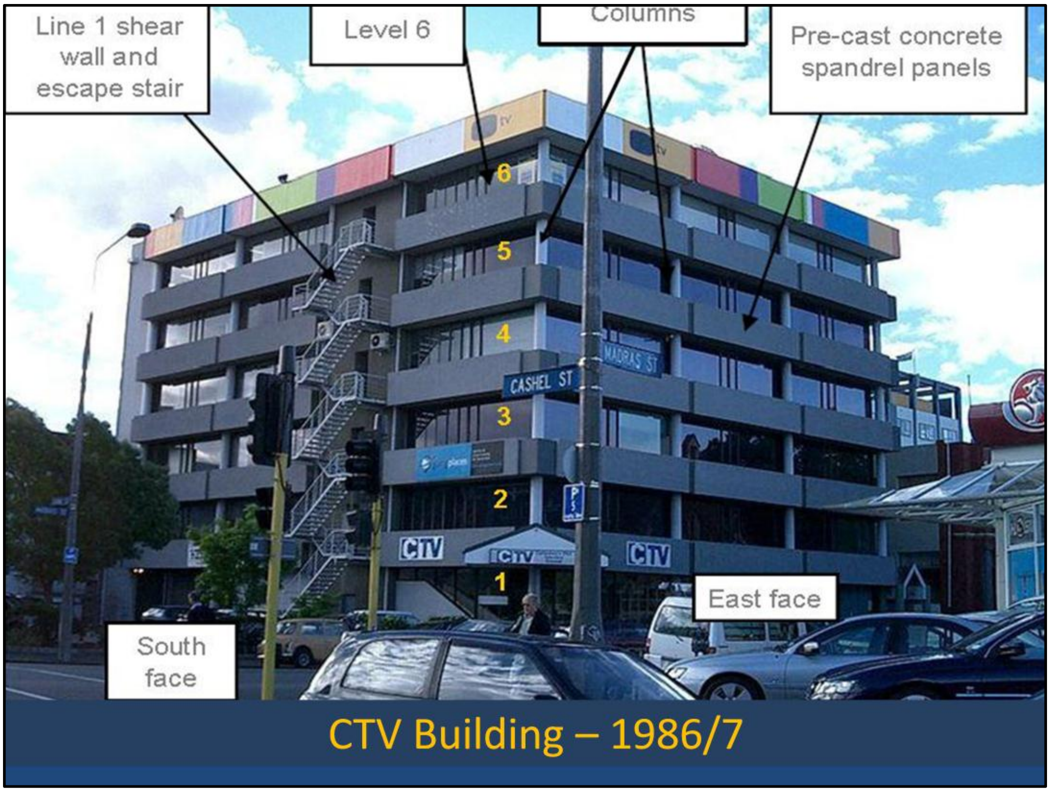
The lower support detail provides only limited ability to move horizontally. Closing movement is constrained. Provision for opening movement was limited.

The collapse is thought to have occurred when a stair unit buckled in a closing movement. This reduced the overall length of the unit. This reduced the available seating for the opening movement that followed.

The clear lesson is to be very conservative in detailing supports of this nature. The cost of providing greater seating is minimal. There are better details than this that are used by many engineers in New Zealand.

It is fortunate that the stairs collapsed during the earthquake shaking and not half a minute later when loaded with escaping occupants.

Footnote: The Department of Building and Housing has issued a Practice Advisory calling for checks on existing buildings with stairs that ay be detailed to allow for horizontal movement when subject to earthquake actions.



CTV Building – 1986/7

The presentation follows closely that given to the next of kin, the survivors, tenants and the media in conjunction with the release in February 2012 of the DBH Reports on the CTV Building. It is pitched at a lay audience but touches on the key points of interest to structural engineers.

The building was 6 floors, counting the ground as the 1st floor. Floors, columns, beams and walls were all of reinforced concrete.

Key features are marked on the photograph.

Note that the south face contains a stabilising (or shear) wall behind the escape stair. A larger set of walls was located around the lift and stairs on the north side.

Note the east face on Madras St. The circular columns can be seen clearly. Similar columns support the floors on the south face and inside the building.

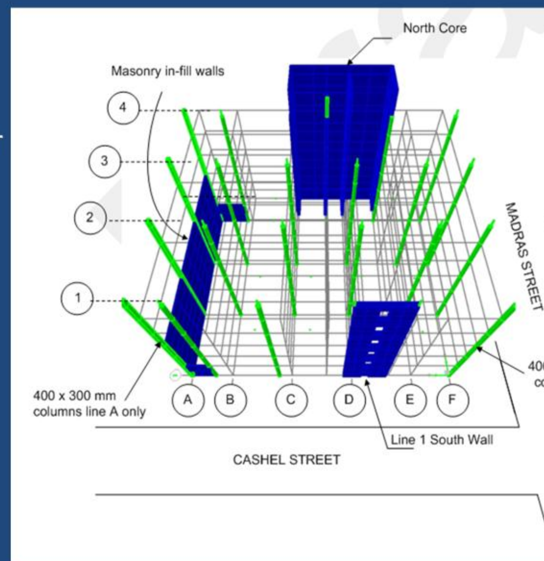
The precast concrete spandrel panels are prominent. They provide fire separation between floors, assist in sun control and contribute to the architectural appearance of the building.

These panels are an inverted ‘L’ shape with a vertical face clearly visible.

Importantly, there is a horizontal leg that extends back to the column line. The gap between these panels and the columns was nominally 10mm.

Structural features

- Designed / Built in 1986 – 87
- North core walls and south wall for stability (Blue shading)
- Columns (Green shading)
- Masonry in-fill walls on the west face to Level 3 (Blue)



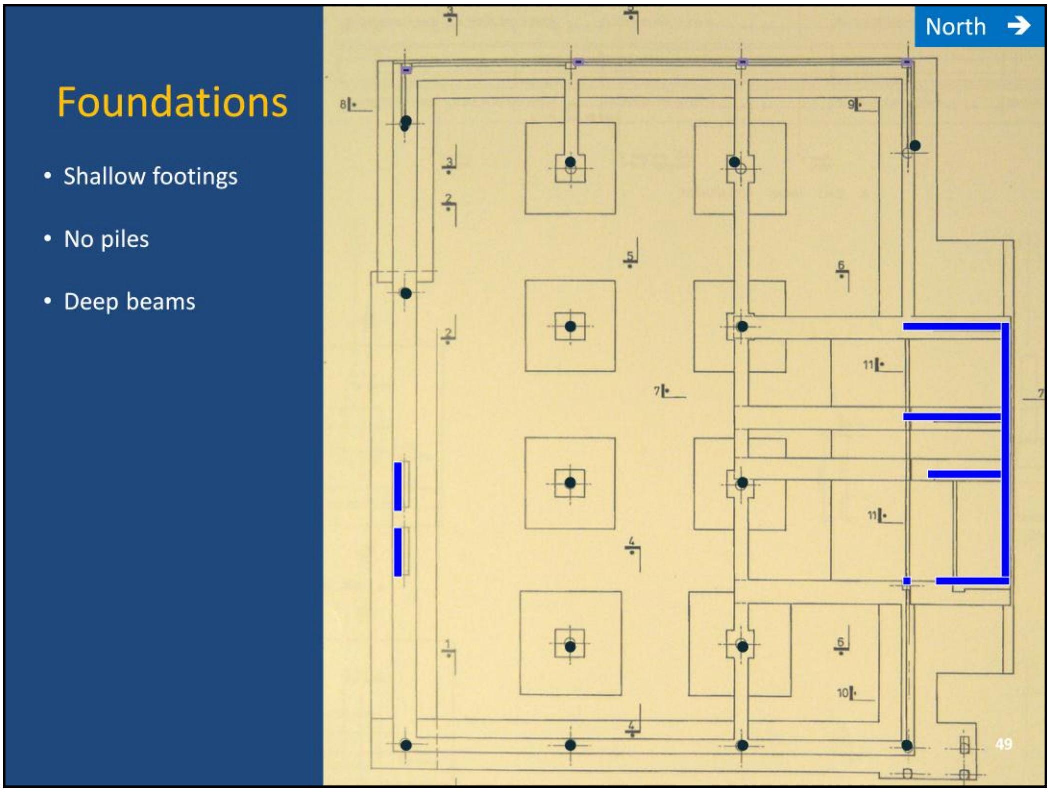
Structural features (CTV)

The building was designed and built in 1986-7. In this view of the structural analysis model the stabilising walls at the north core and the south wall are highlighted in blue.

The lateral stability of the building relied on these two sets of walls.

The other walls highlighted in blue are masonry walls up to level 3 on the west face. These were not intended to be part of the structural system.

The mainly circular columns are shown in green. They were detailed to have identical reinforcing details at all levels and locations. The specified minimum concrete strength was greater for the lower levels to cope with the higher loads.



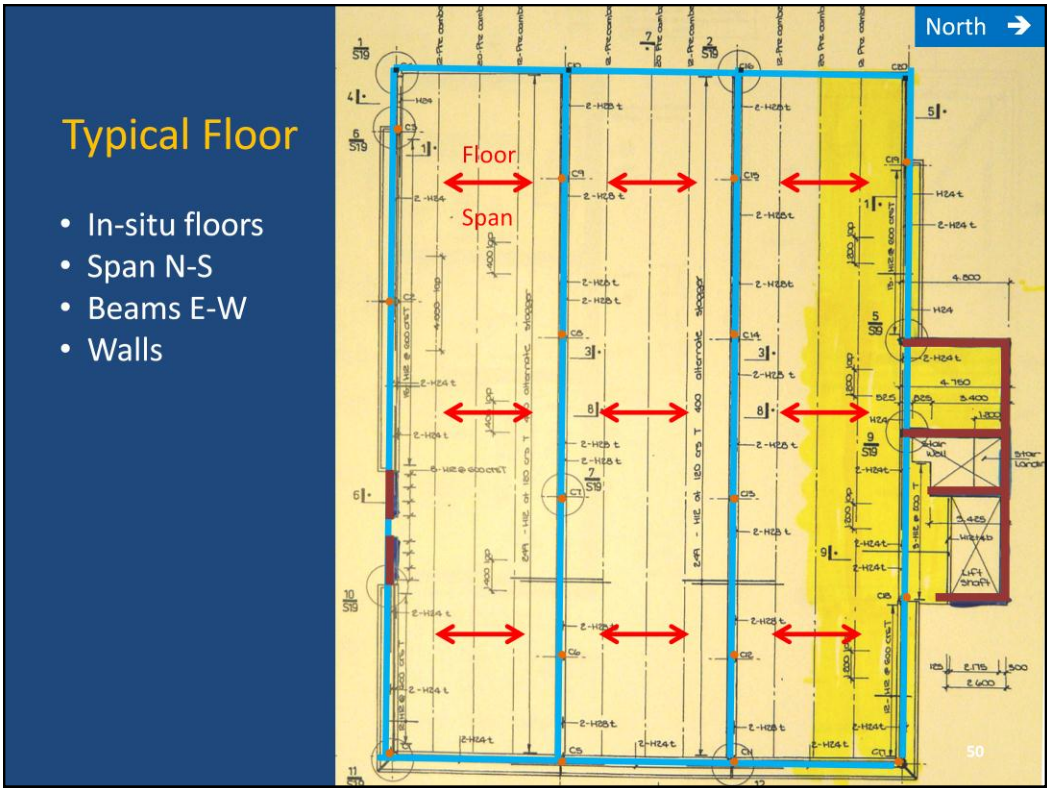
Foundations (CTV)

Foundations were shallow – up to 2 metres deep. There were no piles.

Deep beams anchored the stabilising walls which are highlighted in dark blue.

The north core is considerably stiffer in the east-west direction than the coupled shear walls to the south.

Columns positions are shown as dark dots.



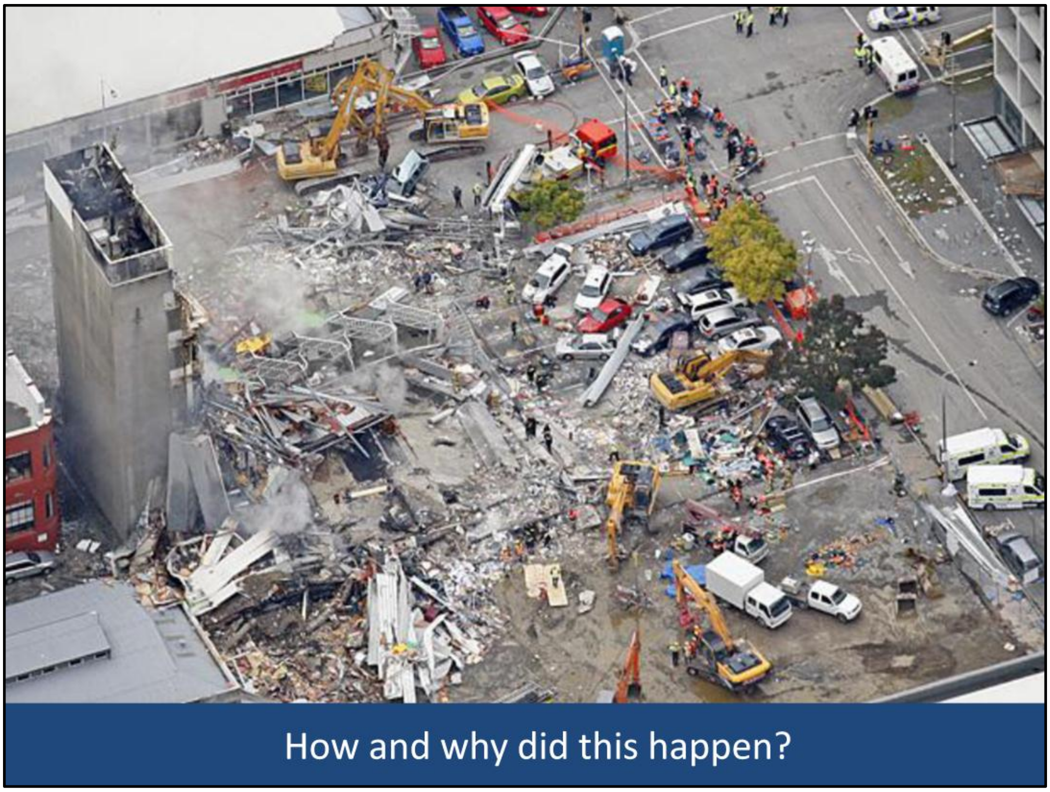
Typical floor plan. (CTV)

This plan shows the reinforced concrete in-situ (poured in place) floors spanning between reinforced concrete beams (light blue)

The stabilising walls are highlighted in deep red.

All floors were the same at each level.

The voids created by the lift and stair well provide limited direct connection between the floor slab and the north core walls.



How and why did this happen?

By the time the Department's investigations started work had already begun on removing the debris.

The fundamental question for the investigation was:

How and Why did this happen?

Investigation outline

- Eyewitness accounts
- Examination of collapsed building
  - Photos
  - Physical examination
  - Materials sampling and testing
- Structural analysis
  - Earthquake effects on the building
  - Strength of critical structural members
- How and why did the building collapse?
  - Possible collapse scenarios (How?)
  - Factors causing collapse (Why?)
- Lessons for future design and construction

Investigation outline

Eyewitness accounts were actively sought and obtained from interviews.

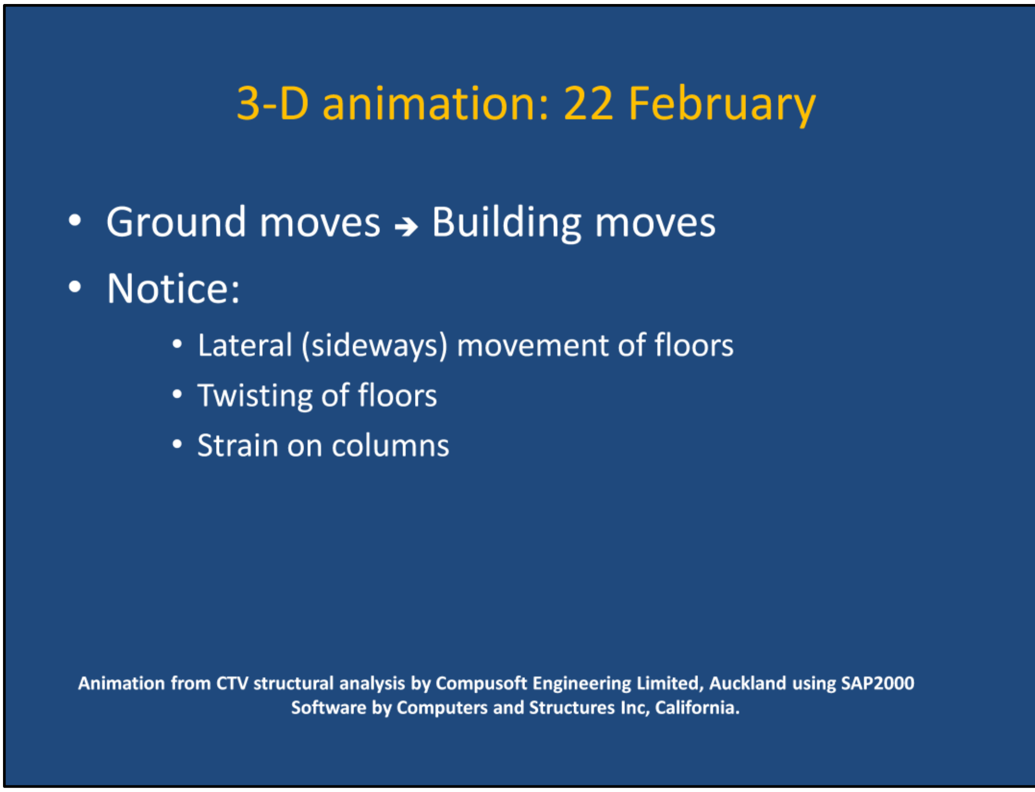
Examination of the building included photos, physical examination on site and at the Burwood Landfill.

Sampling and testing was mainly for concrete and reinforcing steel.

Structural analysis looked at two main aspects:

- estimating the effects of the ground shaking on the building
- comparing these with the assessed strengths of critical members, particularly columns.

This work enabled the development of possible collapse scenarios (How?) and identification of vulnerable elements and factors that may have contributed to the collapse (Why?).



3-D Animation

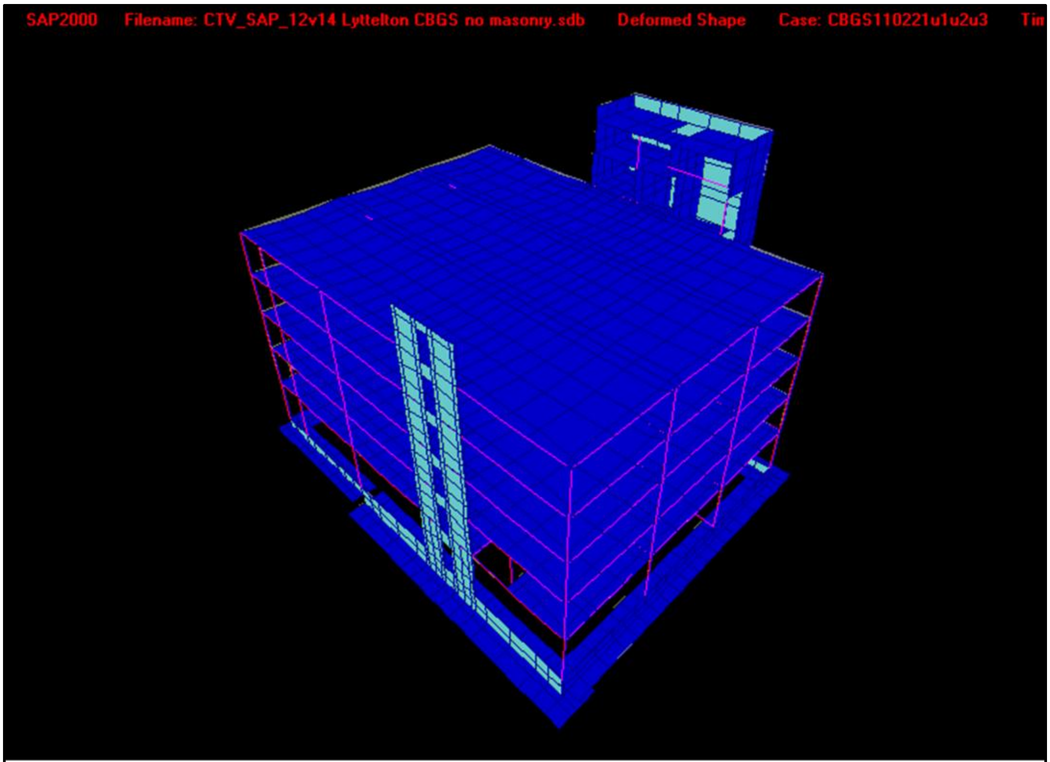
The following is an animation from the non-linear structural analysis which shows how the CTV Building may have moved about in response to the ground accelerations – horizontal and vertical – on 22 February 2011.

The animation does not show the collapse, but the following should be noted:

- Overall sideways movement of the floors
- The twisting of the floors
- The strain on the columns as shown by the appearance of dots.

The dots on the columns appear when they have come under high strain.
(The colours of the dots do not mean anything.)

It is noticeable how much less the north core walls sway compared with the south wall. This accounts for the large twisting.



Animation from CTV structural analysis by Compusoft Engineering Limited, Auckland using SAP2000 Software by Computers and Structures Inc, California.

This animation is derived from output from one of several analyses done on a structural model of the CTV Building.

Several different ground motion records were used in different analyses.

Separate sets of analyses were done firstly assuming that the block walls to the west affected structural behaviour and then assuming that they did not.

How did the building collapse?

- A number of possibilities were examined.
- In all cases the collapse started with *failure of a column*
- A possible sequence is
  - Failure of a column on the east face at mid to upper level
  - Failure of internal columns at mid to low level
  - Progressive failure of columns and collapse of floors

How did the (CTV) building collapse?

A number of possible collapse scenarios were examined.

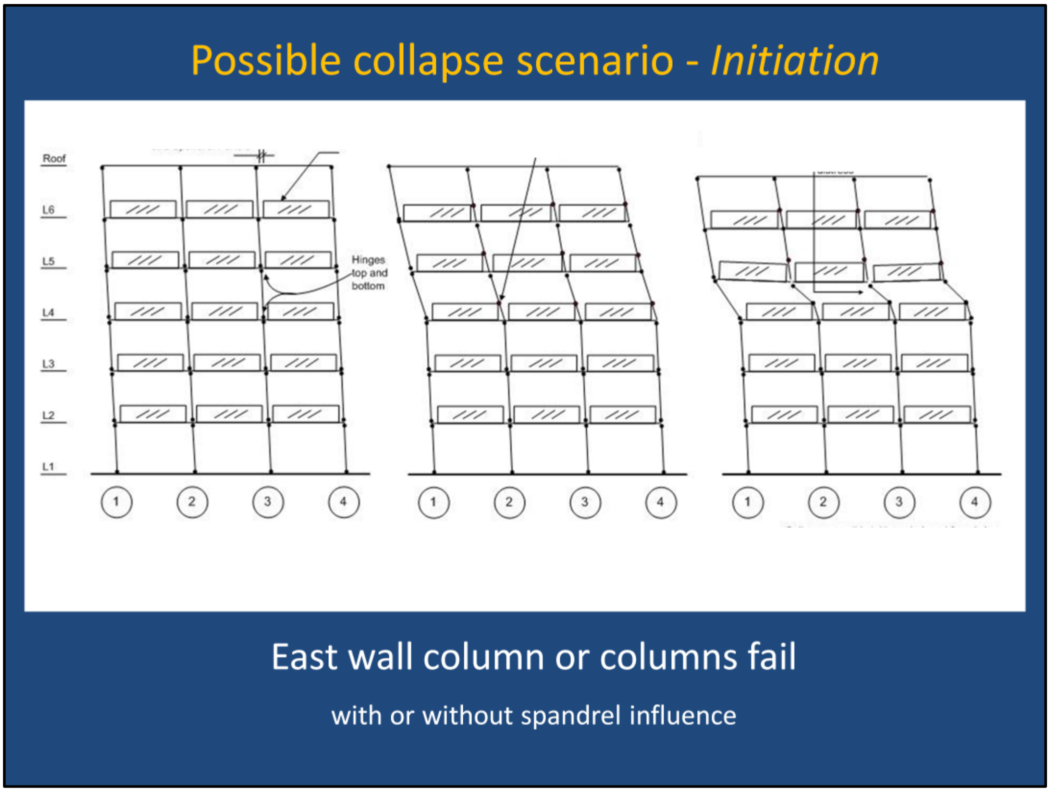
In all cases the collapse started with *failure of a column*.

(Note: The analyses focused on columns but it was recognised that failure of a beam-column joint rather than a column could have triggered the collapse.)

A possible sequence is:

- Failure of a column on the east face at mid to upper level
- Failure of internal columns at mid to low level
- Progressive failure of columns and collapse of floors

The next three slides show this *possible* collapse sequence.



Possible collapse scenario - initiation

In this scenario a column on an upper level on the east side is taken to have initiated the collapse.

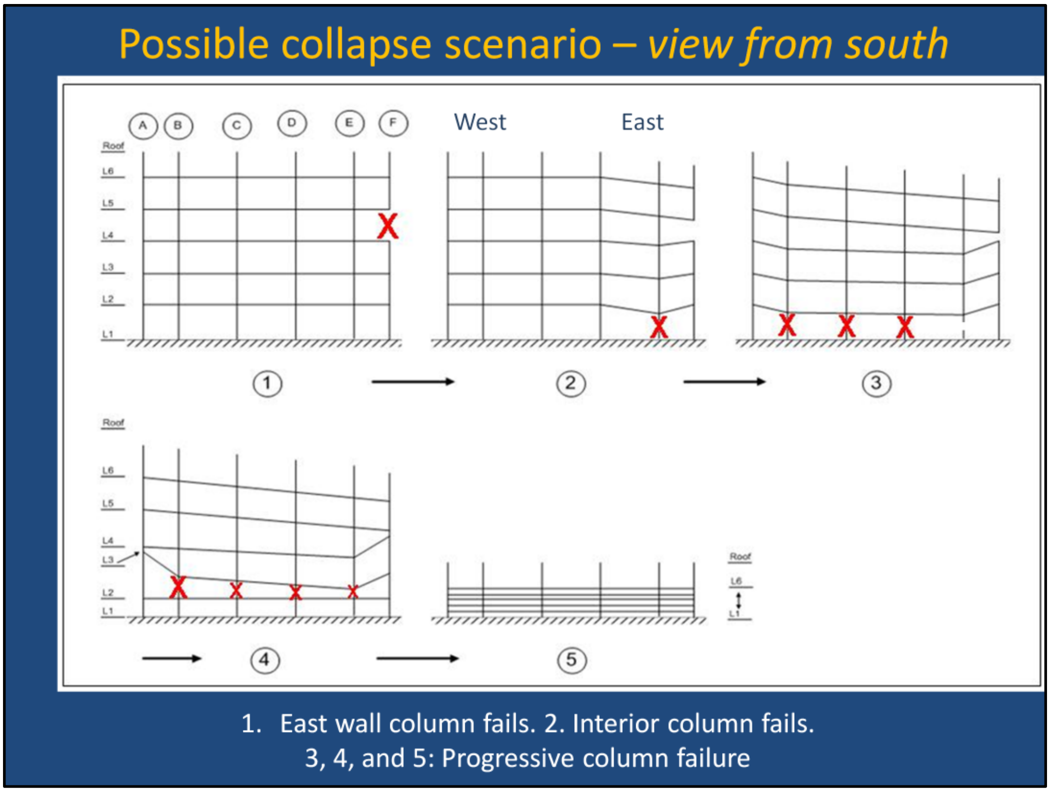
This is consistent with eye-witness reports of an initial tilt to the east before the building collapsed “straight down”.

The building swayed and this put strain on the columns.

(The spandrel panels may have restrained the movement and caused early column failure.)

Once one column failed, others quickly followed.

Weight was transferred to the internal columns ......



Possible collapse scenario – view from south

Here is a view from the south.

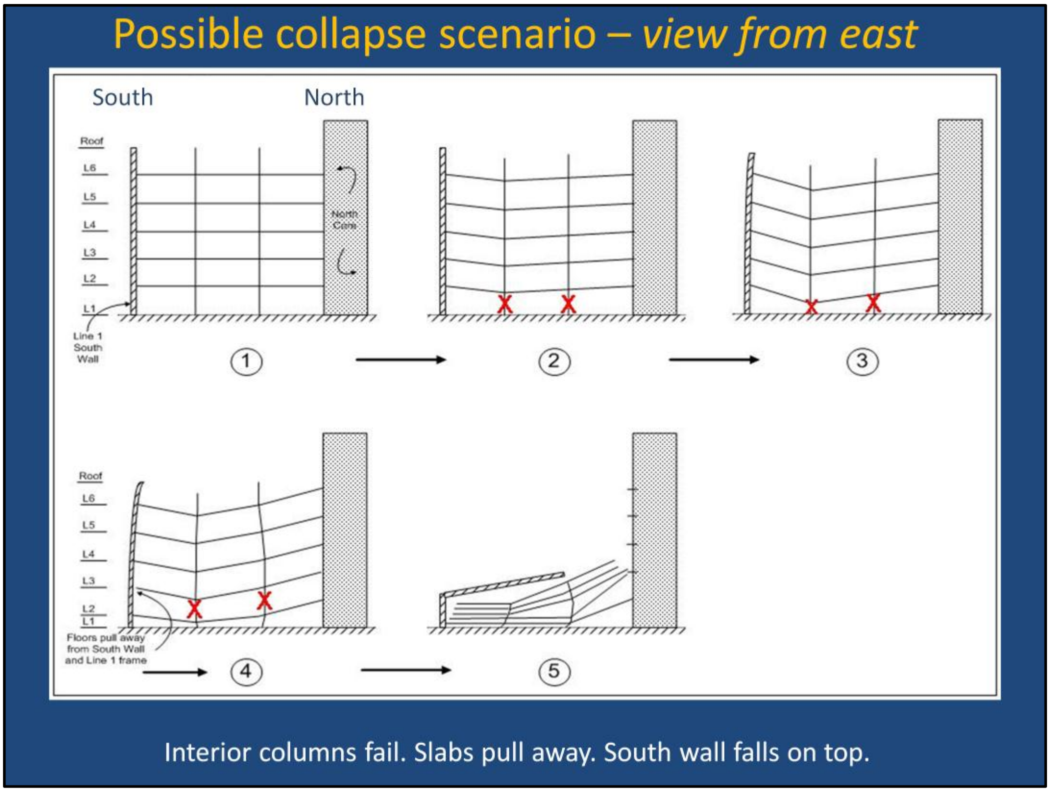
1. East columns fail as just described

2. Weight transfers to internal columns and causes failure of one of these at a low level

3. Failure spreads rapidly to adjacent columns

4. Slabs and beams drop, distorting the columns. Column failure spreads

5. The floors collapse onto one another, particularly in the sections to the south.



Possible collapse scenario – view from east

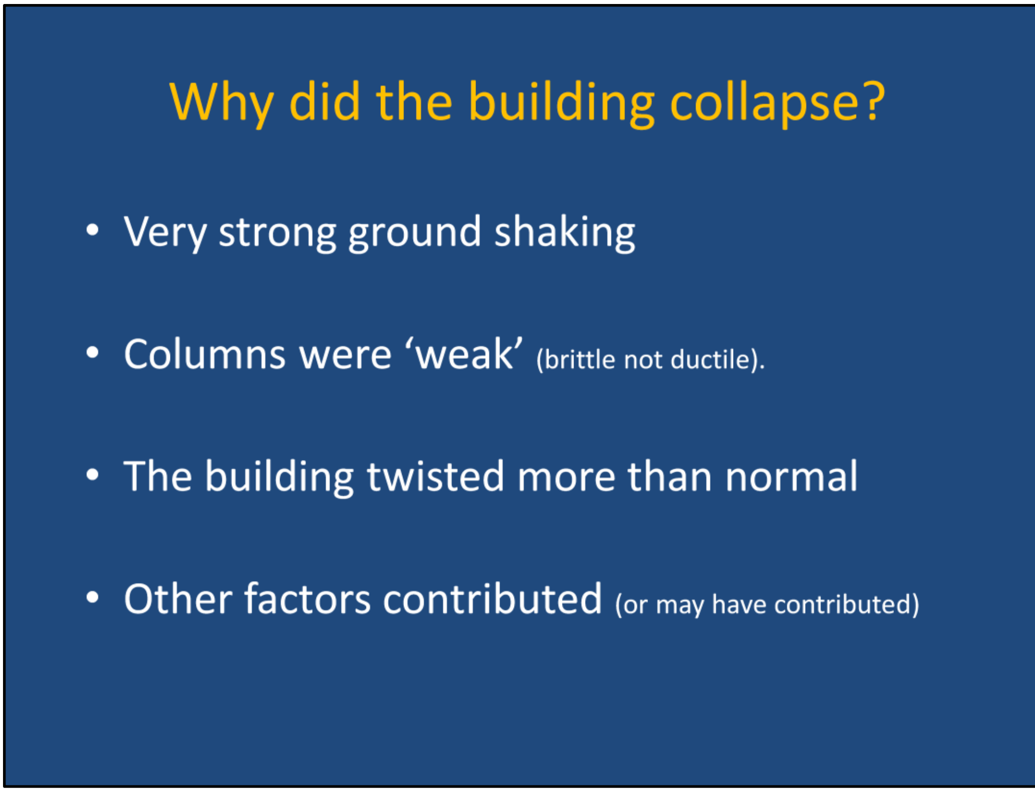
This is a view from the east, but looking at an ***internal column line*** – in line with the north core and south wall:

1. East wall columns fail – no visible effect on internal columns, but load increases on them
2. Internal columns fail at low level
3. Floors drop, distorting and straining the remaining columns and beams
4. More columns fail and the floors start to pull off the north core and south wall
5. Floors collapse. South wall pulled over on top of slabs

The investigation team was well aware that other scenarios could fit the circumstances as well.

Other possible scenarios included:

- Failure of a lower column on line D due to high load and high displacement demand. This does not fully explain the initial tilt to the east, but fits another eye-witness report.
- Failure of an external beam-column joint.
- Failure of an internal column due to increased demand due to high lateral displacements at one level caused by failure of the connections to the north core.



Why did the building collapse?

A range of factors contributed to the collapse.

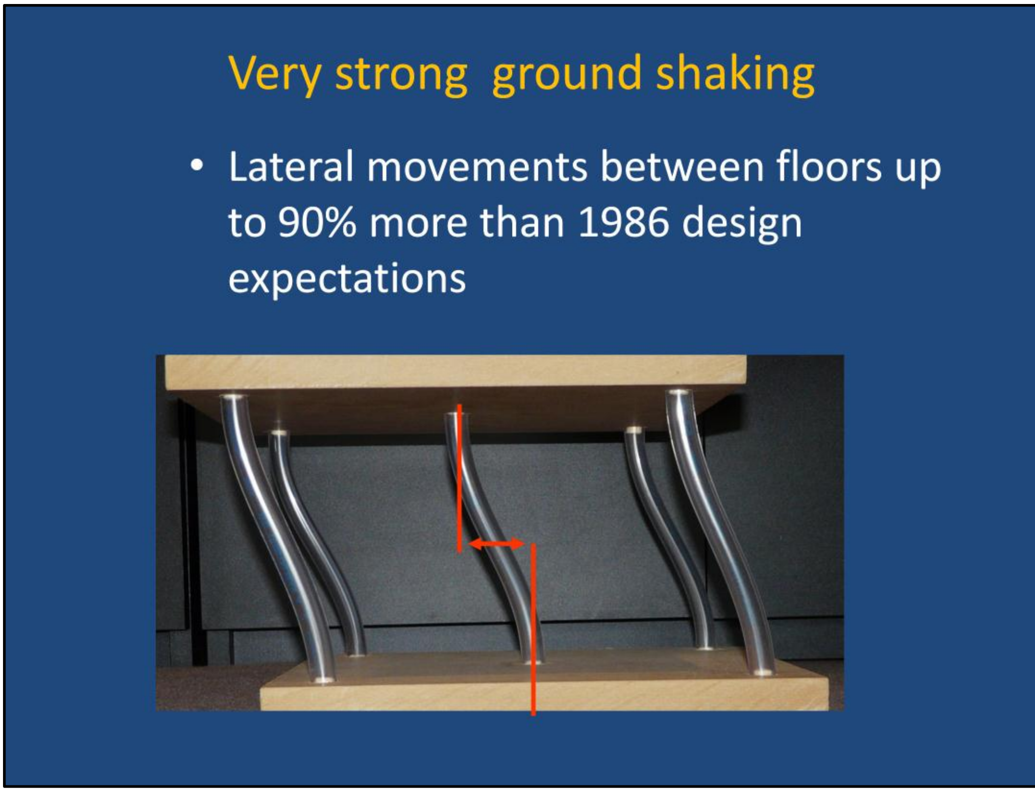
We know that together they brought about the collapse.

It is not possible to say exactly the contribution of each, but ...

Three factors were critical:

- The horizontal ground shaking was very strong
- The columns were brittle rather than ductile.
- The building twisted more than is normal

Other factors contributed or may have contributed.

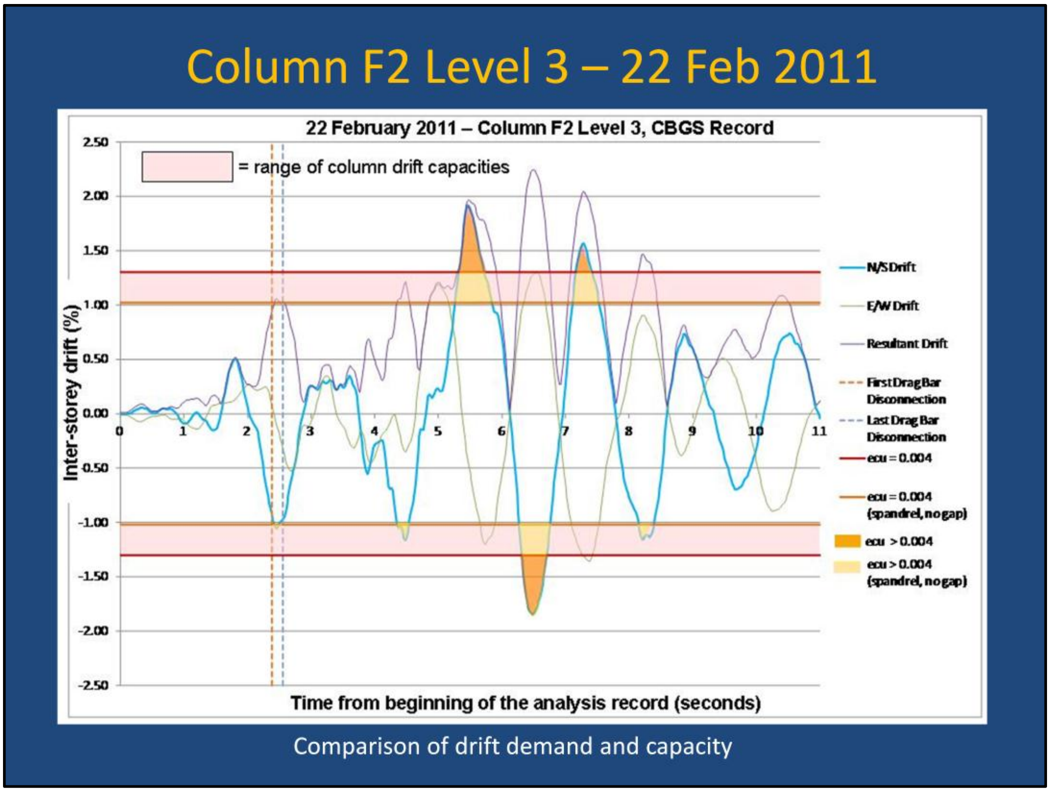


Very strong ground shaking

This simple model demonstrates the lateral displacement between floors that occurs as a building sways and shows the resulting distortion of the columns.

There can be no doubt that strong ground shaking was a factor. The building survived quite strong shaking on 4 September, but the ground shaking on 22 February was much more intense, and the demands on the columns much larger.

The strong horizontal ground accelerations on 22 February caused the floor movement (predicted by the analyses for one critical column) to be 90% more than 1986 design expectations and about double that in the 4 September event.



Column F2 Level 3 22 Feb 2011

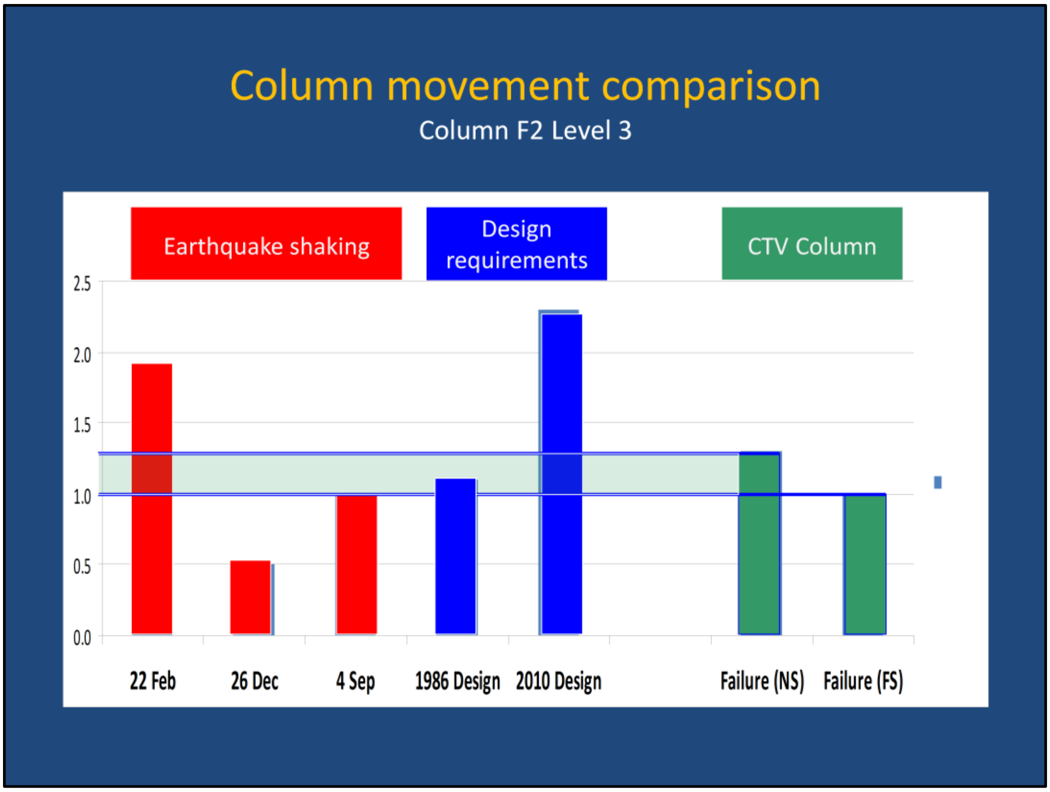
This plot from the non-linear time history analysis (NTHA) provides some insights into possible column failure.

It shows the sideways movement estimated for the column (vertical axis and squiggly lines) against the duration of strong shaking (horizontal axis).

The pink band shows the estimated capacity of this column to sustain lateral displacement, with and without the effect of spandrel restraint.

The vertical dotted lines at 2.5 seconds (the record starts at about 17 seconds from the actual start of noticeable shaking) indicate that the drag bars capacities are calculated to be exceeded at this point.

There is considerable debate about the failure of the drag bars and the possible separation of the diaphragm from the north core walls.



Column movement comparison

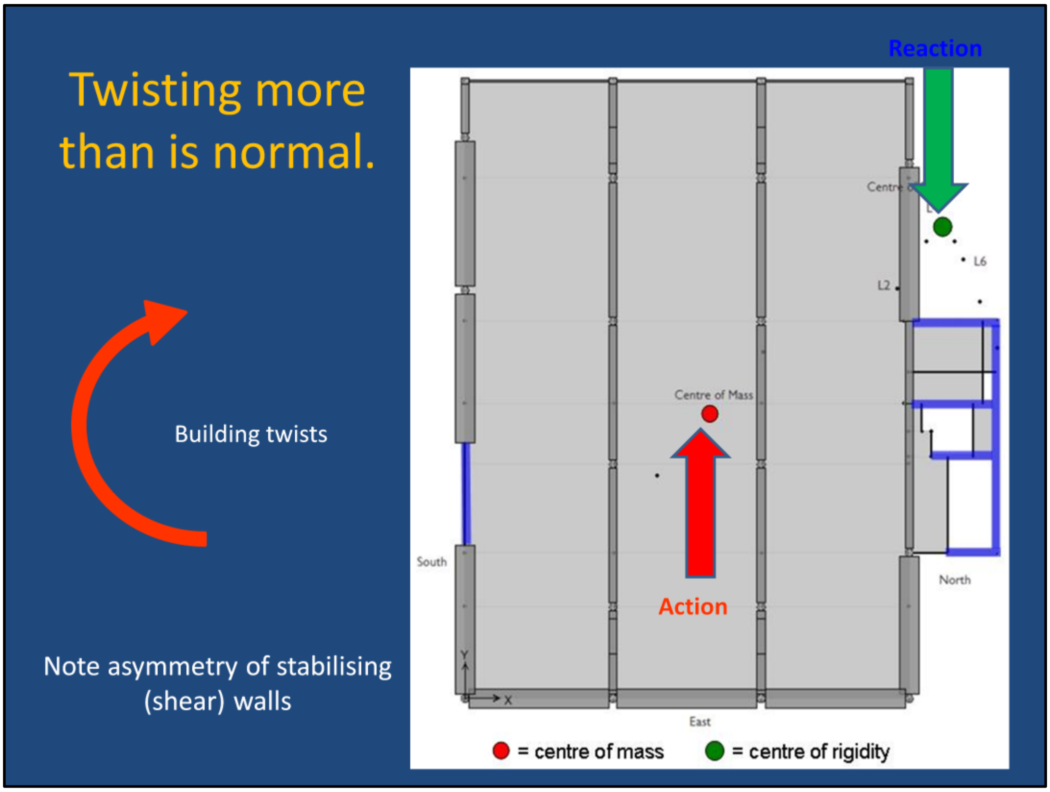
This diagram compares estimated movements, capacities and design requirements for a potentially critical column. It provides some very useful insights into the collapse.

The green represents the estimated movement capacity of the column – with and without spandrel interaction. FS = Full spandrel interaction. NS = No spandrel interaction.

Estimated earthquake shaking movements for this column are shown in red. For the 22 February event the estimated movements are about double those estimated for the 4 September event. These, in turn, are about double the estimated movements in the 26 December 2010 event.

Design requirements for 1986 and 2010 for this column are shown in blue.

When compared to design levels it can be seen that 22 February event was much more demanding than anticipated by the 1986 Standard, but that current standards require allowance for displacements similar to those estimated for the 22 February event.



Twisting was more than normal

The **second critical factor** was the greater than normal twisting of the building.

The twisting effect is included in the 90% extra floor movement mentioned previously and was estimated to contribute about one-third of the overall movement to the worst-affected columns.

The diagram indicates the basic cause of this twisting:

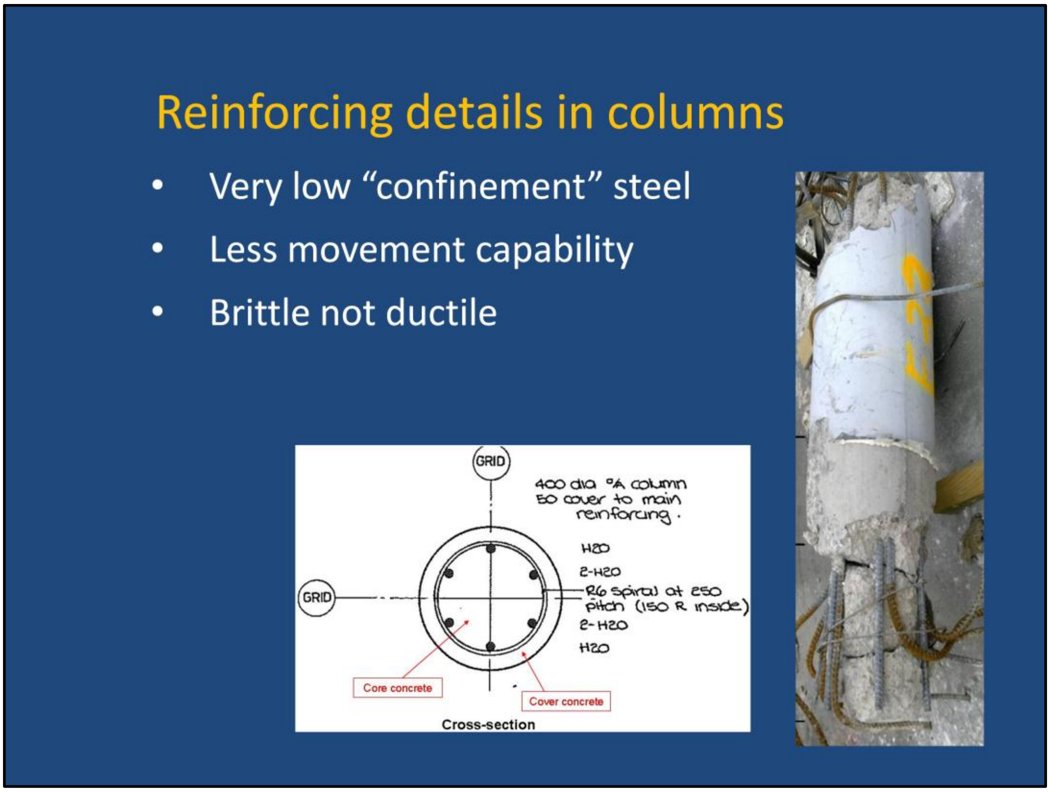
- Forces act at the centre of mass – near the middle of the floors. (Red arrow)

- Forces are resisted at the centre of rigidity . (Green arrow)

- The centre of rigidity in this case is well to the north as shown because the north wall is so much stiffer than the south wall.

- The action is on the red arrow and the reaction is on the green arrow.

This means the building twists (as well as moving sideways)



Reinforcing details in columns

The **third critical factor** is the weak (non-ductile) columns.

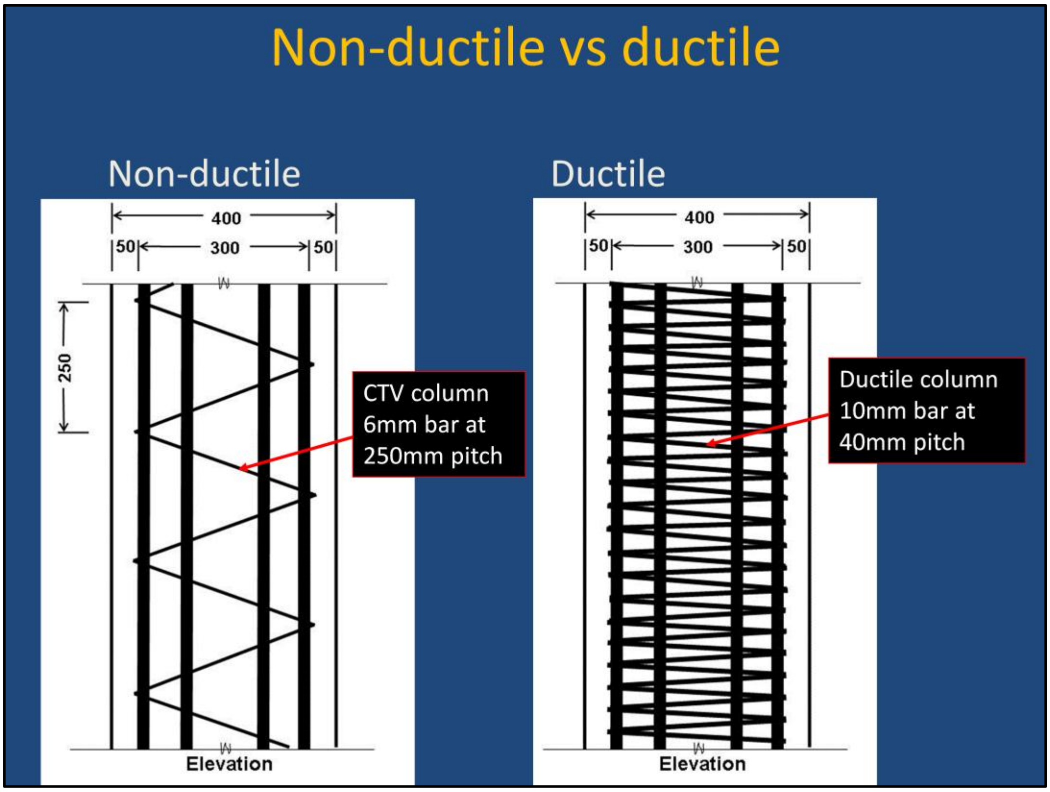
Most of the columns in the CTV building were circular and were 400mm diameter with six 20mm diameter bars lengthways. These bars were set a minimum of 50mm back from the surface of the column. This provides "cover concrete" for corrosion and fire protection.

For engineers, the important part of the column is the "core" – the concrete inside the ring of main bars.

This core (and the column overall) can be made "ductile" by binding it tightly with "lateral" reinforcement such as a spiral.

The spiral for the CTV columns was 6mm diameter at a pitch of 250mm.

This is not enough to make it ductile.



Non-ductile vs ductile

A pencil will flex when subject to bending action. But soon it snaps and has no strength. It is brittle.

A piece of wire is ductile. When subject to bending action it retains its strength even when bent through a large angle.

A structural member, such as a column, that is ductile is able to retain its load-carrying ability (and integrity – it holds together) even when it undergoes large movement.

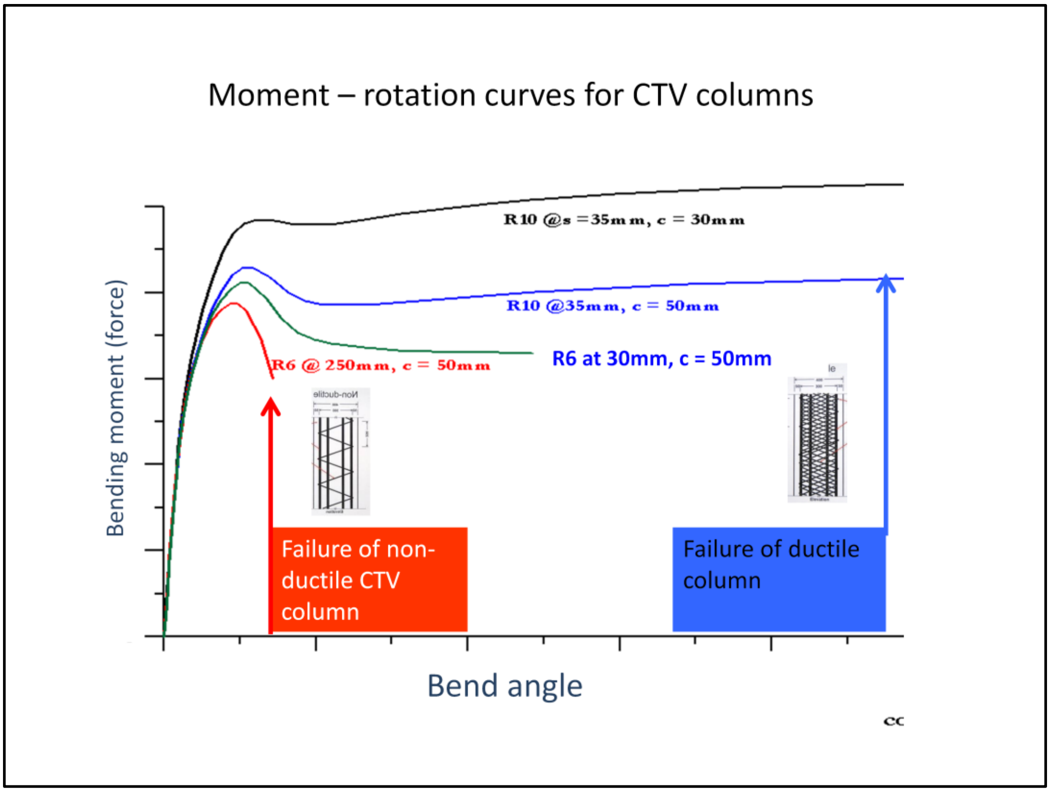
The diagrams above show the non-ductile CTV column and a ductile column of the same size. The ductile column has much more steel in the spiral. It has a larger diameter bar at closer spacing.

The investigation team concluded that the CTV columns should have been detailed for ductility.

The columns would have been much stronger and absorbed much more movement if they had been ductile.

Even in the non-ductile state a typical critical column was shown to be capable of absorbing about two-thirds of the floor movements caused by the 22 February event.

However, once the limit was exceeded, there is nothing to hold the column together and it immediately loses all capacity to carry vertical load.



Moment-rotation curves for CTV columns

This slide shows how much more bend angle a ductile CTV column will sustain without losing strength when compared with a non-ductile CTV column.

The red line shows the CTV columns as designed and built. They fail at a low bend angle.

The green and blue and black lines show the CTV columns with substantially increased amounts of spiral "confining" steel.

The black line indicates the case of more spiral steel and less cover concrete – 30mm rather than 50mm. This greatly increases the core area and thus the strength of it.

Other factors

May have contributed:

- *Vertical ground accelerations*
- *Interaction of columns and spandrels*
- *Low concrete strength in critical columns*
- *Separation of floor slabs from north core*
- *Structural effect of masonry walls*

Not significant in starting collapse:

- *Foundations / site conditions*
- *Limited robustness / integrity*
  (caused collapse to be more extensive).

Other factors

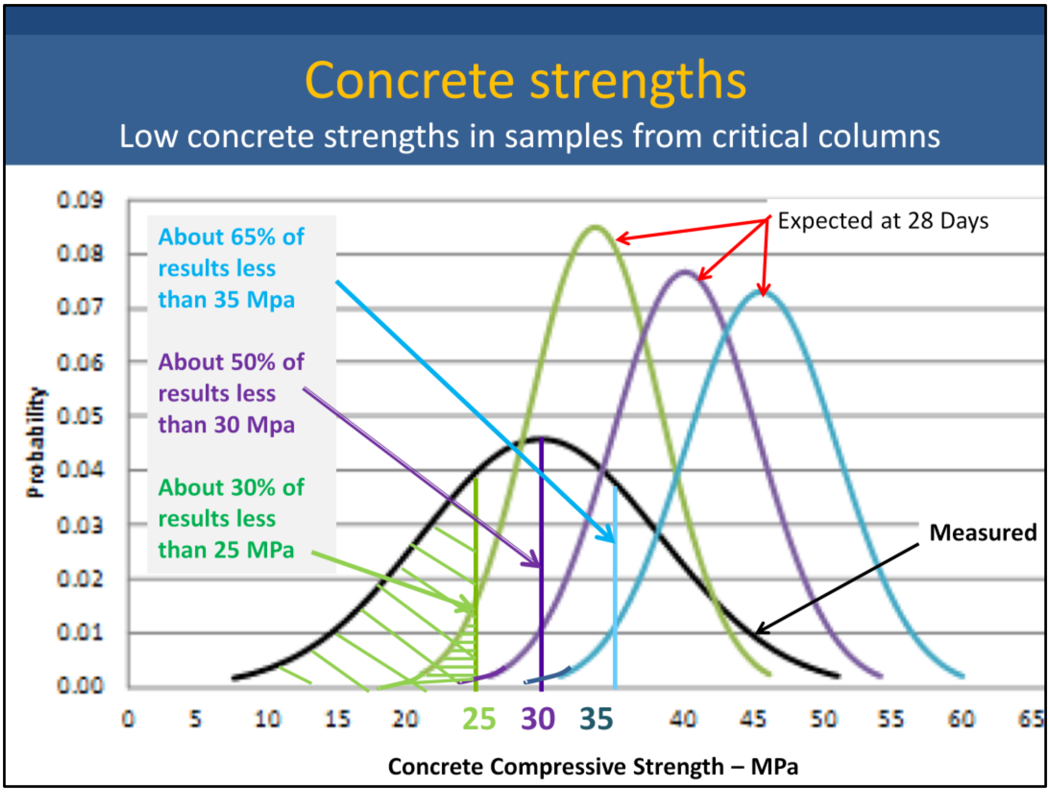
Other factors that contributed to or *may* have contributed to the collapse include:

- ***Vertical accelerations*** are likely to have affected the loads in the columns. Additional compression in the columns (up to 100% of the original load) was estimated to reduce the capacity to absorb floor movements by up to 35%.
- Full ***Interaction with the spandrels*** was estimated to reduce the movement capability of critical columns by about 30%. Some interaction could have occurred but it would not have been necessary to cause column failure.
- ***Measured concrete strengths*** were lower than expected. It may be that results were affected by the collapse or the sampling process. Lower concrete strengths would mean reduced ability for columns to sustain the lateral displacements.
- ***Separation of the slabs from the north core*** may have occurred but it is hard to be certain. In any case, the floor movements used in the assessments assumed separation did not occur. If it did, the movements would have been larger.
- ***The masonry walls*** were found to have some effect on floor movements but these were not as significant as expected.

Other factors considered included:

- ***Foundations and site conditions.*** These were not considered to be a factor. Surveys of the site after the collapse showed no movement or evidence of liquefaction

- The ***limited robustness / integrity*** (tying together) of the building. Except for the possible slab separation from the North Core, the limited robustness was not considered to have caused the collapse. However, greater robustness may well have reduced the extent of the collapse.



Concrete strengths

Tests on 26 column samples showed lower than expected results. It is not known which levels the samples came from – the reinforcing details were the same at each level.

The ground floor was specified to have 35 MPa, the second and third floors 30 MPa and the remainder 25 MPa concrete.

This slide shows the distribution of test results compared with expected strengths at the time of construction.

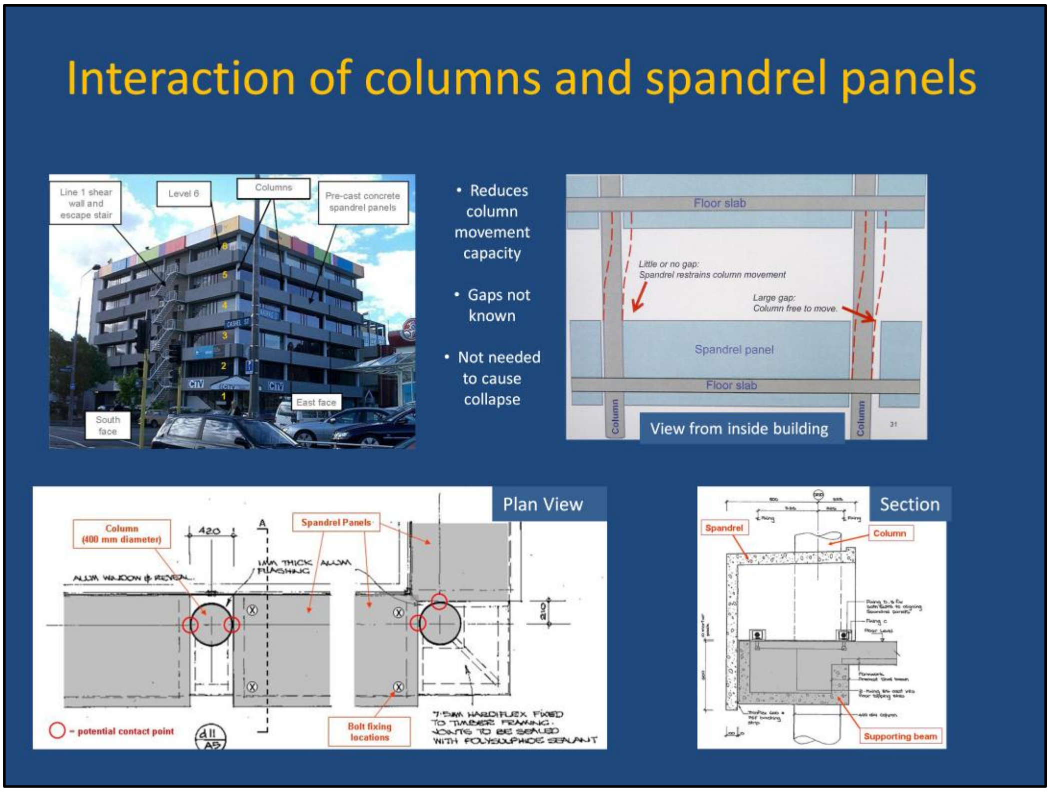
About 30% of results are less than 25 MPa, 50% less than 30MPa, and 65% less than 35MPa.

There is considerable debate as to whether these represent the real concrete strengths prior to the earthquakes since the results could reflect the sampling process and damage to the concrete during the collapse.

Clearly, low concrete strengths would affect the capacities of columns to sustain load.

The fact that collapse did not occur in the 4 September event suggests that the concrete strengths may not have been as low as the graph suggests.

Nevertheless, the Expert Panel recommended follow-up investigations to provide confidence in *as-built* concrete strengths in existing buildings and future construction.



Spandrel context and details

This diagram illustrates how the spandrels could have produced a "short-column" effect and so reduced the capacity of columns to absorb the imposed lateral displacement between floors.

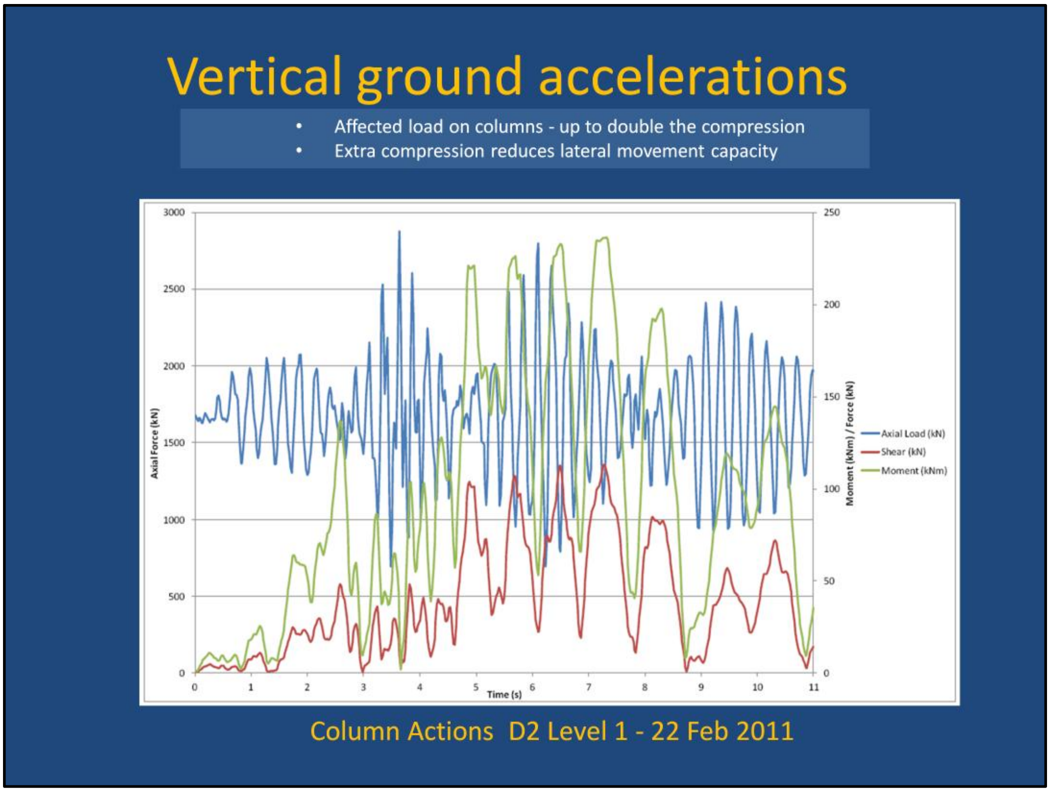
The specified gap was 10mm, but in practice may have been less. It is not possible to know what they were in reality.

The capacity of the spandrels and their fixing was limited so that the full (rigid) restraint would not have been possible.

However, calculations were made assuming rigid restraint or none at all. This provided results for the extremes of the possible effects of spandrel interaction.

Note that the spandrels appear not to be in the column line, but in fact were. This is because there is a horizontal ledge extending back beyond the column line. The red circles on the plan view show the possible points of impact.

Restraint is not as rigid as it would have been had the vertical section of the spandrels been in line with the column.



Column actions D2 Level 3 – 22 Feb 2011

The exceptionally large vertical ground accelerations will have affected the columns, producing fluctuating compression loads in them.

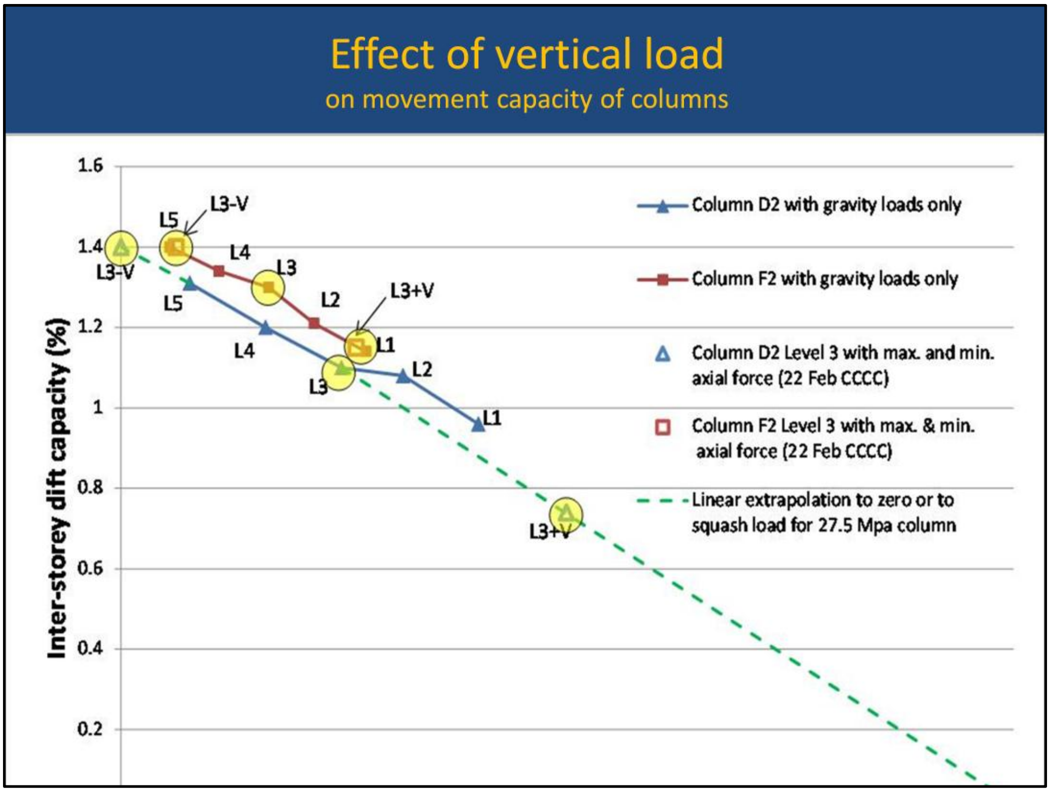
This graph shows various effects in this Level 3 column plotted against time.

The blue line shows axial compression starting at 1700kN and fluctuating between about 700kN and 2800kN. For lower levels, this fluctuation would have been a smaller proportion of the higher initial load.

The green and red lines show moment and shear variation, both of which are measures of the lateral displacement imposed at any instant.

Not surprisingly the oscillation of compression is much more rapid than that of moment and shear.

This means that there is a high chance of having large compression at the time of large displacement – even if the peak values do not coincide.



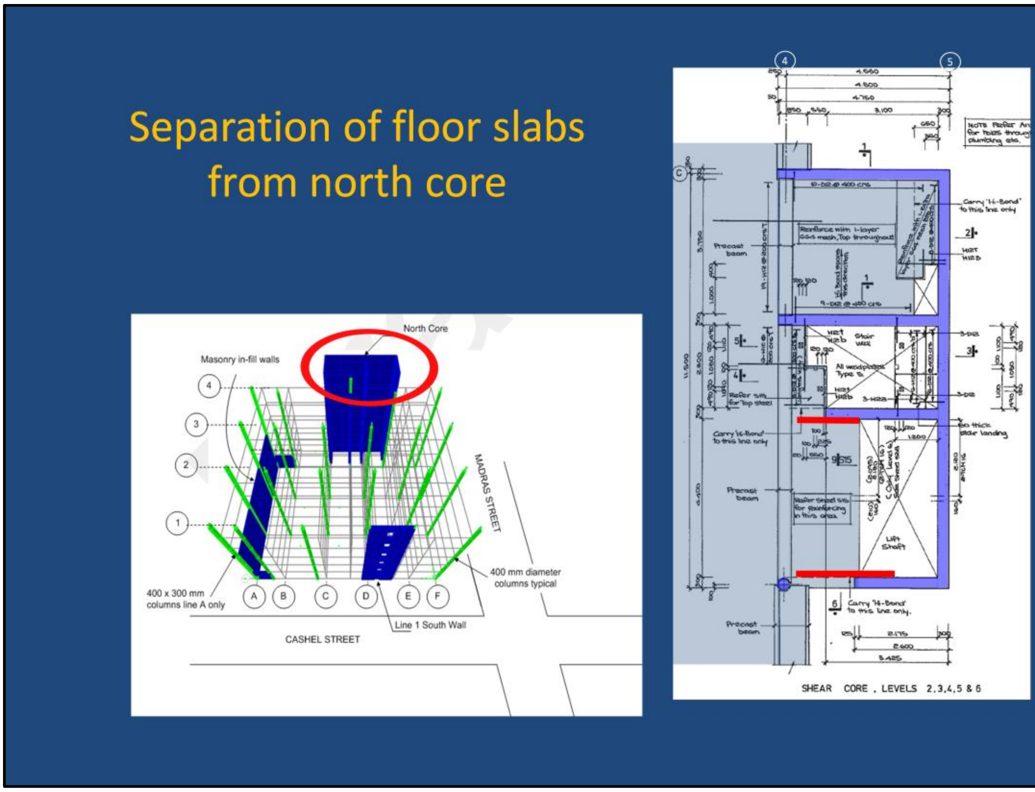
Effect of Vertical Load

This graph was made to illustrate the effect of vertical load on the capacity of the columns to sustain lateral displacement.

The bottom line plots column F2 at Level 3 (an exterior column) showing capacity to vary by about 30%.

The top line shows a similar situation for column D2 (an interior column) at the same level. The effect of vertical acceleration is much less at around 7%.

These comparisons act as a reminder that choosing a particular column and Level as the initiator of the collapse (assuming that it was a column) is practically impossible.



Separation of slabs from the North Core

The limited connections between the slab and the North Core shear walls made separation in the earthquake a strong possibility, and this was recognised from the start.

A pre-purchase inspection of the building identified a weakness here and in 1991 additional connections were installed at Levels 4, 5 and 6.

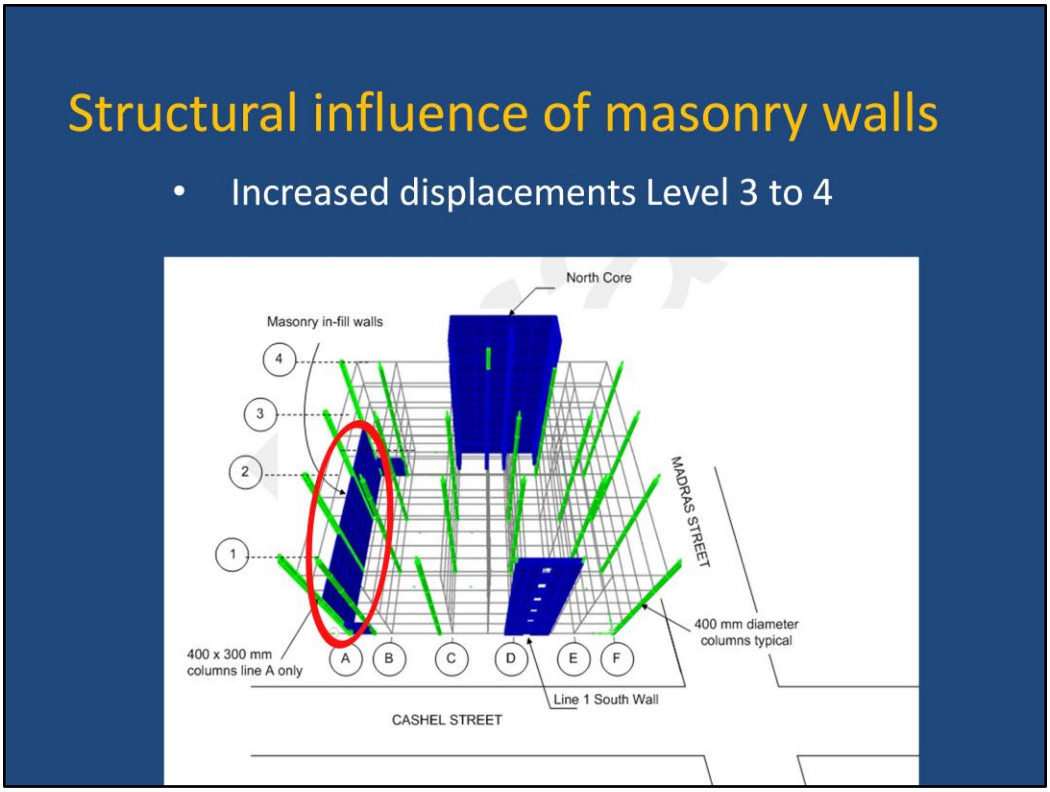
These are the “drag bars” shown in red. They consist of large steel angles bolted to the walls and the slabs.

The non-linear time history analysis (NTHA) showed that drag bar capacities were reached early on in the shaking record.

However, it was recognised that calculation of the instantaneous force in these bars was subject to considerable uncertainty. For example, the true strength and stiffness of the drag bar/bolted connection assembly were difficult to estimate.

Analyses were made assuming no separation and these analyses produced sufficient displacements to fail the columns.

Had the slab separated at any level, the displacement demands would have been greater and failure of a critical column would have occurred sooner.



Structural influence of masonry walls

These walls were intended to be separated from the structure but this was not fully achieved.

The NTHA showed that they had less effect than expected but the potential negative influence was a reminder of the need to make separation of such elements fully effective in practice.

To achieve separation in practice requires careful specification followed up by thorough site inspections.

Issues for follow-up

- Ground shaking allowances for design
- Ductile detailing in columns
- Concrete strengths
- Separation of spandrels and similar elements
- Diaphragm (slab) connections
- Asymmetrical response of structures
- Robustness / integrity (tying together)

Issues for follow-up

The issues identified for follow-up are those where the investigators consider current and / or past standards or practices should be examined and changed if found necessary.

Not surprisingly, the list follows closely the factors identified that contributed or may have contributed to the collapse of the CTV building.

The Expert Panel and consultants feel strongly that every effort must be made to learn from issues raised by the collapse of the CTV Building on 22 February 2011.

CTV Observations

- Forensic work harder than design
- A lot of questions raised
- Highlighted many assumptions that are taken for granted
- Variability and uncertainty
- Calculations give a rough guide to actual behaviour
- ~~"Calculate"~~ → "Estimate"
- Royal Commission hearings

CTV Observations

Here are a few general observations resulting from the CTV investigations:

Forensic work is much harder than design. There are so many unknowns and assumptions to be made when trying to work out what actually happened.

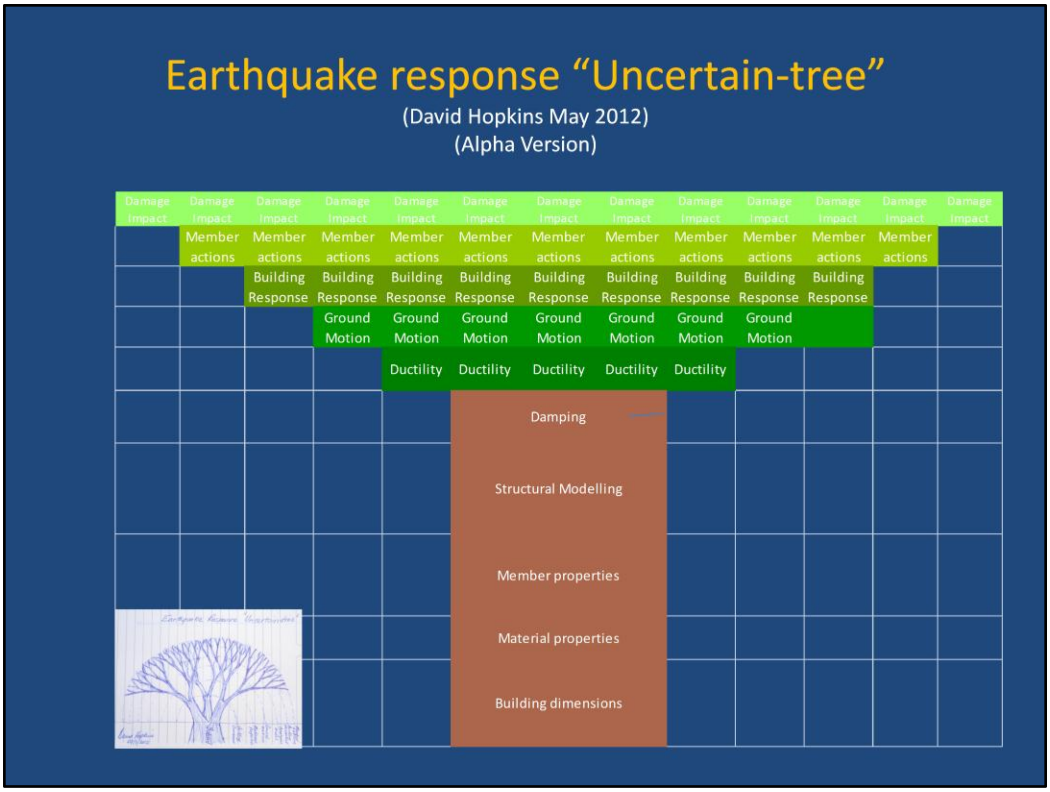
The investigation process and back-analysis involved lays bare the many inherent assumptions in design standards and procedures

Every parameter affecting the structural performance / response is subject to variability and uncertainty. This is compounded when there are multiple vulnerabilities in the building.

These considerations were a reminder that even the most sophisticated calculations should be regarded as a rough estimate of what might happen in reality.

This thinking needs to be included in all design standards and procedures.

The upcoming Royal Commission hearings are likely to indicate a wide range of views and interpretations of the various parameters and vulnerabilities of the CTV Building.



Earthquake response "Uncertain-tree"

This is a crude representation of an important point that was particularly evident from the investigations into the collapse of the CTV building.

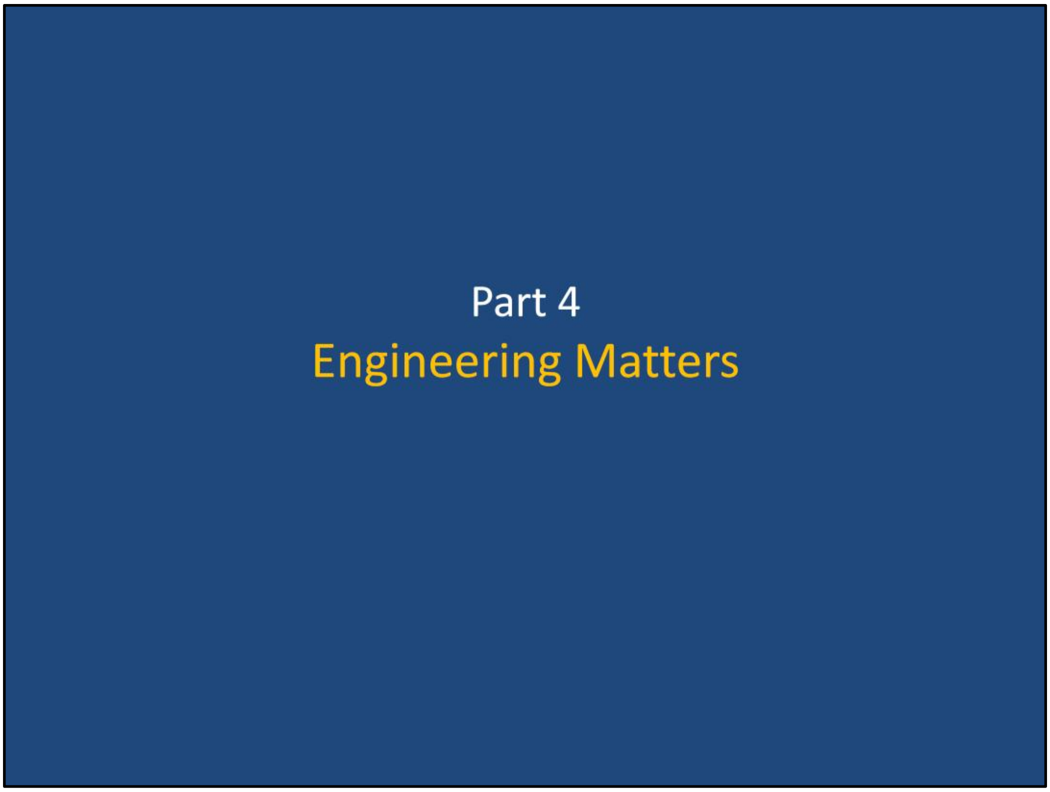
It is intended to show that each assumption is equivalent to a fork in a tree.

This means that the estimated effect on a particular element depends on the assumptions made in the analysis.

It is important to remember this dependence on assumptions when designing for any particular calculated result. Reality could be markedly different.

Development of a more elegant and powerful representation of this concept is ongoing – refer sketch at bottom left.

But no matter how elegant or crude the diagram, the underlying message cannot be ignored and must be taken on board by all those involved in earthquake engineering design.



Part 4 Engineering Matters

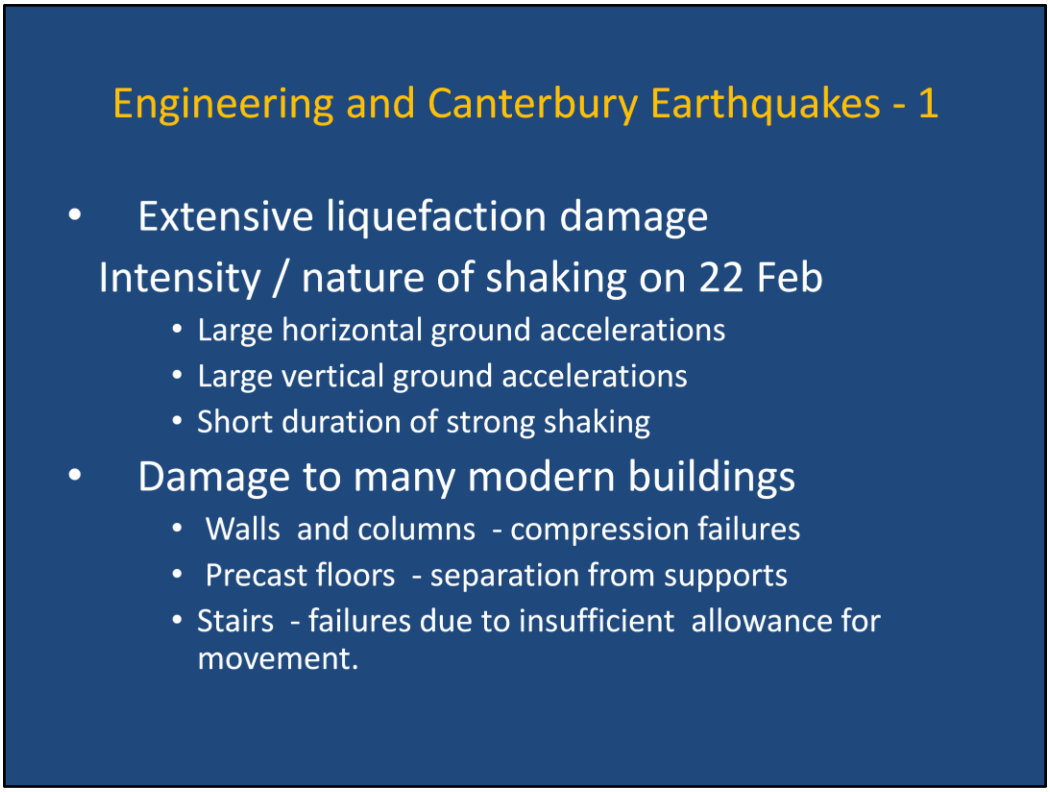
These two words capture the fundamental lessons from the Canterbury Earthquakes

Engineering matters in two ways:

Engineering matters to the community.

Engineering matters to engineers in the design of buildings for earthquake.

It is vital that both the community and engineers understand that engineering matters.



Engineering and Canterbury Earthquakes 1

Here are some of the highlights of engineering aspects of the Canterbury Earthquakes and their effect on buildings and infrastructure.

Engineering and Canterbury Earthquakes - 2

- Small damage ➔ total loss – much demolition
- URM buildings mixed bag. Securing helps
- Non-damage to well-designed buildings
  - Clear, simple concepts
  - Conservative design - well designed and built
  - Attention to foundations
  - High integrity – tying together
- Serious questions re loading / design approaches
  - Probability –based loading
  - Assumptions in design and analysis
  - Complexity of standards

Engineering and Canterbury Earthquakes 2

Here are some more observations on the damage - plus some observations on non-damage and questions on what building performance in the Canterbury earthquakes means for loading and design standards.

Engineering matters to the *community*

This must be reflected in the adequacy of:

1. Engineering resources at central and local government level
2. Technical resources applied to issues of building quality
3. Resources for engineering education and research
4. Resources applied to professional development training
5. Resources applied to the development of engineering standards

Greater recognition is needed of the importance of engineering in the planning, design, consenting and construction of important infrastructure and buildings.

Engineering matters to the community

There are some important questions to ask:

Are there adequate engineering resources applied:

1. at central and local government level to provide confidence in design standards and their implementation with proper recognition of engineering issues?
2. to issues of building quality?
3. to engineering education and research?
4. to professional development training?
5. to the development of engineering standards for design and construction?

Greater recognition is needed of the importance of engineering in the planning, design, consenting and construction of important infrastructure and buildings

Engineering matters for engineers

Good concepts and detailing are (still) vital for good earthquake performance

1. More attention needed to fundamentals
   - clear load paths
   - high integrity,
   - simple concepts
   - capacity design
2. Beware sophisticated structural analysis
3. Beware complex standards and requirements
4. Beware probability-based earthquake design actions

Engineering matters for engineers

There is a growing tendency to regard engineering as the ability to do calculations on structural models or apply formulae in design standards.

More attention needs to be paid to fundamentals

- clear load paths
- high structural integrity (tying together)
- simple concepts
- capacity design (eg. Weak beam / strong column)

Sophisticated computer-aided structural analysis can give the impression of precision in predicting the response of a building to earthquake shaking. The Canterbury earthquakes reminded us not to rely on these analyses totally.

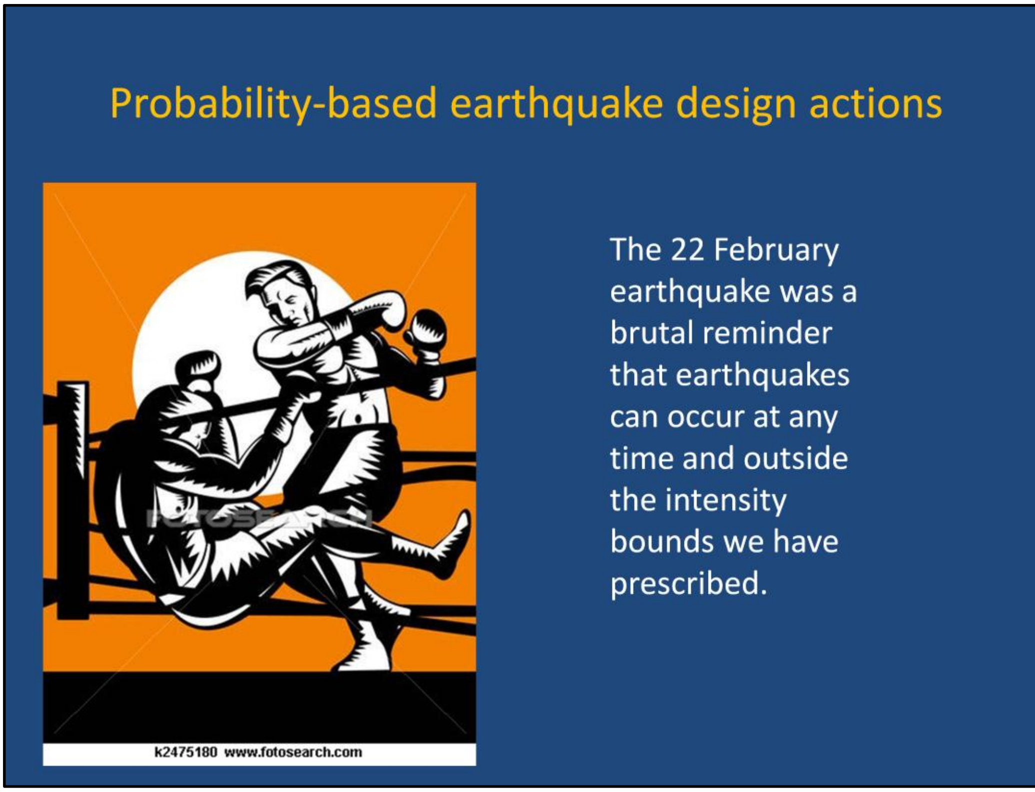
Our structural loading and materials standards have become too complex. There is a concern that many who apply these standards do not fully understand the engineering fundamentals behind them.

Work over the last 30 or 40 years has produced probability-based earthquake design standards and numerous other insights into the probability of earthquake shaking.

We as structural engineers need to step back when we apply this knowledge to our design standards.

For example, given the wide range of performance of real buildings in the Canterbury earthquakes, are contours of seismic hazard factor really justified as a basis for design of buildings?

Do we really believe that such fine distinctions in this one variable will make a difference to building performance – given all the other factors involved?



Earthquake probabilities

The 22 February earthquake was a brutal reminder that earthquakes can occur at any time and outside the intensity bounds we have prescribed.

Consider the frequent use of terms such as:
- 500-year shaking
- 2500-year shaking
- Uniform hazard spectrum
- Annualised earthquake loss
- 10% probability of exceedance
- Loss-optimised design

Elegant analyses of probabilities of earthquake shaking (and their many derivatives) have tricked engineers and code writers into thinking that earthquakes will obey the rules that engineers and code-writers make up.

It is as though we have convinced ourselves absolutely that boxers always punch above the waist, with the gloves on – and only when in the ring!

Probability-based analyses are useful to identify possible trends but should not be relied upon exclusively for setting design parameters.

There needs to be a “what-if?” scenario included when setting requirements and in all design thinking.

Structural design is an art

Engineering matters for engineers!

Overall, these aspects are a reminder that structural design is an art not a just a calculation process.

This fact needs to be evident in the education and training of structural engineers and in the tools used in design for earthquake effects.

Structural engineers must develop a keen insight into the fundamentals of structural behaviour.

They must rely on this insight when using analysis results to determine design details.

Technicians can do calculations and follow prescriptive standards and guidelines

Structural engineers with insight into the fundamentals are needed to carry the overall responsibility for the design of important buildings and infrastructure.

Engineering Matters

1. *The community* must properly recognise the importance of engineering in planning, design, construction and alteration of buildings and infrastructure

2. *Engineers* must reverse a growing trend to obscure structural engineering fundamentals with sophisticated analyses and with overly complex ‘design’ standards.

Engineering matters

“Engineering matters” is a deliberate play on words.

This lecture has touched on some engineering matters related to the Canterbury Earthquakes which I trust have been thought-provoking.

The underlying message is that engineering matters:

- to the community

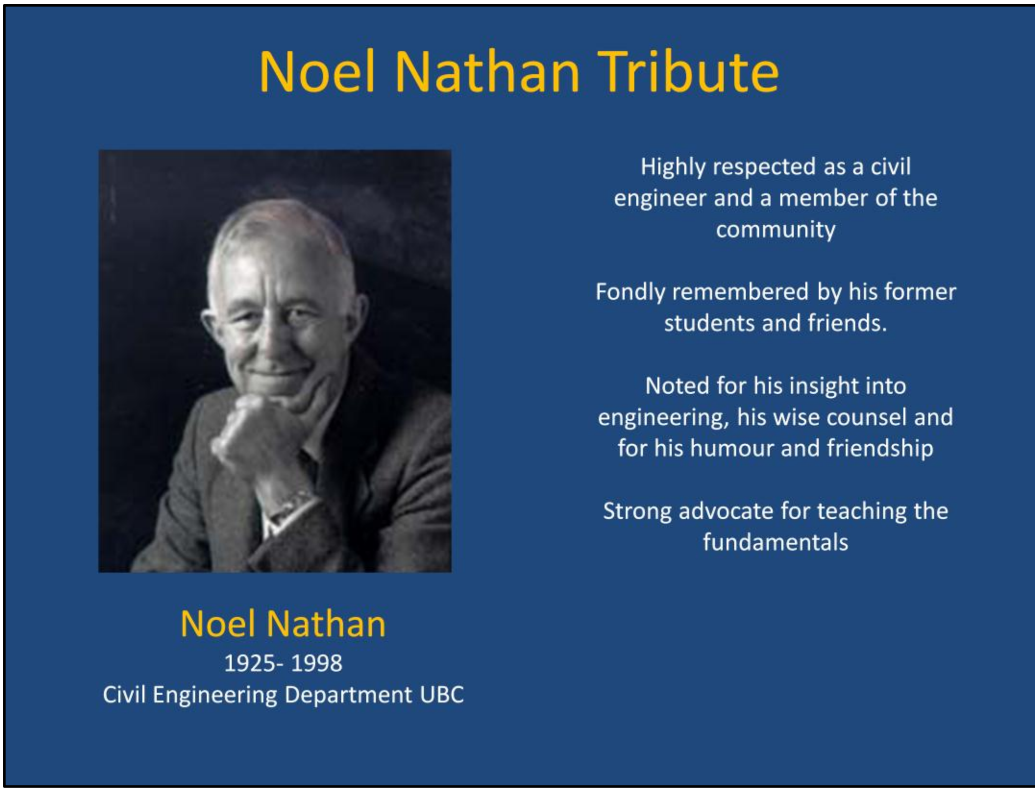
and

- to engineers

This is the most important lesson from the Canterbury Earthquakes.

The lesson needs to be applied in New Zealand.

Other earthquake-prone countries should put the lesson to the test.

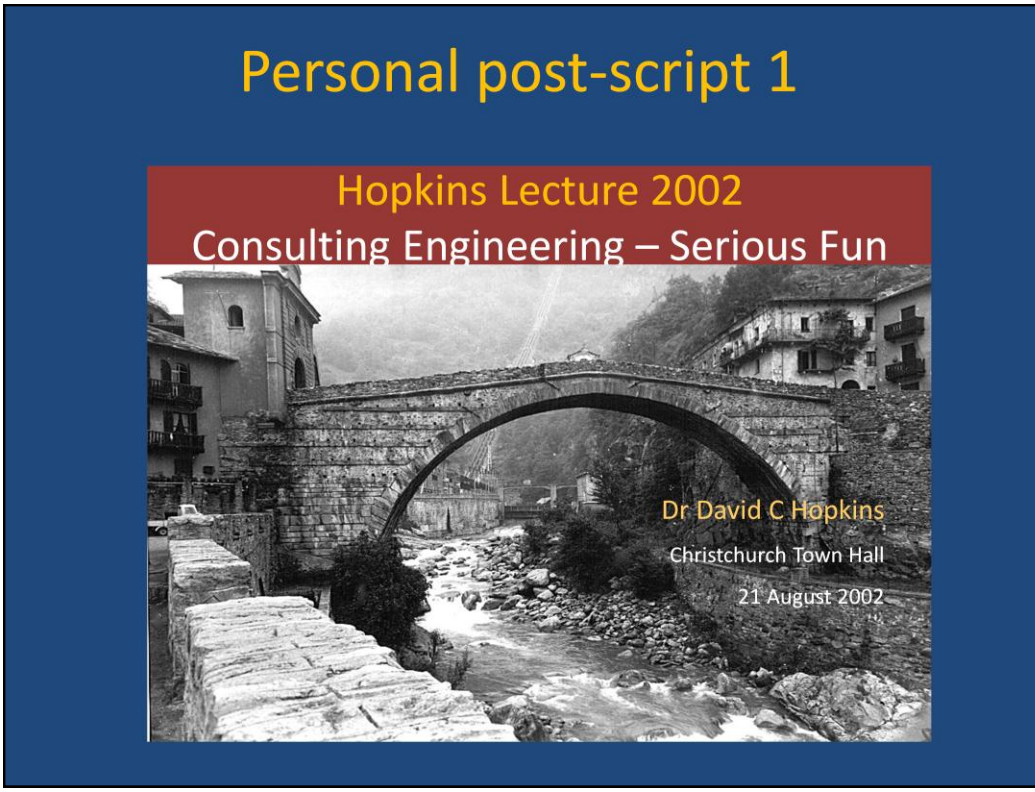


It has been a pleasure and privilege to present this 2012 Noel Nathan Memorial Lecture in Structural Engineering.

Don Anderson said of Noel Nathan that he

".....always pushed for a teaching of the fundamentals. He also was very active in the local structural engineering community, especially in the area of concrete design."

I would like to think that, because of this, Noel Nathan would agree that "engineering matters".



Hopkins Lecture at University of Canterbury

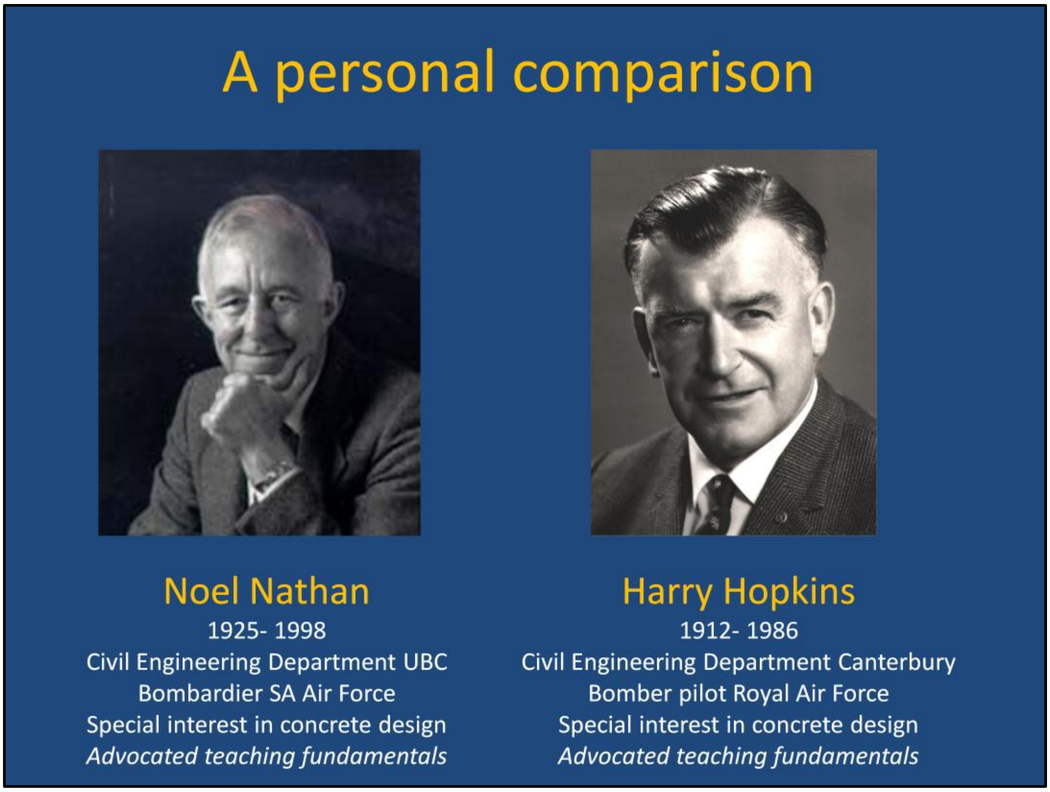
In 1978 an annual lecture on an engineering topic was instituted at the University of Canterbury in honour of my father's 28 years as head of the Civil Engineering Department.

Harry Hopkins died in 1986 and I was privileged to give the Hopkins Lecture in 2002.

Harry Hopkins was known for his advocacy of understanding fundamentals, and taking lessons from the past.

He was widely respected for his leadership of the Department, interest in bridges and concrete design and his involvement in the engineering and wider community.

These strong parallels with the qualities of Noel Nathan add to the privilege it is for me to have given the 2012 Noel Nathan Lecture.



A personal comparison

Don Anderson said of Noel Nathan that he

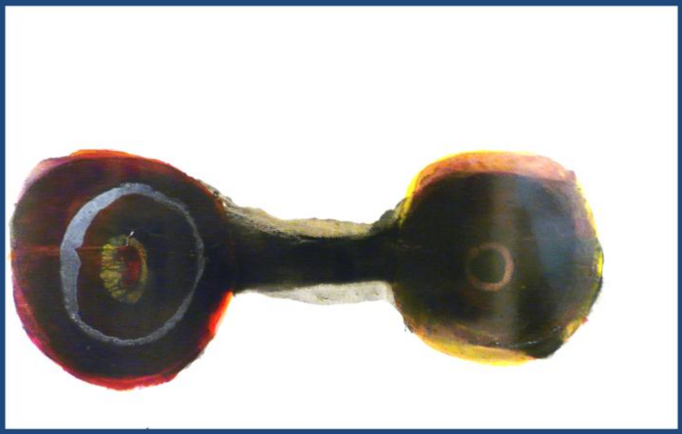
".....always pushed for a teaching of the fundamentals. He also was very active in the local structural engineering community, especially in the area of concrete design."

Harry Hopkins was known for his advocacy of understanding fundamentals, and taking lessons from the past.

Could this common advocacy of understanding the fundamentals of engineering stem from their air force experiences?

Wherever it comes from, the advice is very sound and in need of constant emphasis.

Personal post-script - 2



Abstract painting, *Moving Target,* by Marie Le Lievre, Christchurch 2011
(1670mm x 1170mm)

- Trapped in damaged building in Cathedral Square 22 February 2011
- Retrieved several months later with help of building owner and DH
- Purchased by David Hopkins (Marie's uncle) May 2012
- A moving memento of some extraordinary events and experiences

This painting is special to me because:

- The painting was trapped in the Government Life Building in Cathedral Square for several months – under threat of destruction due to demolition.
- I interacted with the building owner and Marie in the days after the earthquake to help secure its eventual release.
- The building owner, Philip Carter, made special efforts on behalf of Marie and other artist tenants to retrieve their valuable artwork – even though he had suffered the loss of several major buildings in Christchurch.
- The two sets of circular rings remind me of isoseismal maps which are used by seismologists and engineers, and which I use extensively in my work.
- I see the Darfield (right) and Lyttelton (left) events in these representations
- The extensive use of brown and black represent the extensive liquefaction as well as the overall impact of all the Canterbury Earthquakes. The black linking the two sets indicates that extensive liquefaction was common to both events – and occurred in many lesser aftershocks.
- The inclusion of red in the left set of circles reminds me of the tragic loss of life in the Lyttelton event.
- The light brown forms between the sets of rings look like the Port Hills and remind me of the landslip and rock-falls that caused so much damage and loss of prime real estate.
- It has considerable artistic merit regardless of these interpretations.
- Marie Le Lievre is my niece

All in all, this work reminds me of the Canterbury Earthquakes.

It brings to mind all the worst of destruction and disruption they brought, and all the best of human responses evident in the hours, days, months and years that have followed.

Moving Target is a fitting name. The target is Christchurch City, which, together with many parts of Canterbury, has been moving ever since 4 September 2010. So, too, have the epicentres of the thousands of aftershocks.