CTV BUILDING

CTV BUILDING COLLAPSE INVESTIGATION

FOR DEPARTMENT OF BUILDING AND HOUSING
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Part 3 of 3

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APPENDIX A – EYEWITNESS SUMMARIES

INTERVIEWS WITH EYEWITNESSES

Interviews were undertaken with those who were willing to speak of their experiences and what they observed. The names of the witnesses are not revealed for privacy reasons. Their locations are shown on the Eyewitness location map (Figure 57).

Some were inside the building at the time; others were in the street or in other buildings next door with a clear line of site to portions of the CTV Building as it collapsed.

The information gathered from the interviews has been collated into common categories and summarised to identify consistent observations for further technical analysis.

EYEWITNESS LOCATIONS

Eyewitnesses inside the CTV Building

1. Level 6: East side of the southwest corner.

2. Level 1: Ran south out from Reception on the East Side of the building.

3. Level 4: North at the right edge of the building.

4. Level 6: Sitting on the side wall next to the demolition site; farthest away from the front area.

Eyewitnesses outside the CTV Building

5. Les Mills building.

6. IRD building.

7. IRD building.

8. In front of CTV driveway on Cashel Street.

9. Unrestricted view from roof of Les Mills building.

10 & 11 Blackwell Motors on Madras Street side opposite CTV.

12 & 13 IRD building.

14. On east side of CTV on Madras Street just past Samoan Church.

15. In front of CTV driveway on Cashel Street.

16. Working on the re-cladding on the CTV at south west corner of CTV building.

EYEWITNESS LOCATION MAP

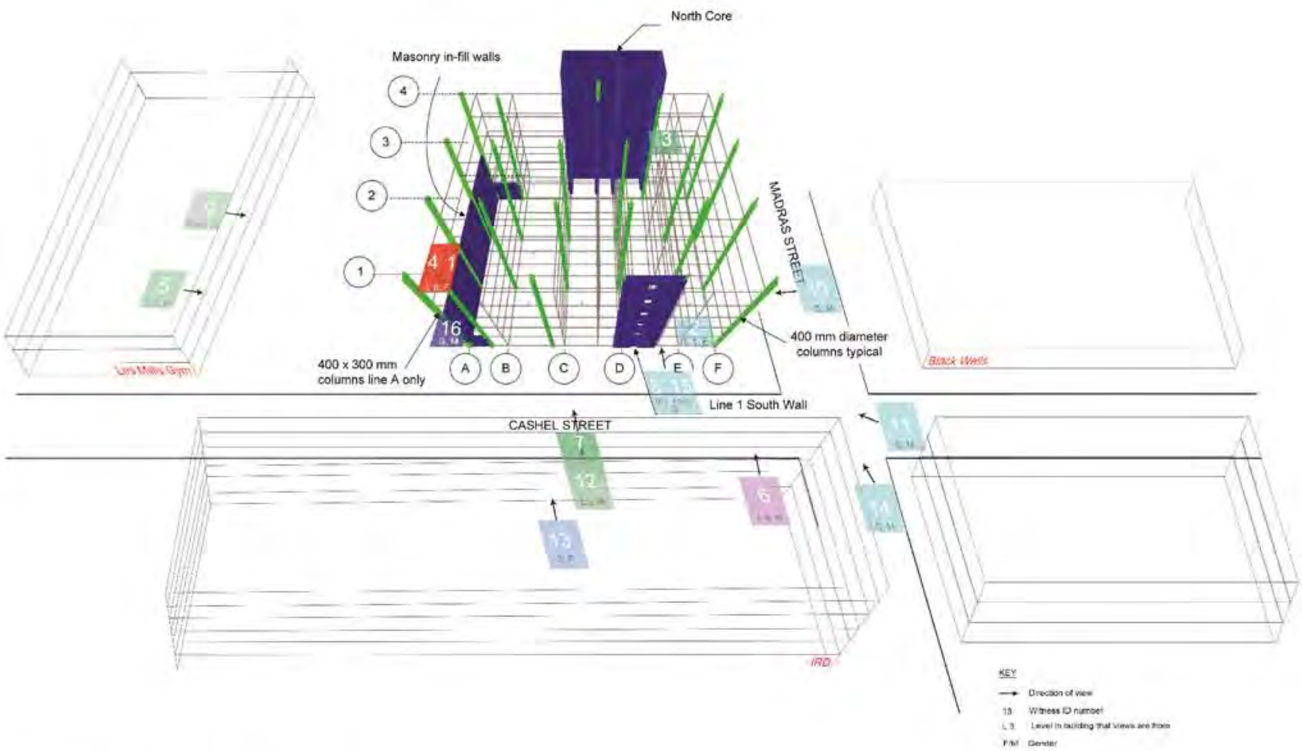


Figure 57 – CTV Building layout and eyewitness locations.


CTV
CTV
1
2
3
5
6
7
8
9
11
12
15
View from south-east

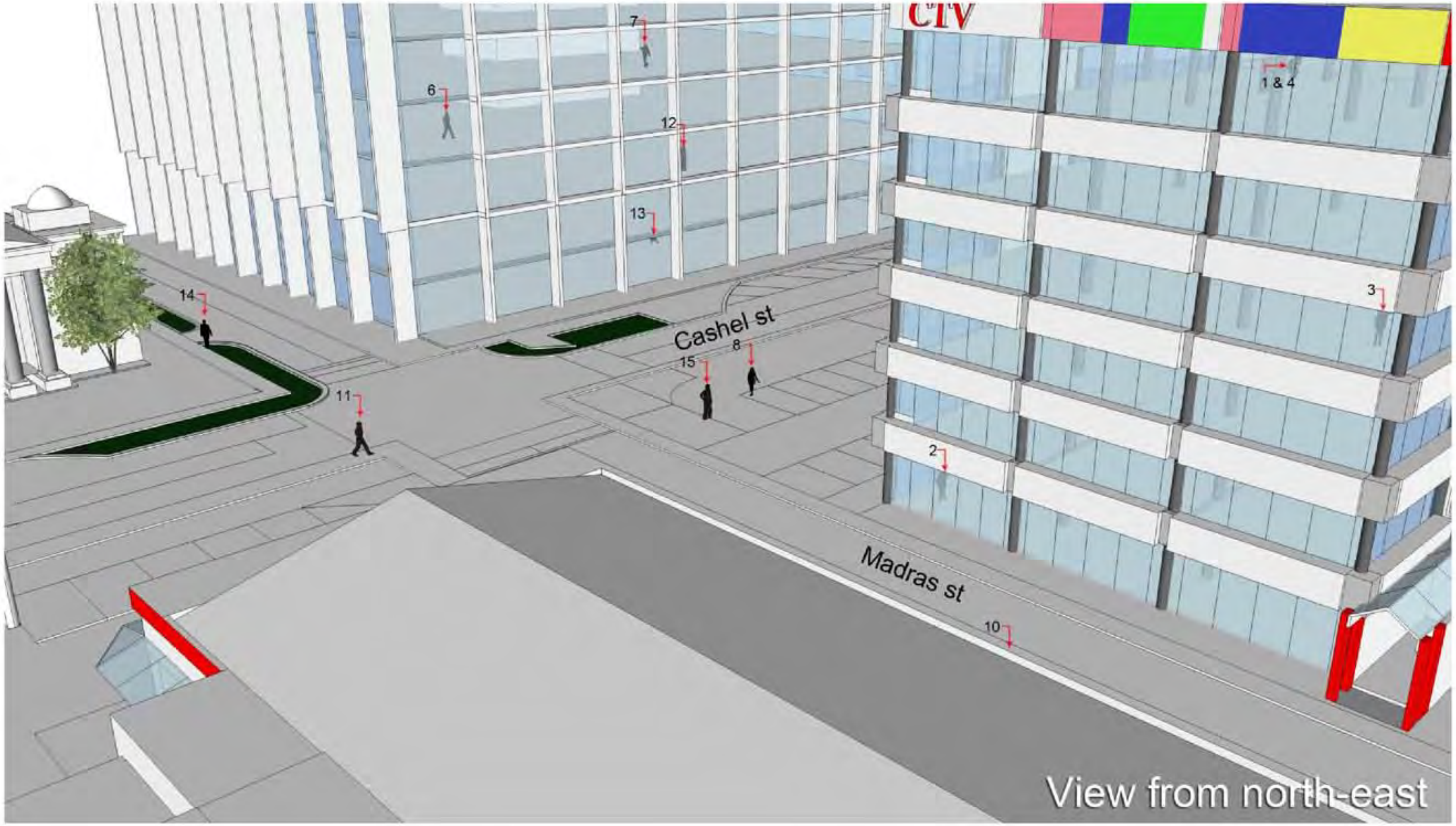

CTV
1 & 4
6
7
12
13
14
3
Cashel st
8
15
11
2
Madras st
10
View from north-east

Figure 58 - Eyewitnesses located on perspective views around the CTV Building.

INTERVIEW SUMMARIES

Eyewitness 1

Eyewitness 1 was in the eastern side of a room on the southwest corner of Level 6 at the time of the earthquake. (See Figure 57 and Figure 59)

Figure 59 - Eyewitness 1 and 4 locations in southwest corner room on Level 6.

She described the quake as a sudden violent lurch – a continuous movement. "Then the building just went joo-joo-joo-joo, and just did not stop. I just felt like we'd gone really far forward and then just kept moving all the time continuously." This she described as not "after" a first jolt, but it being the whole jolt. When it was over, she was on the floor and the ceiling was on her "so what part would have come down first? It would have to be below us – as we just "came down", like floating down. "The whole ceiling collapsed in on us and most of us; in fact all of us I think were pinned to the floor."

• *Direction of fall.*

Where she was there was not a sensation of the floor falling down, more a "sense" of tipping. After the lurch she was "pushed back a little", "A feeling like I was moving in that (east) direction – and then there were just lots of movement, and during that movement the glass exploded on the Cashel Street side. People and furniture also slid towards the eastern wall. One of her colleagues also felt that the whole building was tipping over but she commented that he was standing and she remained seated and felt it differently. For her it was more a sensation of continuous movement and slight tipping.

• *Time frame.*

She said "I am being generous in saying the building was down in less than 30 seconds. Some of my colleagues say it was much quicker than that."

• *Pre-earthquake observations about the building.*

Continual vibration during next door demolition. This eyewitness commented on the demolition that had been going on next door, since September 4th. Some staff had found the continual vibration in the building distressing, particularly in light of nerves around the aftershocks. She referred to a huge vibration on the day when the last part of the demolition occurred. "One day there must have been a wall that either backed on or was semi attached to the back of our building – when that came down a huge vibration went right through the building." She commented that when the demolition ended she returned from the Christmas holidays thinking the vibrations would end. However, the building still continued to vibrate from "the machinery or whatever was going on next door."

Cracks in the lift area. This eyewitness reported what she described as major cracking in the corners by the elevator. "It was cracked from the ceiling all the way down to the floor. This was on the Hereford Street side of the building, at the intersection of the walls.

Eyewitness 2

Eyewitness 2 was on Level One, Reception – running out south from the front door (east side) straight across Madras Street towards Blackwell Motors.

She described the noise and impact of the quake as like a jet plane landing on the roof. "The whole, all the glass, everything was going. The noise was unbelievable. I ran for my life thinking the building was going to get me on the way. I knew it was breaking up. I ran for the doors, everything was coming at me; you know all the windows coming in. I just got through the door. There was no one else on the ground floor at the time… all our other staff were on the first floor and they did not stand a chance. I knew I was the only one that got out, because I knew what was coming down around my ears as I was running."

• *Direction of fall.*

When this eyewitness turned around she was on the corner of Madras and Cashel. She did not actually see the building fall; by the time she got there the building was down. "The building had just pancaked – six floors were down to next to nothing." Inside it had felt "like being pushed around all over the place".

• *What the ground shaking felt like.*

During the aftershocks, when she had made it to Latimer Square, she described the ground as like "jelly". The road was "going up and down… horrific."

• *Time frame.*

"Fortunately I was standing by my desk when it happened. I would not have had time to get up from my chair. By the time I ran across the road really fast and turned around, the building was down; a matter of seconds really. Then, there was another big aftershock and a whole wall of the Samoan Church collapsed over into Madras Street."

• *Pre-earthquake observations about the building.*

The eyewitness commented on the drilling that had been going on inside the building before the earthquake. Every now and again we would get a boom-boom and a shake …no one felt safe in that building. They had already taken a building down next door so I don't know why they were drilling into the side of the building."

The eyewitness also described how in an earlier small earthquake the girls up in the sales office were shaken about it – yet she did not feel it on the ground floor.

She also remarked that she could not remember what the inside staircase (which was right beside her desk) was doing during the earthquake. However she remembered that in a previous 5 earthquake it was like "the whole thing (staircase) was swinging towards me." She could not recall seeing any damage from the two earthquakes before 22 February.

Eyewitness 3

<u>Eyewitness 3 was on Level Four – north on the right edge of the building.</u>

She described her first experience of the earthquake as, a bounce – a jump and then everything moving. She refers also to a second sensation of a definite drop." The analogy I've used in describing how it felt, is like being on an ice rink in flat shoes. Completely just spun from one side of the wall to the other. Then you realise that it wasn't just going to shake, and it wasn't going to stop." She remembers moving towards the underneath of her desk. Then everything went black, everything sort of stopped. The sensation of dust, not being able to breathe. ….the weird sensation that you weren't level, on a slope. I put my hand in the air and realised that the ceiling was actually resting on the top of my desk. Then there was a second movement – a definite downward movement, it went like "choooomf – like on a seat when you drop. She could not be 100% sure of the movement between the first drop and this, as it was already moving….. everything seemed to be dropping constantly – very disorientating – but the second drop feeling was a definite. You suddenly thought "whoa!"and things went downward more."

• *Direction of fall.*

Initially this eyewitness was thrown one way and back again. Against the eastern wall and then thrown back to the west and back east again against her desk. She then got under her desk. "The first initial shake was when it went" – then a feeling of what she called a second drop that where she felt like she was on a "slope". She said "I was pushing with my heels, you felt like you were pushing up hill." First when she was under her desk she had room, but in the second stage "I was sort of on my side."

• *Time frame.*

"It seemed like a long time." But she felt unsure of time – "to be honest, time just – it was very bizarre."

• *Pre-earthquake observations about the building.*

"They demolished a building here behind us – starting pretty much when I started work in the October. "All I do know is we bounced constantly while the digger work was going on. They finished a week before the quake."

Her understanding was that when they took the building down next to the CTV they left a single layer of brick with no bracing. The building that came down only went up to level 3, below level 4.

Eyewitness 4

Eyewitness 4 was inside the CTV against a side wall on Level 6 that comes out to Cashel Street just in front of the IRD (Inland Revenue) building. The side next to where the demolition work was. (Figure 59)

"Usually our meeting would have been in the middle of our premises – but on this particular day we were sitting furthest away from the front area. This decision pretty much saved our lives. I was strategically in a good place because I had no obstruction to access to a door frame. We all eventually came out in the car park. I just felt this "chooo" (vertical feel) a bolt, a "thump" that almost propelled me off my seat. I was like a rocket under the door frame, my colleague and I together as we had rehearsed many times before when the demolition work was really bad. I held on to this flimsy little aluminium doorframe. I was standing up and felt a real sharp jolt from underneath."

• *Direction of fall.*

"I felt a bolt upwards at first, and then it started going sideways. Initially it was really strong with the bolt underneath, like this was very, very fast, real fast, up and down, and then it was swaying, and then it all collapsed, collapsed, collapsed." It started with the usual thump of an aftershock and then accelerated from there. "So there was a thump and I was already under the door, others were still sitting." She felt that she was in line with the doorway as it fell, not sort of falling out of it. "There was a real lion kind of noise, roaring – like cracking. One thing I noticed very quickly was the pink batts coming down on us, so the ceiling must have given pretty soon. The pink batts were the only thing that fell on me. Whatever was collapsing like the other walls caving in, they were just kind of collapsing and nothing really fell on me because everything fell against the frame. Then I remember a little bit of tilting (not steep) to the back from the ground (toward Cashel Street.) It was not much; it wasn't like I had to hold on. I was still standing when we were down 5 floors. I did not have the sensation of freefalling. When it came to a halt I thought we had just come down one floor. When I looked through the open ceiling out I thought I was still high up – then realised oh my God, we're just a metre off the ground…. I was totally surprised that the floor on my side was still in one piece. Nothing had come through." The partition wall she was up against, on the east side of her meeting room side stayed vertical all the way along. On the southwest corner of the floor were the worst injuries. When referring to the tilting of the building – she described it as a slight diagonal lean towards Cashel Street corner demolition site. "My sense is that when the whole building went up and sideways and just went "shhhhoooo" down, leaving the lift shaft still standing. Being in the top of the building where I was saved me. So much more damage happened in the middle part of the building."

• *Time frame.*

"I could not see anything, you know, because the whole walls caved in and – like it was all blocked within seconds, seconds. It was amazing how quickly people stepped into the rubble and got us out, and then the fire broke out in the lift or lift shafts."

• *Pre-earthquake observations about the building.*

This eyewitness mentioned a fear amongst some colleagues that the demolition work was perhaps weakening the building. It was her feeling that it was undermining the building. "This is only my sense, it is not a science."

There used to be two big building complexes next door, and the one adjoining the CTV Building was taken away. Around two weeks before the earthquake they had just freed the area of the building.

"I was right on the outside (of her floor), and when the demolition happened the big diggers, whatever you call them, were pulling that wall. It made a shudder. I don't know for sure – but when they took the building next to us down, I believe it had at least some parts attached to our building." (Lower than her level.)

She described the demolition going on from September to February. On the day of the earthquake they were still coming in with big machinery, flattening it to turn it into car park. "There were constantly machines, and stuff coming down and falling down. Big huge chunks of concrete were just falling to the ground. You could feel it all the time…. Then there were the aftershocks as well. They were horrible as the whole building was just going big sway, big sway."

My sense was "my God, this building is constantly exposed to quite a lot of stress… I thought we're not safe in here…it's not okay, part of it."

She also mentioned that even before the demolition of the building and before the earthquakes when aerobics classes were happening at Les Mills "our building was vibrating." "The outside wall was never very thick I felt."

When asked if she noticed any damage in the building getting worse subsequent to September – she made this comment. "Right at the lift shaft, these big pillars. I noticed like a bigger crack around, I think, the pillar closest to the lift. There was another one – the pillar was intact, but just alongside there was a crack (she moves her hand in an S shape) which just went down." She had hoped when she saw them, that they were just superficial.

Eyewitness 5

Eyewitness 5 was in the Les Mills building next to the CTV Building on the 3rd level (2nd level of the Les Mills gym)

"I was directly opposite (just 10 metres away from) a large window that you could see the CTV Building through. When the earthquake struck I remember turning around and then seeing the CTV come down through the window. I could not see the top of the CTV Building. I saw a portion of it then it all came down. I don't think I could see from edge to edge, but I saw a lot of it…."

- *Direction of fall.*

"I saw the collapse. It was just almost like a level gave way and it just went - whooompf. It was like one of those controlled demos on TV. It was just straight down – and then after when I was down at the site helping out (and as you can see from the TV images) it was really compact, the rubble and that…" The eyewitness found it hard to describe the feeling that its almost like a level was removed and it just all came down. He did not actually see a level collapse – it was just the way it all went down.

- *Time frame.*

"It just fell really quickly. Like ploooop. A couple of seconds. I was on the heavy bags facing away from the window maybe seven, 10 seconds passed as I stabilised myself. I turned around and then another few seconds, then saw the CTV Building come down. The first thing I saw was it coming down." The eyewitness was definite that the CTV was down during that first aftershock, the first tremor. A big aftershock happened minutes after when he was outside Les Mills, and he saw the scaffolding on the Samoan Church come down.

- *Post–earthquake observations at the site.*

He was standing at the front, Cashel Street side. "Everything was just so compact. I remember I just could not believe it was a five-storey building. It was just so tight, the pile, real compact. It was deep down I think the fire. I think it must have caught like this – there were pink batts around, so it must have caught onto that. It was real smoky because the corrugated iron was on top of it. When the digger pulled back some corrugated iron, you did see flames come up."

"Part of the building was still standing. I remember the CTV sign was down." On the Les Mills side, he also remembered seeing the pink batts, and corrugated iron type stuff, sheeting, along the wall. "There were tons of massive puddles, craters with puddles in the graded part between Les Mills and the CTV. There was also a crack in the street where water was flowing out."

Eyewitness 6

Eyewitness 6 was in the IRD (Inland Revenue) building on the third floor as the earthquake hit. (Figure 60).

Figure 60 - Perspective of Eyewitness 6 in IRD building

"I was standing looking out the window at the time that it collapsed so I could see the top half of the building. It started to collapse a few seconds into the quake and what I could see was the top started leaning towards the east, and then basically it just collapsed straight down."

- *Direction of the fall.*

"It was just a slight lean, and it went down vertically. Then we had white dust come up so all we could see for a few seconds was white dust against the windows. Then the Samoan Church opposite us fell down." The third floor of the IRD was the fourth level, so he could see at least the three top levels of the CTV Building. He had no recollection of floors falling into other levels and said "it almost looked like it came down in one piece. It looked like there was something coming up which may have been dust. I was focusing on the top of the building and that, from what I can see, it was going down as a unit."

He pointed out that there seemed to be nothing breaking at the time, but said "I cannot swear to that.... It just looked like something happened below and it was coming down. I did not see anything disintegrating in my field of vision, so whatever was happening was happening further down." Then there was the white out – he could not see anything through the windows at that point. Before the white out, he also recalls a momentary dark flash – but could not tell what it was. "Whether it was smoke or dust or lower floors breaking up, I could not tell. That was only momentary."

• *Time frame.*

The time that this eyewitness felt the first ground movement to the time when he saw the CTV Building collapse was described as seconds. "It would have been a few seconds, but time's pretty elastic in those sorts of things. It probably seemed longer than it was, but it was a few seconds.

• *Post- earthquake observations of the site.*

His observations of the site were few, as he was concentrating on making sure his colleagues were safe, and getting to Latimer Square.

• *Pre-earthquake observations about the building.*

He noted that in the preceding weeks there had been a lot of vibrations from the building that was being demolished next door to the CTV Building. First "when they were knocking down a wall, but I think probably even worse when they were breaking up concrete that was set in the ground and they were using a wrecking ball." He described them as being "like point three earthquakes or something like that – we weren't feeling them, but we were feeling the shocks from the wrecking ball. We'd get vibrations in our building quite often. They were breaking up the concrete approximately one week before the quake, and it was going on for two or three days."

Eyewitness 7

Eyewitness 7 was in the IRD (Inland Revenue) building looking out the window on level five. (Figure 61)

Figure 61 - Perspective of Eyewitness 7 from IRD building.

During the earthquake this eyewitness was under his desk as much as he could be. He was situated right next to the window, and had a full view of the CTV Building. "I could see out to the empty lot and down the side of the building, and I could see the car yard, and yeah, the edge of the entrance way to the CTV reception. I could see the whole lot." He was looking out the window as the CTV Building came down. "I've been in the IRD building plenty of times during aftershocks and I'd never really gone under my desk, I had not felt the need to. But this was quite different. It was super violent, so I was under my desk immediately and it got more and more violent. It was not just a shake. It just kept going with intensity and I was being bounced out of my desk and back again. I don't know how long it went on for but it just stopped, suddenly. There was quite a bit of noise in the office and people upset and so I stood up to call my team together and then looked out the window, and then the CTV Building came down."

• *Direction of fall.*

"A flash of the CTV Building and then it sunk into the ground – you know like the 9/11 buildings, exactly like that. The top floated and was engulfed by a cloud. I probably wasn't even aware that the building had collapsed because it looked like it was engulfed by dust - I realised because we were still in the building for about two or three minutes afterwards that suddenly it was gone. You could just see the lift well…..I don't know how it just sheared off that." His overall impression was that it disappeared – "like sinking into this cloud," "Pretty much as a block."

• *Time frame.*

This eyewitness described seeing the building fall after the quake had stopped, after he stood up from his desk. He felt it did not fall immediately. It was after the earthquake. However he mentions in the same segment of interview that "I've lost some moments in time."

• *Post-earthquake observations about the site.*

The eyewitness remembers the lift well standing, and people helping to lift rubble off with some digging machinery that was on the site. "I ended up over at the lift well at some point where I was fighting a fire. I remember getting to the CTV Building and then suddenly I was on top of the building, so how I got there I do not know, but I was helping get people out for about seven hours or so. "

"I had the expectation that it (the building) would be all over the show. But It fell into a complete square. I mean essentially a seven-storey building had compacted into something that was less than the height of this floor to ceiling." (Referring to the interview room.) Also, "as they were pulling people out, the majority of them were from the 5th floor and they had no idea that the building had collapsed. It was a real shock for them to feel 'how am I on top of this building?' "

•*Pre-earthquake observations about the building.*

"The building next door to the CTV had been damaged in the September earthquake and they were pulling that down. I heard that they were going to turn the land into a car park so they were making it quite flat. So there was a lot of heavy drilling and a lot of demolition ball stuff going on, and often we would be in the IRD building and it would feel like there were tremors – so it it was shaking the IRD building I can only imagine it was having a similar effect on the CTV and the buildings around it because it really felt like the ground was shaking with the work they were doing there."

He recalled no damage to the building being evident after the September and Boxing Day quakes.

Eyewitness 8

Eyewitness 8 was crossing the road to return to the CTV from lunch. As it happened she was standing in front of the CTV driveway. (Figure 62)

Figure 62 - Perspective of Eyewitness 8 in front of CTV Building on Cashel Street.

This eyewitness was halfway over the road with a colleague (in front of the CTV driveway) when it happened. "There was this massive jolt and we grabbed each other. We were looking around us as it started shaking really really bad – and the CTV Building was just sort of swaying back and forward, and the IRD (Inland Revenue) building too – we were looking back and forwards going " which one's going to go" because the IRD windows were coming out just like jelly.... All of a sudden, I think it was the fourth floor (level 5) of the CTV Building just gave way. .."

- *Direction of fall.*

"…The pillars,- like all the glass shattered, and then just - I think the pillars just gave way on the outside. (Moved outwards). And then the fourth floor (what the interviewers call level 5) came down and hit the next floor down. It sort of stopped for a like half a second, and then it dropped again to the next floor down, and it just continued that way down to the ground. But the fifth floor (Level 6), I am pretty sure that stayed intact until it hit the rubble at the bottom." She raced to the rubble at this point – into the car park straight up to the front of the building. "My colleagues car was parked in the second car park across from Madras Street right in front of the building, and it had only really knocked it's bumper off. It came down that straight, it was absolutely crazy. Then I ran around the other side of the building, the back of Madras Street, yelling people's names and stuff. We found a couple of colleagues – but apart from that I do not remember much. But yeah – the fourth floor (Level 5) collapsing…."

• *Time frame.*

The collapse of the building seemed to happen in seconds. . Level 6 dropping as a unit onto Level 5, and then Level 5 onto the ones below. "It must have been the roof that sort of collapsed into level 6 when it hit the rubble."

• *Post-earthquake observations about the site.*

The eyewitness remembers the lift shaft still standing. Also when the building collapsed all the water pipes burst and there was water pouring into the pile of rubble. "Some of it's a blur."

•*Pre-earthquake observations about the building.*

This eyewitness did not bring up any comment about the building before the earthquake.

Eyewitness 9

During the earthquake this eyewitness was on top of the Les Mills checking the air conditioning units – where there is a 360 degrees view of the city. The Les Mills building is four stories so he could not quite see the roof of the CTV, but had a straight view of the collapse. Since a building had been recently knocked down, there was just open space between the CTV and the Les Mills building, so he could see the whole of the west wall. (Figure 63)

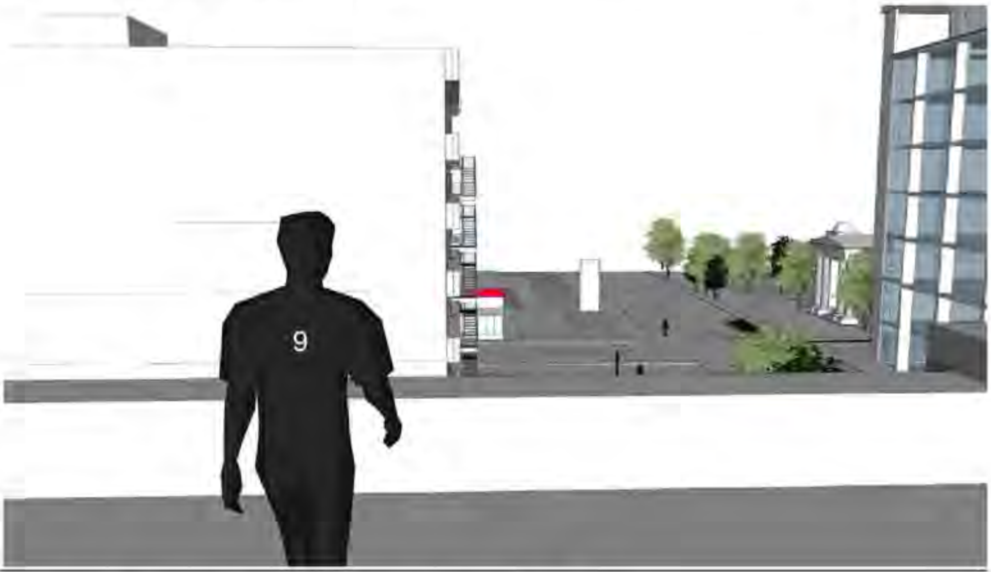


Figure 63 - Perspective of Eyewitness 9 on roof of Les Mills' gym.

• *Direction of fall.*

This eyewitness described it as very harrowing to see the building go down. At the time he was between the air con units. "It was a very vicious punch, as if someone had punched you in your back off your feet, trying to lift you up. I was bashed around a bit and turned around and I was steadying myself on the ends of the units – and I saw the building was there one minute and it just sort of crumpled before my eyes. I can see right down to base level, to ground car park level, and it just folded at the bottom like a pack of cards. The first floor folded in, the second floor followed it milliseconds later, and then it went down like somebody had kicked its legs under it. The fire escape stood up for a few seconds longer. The corner the cherry pickers were working on (south west) just crumpled like a piece of paper. It was just like it were a chair leg and someone had kicked that corner and the whole corner caved, sort of folded under itself and then the next piece. The corner had gone so there was no support."

His experience of the earthquake was one of being lifted, then dropped, then kicked again from all directions. "So if that building did lift up and that got knocked on the next wave, then there was nothing on all that frontage to hold it up. The sheer weight of it brought it down."

"The frontage of the building came away, I presume, because there was nothing attached to the lift after the third floor (Level four) upwards. It just ripped away because you could see the lift shaft, the lift doors, everything. To clarify further – when climbing on the rubble, it wasn't flat, it was at an angle. It spread itself in a line."

The image that sticks with this eyewitness the most is the Cashel Street fire exit stairwell corner disappearing in front of him, and the rest coming down. "It was weird just to see a skeleton for a few seconds; I'd say 5 seconds tops, of the fire escape standing and then sort of crumpling underneath it, because the next floor pulled it down. But it just stood there - and you think that's physically not possible – unreal."

• *Time frame.*

All this happened in seconds "whoof – boomf" and then there was one big cloud of dust and in the corner of the thing there was smoke starting to come up. "I don't think 30 seconds passed by the time it was all over the place, fire alarms, chaos. He did not get the feeling of two shocks. Just the one that went with a bang – and then the sensation of loads of aftershocks. "It is quite possible there were a lot of shocks rippling back." He saw what happened in front of him in the space of a few seconds - then his concern was people.

• *Post-earthquake observations about the site.*

This eyewitness was involved for hours on the site helping get people out. He noticed:

<u>Pieces of the fire escape stairs</u> "You could see sections of the fire escape (Cashel Street end) still left in pieces as it had fallen on to the rubble". He felt that if the building had pancaked (for him meaning 'come down as one') it would have pulled this down, and it would have been twisted metal – "but it wasn't…."

<u>The contained way it landed:</u> "It is amazing that there was not a brick or anything in the car park area, and so many cars parked close by untouched, just covered in dust. The building fell into itself, and was all contained within that area. The building came down within its own space, its own footprint. So if it had come down flat, everything would have spread out. You try and put something down that flat, something's got to go left or right, but it didn't. But as I say, with it coming in on an angle, it was all

still stuck underneath, so it had somewhere to hold, and it just sat on itself – like a big pile of bricks."

The Fire: He did not expect the building to burn as he thought "it's all concrete – it's not going to burn." A big machine had been left there which he used to get the fire people into the lift area with breathing apparatus. The smoke was black and acrid by that time.

Liquefaction: "There was a lot of liquefaction, not around the CTV itself, but where the knocked down building used to be there was a great big hole opened up – and water bubbling up. The liquefaction was all along the front of the Les Mills building, it was pouring in the front door."

Samoan Church coming down: The eyewitness was already helping on the site when the Samoan Church went down.

•*Pre-earthquake observations about the building.*

"The building that was demolished between Les Mills and the CTV was finished on the Friday before the quake. I don't think that would have weakened the structure much. Cherry pickers were in, lads with battens, and they were battening the wall all the way up to put new cladding up the wall. White cladding which was nice. They were doing a good job and they were doing it safely. There were two lads on the end platform that day, and had gone away for dinner. The new cladding crushed it flat."

Eyewitnesses 10 & 11

When the earthquake happened these two colleagues from Blackwell Motors were on the Madras Street side of the CTV Building. Eyewitness 10 was getting into his car parked opposite the CTV and outside Blackwell Motors, Eyewitness 11 was in the Blackwell Motors building. Eyewitness 10 saw from the ground of the building (Figure 64), Eyewitness 11 was looking to the skyline (Figure 65)

Eyewitness 10 had just got into his car parked directly opposite the CTV Building outside his work premises. "I basically shut the door and when the earthquake hit, I actually thought that someone had hit my car initially. Then I realised 'well no' as I watched the parapet from Blackwell Motors fall down beside me. Realising I was not in a good spot, I leapt out of the car to run, then as I turned around to shut the door, I looked across the road and realised 'that building (CTV) is going to come down.' I jumped back in the car again and remember thinking 'this isn't going to be good' then got out of the car again. Then I looked across – it went and it seemed to drop in the reception corner. So it seemed to drop and almost fall around in on itself. Then I remember a couple of our staff (including Eyewitness 11) there disappearing off into the rubble."

Figure 64 - Perspective of Eyewitness 10 from Madras Street.

Eyewitness 11 ran out of the Blackwood Motors building at the time of the quake. When he was in the yard he remembers seeing the Samoan Church fall apart, it was opposite him on diagonal. To him the quake seemed to go 'boom boom boom' for quite a while. He remembers "there was an aftershock and a dome fell off the top of the church right, and landed in Madras Street. That's when I heard this screaming around the side, and I started running around (wiping the dust out of my eyes by this stage) to see what the screaming was. It was a lady in the middle of the road, and as I came around the corner.. I looked up and I could see the top of the building coming down, and it seemed like it was falling away from the lift tower…but I could only see the top of it. I didn't look at the bottom. "

Figure 65 - Perspective of Eyewitness 11 from corner of Cashel and Madras Streets.

• *Direction of fall.*

According to Eyewitness 10 there was definitely a movement downwards. "I'm pretty confident in saying it was the reception corner. (Madras /Cashel end). From my point of view that bottom corner went first, and then it kind of wrapped itself around and down. So when I was sitting in the car it went 'bang' that way (a bit to the east) and almost fell, sort of down then over. Then obviously as it went down, the rest of it went whooooo through.

Eyewitness 11 saw the skyline of the building coming down and it looked like the whole lot was just" booomf boomf boomf" down. He did not see the bottom.

Although Eyewitness 11 felt like it was kind of a blur in some ways – both eyewitnesses felt that the building did not fall down straight away, but in an aftershock. Eyewitness 10, unlike Eyewitness 11 saw the building from the ground – and it seemed to him like it dropped in the reception corner.

• *Time frame.*

These eyewitnesses seemed to experience time differently. For Eyewitness 10 things went very quickly while he was watching the building, for Eyewitness 11 – there was a sense of slow motion. "I looked and I thought – that buildings coming down – and then it was just coming down slowly. Yeah well it seemed like it was coming down slowly. It probably wasn't but it seemed slow at the time. There was so much going on – I don't know if I actually saw that or I actually imagined that. I think something had obviously come down before I got around the corner."

• *Other observations.*

One of their colleagues, standing in the yard – could see waves, the waves coming from the ground in the tar seal; Coming across from the hill, south to north coming across the car park. Eyewitness 10 also remarked that other than being dusty, his car was unmarked, no damage despite being so close to the collapse. Eyewitness 11 also commented on the swaying of the IRD building during the earthquake.

Eyewitness 12 & 13

When the earthquake happened these two people were in the IRD (Inland Revenue) building. Eyewitness 12 was on the 2nd floor (Level three) of the IRD right against the window with an unobstructed view over Cashel Street (Figure 66). Eyewitness 13 was on the ground floor (Figure 67) . She could not quite see to the top – but could see the width of the fall of the building's western wall .(To the side of the CTV rather than right in front of it.)

Eyewitness 12 "It happened so quickly, but the impression I had was that the wall facing the IRD building, the Cashel Street wall, seemed to fall away first – come towards the IRD building, then very quickly afterwards the rest of the floors just seemed to pancake or concertina down towards the same general direction. Leaving just the back wall that seemed to just be standing there with parts of the floor at various stages." His impression was that when the front of the building came towards him, it was actually coming off. "Not the building, just the front piece, and then everything cascaded behind it… It did not actually spill too far towards IRD, it just virtually went straight down I think. The front must have collapsed because I mean I saw floors up behind it, so that suggested to me that the front had just fallen off. And then I saw the rest of the floors. I could see the top level because it had sort of pancaked out. I could see people on the top level sort of trying to get off it – I do not know how high it was, 5 levels? (There are 5 suspended levels). It probably went down to about two levels in height I guess in real terms, and I remember seeing the back wall. There was a floor. It must have been about the third or fourth floor. (Madras Street side.) There was part of the floor (a piece) still sort of sitting off the back wall but without any supports, suspended, no column underneath it. There was a woman up there, alive, yelling."

Figure 66 - Perspective of Eyewitness 12 from the IRD Building.

Eyewitness 13

"I was on the ground floor of the IRD building and our natural way out is through the glass doors, so we were all sort of facing the glass opening doors onto Cashel Street. We saw the CTV come in on itself and then drop like a river. By "coming in on itself" she meant "the building did not fall away going out, it sort of went in on itself and crumbled. Came in and just dropped." She described it as coming in from the top, and then "it just sort of came down, like all the rubble and stuff just flowed down and blew a whole lot of white dust. We couldn't see anything after because it just completely went white."

Figure 67 - Perspective of Eyewitness 13 from the IRD Building.

• *Direction of fall.*

Eyewitness 12 was higher up and right against a window, whilst Eyewitness 13 was on the ground floor, near the lifts, so comparatively quite a distance from the front of the IR building. (Around 10 metres back.) Eyewitness 13 saw no movement out from the CTV, and Eyewitness 12 felt that the front had come out towards their building. Both had the impression of it going down very quickly, not spilling out.

Eyewitness 13 was amazed that it collapsed in. "Like you expect it to go out, but it actually came in – and then just fell really hard. Our building was still rocking when that thing was flat." She also commented about where it went in – "to me it was like two-thirds up and it just went in, and just went down. I remember saying to people – it went in on itself, how does it go in on itself?!"

• *Time frame.*

According to Eyewitness 12 on the third level, "the whole thing happened in 20-odd seconds that earthquake. In the time that our building was rocking the whole building collapsed, so it was really really rapid. Down after the first hit. I watched it for 15 – 20 seconds and we were gone, going down to Latimer Square." Eyewitness 13 also found it very rapid.

• *Other observations.*

Eyewitness 13 considered Eyewitness 12's description of the front of the building falling off. "Maybe what he said explains why we got that huge cloud of whiteness? Totally white. Nobody on the ground floor went out those front doors; we all went through the back."

Eyewitness 14

Eyewitness 14 was in his lunch break at Coffee Supreme at 218A Madras Street, which is just south of the intersection with Cashel Street. When the earthquake hit, he was walking up to the intersection on the east side, just coming up past the Samoan Church, with the IRD building on his left. (20–30 metres from the intersection with an unobstructed view of the CTV.) (Figure 68)

Figure 68 - Perspective of Eyewitness 14 from Madras Street.

"I saw people at the intersection kind of hanging on. You could hear rumbling. I heard scaffolding collapsing on the Church. Realising it was serious, I tried to grab one of the parking poles…I froze and looked up north towards the CTV Building. You could see it all shaking, pretty much around the intersection. Twisting back and forth….then the external cladding…glass shatter and everything, it was just falling off. The floors were just sort of collapsing pretty much from this corner (the southeast corner) working its way back. I could see pillars coming out. It pretty much collapsed from the back, like somebody smashing a wedding cake and a deck of cards. A total catastrophic collapse; shocking to see."

• *Direction of fall.*

Eyewitness 14 recalled the upper columns going initially but it was disintegrating at all levels. Some of the pillars fell out to the side, others fell in the direction of the twisting. (He had a strong recollection of the twisting back and forth along Cashel and back up Madras.) "It was pretty much the outsides falling and behind that the whole building was just falling to bits…. I was probably seeing columns coming out from maybe two or three floors up, but you could see the whole building collapsing in on itself."

• *Time frame.*

"It just collapsed in seconds from the first quake. "There was an aftershock 10 minutes later, but the CTV was a complete pile of rubble after the first shake. There was nothing but the lift shaft still standing." He estimated that it took about 3 – 4 seconds to sense the earthquake happening, then 4 – 5 seconds to grab a parking pole. Then he saw it go 2 – 3 seconds after that.

Eyewitness 15

Eyewitness 15 had gone for lunch from the CTV and was just returning, crossing the road on Cashel Street, just in front of the IRD (Inland Revenue) Building. (Figure 69)

Figure 69 - Perspective of Eyewitness 15 from Cashel Street.

"We (Eyewitness 8 and 15) were in the middle of the road when it happened. It was shaking quite violently and then I looked up at the CTV Building. It was standing at this point, and what we saw was the fourth floor (level 5) collapsed first, so it sort of pancaked down which in turn pancaked the rest of the building down, and then the top floor broke on impact when it hit the ground…..the sound was probably the most horrific thing, everything just sort of crumbling in on each other."

• *Direction of fall*

According to what Eyewitness 15 saw, not the top floor but the next floor down was the one that broke first…"dropping into the slab of level 5 which was still intact until it hit the ground. That whole building collapsed apart from the lift shaft. It fell so straight down, that it only knocked the bumper off my car literally parked right at the front door."

• *Time frame.*

"The earthquake had been shaking violently for about 5 seconds – then the fifth level gave way to the rest of it - and I would say the whole thing was down within say 12 seconds. Only 5 seconds warning."

• *Pre-earthquake observations about the building.*

In the corners of the wall on the second level, on the side where the building had just been knocked down, he had noticed tiny gaps in the brickwork where light was coming through. He said that the day before they had been drilling wooden planks into the side of the building, perhaps the previous week as well. "It wasn't high up, it was the first two levels I think. They were 5 or 6 metres long bits. They weren't covering the whole side of the wall. They had a wrecking ball out the day before also."

Eyewitness 16

Eyewitness 16 was working on the CTV Building at the time the earthquake hit. He was facing the building towards Madras on the corner with a view of the corner column on the Cashel Street edge, out the front.) He and his workmate were hard up against the building (Figure 70).

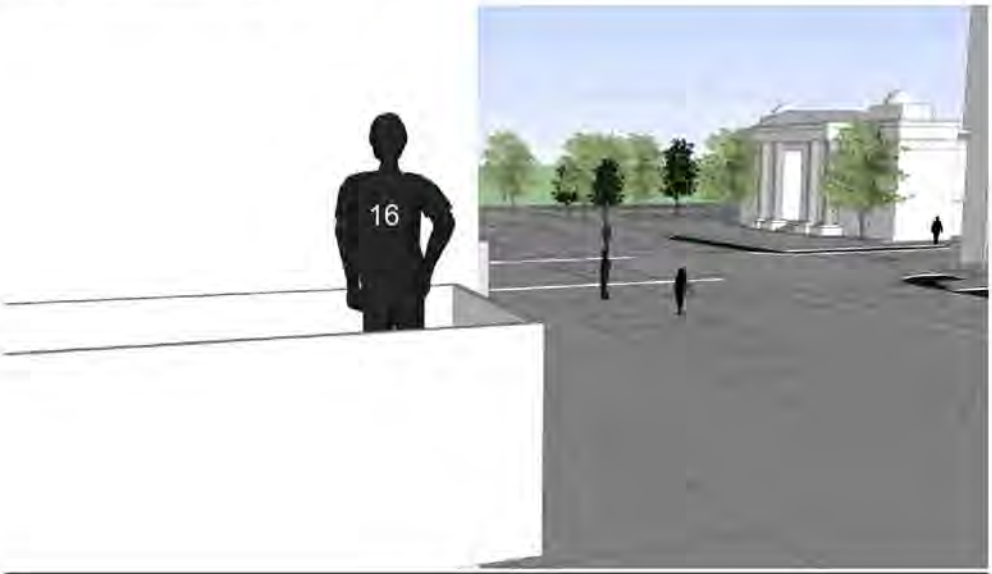


Figure 70 - Perspective of Eyewitness 16 from the elevated work platform at the southeast corner of the CTV Building.

He and his workmate were wall cladding the CTV Building, making it watertight from where the last building had come off it when it was demolished. "We had been working on it for two days... we'd taken the scissor lift about three metres up …when I turned to my workmate to ask him for a drill to drill out the rivet and then everything started shaking away."

• *Direction of fall.*

"It all seemed to jump upwards". He felt a vertical jolt around about a movement of 200mm. I remember looking up and seeing the building pretty much right above my head, so it had obviously swayed from side to side. I threw my workmate off the machine and as I was jumping I had to push myself out of the way of the falling corner pillar. (Southwest) Just out of the corner of my eye I saw the concrete spit out the corner. The pillar came down and brought the machine down to the ground and buried the wheels. It felt like the building moved in the front."

He described seeing the column fracture. "It buckled out. It had cracked and the two bits held still by the steel had spat out, and obviously as the weight got too much, it broke and came down. This was in the middle of the column, between floors. It 'kicked out' in the direction of Les Mills. I remember I was still looking at the corner of the building at that time - it looked like the like the block in front of me came up and back down again. I turned away to the right to throw my workmate off the end of the machine, then I turned back to make sure nothing else was coming and that is when I saw the corner – sticking out around 300mm. It let go – and came down when I was jumping out."

In summary, this Eyewitness was at level 2, and saw it breaking up between level 3 and level 4 columns at the front (southwest) corner. He felt what had happened to the building was like this: "The bottom couple of floors had come out, and the rest of it had come straight down."

• *Time frame.*

All this happened in seconds. He himself was seconds from disaster – saved most likely from his scissor lift holding the debris off when he was sitting beside it.

• *Observations after the quake.*

"The thing that made this side look worst was because it had the security stairway on the outside of the building going up – the emergency exit. That was down, and because there were cars and all sorts there, it made it look like there was lots of debris here, but you could actually physically get to the bottom of the building when we were getting people out."

• *Pre-earthquake observations about the building.*

- Prior to the earthquake, Eyewitness 16, was concerned that people should not have been in it when they were working with the wrecking ball. He noticed that the building was making weird noises.

- He had been working up and down that wall. At about the third level the iron stopped. "Obviously the building that was beside it before had a flashing that went up behind the iron and then it had the rest of the building – but because that had been taken down, all of this was just concrete block façade. We were going to be tucking the iron underneath that. We'd placed 50 x 50 mm timber battens along there and were dyna-bolting about every 400 mm. The wood was so that we had something to screw the iron to instead of having to dyna-bolt every point and put plugs in them. They were 10mm dyna-bolts, and some were 40mm, and 90mm for the random hollow bricks where obviously the grout had not come all the way through. We were using the smaller bolts, just so it was grabbing and the iron wasn't going to come off. The battens went right along horizontally."

- He had not been involved in pulling down the wall that was from the old building away from the CTV. All of that was done, and cleared off the site, before his work started. "It was basically just a work site that had chip stone in it, and it must have been the ground foundations they were working on at the front of the section… pulling out big chunks of concrete that were still left in the ground. We watched them smashing with the big wreckers at lunchtime on the day before. They were doing all sorts of banging on the ground with a digger. It had a big like T-bar that went on the end packing down what they had taken out the day before. They took the pile of concrete debris away and poured crusher steel, or whatever it is, to fill the holes in and used the big arm to pack it down."

We asked some additional questions.

- *"When you were putting the battens on, when you looked at that block work, did it look like it had anything fixed to it in the past?"*

"No. It was roughly mortared as if it was the internal of a brick wall. The building side had obviously been there first. They'd put in the columns and put the bricks on the internal side, because we had to scrape the whole wall off with all the excess mortar that was hanging out of the joints so the battens would sit on it flat."

- *"Any wires sticking out or any sort of tie-backs?"*

"No the only things that you really noticed was that all of these columns were out probably 20mm proud of all these internal block walls."

"..Across the top, underneath each of those beams, you say there were some hollow blocks, but on top was there a gap?"

"No-ah, a couple of floors had gaps actually. I couldn't tell you offhand which ones they were…"

- *"If you looked at the columns and saw the block work, did you see any gap between block work and the column?"*

"No. It was all mortared."

- *"Are you sure it was mortar and not a flexible sealant?"*

"It looked like mortar because we scraped it all. That wall went to the beginning of level 4. It had wall cladding all the way along there. Three levels of block work."

- *"Can you remember what the shape of the column was?"*

"They were square with squared corners. It wasn't like the days now of precast. It looked like it had been boxed up where you could see the joins where the concrete had come out of the edges – as if it were boxed in with wood. There might have been slight gaps. You could see the inch sort of lines in the concrete where the joins were. Just a mould they'd made. That was one thing I did not think I was going to see."

APPENDIX B – PHOTOS OF COLLAPSED BUILDING

B.1 IMMEDIATELY AFTER COLLAPSE

The state of the structure immediately after collapse has been derived from photos supplied by the public and others. Debris began to be moved very shortly after the collapse by heavy machinery that was next door to the building at the time.

West Wall (Line A)

Figure 71 - West side of building with North Core partially obscured by smoke, prior to heavy machinery removing debris. No liquefaction evident.

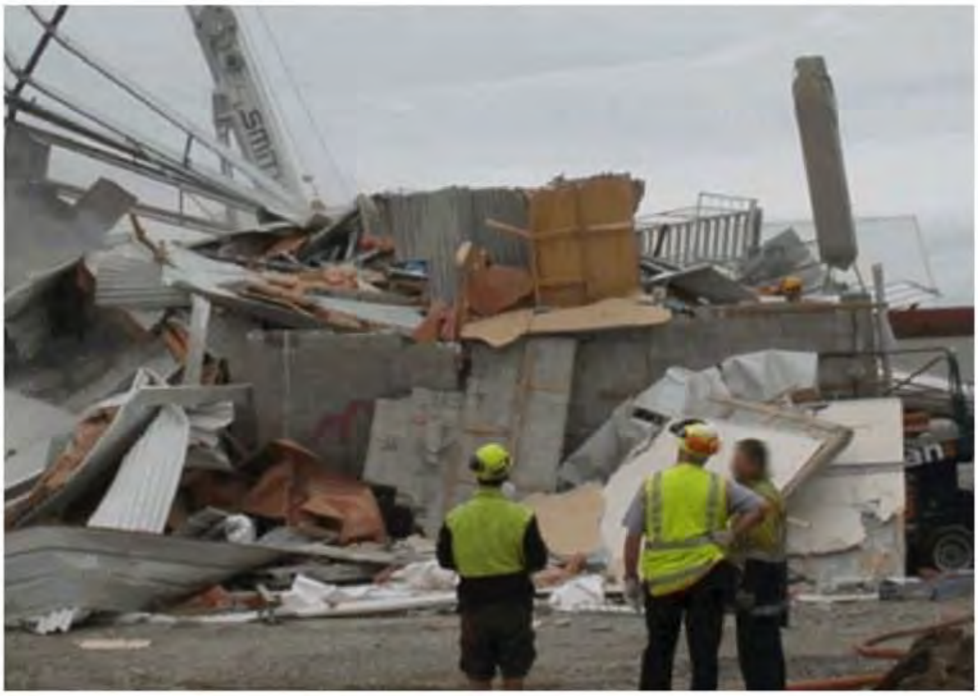


Figure 72 - Southwest corner (Grid A/1) with corner column still standing. Collapsed work platform under wall panels can be seen at the right on which Eyewitness 16 was working.

Cashel St (South, Line 1)

Figure 73 - Cashel St. face with North Core tower in background prior to fire starting.

Figure 74 - Western end of south face (Line 1). Collapsed Line shear wall with escape stair to the right.

Figure 75 – View southeast corner of the CTV Building looking northwest. The South Wall collapsed onto the top of the debris from Level 2 can be seen, identifiable by the white fire escape stair still attached to it.

Figure 76 Madras St with precast Spandrel Panels fallen onto cars.

Madras Street (East, Line F)

Figure 77 - Corner of Cashel and Madras Streets looking towards North Core. Fractured columns and fallen Spandrel Panels are visible (MSN photo).

Figure 78 – View from southeast corner of CTV Building along Madras Street. This shows Line F Spandrel Panels fallen onto cars parked in the street indicating a tilt to the east during collapse.

Figure 79 – View from looking west across Madras Street. A Line F/3 column is highlighted showing conical fracture in the painted portion above unpainted portion which had been enclosed by Spandrel Panels.

B.2 DEBRIS REMOVAL SEQUENCE

The debris from the collapse was removed from site and taken to a secure designated area at the Burwood Eco Landfill. The photos show stages in the sequence of debris removal. The identities of personnel have been blocked out.

Overhead Views

Figure 80 - Aerial view from southeast with debris being removed by heavy machinery. Fire has blackened the North Core. The Samoan church is damaged in the foreground (Dominion Post).

Figure 81 - Aerial view from northwest with heavy machinery removing debris. Water puddles on the vacant site may be due to fire fighting based on comparison with Figure 71 which showed no surface water immediately after the collapse (NZ Herald).

Figure 82 - Spandrel Panels and beams at Cashel Street Line 1 and on Line 4 in background standing vertical. Roof steelwork debris is visible.

Figure 83 - Concrete Spandrel Panels, perimeter beams and columns on Cashel Street face (Line 1 / B-D).

Figure 84 - Line 4 / B-C Spandrel Panels against tower wall, showing (left to right) a) View from north face; b) View from west showing timber framing for wall linings.

Figure 85 - Debris being cleared from Madras Street face.

Figure 87 - View from Cashel Street with debris being cleared away from west wall.

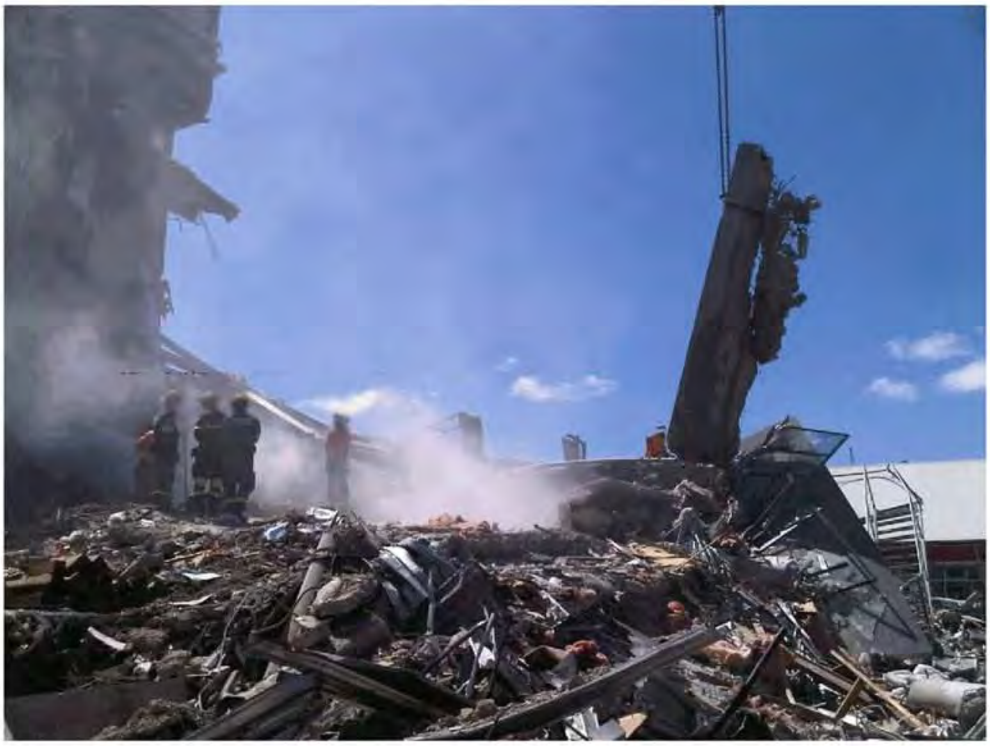


Figure 88 – Precast concrete beam being lifted from the debris near the South Wall.

Figure 89 - Perimeter 400 mm diameter column with spalled base and bar lapping zone at left unpainted portion that would have been located at Spandrel Panel infill areas.

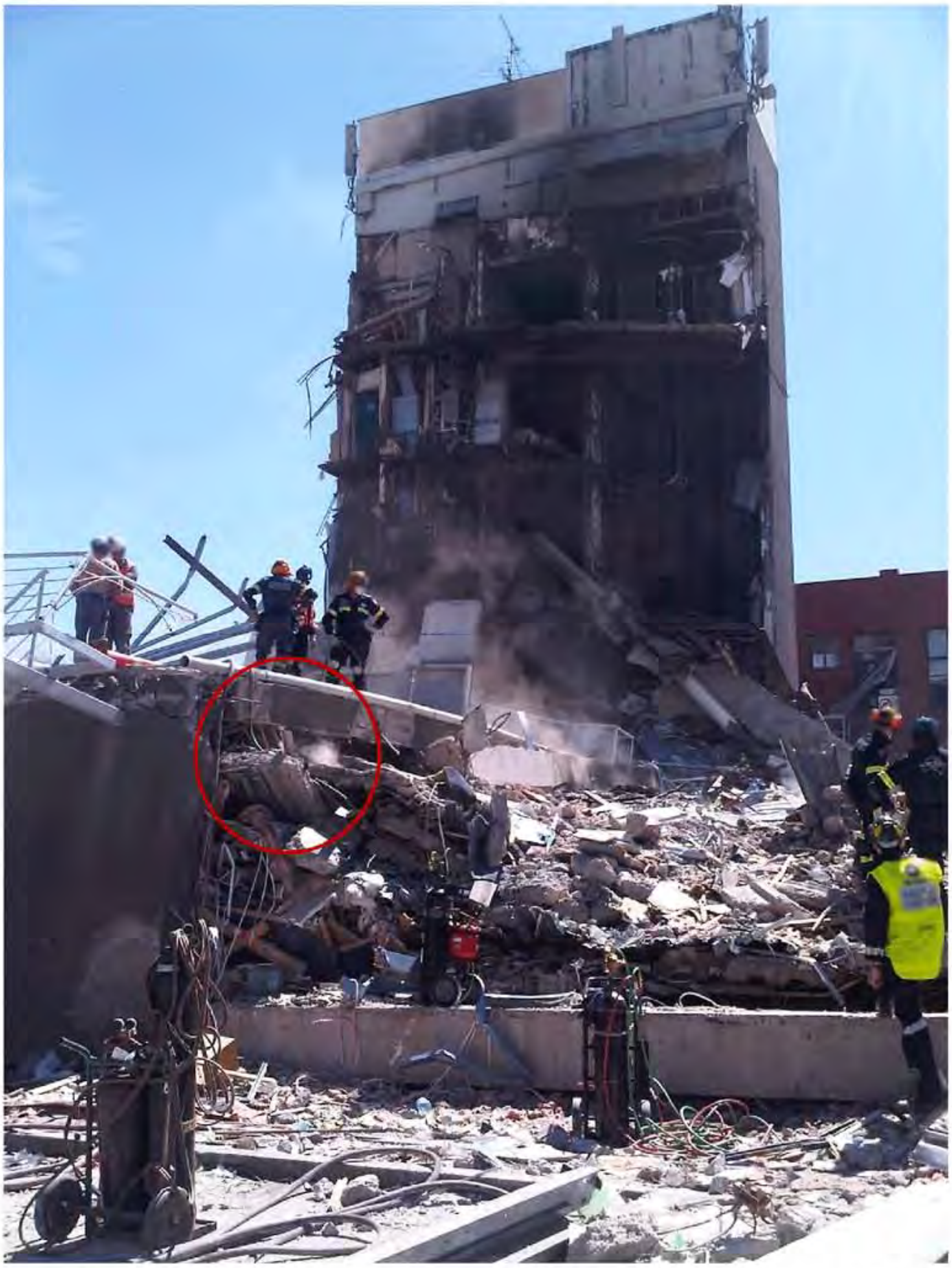


Figure 90 - View from Cashel Street east side of Line 1 with Line 1 South Wall lying on debris at left; trapezoidal end profile of floor slabs laying on top of each other in foreground; A portion of floor slab highlighted, appears to be still in contact with the South Wall at Level 2. Remnants of North Core slabs and the collapsed column on Line 4 D/E can be seen at the rear. The Level 6 slab in front of the lift well can be seen still suspended in mid-air by its Drag Bars connected at Walls D and D/E, even after loss of support from column 4 D/E.

Figure 91 - Portion of Line 1 South Wall being lifted out by crane.. The Level 6 slab in front of North Core has been removed for safety reasons.

Figure 92 – Upper portion of South Wall being prepared for removal. A portion of slab can be seen highlighted in the foreground in contact with the South Wall on this side at Level 2. It may have forced the wall to pivot against it, preventing it breaking over at its base at Level 1.

Figure 93 - Line 1 South Wall at Level 1 showing masonry in-fill at door opening, in-plane flexural fan-like cracking and spalling of concrete at right (east) end. A portion of profile slab can be seen end on through the opening

Figure 94 – Line 1 South wall at Level 4 showing severe diagonal cracking in east panel.

Figure 95 – View of North Core showing Level 4 slab lying diagonally on top of Level 3 slab. This indicates that the Level 3 and 4 slabs lost their vertical support on Line 3 prior to breaking away from the North Core.

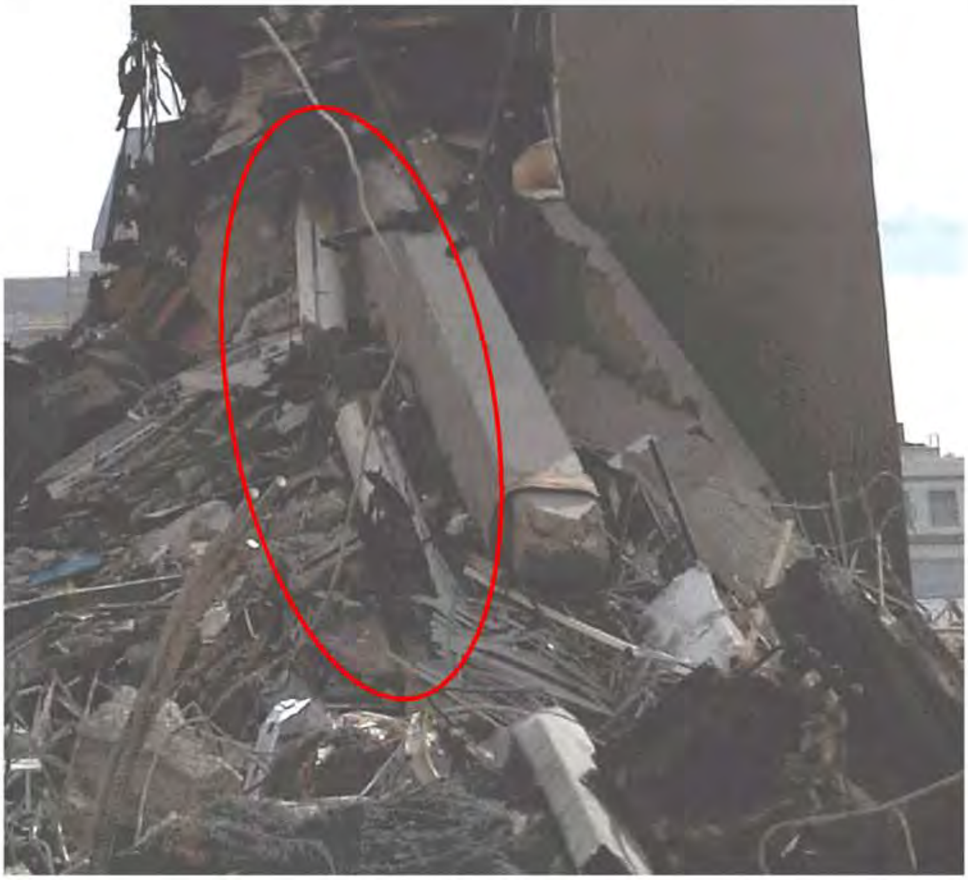


Figure 96 - North Core column 4 D/E highlighted amongst the debris. Hinging can be seen above and below the beam column joint.

Figure 97 – View from southwest of North Core. The Level 4 slab can be seen lying diagonally against the North Core.

Figure 98 - Line 2 beams lying rotated northwards.

Figure 99 - Line 3 beams lying rotated southwards

Figure 100 - Perimeter columns at beam-column joint with shell beam on right side.

Figure 101 - Line 4 / B column with B22 precast log beam in foreground and B23 shell beam at rear. No hinging is apparent at the base of the column compared to the perimeter column Item E33.

APPENDIX B – PHOTOS OF COLLAPSED BUILDINGCONTINUED

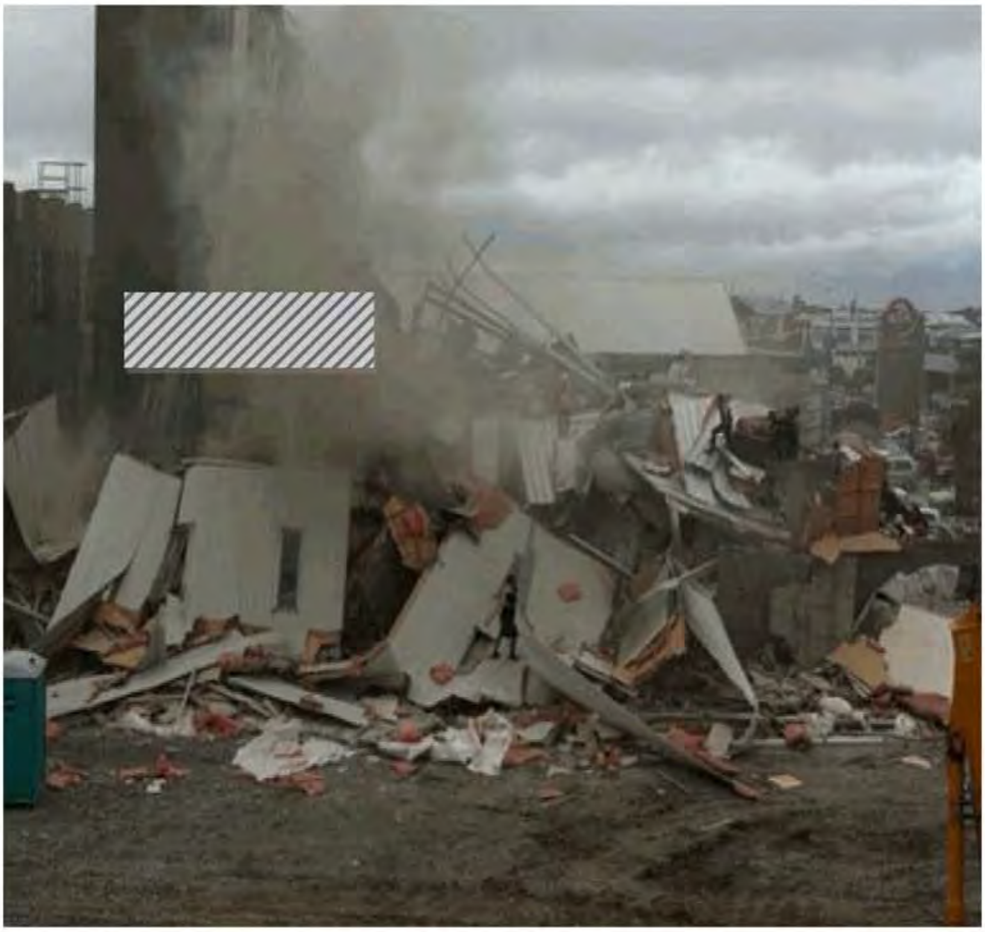


Figure 102 - North Core slabs remaining to be removed.

Figure 103 - North Core slabs removed.

Figure 104 - All debris removed leaving the Level 1 slab on grade and remnants of the North Core.

APPENDIX C - SUMMARY OF SITE EXAMINATION AND MATERIALS TESTING RESULTS

INTRODUCTION

The following summarises observations and material properties from the Site Examination and Materials Testing. A more detailed account is found in the Site Examination and Materials Testing Report (Hyland 2012).

PROFILED METAL DECK AND CONCRETE SUSPENDED SLAB

The profiled metal deck that formed the 200 mm thick slab had de-bonded from the underside of the concrete in many cases during the collapse. This is not unexpected as it is recognised by engineers that profiled metal decking does not rely on chemical adhesion with the concrete to develop the properties of composite profiled metal deck concrete slabs.

The steel decking had pulled away from the supporting beams in all cases except at the pre-cast beam support on Line 4 at the North Core. In that case the steel decking appeared to have fractured in tension.

A portion of the decking was tensile tested and found to exceed the minimum specified yield stress of 550 MPa

PRE-CAST CONCRETE SHELL BEAMS

The pre-cast concrete shell beams were found to have no reinforcement in the in-situ in fill concrete.

There was no roughening of the precast surface on the inside of the shell beams to encourage composite behaviour of the shell and the in-fill concrete. Composite behaviour between the shell and the infill concrete would have increased the ability of the beams to resist the demands placed on them.

The slab on the shell beam on Line 4 that connected into the shear core wall had fractured along the inside edge of the beam.

The bottom reinforcing steel in the shell beams had not been developed fully into the Grid C core wall on Line 4 as specified, except at Level 2. The bars had been bent back into the concrete infill in the shell beam (Figure 105).

Figure 105 - Precast shell beam (Item E14) from northern face Grid 4, west side of North Core (DENG B23 Dwg S18). (clockwise from top right) (a) to (b) Fractured slab outstand remnant at east end from which slab concrete cores were extracted. The bottom H24 bars from shell beam have been turned back into the concrete infill rather than embedded in shear wall as specified (DENG Detail 5 Dwg S19). Notice the bar imprint on wall at the connection seen in (c) at Level 4 and at Level 3.. This meant that these beams would not have performed as intended.

400 MM DIAMETER COLUMNS

The exterior 400 mm diameter column (Item E33) had flexural failure at the floor level lap joint of the vertical reinforcing steel, and compression-flexural fracture at the upper end of the column (Figure 106)

The lap joint in the exterior columns was concealed by the external Spandrel Panels and interior linings.

Figure 106 - 400 Diameter Exterior Column Item E33. (DENG C5 or C11, Dwg S15). Left end is bottom of column at floor level with concrete spalling over lapped vertical reinforcing. Horizontal cracking in core confined by R6 spiral which had fractured. The unpainted portion was protected by Spandrel Panels.. Right-hand end fracture occurred below beam-column joint.

INTERNAL PRE-CAST LOG BEAMS ON LINE 2 AND 3

The ends of the pre-cast internal log beams that supported the 200 mm thick profiled metal deck slab, had smooth formed un-roughened ends at the interface with the beam–column joint zone. This would have reduced beam-column joint shear capacity weakening its ability to hold together during earthquakes (Figure 107).

EXTERNAL PRE-CAST LOG BEAM ON LINE 1 AND 4

The ends of the pre-cast log beams supported by the corner columns on Grid A had a smooth un-roughened end where it connected into the columns. This would have reduced the beam-column joint shear capacity weakening its ability to hold together during earthquakes.

No starter bars connected the log beam into the 200 mm slab that was supported on the shell beams. This is not considered to be usual design practice (Figure 108).

Figure 107 - Interior Pre-cast Log Beams from Line 2 and 3 (DENG Section 3 Dwg S15) showing smooth concrete formed for beam-column joint, and bottom hooked bars that have pulled out of beam-column joints without any obvious straightening;

Figure 108 - Item E18 Pre-cast edge beam north-west corner (DENG B22 Dwg S18 (from left to right) (a) Smooth form finish at attachment to column 4A (DENG Detail 1 Dwg S19); (b) No starters (reinforcing bars) from pre-cast beam into slab to prevent the profiled metal deck slab pulling away (DENG Section 4 Dwg S15). If roughened these joints may have slowed down development of progressive collapse.

LINE 1 SOUTH WALL

The Line 1 South Wall that extended from Level 1 on the ground to the roof had been broken up into single story components during de-construction.

Level 1 to 2 (Item E1)

This panel showed flexural cracking patterns typical of cantilever shear walls rather than coupled shear walls (Figure 109). This was likely due to the effect of the Level 1 doorway having been in-filled with reinforced concrete masonry.

Reinforcing steel taken from the east end of the wall was found to have yielded and elongated prior to the collapse of the building.

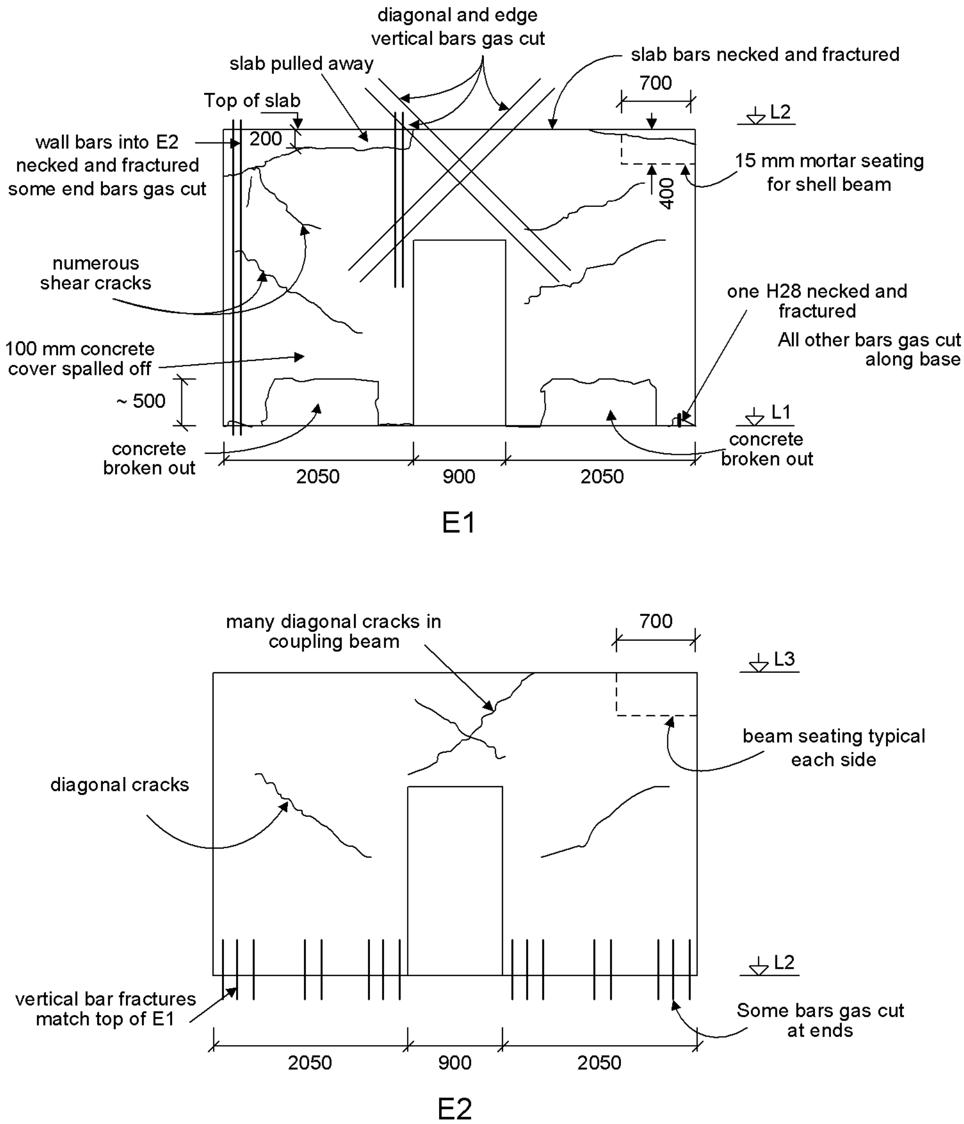


Figure 109 - Line 1 South Wall remnants (top) E1 Level 1 to 2; (Bot) E2 Level 2 to 3.

Level 2 to 3 (Item E2)

This panel had diagonal cracking in the piers consistent with cantilever wall behaviour and two way diagonal cracking in the door head coupling beam (Figure 109).

Level 3 to 4 (Item E3)

This panel had dominant uni-directional diagonal cracking running from the bottom west corner to the top east end (Figure 110).

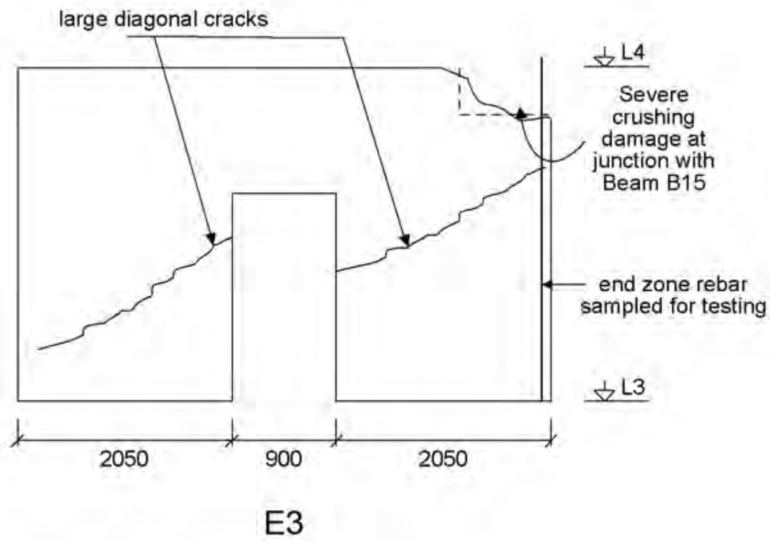


Figure 110 - Line 1 South Wall remnant E3, Level 3 to 4.

Level 4 to 5 (Item E4)

Severe two-way diagonal cracking in east pier and loss of cover to vertical reinforcing steel on east edge.

Smooth mortar construction joints rather than roughened at junctions with pre-cast shell beams B15 and B16 (Figure 111).

The cracking may have been caused on impact with the ground during the collapse.

Level 5 to 6 (Item E5)

Weak concrete in west pier adjacent to top of doorway that was able to be dislodged by boot (Figure 111).

The top surface of wall was smooth rather than a roughened construction joint for slab seating.

This may have led to slippage on these joints potentially contributing to greater than intended inter-storey drifts.

Bars from wall into attached pre-cast beam had fractured.

No obvious cracking had occurred in the wall or the door head coupling beam.

Level 6 to Roof (Item E5A)

No obvious cracking had occurred in the wall piers or door head coupling beam.

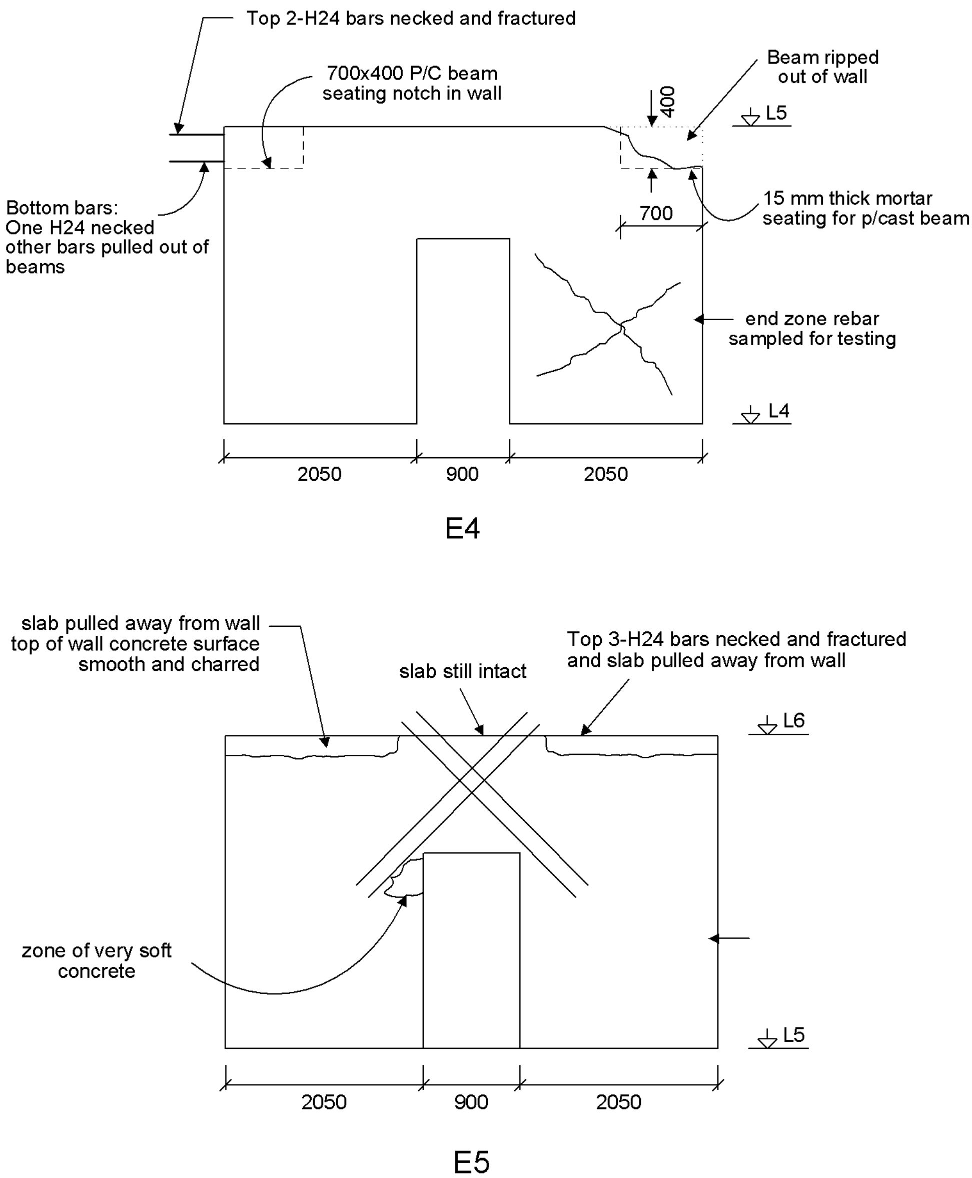


Figure 111 - Line 1 South Wall remnants E4 Level 4 to 5 and E4 level 5 to 6.

Figure 112 - Line 1 South Wall Level 5 to Level 6 (Item E5) (clockwise from top left) (a) Crumbly concrete at door edge of west pier able to be dislodged by boot; (b) Smooth and charred construction joint on top west surface looking east; (c) Charred construction joint above west pier. Door sill on left; (d); Top east corner with fractured top 3-H24 bars. Floor 664 mesh exposed.

NORTH CORE WALLS

Only fine cracking was found on the North Core walls after collapse (Figure 113).

No obvious cracking on Line D/E wall.

Horizontal flexural cracking on west and north west face of Line 5 wall and north end of Line C wall at Line C/5.

Fine two-way diagonal cracking on the inside faces of Level 1 to 2 Line 5 and D walls in North Core.

Figure 113 - North Core cracking (clockwise from top left) (a) No obvious cracking on Line D/E wall; (b) Horizontal flexural cracking on west and north west face of Line 5 wall and north end of Line C wall at Line C/5; (c) Fine two-way diagonal cracking on the inside faces of Level 1 to 2 Line 5; (d) and D walls in North Core.

SLAB AND BEAM REMNANTS ON LINE 4 OF NORTH CORE

The extent of the slabs at the time of examination was measured (Figure 37).

Portions of the level 6 and Level 5 slabs that were still attached immediately after the February Aftershock were removed during deconstruction for safety reasons. The slab at level 2 had also been broken back. The rest of the slab was in the condition it was left after the event.

Level 6 Slab

The slab had a vertical fracture face that coincided with the ends of the H12 saddle bars from the support beam on Line 4 (Figure 114).

664 mesh in the slab had fractured in a ductile manner which is the way it was intended to.

The profiled metal deck steel decking had fractured in tension adjacent to the edge of the fractured slab edge.

Level 5 Slab

The fractured edge of the slab was similar to that at level 6.

Reinforcing was located in the bottom of the slab rather than as specified near the top surface.

Cracks were found running from cores drilled in the slab for pipes.

Level 4 Slab

The imprint of the bent back bottom bars from the pre-cast shell beams (Figure 105) was visible in the cover concrete of the wall.

The profiled metal deck decking of the fractured slab was still clamped to the support beam on Line 4 and fractured in tension.

Level 3 Slab

Similar to Level 4

Level 2 Slab

Bottom bars of pre-cast shell beam had been developed into the core wall on this level only, and beam-column joint type diagonal cracking was seen on the end of the wall. This was consistent with cyclic demands having occurred there during the February Aftershock.

Figure 114 - Line 4 Core Wall Slab Remnant at Level 6 amenity area (clockwise from top left) (a) Slab edge on stairwell wall looking west with H12 saddle bar exposed and ends of mesh below it; (b) Vertical concrete fracture surface with reinforcing mesh fractured; (c) Slab looking west with cores cut in floor for amenities; (d) Fractured mesh angled downwards; (e) Fractured slab edge looking east. Torn metal decking aligned approximately with concrete fracture edge; mesh at varying height within slab; (f) Cores for amenities at fracture edge can be seen and are a small proportion of the total fracture surface length.

SLAB DIAPHRAGM CONNECTIONS TO NORTH CORE WING WALLS ON GRID D AND D/E

After the original construction of the building had been completed, Drag Bars were fixed into the slab and into the walls at Levels 4, 5 and 6 on Lines D and D/E with epoxy grouted threaded rods.

Level 2 Connection of Slab to Walls

No reinforcing steel connected the slab to the east wing wall D/E.

A 20mm hole was found in the west wing wall D where a reinforcing bar had pulled out.

Level 3 Connection of Slab to Walls

An H12 bar was found fractured at the end of the west wall D.

No reinforcing steel was found to have connected the east wing wall D/E to the slab.

Level 4 Connection of Slab to Walls

The Drag Bars on both the west and east wing walls had partially fractured in bending and tension. The epoxy grouted 20 mm threaded rods that were fixed vertically into the slab and into the Drag Bar on the west wall appeared to have pulled out in tension. This occurred as the slab between Lines D and D/E rotated downwards, pivoting about its Drag Bar supports at the ends of the lift shaft walls.

The 20 mm diameter Drag Bar threaded rods were hardness tested by MTL and found to have Rockwell Hardness HRB. This conformed with the minimum requirements of AS 4291.1:2000 (SAA 2000) for Property Class 5.8 threaded rods.

Level 5 and 6 Connection of Slab to Walls

Similar to what was seen at Level 4 (Figure 115).

Figure 115 - Level 5 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top) (a) the Drag Bar consisted of a 150x150x10 L steel angle with a 51 x 3.2 SHS welded to it; 4-M24 anchors were epoxied into the wall and 6–M20 threaded anchor rods 350 mm long were epoxied into the slab at the profiled metal deck rib. 3-M20 threaded rods remained upright on the Grid D Drag Bar. The 51x3.2 SHS had fractured in bending and tension at the bolt hole adjacent to last bolt into wall and twisted with the slab; This shows that the slab that had been fixed to the Drag Bar had rotated downwards as the column on Line 4 D/E collapsed, (b) Epoxy grout can be seen around the threaded anchor rod that had been in the slab; (c) The Drag Bar is bent downwards and holes where 3-M20 threaded anchor rods had been can be seen; d) On Wall D/E a 150x75x10 L steel Drag Bar was still fixed into the wall D/E with 5-M24 threaded rod anchors. The end of the Drag Bar had been gas cut during deconstruction.

CONNECTION OF COLUMN D/E 4 TO NORTH CORE AT LEVEL 7

The column had pulled away from its connection to the North Core wall D/E. Four H20 bars were specified on the drawings to be bent into the wall (Figure 116). However only three 20 to 24 mm diameter holes were found in the location where the column bars had pulled out. This indicates that one bar had not been placed as specified. Though not considered to have initiated the collapse, if all these bars had been present they may have prevented the collapse of column D/E 4. Even so this would not have prevented the collapse of the other columns in the building.

Figure 116 - Lift Well Wing Wall D/E: Column D/E 4 Connection (DENG Dwg S14); 3 x 20 to 24 mm diameter holes can be seen where reinforcing bars from column have pulled out. The drawing shows that 4-H20 bars were required to be bent in to the wall.

LEVELS AND POSITIONAL SURVEY

The floor slab, slab overlay and foundation beams were found to have levels consistent with original construction tolerances and practice.

No evidence of long term foundation settlement or settlement induced by the February Aftershock could therefore be inferred.

The core walls on Line 5 were surveyed for verticality by sighting on the eastern and western corners of the north face of the wall. It was found that there was a northwards out-of-vertical measurement of 91 mm over 18.53 m between Level 1 and Level 7 at the northeast corner, and 68 mm over 18.53 m at the northwest corner.

This is much greater than the straightness limit of 30 mm for structures greater than 10m high or position plan tolerance of 10 mm in NZS 3109.

The company that maintained the lifts at the CTV Building advised that they had no records of the inside faces of the walls being out-of vertical alignment after construction.

REINFORCING STEEL PROPERTIES

Reinforcing steel samples were extracted from the Line 1 South Wall and tested to determine tensile properties, production uniformity and work hardening during the February Aftershock.

The reinforcing steel from the South Wall was found to conform to the standards of the day.

The H28 steel extracted from the lower portion of the South Wall item E1 was found to have elongated 3.3 % more than the other 16 to 28 mm bars extracted. It also had an elevated yield stress and ultimate tensile strength. This is known to occur in constructional steels that have been work hardened and have subsequently strain aged (Hyland, Ferguson et al. 2003).

This is evidence that the bar appeared to have "work-hardened: during the February Aftershock and prior to the collapse of the building.

The chemical analysis of the 16 to 28 mm bars found that they had chemical compositions consistent with them being from the same or similar production runs.

The suspended slabs were reinforced with hard drawn steel 664 mesh sheets with wires spaced at 150 mm cross centres. The 664 steel mesh from the suspended slab was sampled and tested.

The 664 steel mesh was found to conform to the standards of the day.

CONCRETE PROPERTIES

Cores were extracted from remnants of columns, beams, slabs and walls for compressive strength testing. The chord modulus of elasticity was also determined for the South Wall and North Core concrete.

The sample means of the test results for a particular member were adjusted up by a factor of 8% where required, to allow for test orientation effects where testing had been done transverse to the direction of casting (Figure 117). This was in accordance with the recommendations of the Concrete Society Technical Report 11 (GBCS 1987).

The concrete test results need to be interpreted in light of the fact that the samples were extracted from components that had been damaged in the collapse. Care was taken however to avoid coring in portions of concrete with obvious cracks and samples were visually scanned before testing for signs of cracking.

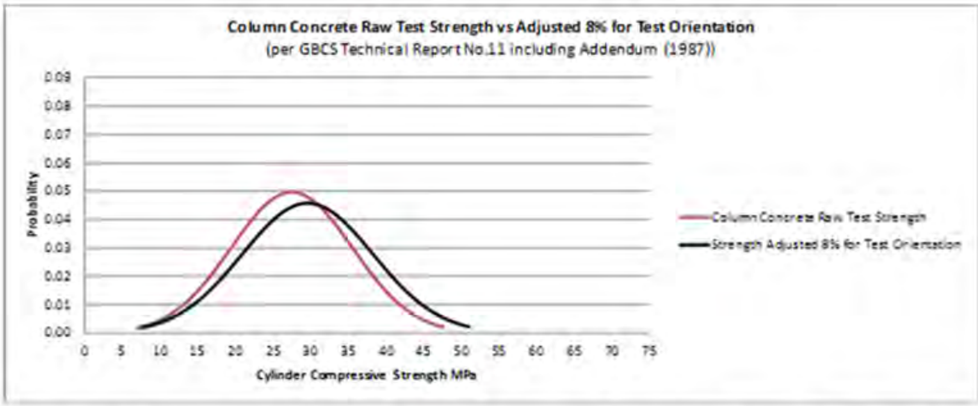


Figure 117 - Column concrete test strengths compared to strengths adjusted 8% for test orientation being transverse to direction of concrete casting. This adjustment in test strength was recommended by the Concrete Society Technical Report 11 "Concrete Core Testing for Strength".

The adjusted sample means were then assessed against the known means of concrete properties with 28-day strengths conforming to NZS 3104:1983.

A lower 0.1% acceptance limit was applied to identify upper bound conformity with a specific strength category. Where the sample size was sufficiently large an upper 0.1% rejection limit was also applied to identify non-conformity with a lower strength category.

Suspended Slab Concrete Properties

The suspended slab concrete was core tested in two locations. The average strength at test was 24.6 MPa.

In conclusion, at the time of the collapse the concrete in the suspended slab had mean strength not greater than that of concrete with 28-day strength of 25 MPa.

This indicates that at the time of the collapse the concrete in the slab may have been consistent with concrete with the specified 28-day strength of 25 MPa.

The mean strength of the concrete was also not greater than that with 28-day strength of 20 MPa Aged by 25%. This indicates that the slab concrete may not have achieved the specified 28-day strength at the time of construction.

South Wall and North Core Concrete Properties

Concrete cores extracted from one location each in the South Wall and the North Core found an average strength of the walls of 33.8 MPa.

This was adjusted 8% for testing orientation transverse to casting direction, to give 36.5 MPa.

In conclusion, at the time of testing, the concrete in the shear walls had mean strength not greater than that of concrete with 28-day strength of 35 MPa. This seems consistent with it having had strength satisfying the minimum specified 28-day strength of 25 MPa.

The concrete in the shear walls also had mean strength not greater than that of concrete with, 28-day strength of 30 MPa Aged by 25% or less. This seems consistent with the concrete having achieved the minimum specified 28-day strength of 25 MPa at the time of construction.

The chord modulus of elasticity of the shear wall concrete was found to be an average of 27,600 MPa. This was consistent with what would be expected for concrete with that strength

The calculated average secant modulus of elasticity was 26,100 MPa.

Column Concrete Properties Summary

The concrete column test strengths derived from core and rebound hammer tests are shown in Table 6. These are also shown factored up by 8% to allow for the effect of testing transverse to the direction of casting (GBCS 1987).

Cores taken from the Line 4-D/E column were found to have some discolouration that may or may not have been due to silt, had low density and did not exhibit the degree of aggregate breakage expected for concrete of the specified strength.

The strength of the concrete in the column remnants tested was based on the testing of a statistically significant sample of 26 column remnants selected at random from the debris at the Burwood Eco Landfill.

The adjusted sample mean of all column remnants tested was 29.6 MPa.

This indicates that at the time of testing the column remnants from Levels 1 to 6 had mean concrete strength equivalent to that of concrete with specified 28-day strength of 20 MPa (Figure 107). This is less than the minimum concrete 28-day strength of 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

This also indicates that the columns in Levels 1 to 6 may not have achieved the specified 28-day strength at the time of construction (Figure 108). The specified concrete 28-day strength was 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

	As-Tested	Adjusted 8% for Test Orientation
Sample Size (n)	26	26
Minimum (MPa)	16.0	17.3
Maximum (MPa)	46.6	50.3
Lower 5% (MPa)	14.2	15.3
Mean (MPa)	27.4	29.6
Upper 95% (MPa)	40.6	43.8
Coefficient of Variation (cov)	0.293	0.293
Standard Deviation (MPa)	8.04	8.68

Table 6 Column concrete test properties statistics

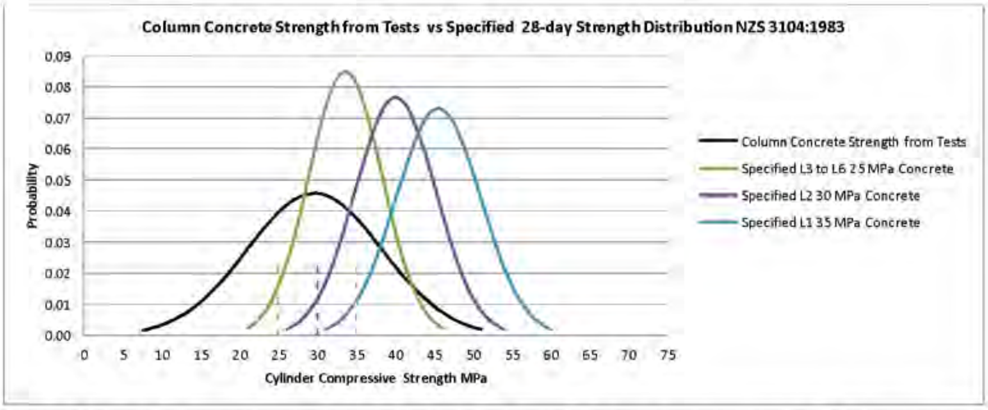


Figure 118 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS3104:1983. This indicates that the concrete in a significant proportion of the columns may have had strengths less than the minimum specified.

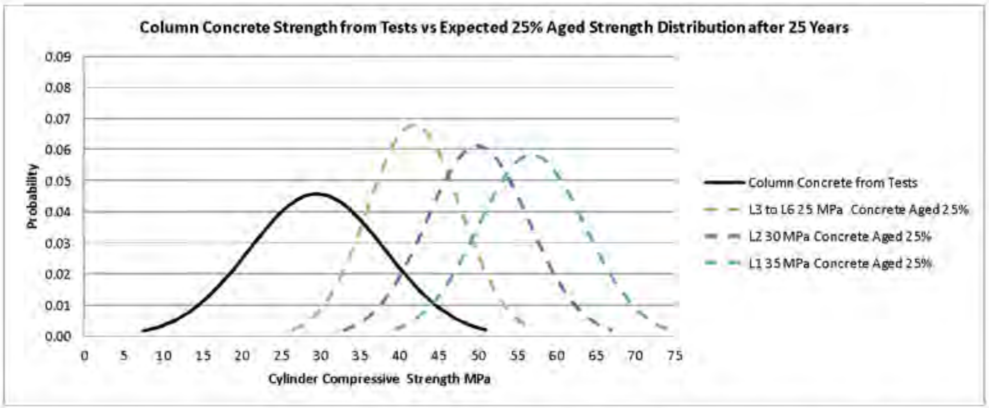


Figure 119 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS 3014:1983 strength-aged by 25%. This indicates that the concrete in the column remnants may have had a significantly lower strength distribution compared to the lowest concrete strength specified, when a 25% allowance is made for the strengthening of concrete with age.

APPENDIX D - NON-LINEAR TIME HISTORY ANALYSIS

INTRODUCTION

Non-linear time history analysis (NTHA) were used to evaluate the response of the CTV Building to the ground motions that had been recorded at three other similar sites in the Christchurch CBD for the September Earthquake and February Aftershock.

With NTHA, changes in building stiffness resulting from the non-linear behaviour of structural elements are calculated at each time step, which allows the deformations and structural actions to be determined as accurately as possible based on current engineering knowledge. It should be noted however, that the results should be interpreted as being an estimation of the response rather than an accurate representation of the actual response during the earthquake at every step in time.

The primary objective with the NTHA for the CTV building has been to model the overall lateral stiffness and strength of the building as accurately as possible. The vertical stiffness of floors and beams has also been modelled to enable quantification of the effects of vertical accelerations.

The main findings from the analysis are described in the following sections. Floor diaphragm connections and columns are a focus, since they are potentially critical failure mechanisms under seismic loading. Irregularity of the building structure and the resulting torsional response is a significant influence. The fragility of beam-column joints is also discussed.

The load demands on floor diaphragm connections to shear walls were obtained directly from the analysis. For columns, the inter-storey drifts output from the NTHA were considered to represent the earthquake demand, against which various potential failure mechanisms were then assessed post-analysis.

ANALYSIS OVERVIEW

The three dimensional model shown in Figure 120 was created using the SAP2000 finite element program. Non-linear static pushover analyses and non-linear time history analysis were carried out using this model to evaluate seismic actions on the structure.

The basis of the non-linear analysis is reported in more detail in the referenced 'Non-Linear Seismic Analysis Report' by Compusoft Engineering, (Bradley, Stuart et al. 2011) who were engaged by StructureSmith to undertake the analysis. Key points from that report and interpretation of results are summarised below.

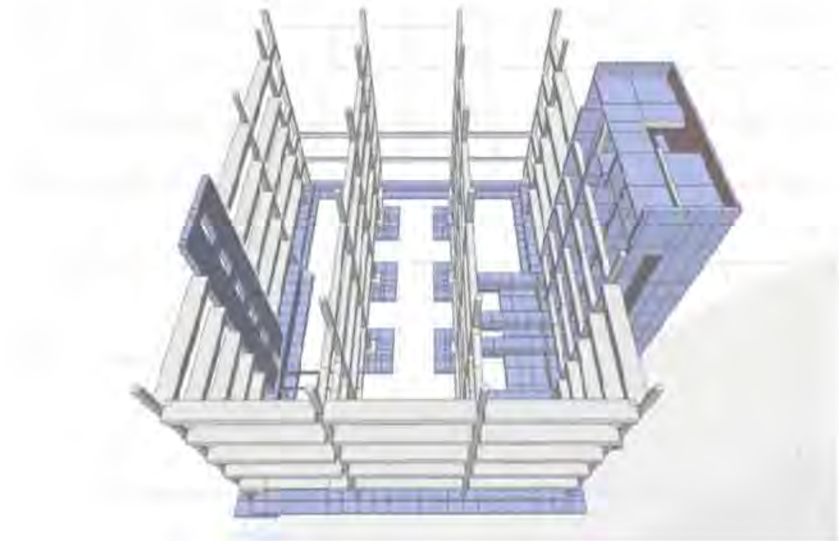


Figure 120 - SAP non-linear analysis model viewed from east side.

The analysis of the CTV structure investigated two different structural configurations denoted '*MODEL A*', and '*MODEL B*', as outlined below.

- **MODEL A (no masonry)**

  This was the structure 'as detailed' and included the contribution from the primary seismic force resisting system, the North Core and South Wall, and the secondary structural elements that were not detailed for separation (the concrete frames) only. The masonry infill walls and precast concrete spandrels were assumed to be effectively isolated from the structure so as not to participate in the seismic response. A variation of this configuration where the precast spandrels engaged the perimeter columns was run as a pushover analysis only to enable assessment of the perimeter columns under that condition.

- **MODEL B (with masonry)**

  The structural form described above, but with the masonry infill walls not effectively isolated from the identified structure and so contributing to the seismic response. This modelled the masonry with no gap to the adjacent columns and with an assessed upper bound stiffness and strength based on flexural yielding of each individual masonry panel.

Note: The interpretation of the primary and secondary structure in Models A and B above was based on review of the original design calculations, which appear to have considered the concrete frames as 'gravity only' secondary elements, with the masonry intended to be separated.

The overall procedure for the non-linear analysis consisted of the following stages:

1. A gravity analysis on the structure using appropriate imposed loading allowances.

2. A nonlinear static pushover analysis of the structure for the two primary directions starting from the end state of the gravity analysis. This enabled the non-linear performance of the individual lateral load resisting structural components to be verified and then combined together in the model to be used for the NTHA.

3. The axes of the adopted ground acceleration records from the September Earthquake and February Aftershock were aligned to the principal axes of the CTV Building, which are essentially north-south and east-west.

4. Non-linear time history analyses using the three adopted ground acceleration time history records of the September Earthquake and February Aftershock. This process was carried out for both structural forms *MODEL A* and *MODEL B* for one record of the September Earthquake and then for Model A only for three records of the February Aftershock. All components of the acceleration time history were incorporated simultaneously including north-south, east-west and vertical components.

5. The results were then processed and the performance reviewed.

KEY ASSUMPTIONS

Key assumptions and features in the SAP non-linear model included the following:

1. Reinforcing steel properties were taken from tests reported by HCL (Hyland 2012). Expected concrete strengths for columns were taken as equal to the specified 28-day strength + 2.5 MPa. In fact concrete strengths were found to vary considerably.

2. Foundations were modelled with non-linear soil spring supports, with compressive stiffnesses evaluated by Tonkin and Taylor Ltd, and with gapping under uplift conditions to model the potential rocking of foundations. Rocking of the north foundation was indicated by the NTHA, and its effects in terms of drifts are included in the analysis output and have been used in the demand / capacity comparisons.

3. The yielding portions of shear walls were modelled using nonlinear layered shell elements which incorporated inelastic material effects at a fibre level. Where there was no significant inelastic demand the walls were modelled using linear elastic shell elements with stiffness modifiers determined from moment-curvature analyses. Modelling of the diagonally reinforced coupling beams in the South Wall used non-linear links substituted for the fibre elements to reduce computation times.

4. Beams and columns were modelled as elastically responding frame elements, with stiffness modifiers determined from moment-curvature relationships. Figure 121 below shows the effective column stiffness relationships used in the non-linear analysis model, with the effective stiffness properties from

NZS3101:1982 and NZS3101:2006 also shown for comparison. Inelastic behaviour of the beams and columns was incorporated by the way of discrete hinges. These hinges considered stiffness degradation but not strength degradation during hysteretic cycling and had no plastic rotation limits applied. Hinges were located at the face of the connecting member (i.e. at face of beam for column hinges and at face of column for beam hinges). For columns, rigid-plastic interacting M-M hinges were used, calibrated to commence hinging at the point where nominal flexural strength is reached, for the average gravity axial compression action on the column at the time of the earthquake. M-M refers here to the bending moment in two orthogonal directions.

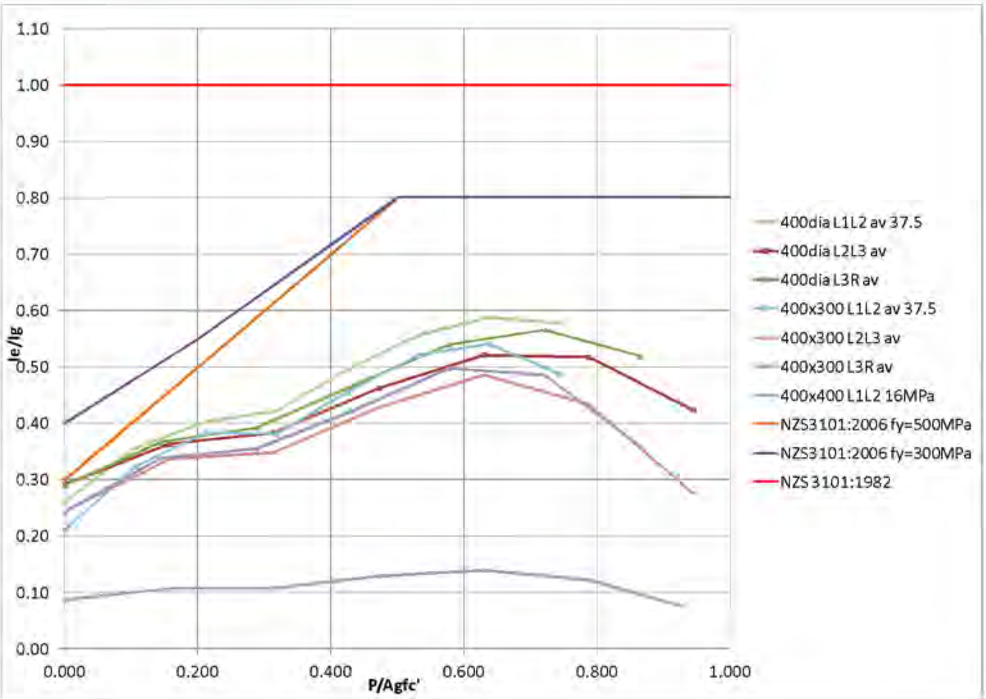


Figure 121 - Effective column stiffness relationships used in the non-linear analysis model

5. It has been assumed for the purposes of the NTHA that beam hinge formation is not limited by the capacity of the beam bar end anchorages or beam-column joint shear capacity.

6. Examination of the detailing for the connection between the top of the column at grid 4 D/E and the overhanging core wall indicated that it was not capable of transferring the significant axial forces that would result from 1986 Code seismic demands. For that reason the connection was released in the non-linear model.

7. Physical evidence indicated that the positive moment (bottom) reinforcement of the beam along gridline 4 between grids B and C was not effectively anchored into the North Core wall on grid C at levels 1,3, 4, 5, and 6. No positive moment capacity was provided at these locations in the model at Levels 3 and 4 which had been confirmed at the time the NTHA was carried out.

8. Floor diaphragm connections to the North Core walls on grids D and D/E were identified as an area of potential connection failure. As a consequence of a lack of tie reinforcement it was assumed that there was no tensile or gravity connection between the slab and these walls at levels 2 and 3. At levels 4 to 6 a retrofitted steel angle tie (or 'Drag Bar') provided limited tensile and gravity connection to the slab at the tips of the walls on grids D and D/E. The Drag Bars were modelled using fuse tension links incorporating 2 mm initial slip in connections and the calculated elastic stiffness of the steel angle section. At actions equal to the calculated limit state tensile capacity of the Drag Bar and its connections (based on design documentation and tested properties of anchor bolts and slab concrete) the fuse links would disconnect. No limitation was placed on the compressive load capacity. A further analysis, with the CBGS February Aftershock record was carried out with the drag bars remaining connected throughout. Gravity load transfer at this interface is expected to be limited to a low value by slab reinforcement yielding and so was taken as zero for the purposes of the seismic analysis. Floor diaphragm connections to other walls were assumed to remain connected for the purposes of the analysis, including when Drag Bars had become disconnected, irrespective of the demands placed on them.

9. In-plane stiffness of the floors was modelled as 0.5 Agross for an average slab thickness of 173mm to allow for nominal cracking. For out-of-plane demands the floors were considered to have effective stiffness corresponding to 0.5 Igross at midspan. The effective out-of-plane stiffness adjacent to beam lines was taken as the average of the positive and negative stiffness. This was determined from moment-curvature analyses considering the reinforcement present (it appeared there was no bottom reinforcement from the floor slab into the supporting beams). The effect of the profiled metal deck was not incorporated into the model.

10. In Model B, the stiffness and strength of the masonry infill were modelled using elastic shell elements, with non-linear link elements connecting each masonry panel to the underside of the floor or beam above at each level. Based on a calculation of the flexural capacity of a typical masonry panel, the non-linear links transferred a maximum of 100kN shear from each 2.3m wide masonry panel at up to 20mm lateral displacement, degrading to zero shear after 35mm lateral displacement. (This is less than the shear strength of the masonry that could be developed if the panels were fully constrained by the beams and columns around them.)

11. The NTHA did not include the potential effects of variation of concrete strength or the potential interaction of the precast façade spandrels with perimeter columns directly. The reason the effects of the spandrels and varying concrete strength were not explicitly modelled is that they were considered not to alter significantly the overall building response to earthquake shaking. However, these effects were considered in the assessment of individual elements such as columns post-analysis. The upper bound effect of the spandrels was modelled in an additional static pushover analysis.

NONLINEAR STATIC PUSHOVER ANALYSIS

Nonlinear static pushover analyses were carried out to verify the lateral stiffness and strength of the various components of the lateral load resisting structure - before the components were combined in the full model for the NTHA. Static pushover analyses were also used to determine estimates of the column first yield and nominal strength drifts and to assess the potential effects on the columns of the precast Spandrel Panels.

The pushover curves for model A, with the masonry infill walls effectively separated, are shown in Figure 122 and Figure 123 below. In these figures, displacements were at a node located approximately at the centre of mass of level 6, and the base shear components were at the top of the foundation beams.

A feature that can be seen is the significant difference in displacement and base shear between the North Core and the South Wall in the east-west direction. This represents a severe plan irregularity in the seismic resisting system.

It can be seen in Figure 123 that the plots for the eastward and westward pushovers are almost identical, indicating a similar response in both these directions. By comparison in Figure 122 the initial response of the building in the northward direction is stiffer than in the southward direction, which can be attributed to the differences in foundation stiffness under the North Core.

Greater base shear is carried by the North Core for a northward push than for a southward push. This is due mainly to the mobilisation of the gravity loads on beams along gridline 4 to resist overturning as the core walls rock and move upward beneath the beams. This behaviour is not as significant in the southward direction because of the restraining effect of the foundations to downward loads on gridline 4.

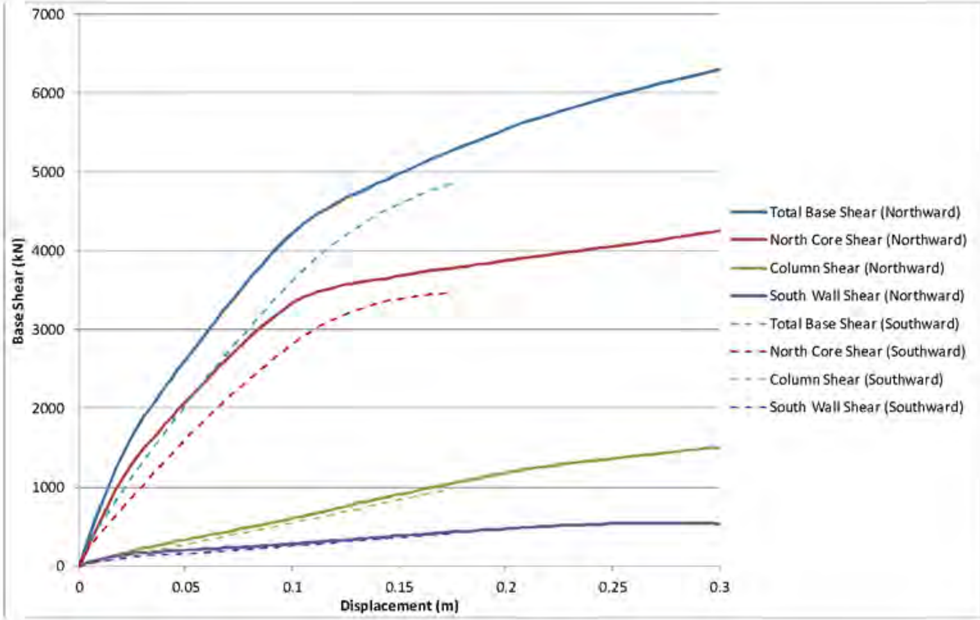


Figure 122 - Pushover curves, north-south, no masonry

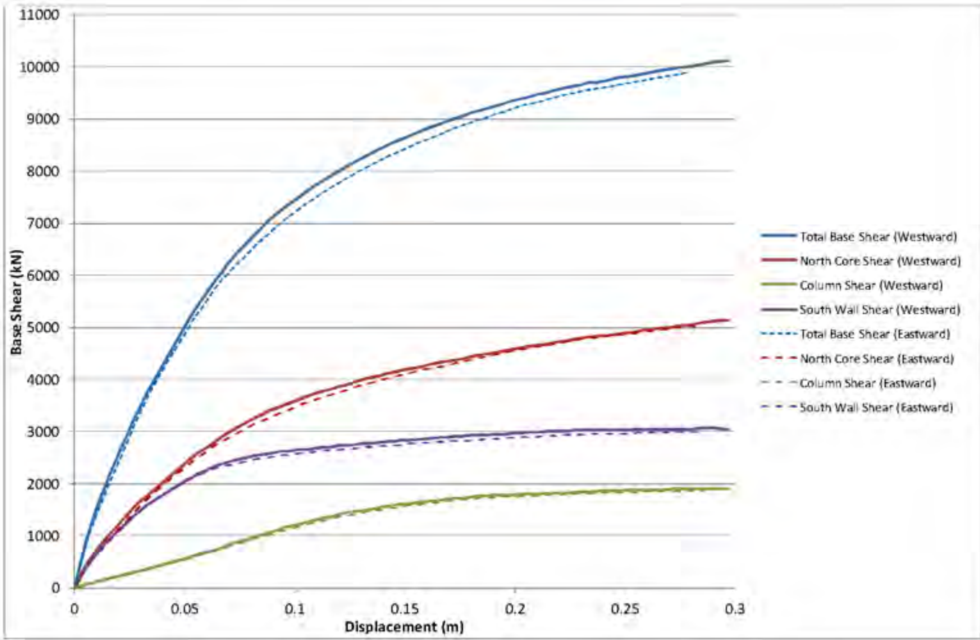


Figure 123 - Pushover curves, east-west, no masonry

In

Figure 124 plots of the pushover curves for Model A (no masonry) are shown by the dashed lines and Model B (with masonry) are shown by the solid lines. The stiffening effect of the masonry can be seen. Also, slight strength degradation of the masonry is evident in the pushover curves, where the solid and the dashed lines converge near the right-hand side

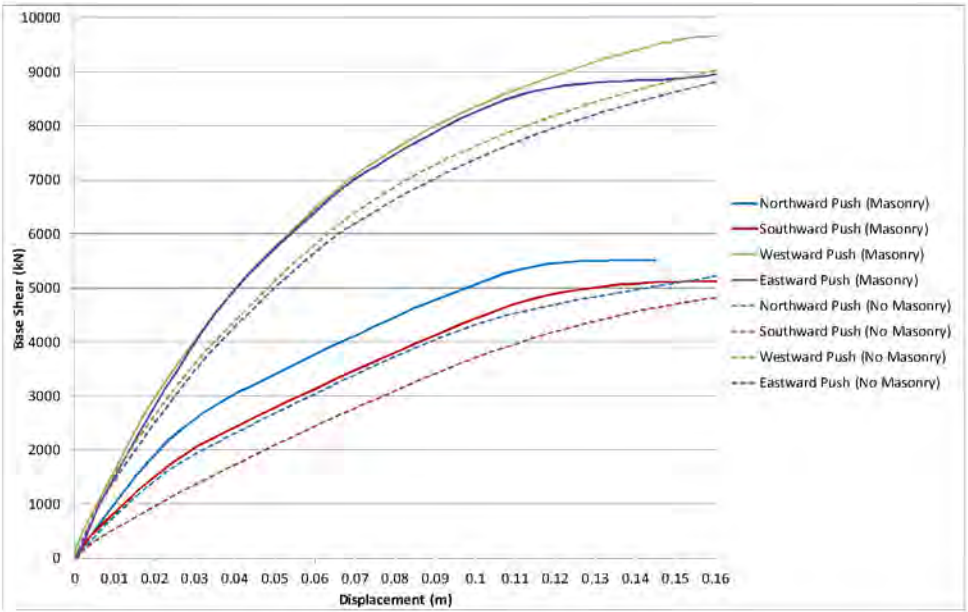


Figure 124 - Pushover curves, total base shear, with and without masonry

STRUCTURAL MODELS AND EARTHQUAKE RECORDS

The NTHA analysis runs that have been carried out are shown in Table 7 below:

Event	4 Sep Darfield	4 Sep Darfield	22 Feb Lyttelton
Structural Model	Model A – no masonry infill	Model B – with masonry infill	Model A – no masonry infill
Earthquake record:			
CBGS	☑	☑	☑
CCCC			☑
CHHC			☑

Table 7 - Summary of NTHA cases

The above analysis runs were all carried out with the diaphragm Drag Bar fuse elements described above. This enabled the comparison of results for both structural models, A and B, using the same earthquake record; and also the comparison of results for three different earthquake records using the structural model A.

A further NTHA was then carried out using Model A with the CBGS February Aftershock record, and with the Drag Bars at levels 4 to 6 remaining connected, i.e. not fused, to enable the upper bound diaphragm connection forces to be quantified.

Most of the NTHA's were carried out with all the earthquake direction components acting simultaneously, i.e. north-south, east-west and vertical. However to assess the effect of vertical accelerations separate NTHA's were also undertaken using the vertical components only of the ground accelerations from the CBGS and CCCC February Aftershock records.

The analyses for the September Earthquake and February Aftershock both assumed an undamaged structural state at the start of the earthquake record.

The input ground motions used were those recorded at GNS sites in the Christchurch CBD, located between 650m and 1500m from the CTV site. Tonkin and Taylor advised that the sites where these recorders were located have broadly similar geological profiles to CTV and that the results from the three suitable records (Christchurch Cathedral College CCCC, Christchurch Hospital CHHC and Christchurch Botanic Gardens CBGS) should be averaged when estimating the response at the CTV site.

For the purposes of the NTHA, reduced length ground motion records were used to reduce computation times. Record start and finish times were selected to ensure that all significant shaking was captured by the analysis and these times are presented in Table 8. All results reported in this document have been presented relative to the adopted start time for each acceleration time history record.

Station Name	Event	Start Time (sec)	Finish Time (sec)
Christchurch Botanic Gardens (CBGS)	4 Sep Darfield	28.90	40.90
Christchurch Cathedral College (CCCC)	22 Feb Lyttelton	15.04	23.90
Christchurch Hospital (CHHC)	22 Feb Lyttelton	16.00	27.20
Christchurch Botanic Gardens (CBGS)	22 Feb Lyttelton	16.50	25.50

Table 8 - Adopted earthquake records, start and finish times

BASE SHEARS

Peak base shears were recorded during the NTHA as shown in Table 9 and Table 10. Results have been recorded at the top of the foundation beams and are presented in units of gravitational acceleration (g), with the total seismic weight above that level being approximately 33,300 kN.

Direction	Model A Base Shear (g)	Model B Base Shear (g)
Northward	0.13	0.14
Southward	0.16	0.15
Westward	0.21	0.22
Eastward	0.22	0.22

Table 9 - Peak Base Shear, 4 September Darfield Earthquake, CBGS record

Direction	CCCC Base Shear (g)	CHHC Base Shear (g)	CBGS Base Shear (g)
Northward	0.28	0.20	0.26
Southward	0.18	0.21	0.22
Westward	0.38	0.31	0.34
Eastward	0.40	0.39	0.39

Table 10 - Peak Base Shear, 22 February, Lyttelton Aftershock, various records as shown

The peak base shears above are the overall lateral forces that had to be resisted by the seismic resisting system. Comparing the base shears obtained from the NTHA for Model A and Model B for the September Earthquake event, as shown in Table 9, it can be seen that there is little difference. In Model B the masonry was found to have reached its maximum shear resistance limited by the flexural capacity during the September Earthquake record as shown in Figure 133.

In Table 10 there is seen to be more variation across the three adopted seismic records. This is particularly so for CHHC in the northward and westward directions compared to CCCC and CBGS.

Base shears are greater in the east-west direction than north-south because of the greater lateral stiffness and strength in that direction.

STOREY DRIFTS

Figure 125 to Figure 128 show maximum storey drifts predicted by the NTHA for the September Earthquake are around 1.1% (+/-35mm) in the north-south direction on Line F, 0.61% (+/-21mm) in the east-west direction along grid 1 and 0.3% (+/-10mm) in the east-west direction along grid 4. Storey drifts were less along grid 4 because of the greater stiffness of the North Core in the east-west direction when compared with the South Wall. (Northward drifts are positive.)

The predicted drifts for the September Earthquake would have been sufficient to cause interaction with the masonry infill walls and for the precast spandrels to interact with perimeter columns on Line F and the south side on Line I if the effective separation gap was less than the 10mm nominal gap indicated on the drawings.

Comparison of the results for storey drifts for the masonry and non-masonry models for the September Earthquake can also be seen in Figure 125 to Figure 128. This shows that the masonry walls have minimal effect on the drifts in the east-west direction. In the north-south direction the masonry walls had a stiffening effect, reducing the storey drifts on line A and also to a lesser extent on line F.

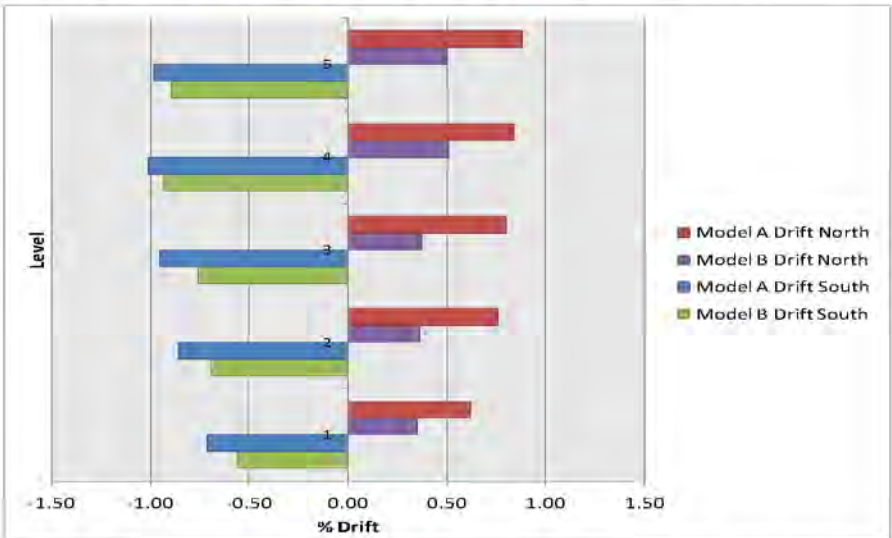


Figure 125 - Frame A north/south maximum storey drifts - 4 September Darfield Earthquake

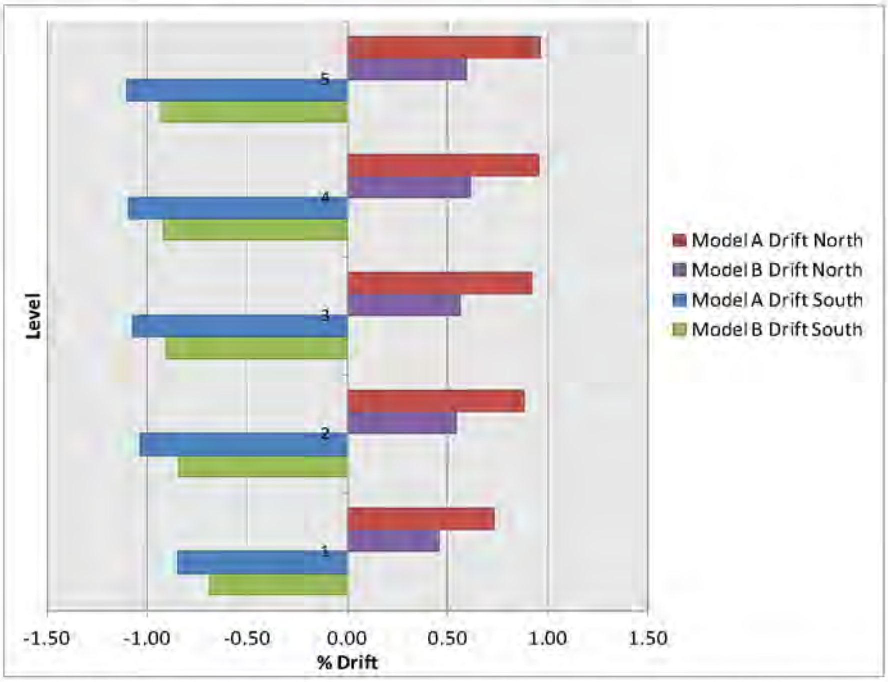


Figure 126 - Frame F north/south maximum storey drifts - 4 September Darfield Earthquake

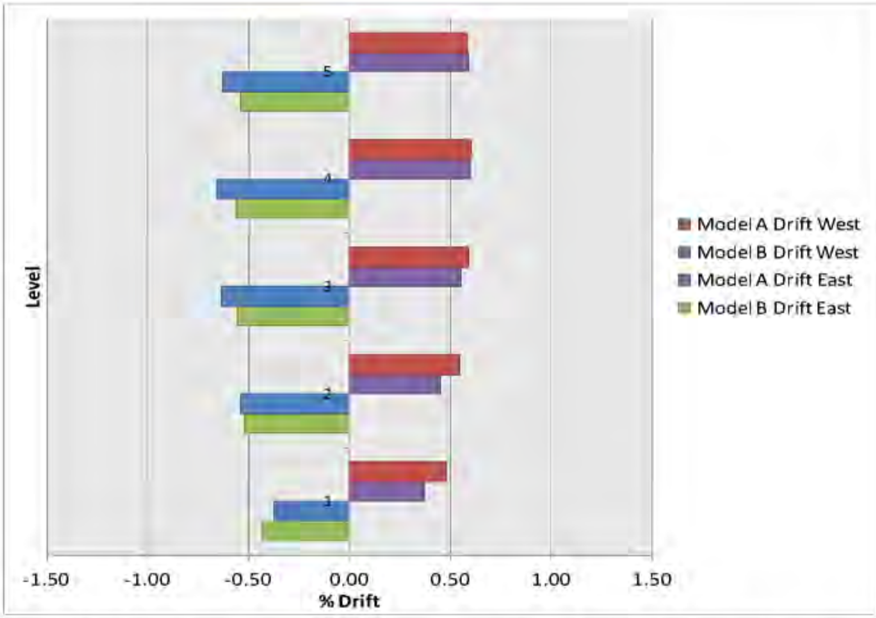


Figure 127 - Frame 1 - east/west maximum storey drifts - 4 September Darfield Earthquake

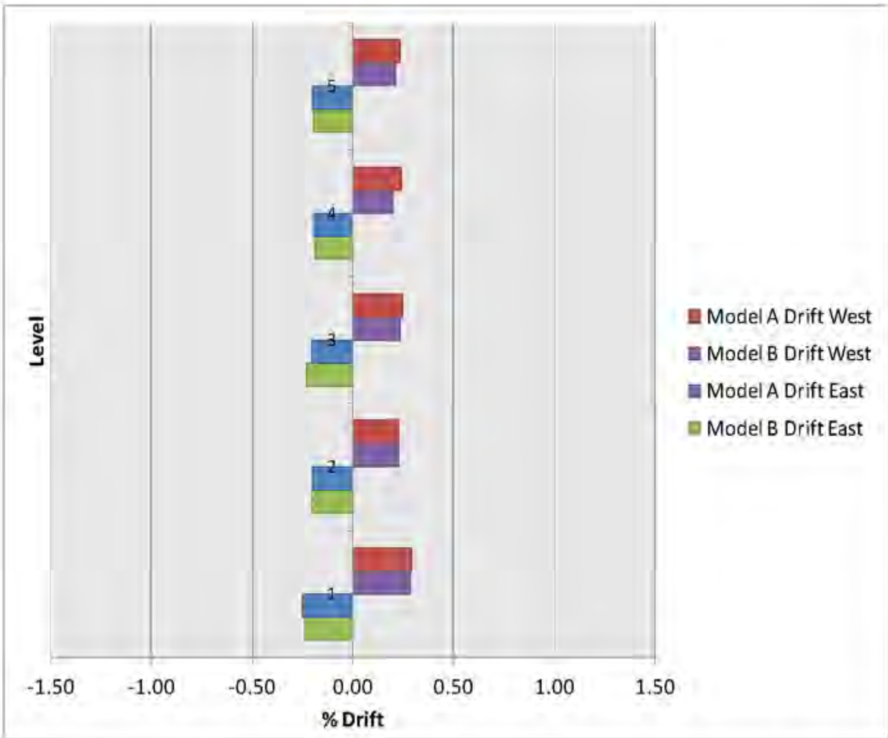


Figure 128 - Frame 4 east/west maximum storey drifts - 4 September Darfield Earthquake

Comparison of the results for storey drifts for the three different earthquake records adopted for the February Aftershock can be seen in Figure 129 to Figure 131. Storey drifts are shown to be somewhat higher in the north-south direction for the CHHC record and higher in the east-west direction for the CCCC record. It was recommended by T&T when carrying out time history analysis for assessments to average these results.

For the February Aftershock maximum storey drifts predicted by the NTHA are shown in Figure 129 to Figure 132 to be up around 3% (+/-100mm) in the north-south direction and also in the east-west direction along grid 1.

In the east-west direction along grid 4 the storey drifts are predicted to be around 1% (+/-32mm). This variation in drift between grids 1 and 4 reflects the severe plan irregularity and the torsional behaviour discussed earlier.

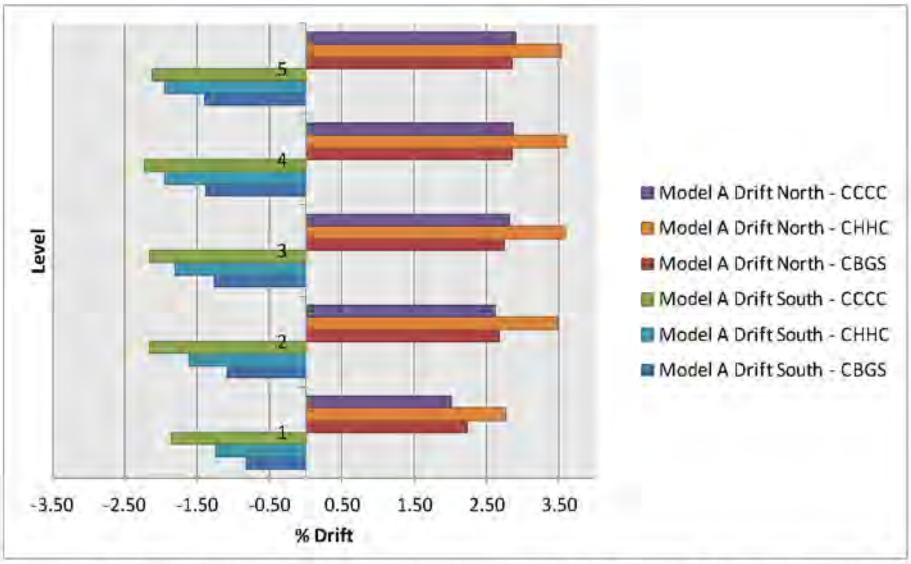


Figure 129 - Frame A north/south maximum storey drifts - 22 February Lyttelton Aftershock

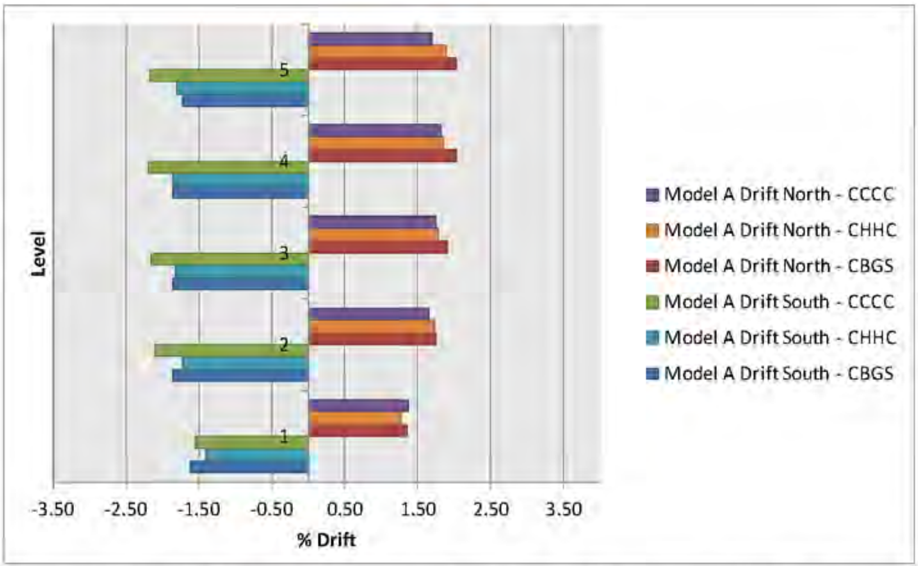


Figure 130 - Frame F north/south maximum storey drifts - 22 February Lyttelton Aftershock

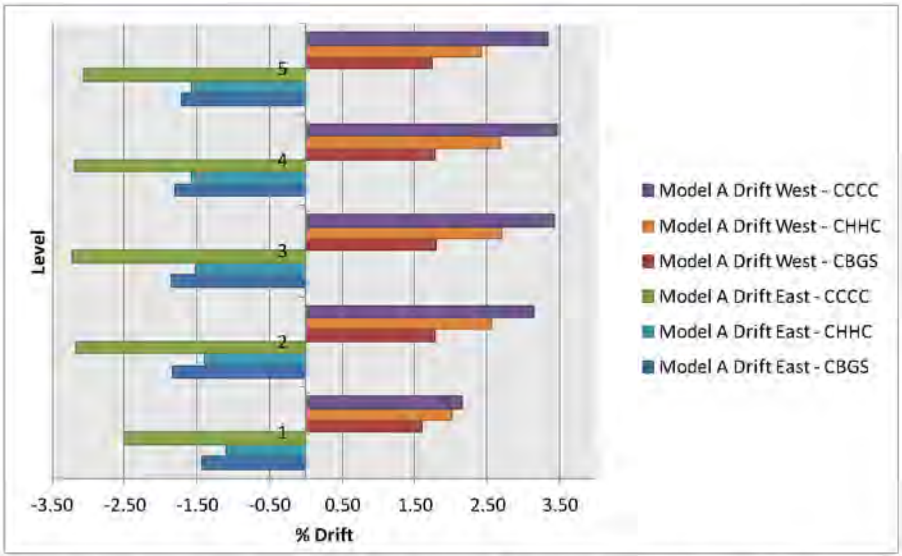


Figure 131 - Frame 1 east/west maximum storey drifts - 22 February Lyttelton Aftershock

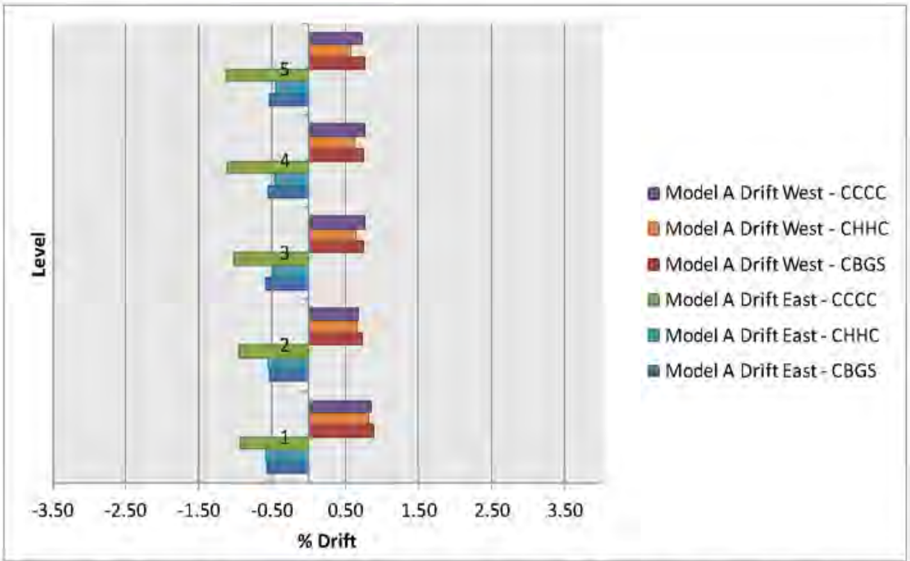


Figure 132 - Frame 4 east/west maximum storey drifts - 22 February Lyttelton Aftershock

Overall, taking into account the large number of input variables and assumptions the level of variation between the results from the three earthquake records used is considered to be not unusual for NTHA. Most of the following detailed results for the February Aftershock are reported for the CBGS record only although comparative results for the other two records were studied and the average result from the three records used where appropriate.

EFFECTS OF MASONRY INFILL WALLS

It has been shown in Figure 125 to Figure 128 that the masonry infill walls, if engaged with no effective separation, would generally have caused a reduction in storey drifts for the September Earthquake CBGS record. This reduction in drift occurs because of the additional torsional resistance of the masonry acting in tandem with the concrete shear walls and the concrete frame.

Figure 133 below is a plot showing the shear force in a typical 2.3m wide masonry infill panel over the duration of the September Earthquake CBGS record. It shows that the masonry panel, as modelled, would have reached its bending limited shear capacity of ±100 kN in a number of cycles. There were nine masonry panels on grid A at each floor level, meaning that the total storey shear contribution from the masonry was limited to 900kN (0.027g) in the model, limited by cantilever bending capacity of the individual panels.

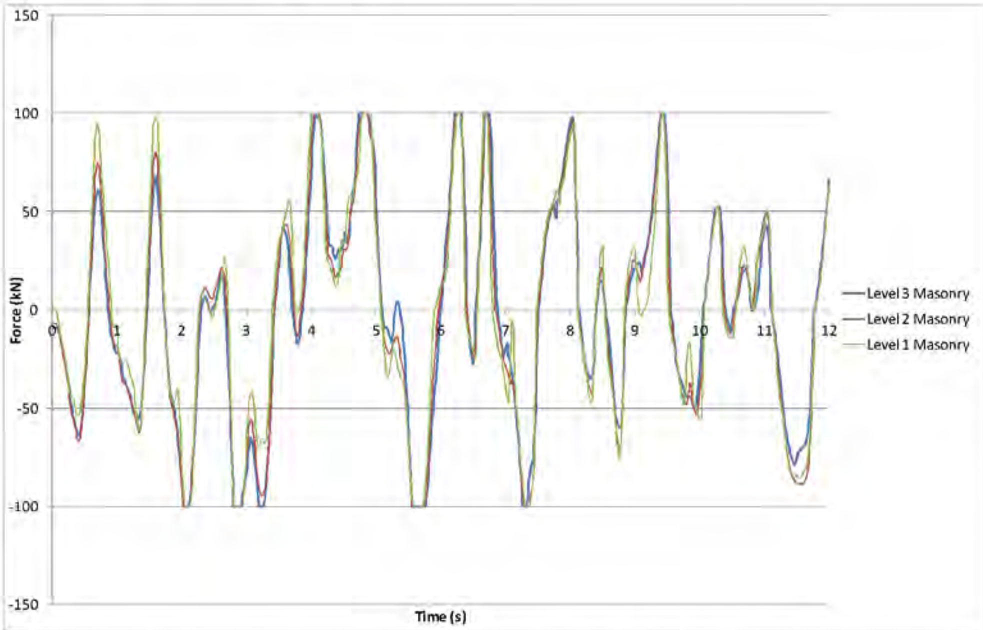


Figure 133 - Shear Force in typical 2.3m wide masonry infill panel, September Darfield Earthquake CBGS. This shows that he masonry panels reached their shear resistance of ±100kN as limited by flexural capacity on a number of cycles.

INELASTIC DEMANDS FOR THE SEPTEMBER EARTHQUAKE

The inelastic demands indicated by the NTHA for the September Earthquake, sometimes referred to as the September earthquake, were compared to the damage reported by the OIE, who carried out the post-September Earthquake damage assessment for the building owner.

The results of the NTHA indicated that inelastic demand from axial actions and bending of the North Core and the South Wall may have occurred in the lower part of level 1. From the NTHA for September Earthquake CBGS, the maximum vertical tensile strain predicted by the analysis in the bottom metre of the grid D wall was 9.7mm/m, and for the South Wall was 6.7mm/m. In other words the steel in the bottom metre of the South Wall was predicted to stretch by up to 6.7mm during the earthquake, which would lead to cracking in the concrete with the sum of all the crack widths over that bottom metre also adding up to 6.7mm. Yield strain in the steel would be approximately 2.2 mm/m. With Model B the corresponding maximum strain at the bottom of the South Wall was 4.7mm.

To correlate damage (crack widths) reported after the event with the NTHA results it is also instructive to a review the variation in strain over time. Figure 134 below, is a plot of the strain at the base of the southern coupled shear wall at the eastern face, for Models A (no masonry) and B (with masonry). It shows the peak strains of 6.7mm/m and 4.7mm/m described above at between nine and ten seconds into the record. It also shows the variation of strain over time and indicates a much reduced strain, between 0.5mm/m and 1.3mm/m at the end of the portion of record analysed.

After the September Earthquake the OIE reported that diagonal shear cracking and cracking of construction joints has occurred in the shear walls. The OIE believed there had been no yielding of the reinforcement in the walls and that structurally their integrity was still sound.

For the South Wall, one diagonal crack was reported as being visible on the outside of the ground storey, just below the fire escape landing (approximately 1m above ground level). The OIE also reported that the rough texture of the finish on the wall made it difficult to detect any cracking on the outside face, and that the inside of the wall at level 1 was strapped and lined with plaster board. On level 2, the OIE reported that the inside of the South Wall was finished with a thin skim coat of painted gypsum plaster, and that some diagonal cracks could be clearly seen in the gypsum plaster and measured up to approximately 0.2mm in width.

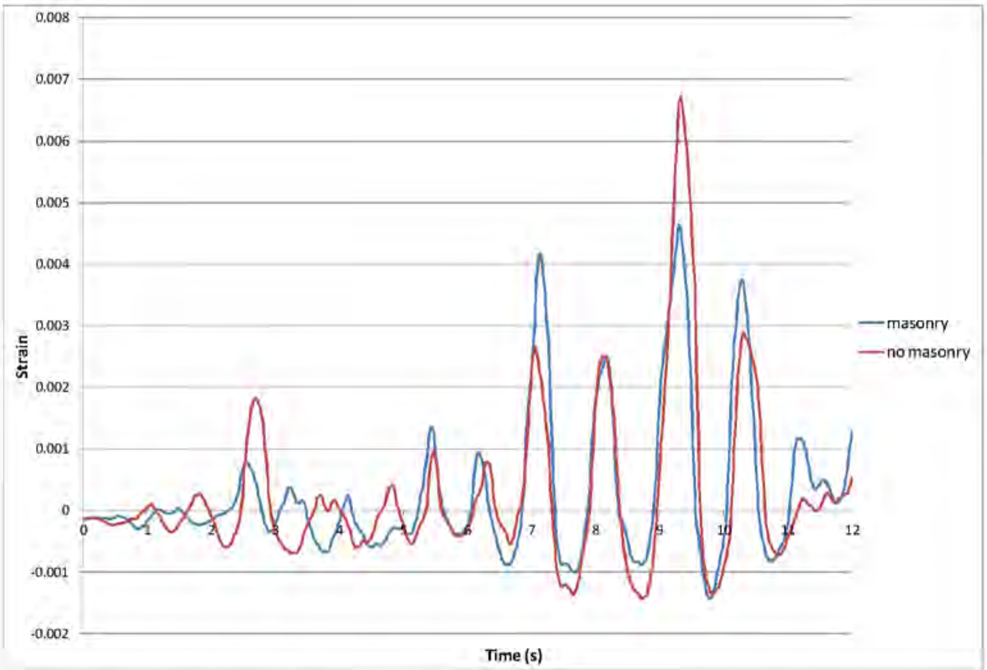


Figure 134 - Strain at base of South Wall at eastern face, September Darfield Earthquake CBGS

The NTHA for the CBGS September Earthquake indicated that one of the coupling beams in the South Wall was predicted to yield at level 2, with a maximum strain of 3.5mm/m for Model A and 2.5mm/m for Model B. The OIE did not report any cracking to coupling beams after the September Earthquake.

The development of column 'hinging' at nominal bending strength, was also predicted from the CBGS September Earthquake NTHA, with column hinges developing in the upper level columns on grid F first, and then progressing to other locations and to lower levels. Up to 10 columns were predicted to develop their nominal strengths in Model A and three columns in Model B. The same effect as can be seen for the South Wall in Figure 134 would also apply for gravity loaded columns, with strains (or crack widths) at the end of the earthquake likely to be less than the peak response.

ASSESSMENT OF FLOOR DIAPHRAGM CONNECTIONS

The NTHA for CBGS September Earthquake predicted that several of the Drag Bars on grids D and D/E would disconnect from the floor diaphragms, starting at around nine seconds into the record. The connection forces were found to have exceeded the modelled upper bound tensile strength of the Drag Bar connections on grid D and D/E.

The floors surrounding the lift core and the Drag Bars were not reported as having been inspected following the September Earthquake and so this aspect of the damage prediction cannot be verified against the actual damage that occurred, if any; Vertical cracking in the junction between the lift walls and partitioning was observed.

For the February Aftershock CBGS event the NTHA predicted that Drag Bar connections on grids D and D/E all disconnected at between 2.3 and 2.6 seconds into the record. The remaining slab connections to walls C and C/D were also found to be over-stressed once the Drag Bar disconnections occurred, and so the floor diaphragms may have disconnected from the North Core completely had this failure mechanism been modelled.

Although the NTHA predicts the disconnection of the Drag Bars from the floors, this needs to be considered in light of the particular structural configuration and the analysis assumptions and reconciled with observations of the collapse debris on site. The NTHA model indicates that there would have been considerable interaction between the individual walls in the North Core and the connecting floor diaphragms. Therefore the analysis results were sensitive to the assumptions made about the stiffnesses and strength of these connections. Also, in practice the 'disconnection' of the floors from the Drag Bars may have required considerably more elongation and slip than the 2mm to 3mm modelled.

To investigate the behaviour of the structure, without disconnection from the North Core, and to enable quantification of the peak diaphragm actions, another NTHA run was completed using the CBGS February Aftershock record. Here the Drag Bars remained connected with unlimited tensile capacity at levels 4 to 6. When the results from this analysis were compared with the original analysis (i.e. with the fused Drag Bars), it was found that the differences were small as far as storey drifts were concerned.

It is interesting to see in Figure 135 and Figure 136 that the maximum calculated total diaphragm connection force to the North Core exceeded 3500kN in tension (which is negative in the figures) at level 3 (0.61g) using the full record and without Line A masonry. This is nearly five times the design tension diaphragm connection force of 0.125g that would have applied for level 3 from the Loadings Standard of the day NZS4203:1984. It is also noted from the figure that the diaphragm forces fluctuate at very high frequency, much higher than the natural periods of lateral vibration that varied between 1.0 and 1.3 seconds..

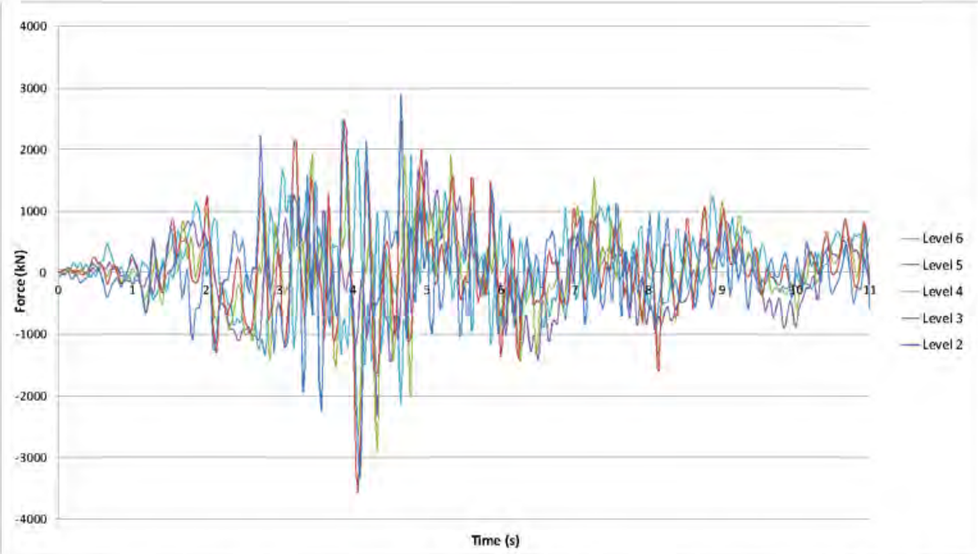


Figure 135 - North Core total diaphragm connection N/S actions (no disconnection), CBGS 22 February Lyttelton Aftershock.

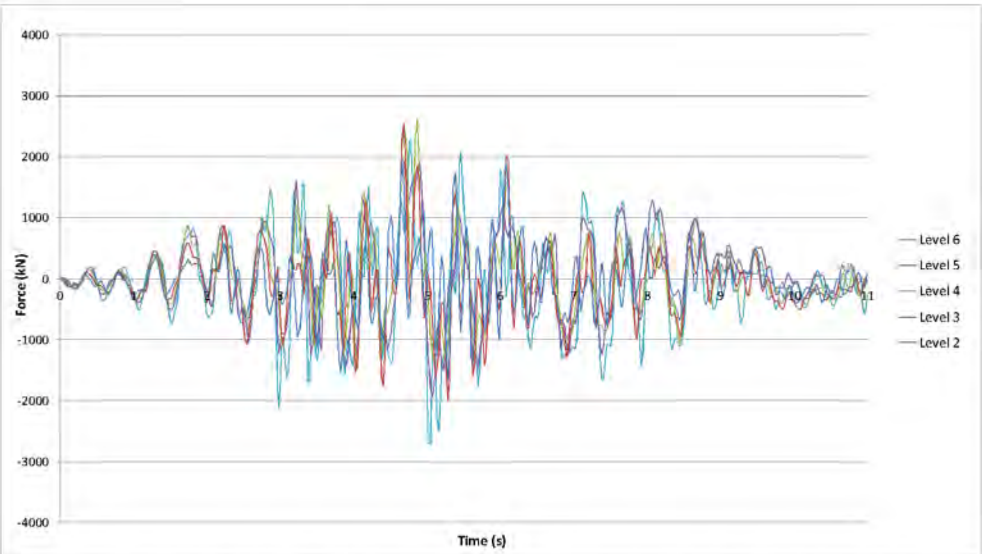


Figure 136 - North core total diaphragm connection E/W actions (no disconnection), CBGS 22 February Lyttelton Aftershock

VERTICAL EARTHQUAKE EFFECTS

Most of the NTHA's were carried out with all the earthquake direction components acting simultaneously, i.e. north-south, east-west and vertical. To assess the effect of vertical accelerations separate NTHA's were undertaken using only the vertical ground acceleration components of the CBGS and CCCC February Aftershock records.

The maximum variation in axial force was obtained during the analysis for a selection of columns under the CBGS and CCCC records. These analyses showed up to +/- 100% variation in axial actions for the most heavily loaded columns at Level 1with the CCCC record generally giving higher column axial forces due to vertical accelerations than CBGS.. The maximum axial action variation generally does not occur at the same time as the maximum horizontal actions, however the interaction of vertical and horizontal components is likely to occur at various times and could affect the column behaviour, in particular it could reduce the maximum storey drift that the columns can sustain.

In Figure 137 below the variation of axial action, bending moment and shear force for the 22 February CBGS record is plotted over time for one of the most heavily loaded columns at grid D2 at level 1. This shows the bending moment and the shear force in phase at around one cycle per second, and the vertical component superimposed at higher frequency. The flat spots on the bending moment and shear force curves represent the times when column hinging is occurring in the model. From this plot it will be appreciated that assessing the demand on, and capacity of the column at any particular instant in time is difficult.

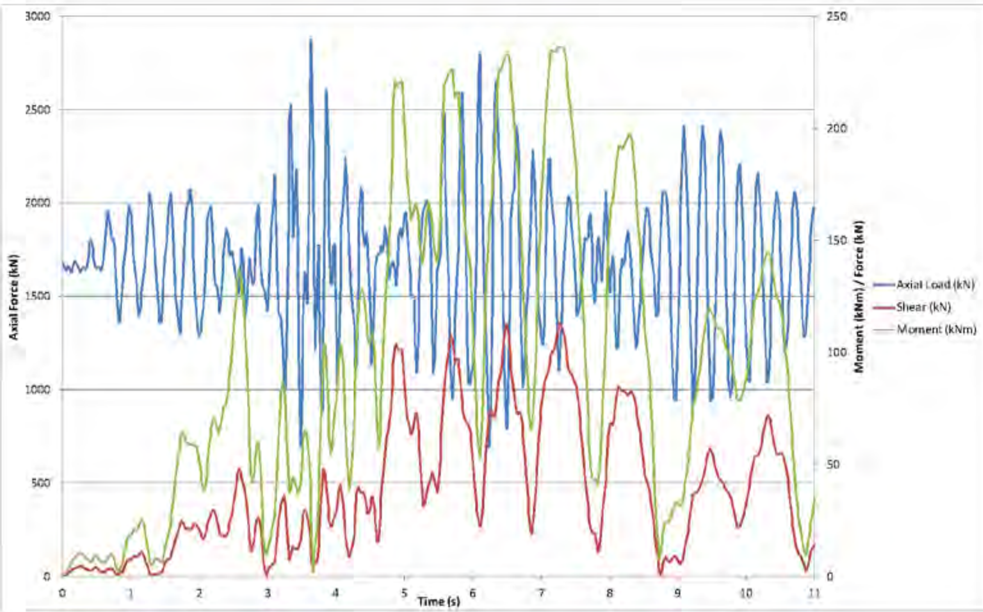


Figure 137 - Column Actions D2 Level 1, 22 February Lyttelton Aftershock, CBGS

To gauge the significance of the vertical accelerations in relation to column strength the M (moment) – N (axial action) interaction diagram shown in Figure 138 was generated. It should be noted that Figure 137 and Figure 138 show the actions over time for the CBGS record, and that the CCCC record exhibited somewhat higher sensitivity to vertical accelerations.

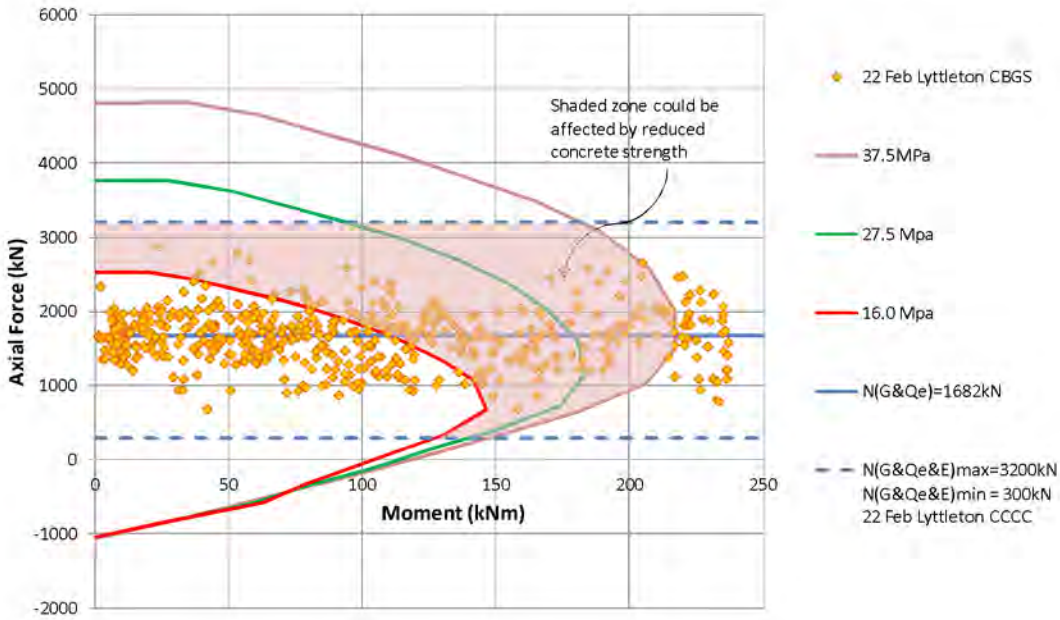


Figure 138 - Column D2 Level 1 M-N Interaction diagram. (Fy=448 MPa, phi=1.0, no masonry.)

February Aftershock CBGS

Points that can be observed from Figure 138 include:

- The M-N interaction curves have been drawn for three different concrete strengths 37.5 MPa, 27.5 MPa and 16 MPa, representing a range of concrete compression test results. The specified concrete strength for this column was 35 MPa, for which an expected strength of 37.5 MPa was used in the NTHA. The lowest strength found from material testing of column remnants was 16.0 MPa.

- The solid horizontal blue line is the gravity axial action on the column, 1682 kN. Note column D2 is one of four columns in the building with the highest gravity compression action.

- The dashed horizontal blue lines show the maximum and minimum axial action from the NTHA (CBGS 22 February Aftershock), including vertical earthquake effects.

- The data points marked by green, red and gold diamonds are the moment and axial action in this column at each time step from the NTHA (CBGS 22 February Aftershock record) prior to, during and following the modelled disconnection of the Drag Bars. It should be noted that the data points plotted are for the column modelled, i.e. with 37.5 MPa concrete, and these will be shown in the correct proportion to the 37.5 MPa interaction curve. If the concrete strength for this column had been lower, for example 27.5 MPa then the response would be slightly less and so the relationship of the data points to the 27.5 MPa interaction curve will have been slightly over-stated in the figure.

- Where the data points go outside the interaction curve at the right hand side of the chart this is mainly due to cyclic behaviour in the column hinges beyond the nominated failure criterion of Ecu = 0.004 where, due to the hinge modelling parameters chosen for the NTHA, the column moments are reported as being slightly higher than the theoretical value if failure had not occurred. Also, column hinge capacities have been based on the average gravity axial load and the hinges do not automatically take account of varying axial loads during the analysis. This does not have a significant effect on the storey response and has been taken into account in the assessment.

- The shaded area represents M-N combinations from the NTHA that would be affected by reduced concrete strength, i.e. there would be further hinging and potential column failures in this area if these reduced concrete strengths had been present.

- The unshaded area within the 16 MPa interaction curve represents M-N combinations that are within the admissible range for either concrete strength.

Figure 138 indicates that vertical earthquake effects alone may not have been enough to fail the columns if they had been constructed at the specified strength. However, vertical earthquake effects in combination with column actions resulting from lateral drift are significant and may have contributed to column failures, particularly if in combination with reduced concrete strengths.

ASSESSMENT OF CRITICAL COLUMNS

General

From the NTHA using the September Earthquake CBGS record, ten columns (out of 120) were predicted to develop nominal strength hinging in Model A, and four columns were predicted to develop nominal strength in Model B (with the masonry infill walls fully engaged). Column hinging was predominantly in the eastern frame, on line F, and initiated in the high level columns, progressing to lower levels as the displacement demand increased. This sequence of column hinge formation was also observed in analyses with the other two earthquake records.

The February Aftershock CBGS NTHA indicated that up to 90 columns (out of 120) yielded and deformed plastically in Model A. Taking into account the non-ductile detailing of the columns, this widespread hinging behaviour points to the potential for column failure. Many columns were calculated to reach their capacity at similar times, and this makes the identification of critical columns very difficult.

Potential Failure Criteria and Critical Columns

Calculations have been carried out in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) detailed assessment guideline of 2006 to assess the storey drifts that could lead to failure of columns by two criteria, as follows:

The first and critical, axial load and flexure failure criterion was taken to be when maximum concrete compression strain ec exceeded 0.004. The 0.004 maximum compression strain figure was assessed as being a critical condition for these columns because the small diameter and widely spaced spiral transverse reinforcement was not sufficient to provide effective confinement.

The spandrels were considered capable of lowering the drift capacity of columns governed by this Ecu = 0.004 criteria.

The second criterion, which in the end was considered less likely to be critical, was the NZSEE lower bound shear strength criterion based on the following formula,:

$V_{LB} = 0.85(Vc + Vs + Vn)$

Where Vc = shear resisted by concrete mechanisms

Vs = shear resisted by transverse reinforcement; and

Vn = shear resisted as a result of the axial action

The lower bound NZSEE shear criterion may have been critical if the columns had been restrained by the Spandrel Panels, but the spandrels were assessed not capable of this.

Also the CTV columns appear to have lower concrete strength, less reinforcing steel and lower core section to gross area ratio than those used to validate the NZSEE shear strength criteria so may display different shear strength characteristics to those used in the validation of the NZSEE guidelines.

Critical Column Identification

Analyses showed that drift (i.e. lateral displacement) demands were generally greater at the upper levels of the structure than at lower levels. For drifts in the north-south direction, the Line F (east side) columns were more vulnerable than columns on other lines because they formed a moment frame with the stiff façade beams and they may also have interacted with the Spandrel Panels. Drift demands in the east-west direction were greater towards the southern side of the building, being more distant from the stiff and strong north core walls. Line 1 (south side) columns also formed a moment frame with the stiff facade beams, and would have been subject to high drift demands in the east-west direction. However the columns on Line 1 were protected to some extent by the South Wall, which was one of the last components of the structure to fall, and so were considered to be less vulnerable than the columns on Line F.

The columns on Line 2 were seen as potentially vulnerable. While the lateral displacements (drifts) may have been less than on Line 1, these internal columns supported additional gravity load (with floor slabs all around). Thus it was recognised that the reduced drift demand could have been matched or exceeded by a reduction in capacity to sustain the drifts imposed. They also may have been more vulnerable to vertical acceleration effects due to the higher axial loads carried.

Taking the above factors into account, critical columns were identified at grid locations F2 and D2 and the ratio of drift demand to column capacity at various levels was examined as shown in Table 11 and Table 12. In these tables the first yield, nominal strength and Ecu = 0.004 drift capacities have been calculated from static pushover analyses and the drift demand has been taken from the NTHA noted. The terms first yield, nominal strength and Ecu = 0.004 are defined in the Glossary.

Level	Axial Load (kN)	Drift Capacity (% of storey height)			E/W Drift Demand 22 Feb CHHC (%)	Ratio Demand / Capacity (Ecu=0.004)
		First Yield	Nominal Strength	Ecu=0.004		
5	324	0.71	0.85	1.31	1.65	1.26
4	681	0.85	0.90	1.20	1.85	1.54
3	1038	no yield	0.89	1.10	1.86	max. 1.69
2	1328	no yield	0.95	1.08	1.76	1.63
1	1682	no yield	0.90	0.96	1.46	1.52

Table 11 - Drift demand vs. capacity for columns at grid D2

This process resulted in the identification of two "indicator" columns – one at level 3 at grid position F2 and one at level 3 at grid position D2. These particular columns were chosen because, based on average drifts from the NTHA, and with expected concrete strengths; the ratio of lateral displacement demand to column capacity would be greatest in these columns.

In making these comparisons, it was recognised that low concrete strength, vertical acceleration effects, diaphragm separation and/or a different level of interaction with a Spandrel Panel could mean that a column in another location could have initiated failure

Analysis Results

Level	Axial Load (kN)	Drift Capacity (% of storey height)			E/W Drift Demand 22 Feb CHHC (%)	Ratio Demand / Capacity (Ecu=0.004)
		First Yield	Nominal Strength	Ecu=0.004		
5	230	0.67	0.86	1.40	2.03	1.45
4	462	0.77	0.94	1.34	2.04	1.52
3	695	0.91	0.98	1.30	1.91	1.47
2	910	0.91	0.96	1.21	1.85	max. 1.53
1	1154	0.96	0.97	1.14	1.62	1.42
Check effect of upper bound spandrel interaction on drift capacity of columns (refer note* below):						
5-S	230	0.57	0.73	1.11	est. 2.03	1.83
4-S	462	0.63	0.76	1.06	est. 2.04	max. 1.92
3-S	695	0.72	0.80	1.02	est. 1.91	1.87
2-S	910	0.81	0.88	1.05	est. 1.85	1.76
1-S	1154	0.95	0.92	1.09	est. 1.62	1.49

Table 12 - Drift demand vs. capacity for columns at grid F2

***Note to Table 12: The drift capacity values in the bottom row, denoted Levels "1-S", are from the pushover analysis with full spandrel interaction (i.e. "S" is for spandrels). Potential beam-column joint failure has also not been included in these Figures.**

The time history plots of column drifts, (Figure 139 to Figure 141), have been developed from the September Earthquake and February Aftershock NTHA's using the CBGS and CHHC ground motion record and Model A (no masonry) as noted below each figure. The potential column and Drag Bar connection failure criteria outlined above are superimposed on the figures. Points to note on these figures include the following:

The column failure analyses here consider no masonry infill, expected concrete strengths and average gravity compression actions on columns. The potential variations due to variable concrete strength and vertical earthquake accelerations are not shown in these plots - but were calculated to result in reduced drift capacities, particularly if considered together.

The vertical axis shows the amount of inter-storey displacement (drift). The horizontal axis is the time from start of shaking (as input into the analysis). The wavy lines plot the drift level over time and are based on application of the full ground shaking record in the analyses. This drift is a key measure of demand on the column.

An important feature of this analysis was that for heavily loaded columns, the displacement to cause yielding of the main column bars was close to the displacement to cause failure. This is significant because it indicates that significant displacements, such as occurred in the September Earthquake, could be sustained with little evidence of distress, yet collapse could occur due to a relatively small additional displacement.

The horizontal lines represent the estimated capacity of this columns to sustain the drift without failing according to the Ecu=0.004 criteria (assuming expected concrete strengths and without vertical earthquake effects). Ranging between "no interaction with the spandrels" (higher value) and "full interaction with the spandrels". The areas where the drift has exceeded the estimated capacity are shown shaded orange or yellow respectively. The band showing the range of capacities would be wider if allowance was made for the effect of variable concrete strength and vertical earthquake forces in the column.

Key points to note from Figure 140 and Figure 141 are that, for the September Earthquake the maximum displacement demands are about half those calculated for the February Aftershock. Although there are two places where the September Earthquake displacements are shaded, only one of these is for the north-south drift. There are no cases where they exceed the maximum assessed capacity. The February Aftershock demands have many "excursions" shown shaded and three that exceed the maximum value by a noticeable margin.

Similar plots were made for column D2 at Level 3 as shown in Figure 139, with similar conclusions being reached regarding the likely performance of this column in the February Aftershock.

Such comparisons provide valuable insights into the relativity of demand and capacity, but must be interpreted with care.

These comparisons give some indication of the challenges of determining which column or mechanism initiated failure. However, the plots indicate clearly that there is a strong likelihood that the demands of the February Aftershock were enough to cause column failure, whereas the demands of September Earthquake were not.

Although the vertical accelerations at the site could have been high during the February Aftershock, the analyses completed indicated column failure was possible without the additional effects from vertical accelerations.

Displacements for column D2 on Level 1 (ground floor) (for the full record) were well below the assessed capacity of this column for the September Earthquake and only marginally exceeded the capacity for the February Aftershock analysis. This is a broad indication that this column is less likely to have been the initiator of the collapse. However, this possibility cannot be ruled out because this column may have had lower than average concrete strength and/or suffered more from the effects of the considerable vertical forces generated in the February Aftershock.

An illustration of the effects of vertical acceleration on column drift capacity is shown in Figure 142.

Figure 139 for the column at Grid D2 at Level 3:

1. The east-west drift is key here (the green wavy line), because that is the direction of the floor beams on line 2. The coincident north-south drift and the resultant drift are also shown superimposed for information..

2. The earthquake record used here is CHHC, because that is the record that gave the average maximum drift in the east-west direction.

3. The maximum concrete compressive strain limit of 0.004, which indicates ultimate curvature for an unconfined column, is calculated to have been reached at a drift of around 1.1% (the horizontal red lines) which was well exceeded in the February Aftershock, as indicated by the orange shading.

4. The times where the floors were calculated to have disconnected from the steel angle Drag Bars on the lift shaft walls at levels 4 to 6 are shown by the dashed vertical lines

Figure 140 and Figure 141 for the Column at Grid F2 at Level 3:

1. The north-south drift is key here (the blue wavy line) because that is the direction of the stiff façade beams that form a moment frame with these columns and it is also the direction of potential interaction with the precast spandrels. The coincident east-west drift and the resultant drift are also shown superimposed for information.

2. The earthquake record used is CBGS because that is representative of the average response in the north-south direction.

3. The maximum concrete compressive strain limit of 0.004 is calculated to have been reached at a drift of around 1.3% (the red horizontal lines) with no spandrel interaction, or at 1.0% (the orange horizontal lines) in the case of a Spandrel Panel adjacent to the column with no initial gap. 1.3% drift was not exceeded using the September Earthquake record, but it was well exceeded using the February Aftershock record, as indicated by the orange shading.

4. The times when the floors were calculated to have disconnected from the steel angle Drag Bars on the lift shaft walls at levels 4 to 6 are shown by the dashed vertical lines.

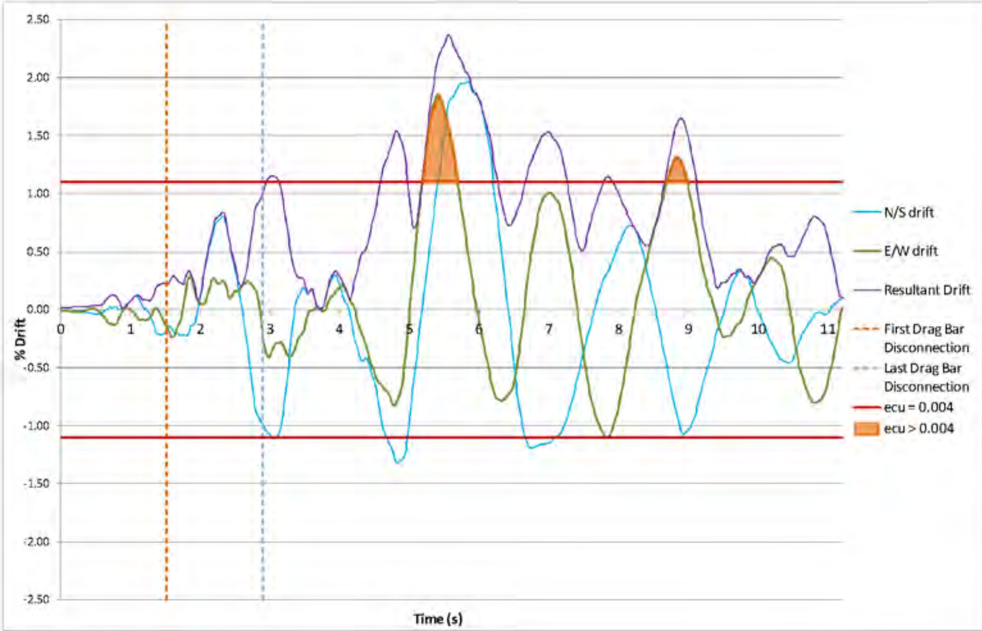


Figure 139 - Column D2 Level 3, CHHC, 22 February Lyttelton Aftershock, no masonry.

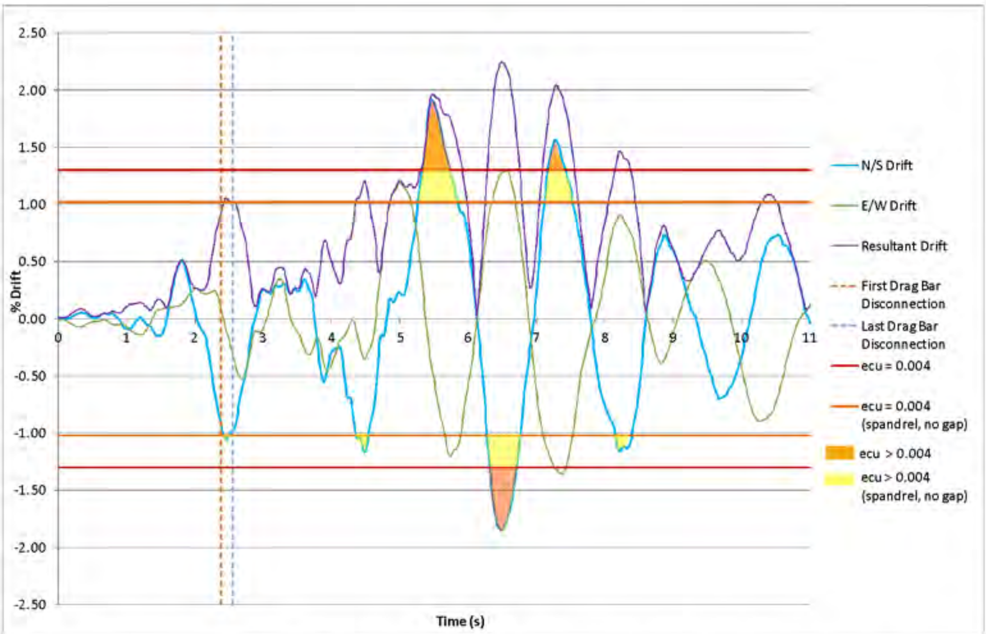


Figure 140 - Column F2 Level 3 Drifts - CBGS, 22 February Lyttelton Aftershock, no masonry.

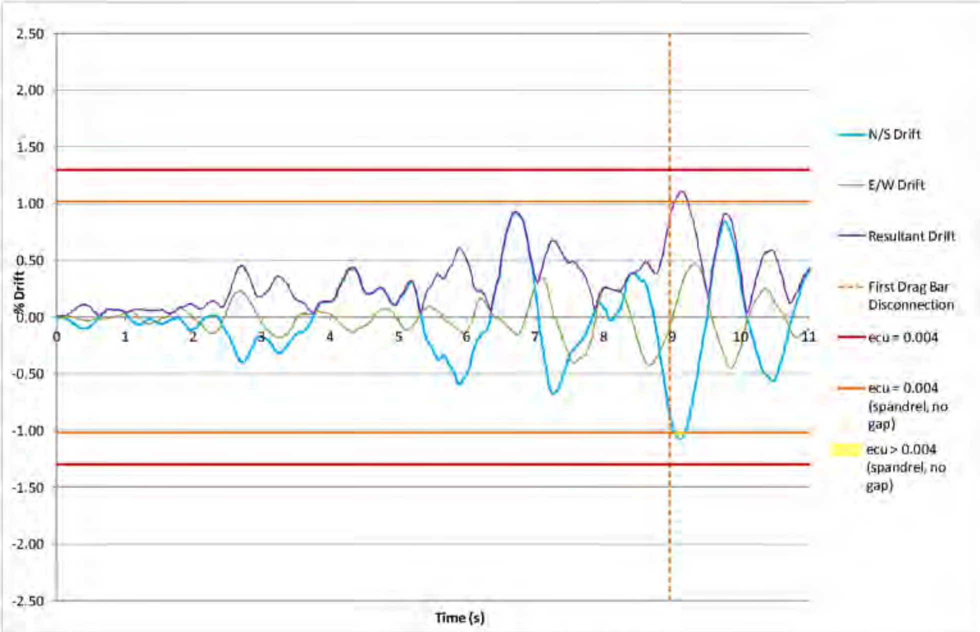


Figure 141 - Column F2 Level 3 Drifts - CBGS, 4 September Darfield Earthquake, no masonry.

The potential effect of vertical accelerations on column drift capacity is indicated in Figure 142 . The solid lines are a plot of the failure drift capacities of the indicator columns at grids F2 and D2, according to the criteria of 0.004 maximum compressive strain as determined from the non-linear pushover analysis of the whole structure (assuming expected concrete strengths and without vertical acceleration effects). The solid coloured markers each represent one level of the particular column, as indicated by the labels and show the drift capacity at the corresponding axial force. The hollow markers, labelled "L3+V" show the revised estimate of drift capacity, found by interpolation or extrapolation as indicated in the figure, for the Level 3 columns when the maximum assessed vertical earthquake axial compression force is included. Similarly the hollow markers labelled "L3-V" show the estimated drift capacity when the minimum axial compression force is included. The maximum and minimum vertical earthquake force is from the NTHA analysis with vertical accelerations only, for the CCCC record of the 22 February 2011 event, which gave the largest variation of axial forces of the two analyses completed (CBGS and CCCC).

Note from Figure 142 that for the grid F2 column at Level 3, an increase in axial force of 60% reduces the drift capacity reduces from 1.3% to 1.15% which is a reduction of around 12% in drift capacity. Similarly for the grid D2 column at level three, a 100% increase in axial force reduces the drift capacity from 1.1% to 0.68% which is a reduction of around 38%. When there is a reduction in the axial force, the drift capacity increases as indicated by the L3-V markers in Figure 142.

Referring to Figure 137, the axial forces in column D2, from the NTHA with vertical accelerations, were seen to fluctuate at much higher frequency than the lateral modes of vibration. There was therefore significant variation in the axial force for any given cycle of lateral drift, with an increase in axial force being detrimental and a reduction in axial force being beneficial to the column drift capacity at any particular time step.

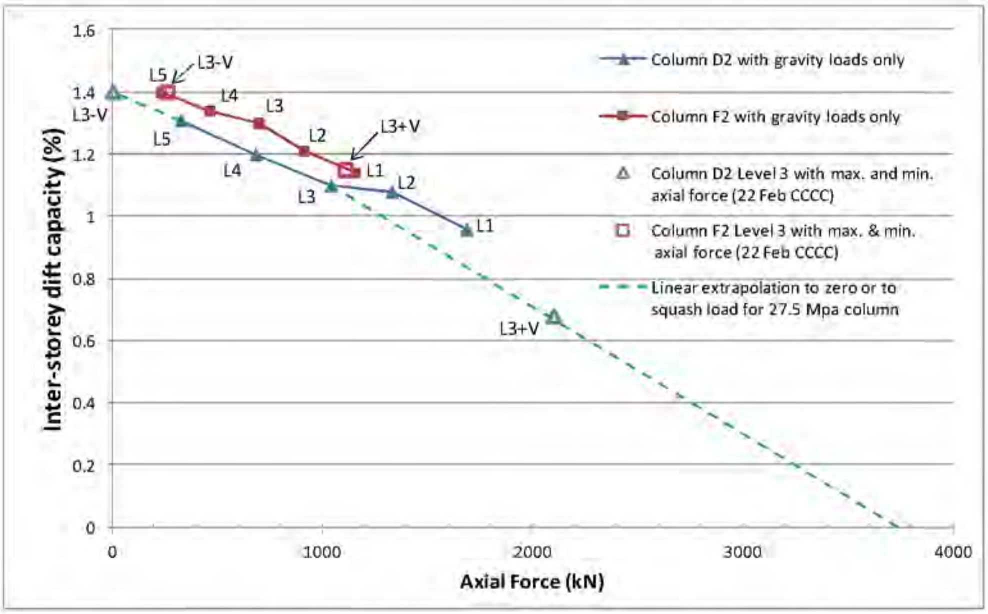


Figure 142 - Column F2 and D2 drift capacities to failure (Ecu = 0.004) (including the effect of maximum vertical earthquake force on Level 3 columns) Notes: Concrete strength as specified + 2.5 MPa. Linear extrapolation is indicative only.

ASSESSMENT OF BEAM-COLUMN JOINTS

Beams column joints were assessed as another potential critical weakness. Two aspects of the joints were considered possible causes of premature failure, these being the joint shear capacity and the potential pull-out of short hook anchorages. The corner column joints were considered to be most critical since they had beams incoming from two orthogonal directions, with top and bottom bar anchorages from each beam placing demands on the column joint zone. However the bottom bar anchorages at the other locations were detailed extremely short and may have been equally vulnerable.

The assessment method for joints is less well defined than for columns and the results are likely to vary more with axial action and concrete strength.

Given the greater uncertainties with analysis of the joints, and given the results that have come out of the column analyses, it was decided that limiting the assessment to columns would be sufficient for the purposes of this investigation.

CONCLUSIONS

NTHA has been used to evaluate the response of the CTV Building to ground motions recorded at nearby sites in the Christchurch CBD on the September Earthquake and February Aftershock. The results are subject to considerable uncertainty due to possible variations in the ground motion at the CTV site, real building response to ground motions, various assumptions made in the analysis, concrete strengths, Spandrel Panel gaps and other variables; however the analysis has indicated the following:

1. Maximum inter-storey drifts around 1% using the September Earthquake record and around 3% using the February Aftershock record. Inter-storey drifts were seen as the major factor leading to column collapse.

2. A highly irregular seismic resisting structure, with drifts for the February Aftershock at the east, south and west sides being two to three times the drifts along the north side of the building.

3. The masonry infill walls, if fully engaged in the seismic response, were seen to introduce additional plan irregularity and vertical irregularity to the system. However, at the same time they also generated additional torsional resistance in tandem with the concrete shear walls and the concrete frame. The overall effect of the masonry as calculated for the September Earthquake was generally to reduce storey drifts.

4. Some of the floor diaphragm connections were predicted to fail using the 4 September Earthquake records, and all of the floor diaphragm connections were predicted to fail early on using the February Aftershock records. However the timing in relation to column failure varied and the analysis was likely to be more sensitive than for columns.

5. Assuming expected concrete strengths, and without the effect of vertical accelerations or Spandrel Panel interference, the columns were predicted to survive the September Earthquake, but were predicted to fail in the February Aftershock by exceeding the drift capacity that would lead to ultimate compressive strain exceeding 0.004. This was considered to be a potentially critical condition given that the columns were essentially unconfined by the widely spaced spiral reinforcement.

6. Overall, the output of the NTHA analyses was not inconsistent with the reported condition of the building after 4 September 2010. The limited available evidence of the building condition after 4 September 2010 leaves room for a range of interpretations of the likely maximum displacements in the 4 September 2010 event. However, the conclusions drawn from the analyses are not particularly sensitive to the level of demand assumed by the NTHA, with indications that collapse could have occurred at lower levels of demand.

7. If full interaction between the columns and the precast spandrels was assumed then column failure could have been initiated earlier, by the 0.004 compressive strain. The compressive strain being a combination of axial and flexural compressive strains.

8. Vertical accelerations alone were considered not likely to have caused columns to fail, unless concrete strength in critical columns was extremely

low. However when combined with lateral drifts, vertical accelerations certainly could have contributed to column failure. Although the vertical accelerations at the site could have been high during the 22 February 2011 event, the analyses completed indicated that column failure was possible without the additional effects from vertical accelerations.

9. Concrete strengths were indicated from testing to vary widely and in many cases may have been below the minimum specified strengths at the time of the collapse. Low concrete strengths in critical columns, particularly if in combination with vertical accelerations would have led to premature failure by any of the above criteria.

APPENDIX E – ELASTIC RESPONSE SPECTRA ANALYSIS

EARTHQUAKE AND AFTERSHOCK RECORDS

Strong Motion Recordings

The nearest strong motion recordings of the three Canterbury earthquakes of 4 September 2010, 26 December 2010 and 22 February 2011 were downloaded from the GeoNet ftp site. (GeoNet is a collaboration between the Earthquake Commission and GNS Science that provides public access to hazards information including earthquake records at **www.geonet.org.nz/earthquake**).

The instruments are located at the following sites, and as shown on the map below in relation to the CTV site:

- Botanical Gardens (CBGS)
- Cathedral College (CCCC)
- Christchurch Hospital (CHHC)
- Rest Home Colombo Street North (REHS)
- Westpac Building (503A)
- Police Station (501A)

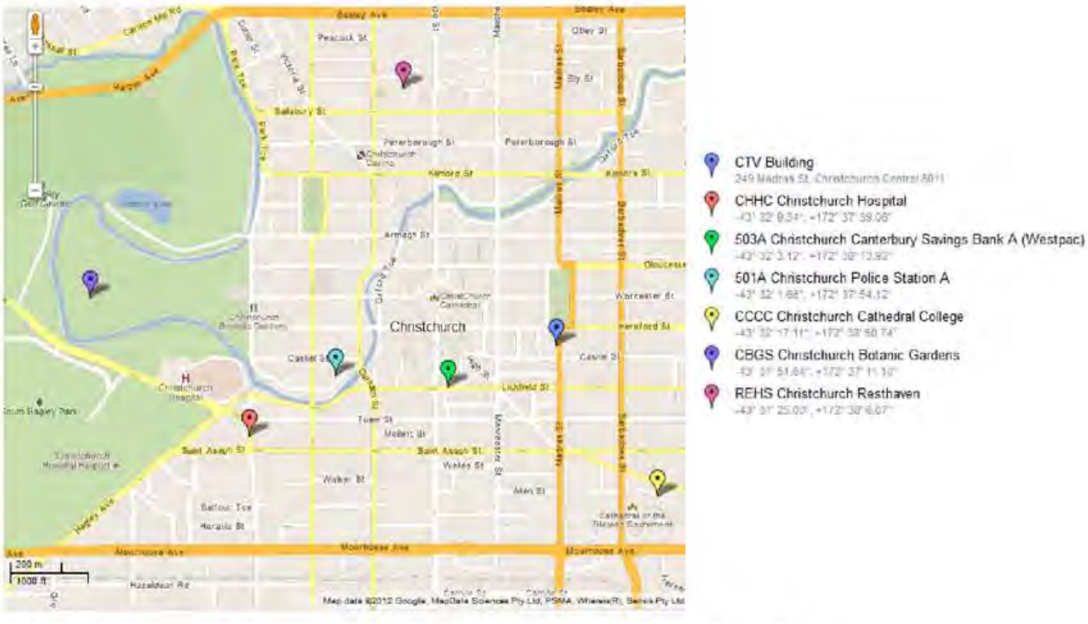


Figure 143 - Locations of Geonet strong motion recorders relative to CTV Building site

For each earthquake, or aftershock, the response spectra records have been converted into a 5% damped response spectrum by GNS.

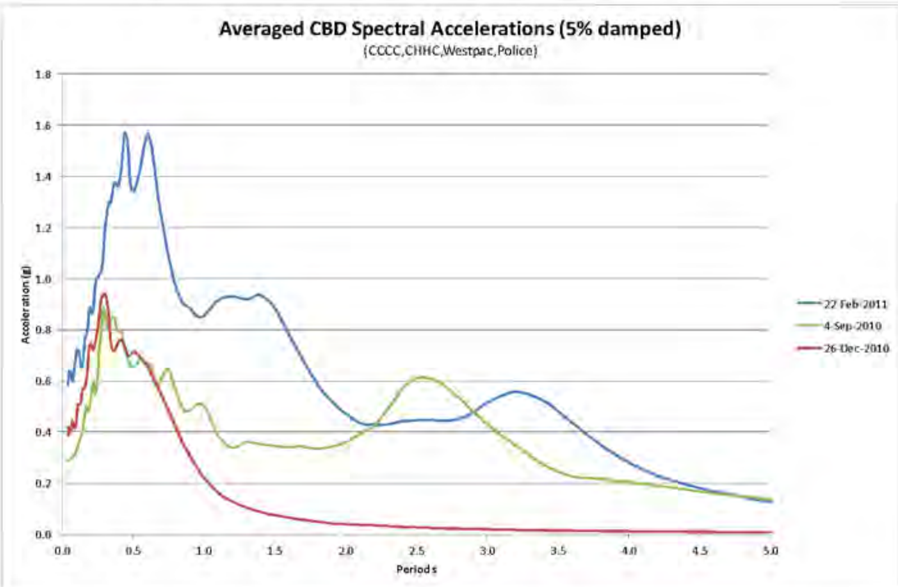
The direction of the axes of the instruments vary.

There is not sufficient detailed information about the ground conditions at the recording stations, or the ground conditions between the recorders and the CTV site, to be totally accurate about an equivalent record for the CTV site. However, since the stations effectively surround the CTV site on three sides at fairly close proximity then the records are very helpful and the best available to demonstrate the average level of ground motion.

It is important to recognise the difference between spectral records and the NZS 4203:1984 design spectra. Design spectra are scaled to provide a minimum specific level of design demand on the structure. The earthquake records on the other hand are not records corresponding to the demands experienced by the actual building It is possible that the building did not respond to the earthquake exactly as the computer model responded. The response of the site may also have been different to that reflected in the records used. It is likely that some level of calibration may be required to determine building response reliably from earthquake ground motion and response spectra. The level of calibration required is difficult to determine and beyond the scope of this investigation. It is likely to be dependent on the specific nature and configuration of each structure.

Averaged Resultant Response Spectra

Averaged resultant spectra were derived from records at Westpac Building, CCCC, CHHC and the Police Station as these were closest to CTV. The resultant of the accelerations from each orthogonal axis of the recording instrument at each recorded period was used as it is not possible to determine the sign of each component. Discretised linear curves were then fitted to the spectral plots to allow input of the spectra into ERSA software (Figure 144).



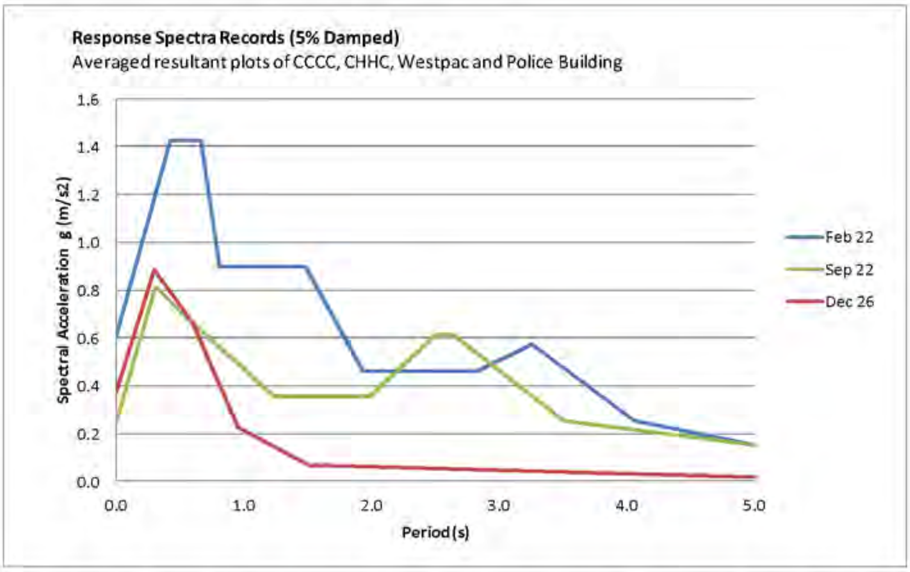


Figure 144 - Averaged resultant response spectral records (5% damped) from CCCC, CHHC, Westpac and Police building GNS records. The lower plot has been discretised into linear steps to facilitate use in ERSA.

The NZS 4203:1984 design spectra have been superimposed on these in Figure 145. The NZS 4203:1984 spectra have been scaled to ensure the base shear from ERSA corresponds to 90% of that derived from a static analysis in accordance with NZS 4203:1984. The earthquake records have been left unscaled.

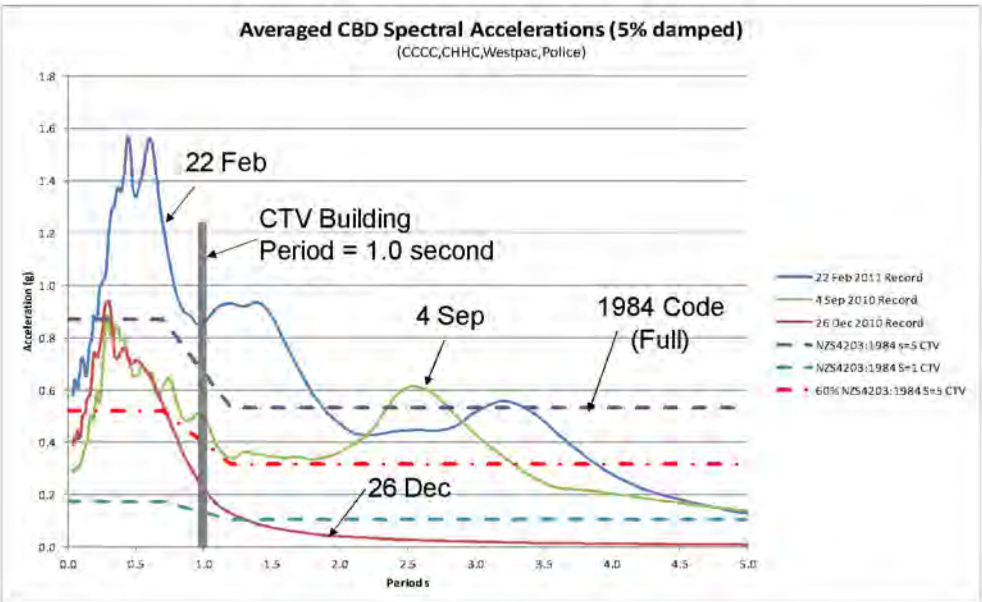


Figure 145 - Averaged CBD response spectra superimposed with design spectra for CTV Building according to NZS 4203:1984. The NZS 4203 spectra have been scaled by around 1.5 to achieve 90% of the first mode base shear derived from a static analysis in accordance with the requirements of NZS 4203:1984. The scaling factor required in NZS 4203:1984 varies depending on the direction of loading and configuration of the structure. The period for the CTV Building shown is approximate only and also varies with the structural configuration, and eccentricity and direction of loading. For the CTV Building this ranged from 0.79 to 1.22 seconds for the ERSA. Both the ERSA and NTHA calculate and combine the structural response in detail accounting for the various modes of vibration.

RESPONSE SPECTRA AND NZS 4203:1984 DESIGN SPECTRA

The February Aftershock spectra were plotted in Figure 146. The effect of damping on the spectra can be seen from the comparative plots.

The 20% damped response spectra drops below the S=5 NZS 4203:1984 ULS spectra.

The ductile design requirements of NZS 4203:1984 and NZS 3101:1982 were that the South Wall and North Core had dependable strength greater than the demands derived from application of the S=1 spectra. However the implication of those standards was that those elements should have been able to sustain the demands imposed from application of the S=5 ULS spectra in terms of inelastic drifts without collapse. This required satisfaction of the capacity design and detailing requirements of the standards.

In other words, if the structure had been regular and symmetrical, the structure was expected to be able to sustain the NZS 4203:1984 ULS drifts shown in Table 13 and

Table 14 without collapsing. However severe damage would be anticipated to have occurred at those drifts.

To calculate the drift demands for compliance with NZS 4203:1984 simply required that the drifts derived on the basis of S=1 demands be factored up by a K/SM factor of 2.75, equivalent to 55% of the S=5 ULS demands and checked against various limiting drift and detailing criteria. The current design standards use a similar approach but would require the S=1 drifts to be factored up by 5 and checked against different limiting drift criteria. In both cases the philosophy is one of equivalent displacement whether the structure responds elastically or inelastically to the imposed earthquake loadings.

The limitation of the equivalent displacement approach is that it assumes that the primary structural elements will develop compatible levels of inelastic demand so that a significant change in the distribution of demand between them does not occur to compromise the design.

The CTV Building is an example of where the significant irregularity and difference in strength and stiffness of the North Core and the South Wall made direct use of the equivalent displacement approach with ERSA potentially unreliable. This is an issue that needs to be better addressed in seismic design standards.

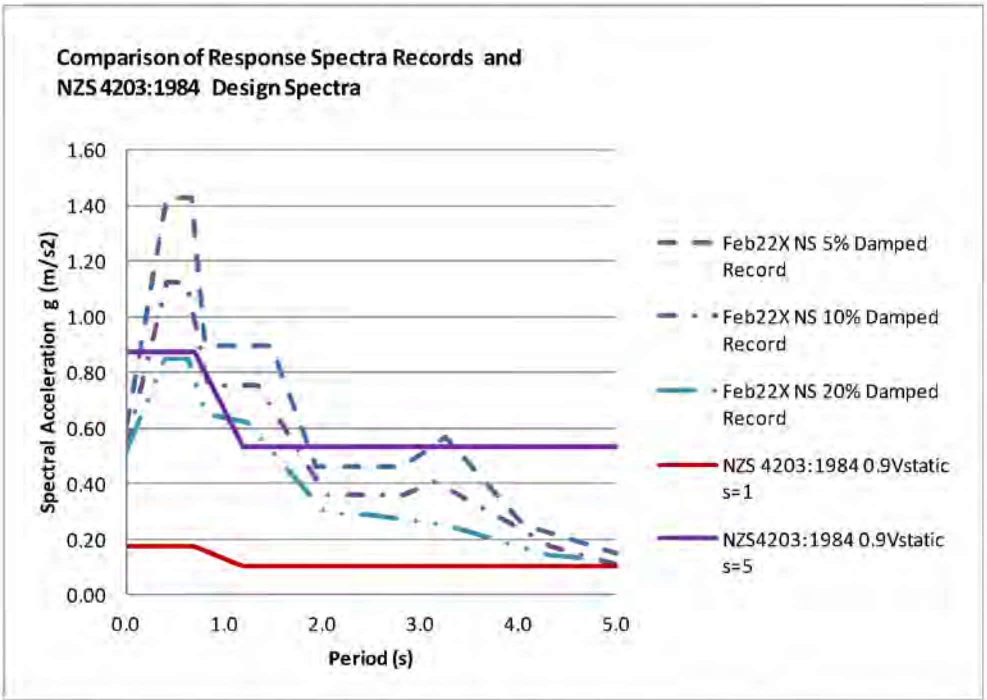


Figure 146 - Response spectra records for various levels of damping are shown in conjunction with the design spectra for the CTV Building from NZS 4203:1984. The North Core and South Wall were designed to have dependable strength exceeding the demands imposed by the S=1 spectra but were expected to be able to survive the drift demands from the S=5 ULS spectra without collapsing due to the provision of ductile detailing.

As a consequence the ERSA has only been used in this investigation for some limited purposes. These included identify compliance or to undertake checks consistent with the requirements of NZS 4203:1984. Comparative assessment of the level of demand implied by the earthquake spectra records compared to the NZS 4203:1984 design spectra was done. It was also used to indicate what the distribution of demand was on the various structural components prior to significant inelastic deformation developing. Beyond that the NTHA has been used to better assess inelastic structural performance.

ERSA MODELLING

Introduction

Linear 3D structural elastic response spectra analysis ("ERSA") using ETABS software was carried out. This was to enable design checks using the methods required in NZS 4203:1984 against Standard loadings and to investigate structural behaviour in various configurations. The ERSA was also used to compare the relative demands of the NZS 4203:1984 loadings to those implied by the response spectra records for the September Earthquake, and the December and February Aftershocks. The configurations included:

- Primary walls alone.
- Primary walls and the masonry infill wall on grid A.
- Primary walls and the masonry infill wall on grid A and secondary frames.

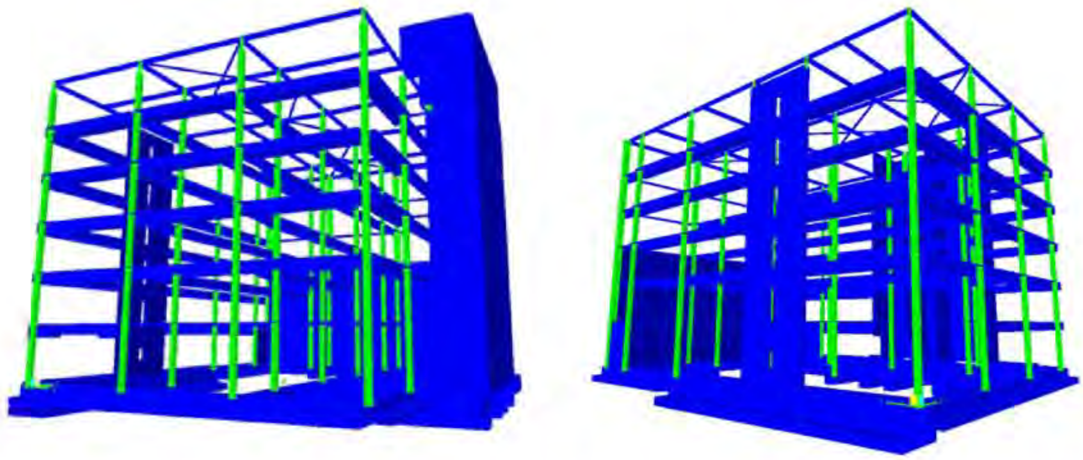


Figure 147 - ETABS computer model - views from north-east and south-east.

ERSA Computer Modelling Assumptions

This method of analysis was required to be used for this structure by NZS 4203:1984 and was reportedly used by the Design Engineer of the CTV Building in 1986. It is a method still commonly used today for the structural design of multi-storey buildings.

The main assumptions in modelling were as follows:

- Upper bound soil stiffness, as recommended by Tonkin & Taylor.

- Concrete walls only as seismic bracing, with secondary frames considered separately.
- Line A block walls as seismic bracing (because of lack of separation).
- Fully ductile response.
- Concentric, +0.1b and -0.1b accidental eccentricity.

Superimposed dead load was estimated as 0.55 kPa throughout.

Live load was taken as 2.5 kPa as applicable for "offices for general use" according to NZS4203.

Seismic live load was calculated to be 0.83kPa in accordance with NZS4203.

Material properties were calculated based 25 MPa for the North Core and South Wall.

Effective section properties of the walls, were calculated in accordance with the recommendations of NZS 4203:1984 and NZS3101:1982, - using the paper titled "The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structure" by T. Paulay and R.L. Williams (NZSEE Bulletin Vol13 No.2 June 1980) as the basis.

Concrete walls and coupling beams in the South Wall were modelled, allowing flexibility in the beam/wall joint zones. Refer Figure 147 and Figure 148.

The ERSA model for the NZS 4203:1984 analyses used cracked section properties of 0.6 Ig for walls and 0.4 Ig and 0.83 Ag for coupling beams. The AS/NZS 1170.5 check used 0.6 Ig and 0.048 Ag to 0.092 Ag for the coupling beams.

The subsoil was considered to be flexible as defined in NZS4203:1984.

In 1986 it appears to have been common practice by many engineers to assume that foundations were rigid, and this was allowed by NZS 4203:1984. However for this investigation, as the building was founded on flexible subsoil, with shear walls cantilevering off foundation beams, the assumption of flexible soil springs was considered to be appropriate to gain a better insight into the behaviour of the structure.

Sensitivity analyses were carried out using a range of values for foundation spring stiffness. The appropriate stiffness of soil springs for seismic analysis were calculated by geotechnical engineers Tonkin & Taylor Limited ("T&T") as outlined in (Sinclair 2011). T&T gave three sets of values for soil spring stiffness; one considered to be a lower bound stiffness, one considered to be the most likely stiffness and one considered to be an upper bound stiffness. For the purposes of this report the upper bound stiffness values (i.e. 1.36k) were used. This was to achieve a conservative estimate of the natural periods of the structure and of the design base shear.

The ERSA analyses used to assess NZS 4203:1984 drift criteria did not incorporate the effects of the internal and perimeter frames columns along Line 1, 2, 3 4 and F in accordance with the primary and secondary frame analysis approach of NZS 4203:1984. The assessment of those frames and the effect of engagement of perimeter columns with the pre-cast concrete Spandrel Panels were accounted for separately in the displacement compatibility analyses described in Appendix F.

The dynamic base shears derived from the ERSA were scaled to 90% of the equivalent static value in accordance with the Code. The analyses were carried out for fully ductile response using a structural type factor S=1.0 and a structural material factor M=0.8, (SM=0.8) in accordance with NZS4203:1984.

One model included only the North Core and the South Wall as the primary seismic resisting system. This reflected the authors understanding of the original design intent based on the structural calculations provided by the Design Engineer. In their calculations the beam/column frames on Lines 1, 2, 3, 4 and F, and the concrete masonry wall on Line A were not included in the seismic analysis as part of the primary seismic resisting structure.

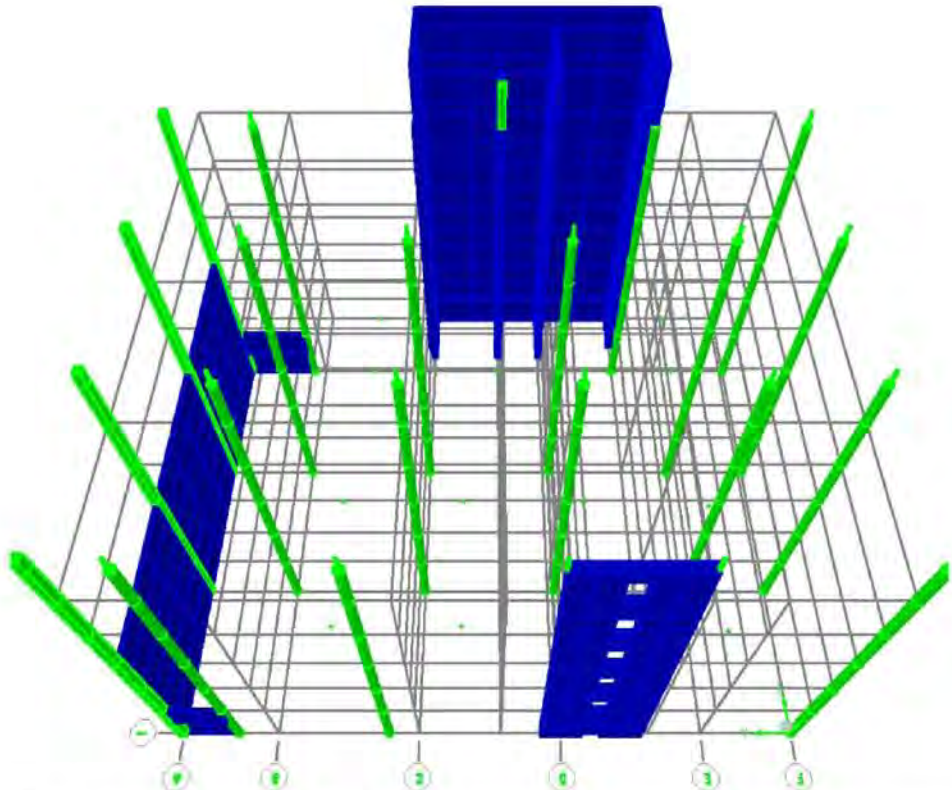


Figure 148 - 3-D view of ETABS model 1c showing layout of concrete shear walls, concrete masonry walls and columns (beams are not shown in this view for clarity).

The recommendation of the Commentary to the Concrete Structures Standard NZS3101:Part2:1982 cl C3.5.14.1 on identifying whether the beam and column frames should be considered as part of the primary seismic resisting system was that "frames in parallel with slender shear walls should be designed as fully participating primary members".

There was no specific guidance as to what a slender shear wall was, however some would consider the South Wall with a height to length ratio of 3.3, when measured to the underside of Level 6, to have been slender.

Commentary recommendations to Standards are recognised however as not requiring mandatory conformance. Displacement compatibility analyses of the secondary frames would have been expected to have ensured that the secondary frames were adequately designed for the anticipated inelastic displacements of the South Wall.

It is likely that other buildings designed to these standards may also have secondary frames that do not satisfy displacement compatibility demands of ductile shear walls or frames.

The column that was located at grid intersection 4-D/E was modelled as part of the North Core. This column was connected to the top of the core wall directly. It was considered to be an integral part of the North Core and therefore part of the primary seismic force resisting system.

Modelling of Line A Masonry Infill Wall

The masonry infill wall on Line A was included in some of the ERSA models as part of the primary seismic resisting system at the time of the September Earthquake. This was to reflect the lack of effective separation to the outer face of the masonry infill prior to the February Aftershock as reported by Eyewitness 16. In other models it was excluded to reflect what appeared to the original design intent.

The design drawings show that the Line A wall should have been fully grouted of the horizontal pre-cast concrete boundary members. Greased starter bars at 600 mm centres were fixed into the under-side of the precast beams along Grid A (Figure 149 DENG Dwg S9 section 6). There was a D12 horizontal bar shown in the shaded top course of the infill masonry in Section 6 of S9, and a note on Dwg S17 required "Grade B masonry all cells filled" indicating that it was to have been filled. As it was Grade B masonry, it required observation by an engineer during construction.

The Design Engineer calculations indicate that the intent may have been to leave the top courses only partially filled to reduce interference with the structure; however this did not get shown on the Drawings.

However based on the statement of Eyewitness 16 complete grout filling of the top courses may not have been achieved. The adjacent building had been demolished after the September Earthquake leaving the wall exposed to the weather. Eyewitness 16 and another worker were required to remove mortar trimmings off the face of the wall in preparation for strapping and cladding the wall a day or so before the February Aftershock. They found that the top courses of the masonry infill at each level were apparently hollow, and no vertical gaps were apparent to them between the masonry and the columns, but some horizontal gaps were found in places between the top courses and the beams. They were able to knock out the face of one top course block on the Level 1 portion of the wall with hammer blows which showed it was hollow (Figure 150).

They also found that the wall wasn't fully grouted when they later drilled holes into it for timber strapping fixings, and that the movement joints in the masonry between the panels and the columns were filled with mortar. The outside face joint appeared to have had a nominal amount of mortar filling the outside edge. This may have been an attempt to ensure the boundary wall fire rating was achieved. This would have limited the ability of the masonry panels to move as three separate panels and increased their collective stiffness.

The rectangular columns sat out proud of the wall face by 20 mm or so. Photos showed vertical separation gaps between the corner column (Grid 1/A) and the short Line 1 return wall on the south face of the building. A horizontal separation gap appeared to be evident between the Line 1 wall and the beam above it (Figure 149). However no vertical or horizontal gaps were evident from the photo on the West wall along Grid A.

An engineering inspection by the OIE after the September Earthquake found sealant on the inside face adjacent to a car park column (Figure 152). This indicated that the specified vertical gap appeared to be as specified when viewed from the inside of the building. At the time of the inspection the boundary wall of the adjacent building would have still been in place. The wall showed no signs of any cracking. The top course of block work can be seen to have been fitted snugly under the precast beam above it as specified. The OIE also reported seeing some light through a gap between the column and the wall in the northwest corner. So there is the possibility there were some gaps in places and not in others along the wall.

The inside of the west wall, after the September Earthquake showed some damage to the linings at Level 2 (Figure 152).

During the collapse (Figure 153) the masonry wall along Grid A broke apart, in some cases as distinct panels, consistent with the design drawings (DENG Dwg S17). Two way diagonal shear fracturing, indicative of severe cyclic demands on the Level 2 masonry was also evident.

This indicates that infill masonry above Level 2 on Line A fully developed its shear capacity prior to the collapse and therefore affected the response of the structure to the February Aftershock (Figure 154).

Gravity actions and confinement of the masonry by the surrounding beams and columns due to the compromised gaps, means it is possible that the masonry infill wall acted as confined masonry making the wall very stiff and strong.

The extent of that interaction with each other and the columns either side through the full earthquake response was difficult to quantify accurately. Therefore, for the ERSA cases in which masonry interaction was assumed, the masonry wall panels were modelled two ways. One as cantilevers pinned to the floor diaphragm above, ignoring the effect of interaction between the sides of the panels and the columns. . The second as a fixed edge panel fixed edge panel using the same masonry material properties.

The level of stiffness introduced into the structure even with pinned cantilever panel approach was sufficient to move the centre of stiffness significantly towards the western wall, compared to that found using the model ignoring the effect of the masonry infill wall. The additional effect of fully locking up the walls as an integral unit would further moved the centre of stiffness westward but by a smaller amount. The effect of the masonry wall seemed to be to change the distribution of drifts around the structure as can be seen in Table 15 to Table 17. The modelling of the Line A masonry wall was therefore difficult to define accurately, but assumptions were made as follows:

- Connection to the floor diaphragm was assumed to occur at the top of the masonry wall, although no vertical load carrying load paths were included.
- The masonry walls were input assuming the 10 mm gap between panels and the 25 mm gap between the masonry and concrete framing was present.
- The masonry material properties were E = 15 GPa.

The masonry walls at level 1 on Line 1 and 4 were not included in the computer modelling as they were specified as separated structurally from the columns each side with reasonable gaps - and had no reinforcing steel connecting them to the floor beam above (DENG Dwg S9 Section 2 and 3) .

For normal design purposes, to allow for various torsional effects, the loadings Standard requires the seismic force to be applied at points +0.1b and -0.1b eccentric from the centre of mass - where b was the breadth of the building perpendicular to the horizontal loading direction under consideration. For assessing NZS 4203:1984 design drifts along Line 1, 2 and F the ERSA used eccentricities to the south and east of the centre of mass.

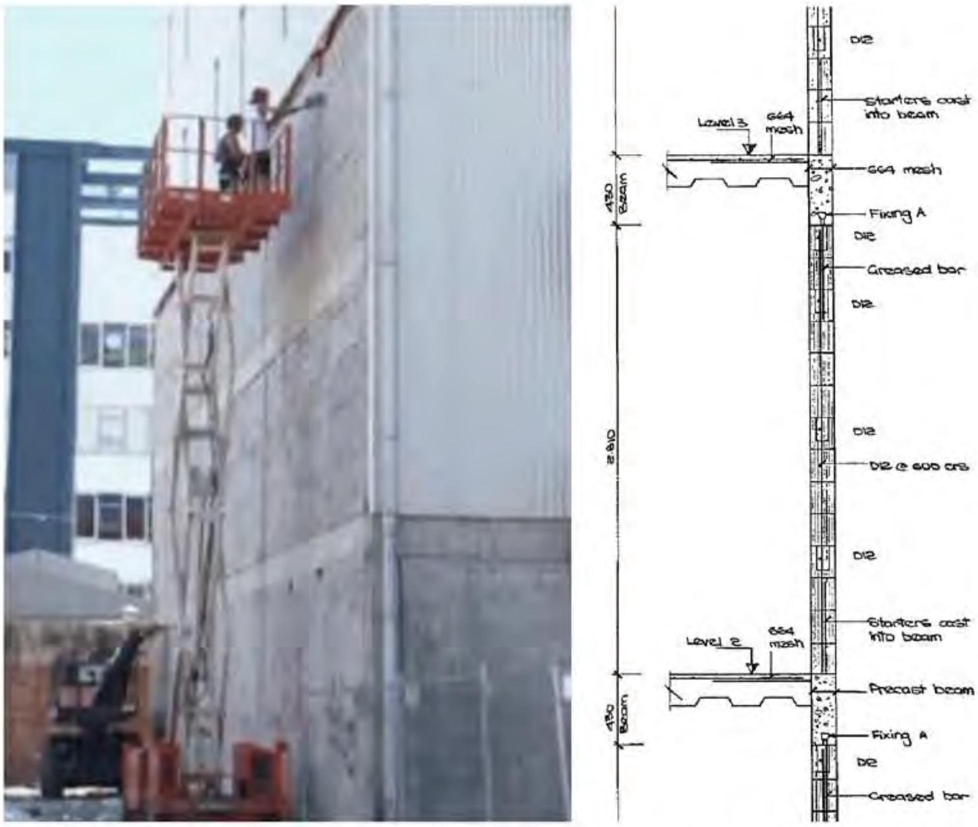


Figure 149 - West wall on Line A (left to right) : Being prepared for strapping and cladding a day or so before collapse on 22nd February; b) Connection of west wall block work into floor beams top and bottom (portion of DENG Dwg S9 Section 6), showing the fixing of the top of the wall into the structure.

Figure 150 – Workers (including Eyewitness 16) hammering face of top course block away on west wall near Line A / 1 corner column. This indicates hollow blocks occurred in the top course and no separation joints on the outer face of the masonry.

Figure 151 - Line A infill masonry wall adjacent to column with no obvious cracking after the 4 September earthquake. Flexible sealant is visible between masonry and column.

Figure 152 - Inside of the west wall at Level 2 after the 4 September 2010 Earthquake shows some damage to the linings.

Figure 153 - West wall on Line A at southwest corner shortly after the collapse.

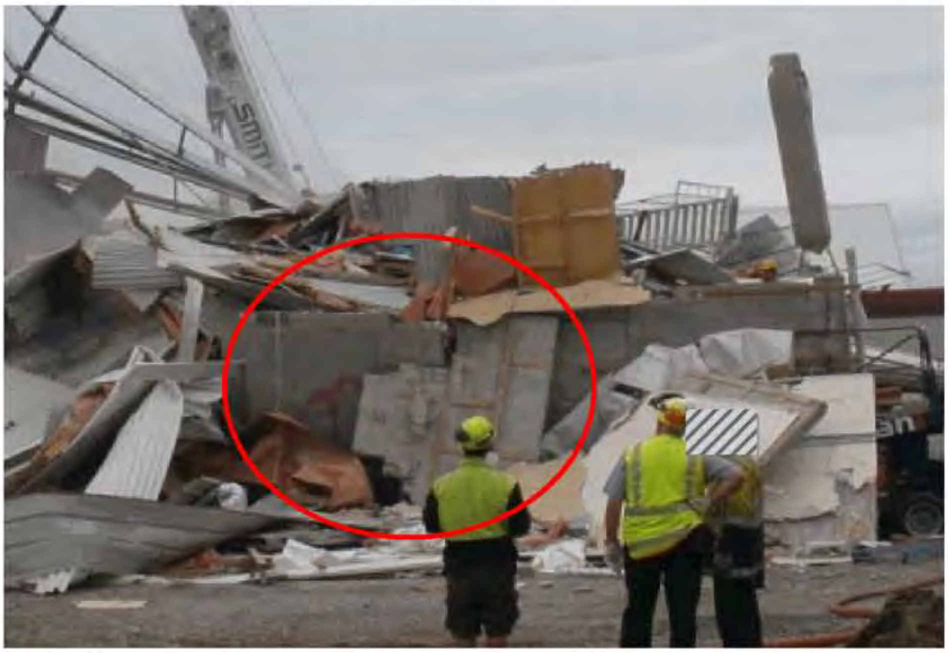


Figure 154 - West wall shortly after collapse. The corner Grid 1/A column is still standing and the wall panels have broken free in panel sections in places. The edges of the panel section are square consistent with the design drawings. Diagonal fracture of the masonry infill that has fallen outwards from level 2 is highlighted. This indicates that the infill masonry above Level 2 fully developed its shear capacity prior to the collapse and therefore affected the response of the structure to the February Aftershock.

ERSA RESULTS

Irregularity and Torsional Response

One of the features of the CTV Building seismic force resisting structure was the asymmetrical plan layout of the concrete bracing walls. The North Core being substantially stiffer and stronger than the southern coupled shear wall in the east-west direction, meaning that the structure had a severe plan irregularity. This can be seen in the following Figure 155 showing the plan location of the centre of mass and the plan location of the centre of rigidity for each of the main floor diaphragms at levels 2 to 6.

Note - The centre of rigidity is defined as follows:

> *When translational lateral loads are applied at the centre of rigidity of a particular floor diaphragm, with no loads applied to any of the other floor diaphragms, the displacements of that diaphragm will have only translational components with no rotations. It should be noted that the resulting displacements of the diaphragms at other levels in general will contain translational as well as rotational components.*

With the concrete masonry walls also participating as part of the seismic force resisting system, the structure was highly irregular in plan in both directions and also a vertical irregularity was introduced at level 4 due to the participation of the masonry walls below that level. The second mode of vibration of the building became much more torsional when masonry was present. With the secondary Group 2 concrete frames added, the irregularities were moderated slightly by the action of the frame which was located more centrally than the walls.

The authors consider that the seismic resisting system in this building was irregular and there are warnings in the loadings Standard NZS4203 that the seismic performance of such irregular structures is less predictable than for equivalent symmetrical structures.

It was permissible according to the New Zealand Concrete Structures Standard NZS 3101:1984 cl. 3.15.14.3 (a) to ignore the seismic requirements of the standard, if the Line A masonry in-fill complied with the provisions for Group 2 Secondary members.

The calculations by the Design Engineer indicate that the intention may have been to more fully isolate it from the structure than what was shown on the Drawings.

A limitation observed of the Group 2 provisions is that they appeared to require that Group 2 elements such as masonry in-fill block work to be protected, but not for their effect on the overall structural response to be considered.

This provision of the standard should be reviewed and could mean that other buildings of the era may have unanticipated responses to earthquakes.

Better guidance is required as to what is acceptable interaction of Group 2 secondary elements with structures.

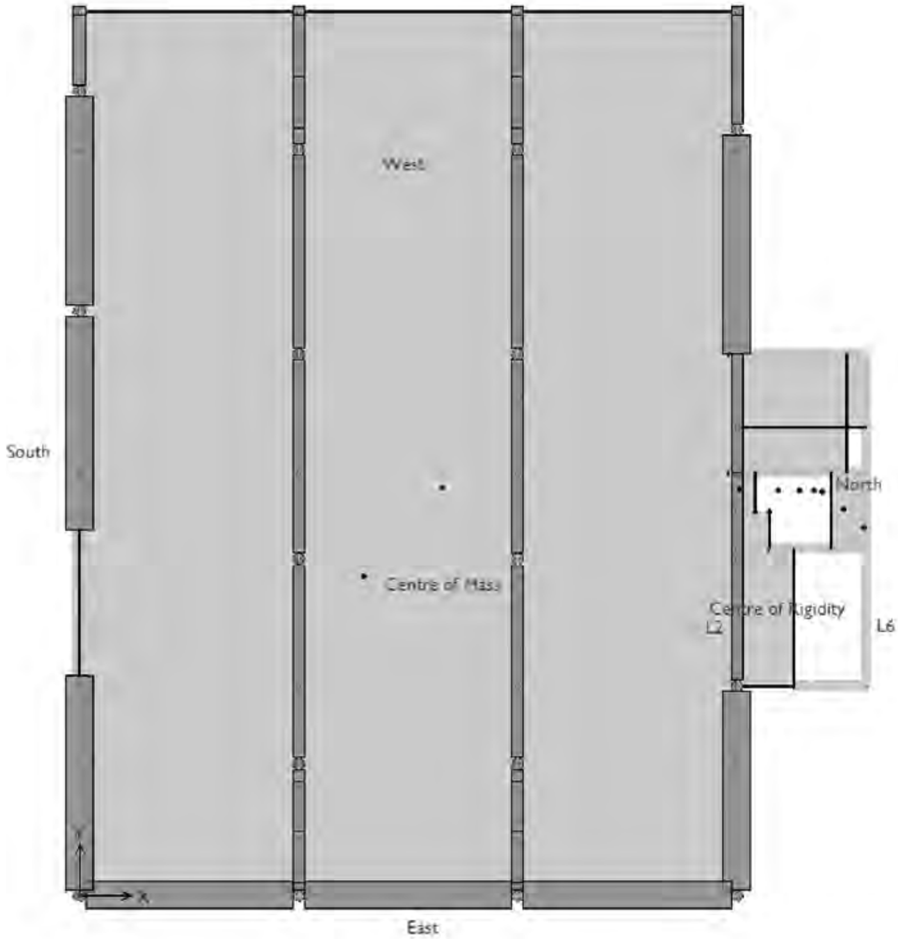


Figure 155 - Centre of Mass and Centres of Rigidity for each Floor (North Core and South Wall only as primary seismic resisting system) The centre of rigidity is close to alignment with the centre of mass for North-South excitation, but highly eccentric from the centre of mass for east-west excitation. This means that the building would have more torsional or twisting response to east-west components of earthquake ground accelerations than to north-south ground accelerations if the Line A masonry infill wall was adequately separated from the structure.

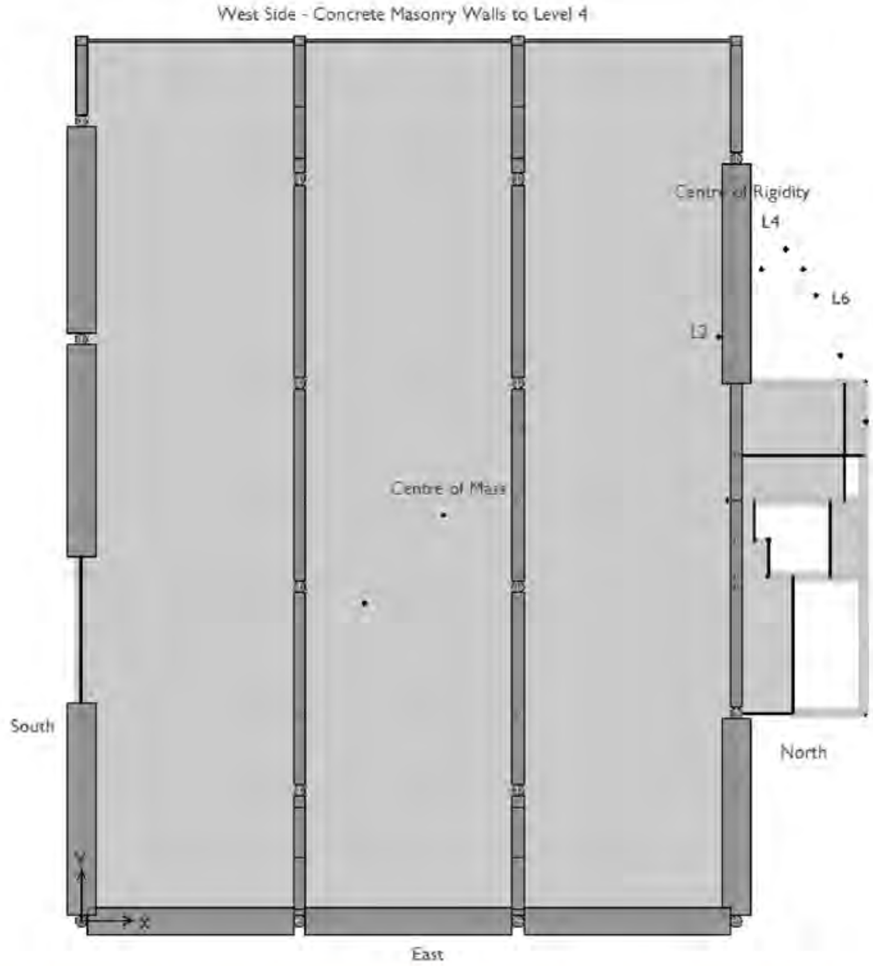


Figure 156 - Centre of Mass and Centres of Rigidity for each Floor (North Core, South Wall and Line A masonry infill wall in contact with structure). The centre of rigidity is highly eccentric from the centre of mass in both directions due to the participation of the west side Line A masonry infill wall below Level 4. The earthquake loads act through the building's centre of mass at each floor level, and the building tries to resist the earthquake actions through its centre of rigidity at each level. The offset between the centre of mass and the centre of rigidity is the eccentricity that determines the level of twist or torsion that results. With Line A masonry wall in full contact with the structure the building will have increased torsional response to north-south earthquake ground motions.

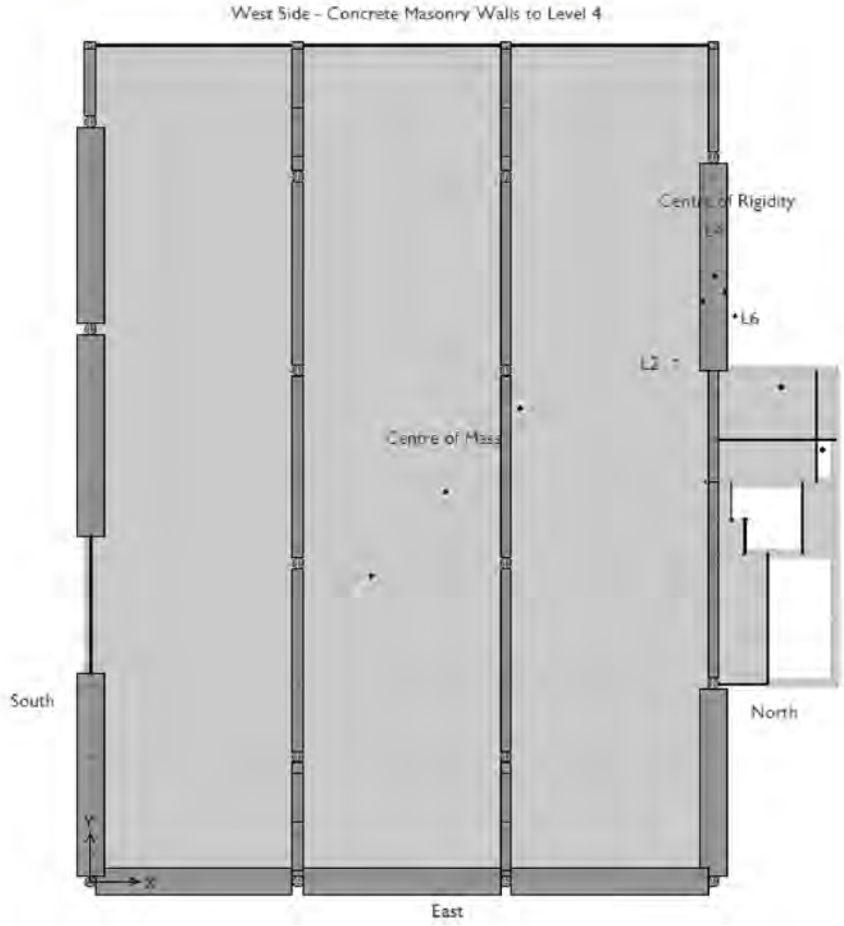


Figure 157 - The changes in the locations of the Centre of Mass and Centres of Rigidity for the building each Floor (North Core, South Wall and Line A masonry infill walls and secondary frames on Lines 1, 2 3, 4 and F). The centre of stiffness moved south and west, reducing the torsional response of the building, due to the effect of the secondary frames.

The effect of the eccentricity between the centre of mass and centre of rigidity on the torsional behaviour on the columns can be seen in the following plots of column shear actions for earthquake shaking in the east-west direction.

Figure 158 is a plot of shear actions (which are related directly to the level of column drift) on the east-west axis of the columns. The columns nearer to the south side and nearer to the top of the building were subject to higher drifts and shear actions (and corresponding bending moments). This is because they were furthest from the centre of rigidity and so experienced more seismic drift, and because the frame attracted a bigger proportion of the total storey shear compared with the walls nearer the top of the building.

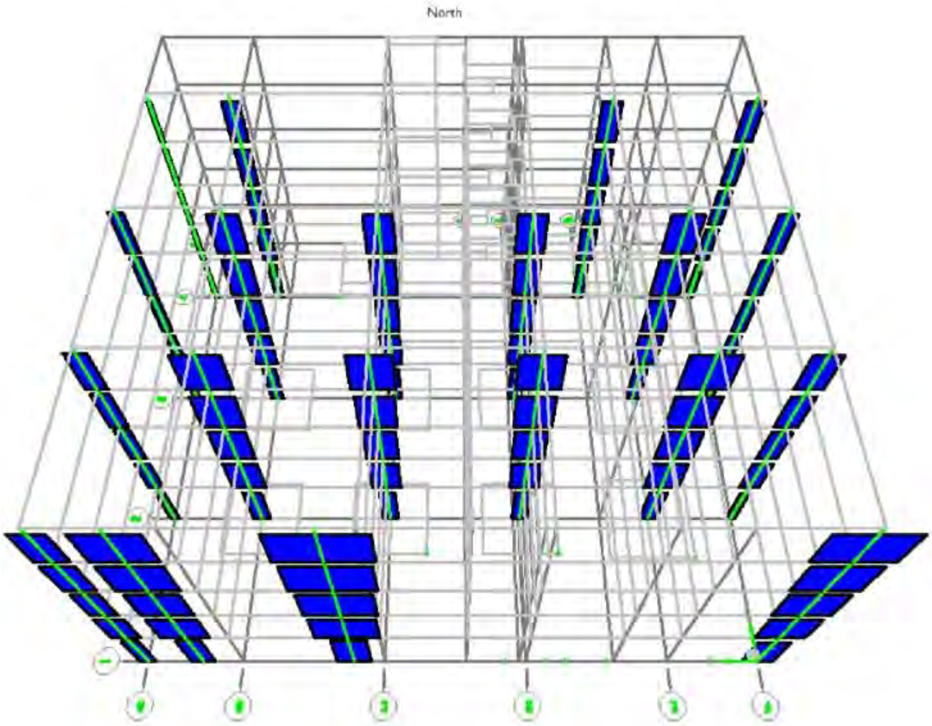


Figure 158 - Plot of column shear actions on east-west column axis for earthquake shaking in east-west direction.

The internal columns did not experience significant shear actions in the north-south direction because the Line 2 and 3 floor beams ran east-west. Similarly the columns at the west side above level 4 did not experience significant shear actions in the north-south direction because there were no beams at the west side above level 4.

The columns at the east side and nearer to the top of the building were subject to the highest drifts (and corresponding shear and bending moments). This was because they were furthest from the centre of rigidity and so experienced more seismic drift. The magnitude of the column drifts in this north-south direction along Line F were of a similar order to column drifts in the east-west direction along Line 1, the direction of earthquake shaking modelled. The effect of torsion was therefore significant. The columns at the east side of the building formed part of a two-way moment frame and so they also experienced concurrent actions in each direction.

South Wall

It was initially thought the damage observed in the shear wall remnants E4 between Levels 4 and 5 discussed in the Site Examination and Materials Tests report may have been due to the influence of the Line A wall (Hyland 2012). However if collapse initiated on Line F at north-south drifts of between 0.9% to 1.5%, the level of shear demand in the South Wall consistent with that level of drift would not appear to have been sufficient to have caused the damage observed. So it was concluded that the damage likely occurred as a consequence of the wall falling onto the debris.

Line A Masonry In-Fill Wall Shear

The shear actions on the Line A wall were greater at level 4 and least at Level 1. The manner of interaction with the Line A reinforced concrete boundary frames would have affected the level of shear capacity able to be developed in the wall. With full contact with the boundary members the wall may have acted as a single stiff shear wall element. Whereas if the wall was only partially connected into the frame as three partially separated cantilever panels per bay flexural behaviour may have reduced the shear capacity and stiffness of the wall significantly.

With full contact the shear capacity was in the order of 2800 kN whereas as partially connected cantilever panels this reduced to around 900 kN.

The ERSA indicated that the wall may have been expected to maintain shear capacity until the South Wall ideal bending capacity developed. It may then have become severely damaged in shear between Levels 2 and the underside of Level 4 prior to diaphragm drifts consistent with north-south drifts of between 0.9% to 1.5% developing on Line F. But given the difficulties of applying the equivalent displacement approach in such circumstances this could not be fully confirmed using ERSA.

Severe diagonal fracture of what appears to be Level 2 wall panels was observed in photos of the debris immediately after the collapse (Figure 72).

South Wall and North Core Flexural Action to Capacity Ratios

The flexural demand to capacity ratios of the walls on Line 1 and 5 in East-West direction were calculated relative to the SM=0.8 response spectrum analysis actions and subject to axial gravity actions of G+Qu in accordance with the design method of the loading standard NZS 4203:1984.

As an isolated element the South Wall was adequately strong in terms of the code requirements. However its large differential in its strength and stiffness compared to the North Core meant that the building was torsionally unsymmetrical. This links back to the issue of the lack of geometrical symmetry in the structure.

As previously noted this raises questions about the adequacy of the provisions for the ductile design of torsionally irregular structures. In this case though the North Core walls were detailed for ductile performance they were so strong it is possible that they in fact responded largely as elastic elements.

Flexural Demands vs Capacity of South Wall

The flexural demand on the Line 1 South Wall was greatest at its base. Partial masonry in-fill to the Level 1 door in the wall appeared to constrain the wall to act as a cantilever wall between level 1 and 2 rather than as intended as a coupled shear wall. This cantilever behaviour at Level 1 was also indicated by the fan-like cracking patterns observed in the wall after the collapse (Figure 93). These are generally considered typical of flexural cracking in cantilever shear walls.

The ideal bending capacity of the wall, without strength reduction factors, was calculated to be 21103 kNm based on the average tested concrete strength from cores in the wall of f'c= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa. The flexural demand on the wall for the structural configuration including partial masonry infill engagement for the S=1 spectra was M*=12605 kNm. The ratio of ideal capacity relative to NZS 4203:1984 design demand of the Line 1 South Wall was:

$$\frac{M_i}{M^*} = \frac{21202}{12605} = 1.7$$

This indicates that the wall satisfied the requirements of NZS 4203:1984 in terms of bending capacity at its base for that configuration.

The displacement of the structure in that configuration as whole appears to have remained largely elastic until the demand on the Line 1 South Wall reached around 1.7 times the loadings from the S=1 spectra. The attainment of the ideal bending capacity of the South Wall was associated with east-west drifts along Line 1 of 0.35% and north-south drifts of 0.35% to 0.40% along Line F, between Level 3 and 4. By reference to Figure 159 the columns along Line 1 and F would not be expected to have reached their yield moments at that level of drift demand. The masonry infill wall on Line A however may have been close to its ideal shear capacity, depending on the orientation of the earthquake loading.

Flexural Demands vs Capacity of Line 5 Wall

The flexural demand on the Line 5 wall was greatest at its base.

The ideal bending capacity of the wall in the east-west direction was calculated to be 167904 kNm based on the average tested concrete strength from cores in the wall of f'c= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa.

The flexural demand on the wall from the NZS4203:1984 S=1 spectra was approximately M*=20500 kNm.

The ratio of ideal capacity over demand for the S=1 spectra was therefore:

$$\frac{M_i}{M^*} = \frac{167900}{20500} = 8.2$$

The North Core was very strong relative to the South Wall in the east-west direction.

It should also be noted that while the South Wall appeared to have suffered in-plane flexural damage prior to collapse, the South Wall was considered to have performed adequately in terms of ductile design expectations.

Limitations of the NZS 4203:1984 and Current ERSA Provisions

The provisions of NZS 4203:1984 for analysing structures like the CTV Building did not adequately anticipate the effect of differential inelastic action developing in different parts of the structure such as occurred in the CTV Building.

While some designers recognise these sorts of issues and compensate for the effects in the way they use the ERSA beyond what the standard required, the provisions of the standard would lead most to under-predict drifts and building response. This is an issue with NZS 4203:1984 and also remains one that needs addressing in current earthquake design standards.

NTHA methods help to deal with these issues. However ERSA is a much more common and economical analysis method for engineering design. Therefore the development and codification of better methods to account for irregular structures using ERSA would likely lead to significant broad industry level improvements in reliable seismic design of structures with moderate to high levels of torsional irregularity.

APPENDIX F - DISPLACEMENT COMPATIBILITY ANALYSIS TO STANDARDS

INTRODUCTION

NZS 4203:1984 and NZS 3101:1982 required that the secondary or Group frames in earthquake resisting structures were able to satisfy certain displacement compatibility criteria. These were intended to ensure that the secondary frames would remain able to sustain load under the specified earthquake demands. In this section the structure was assessed against those criteria.

METHOD

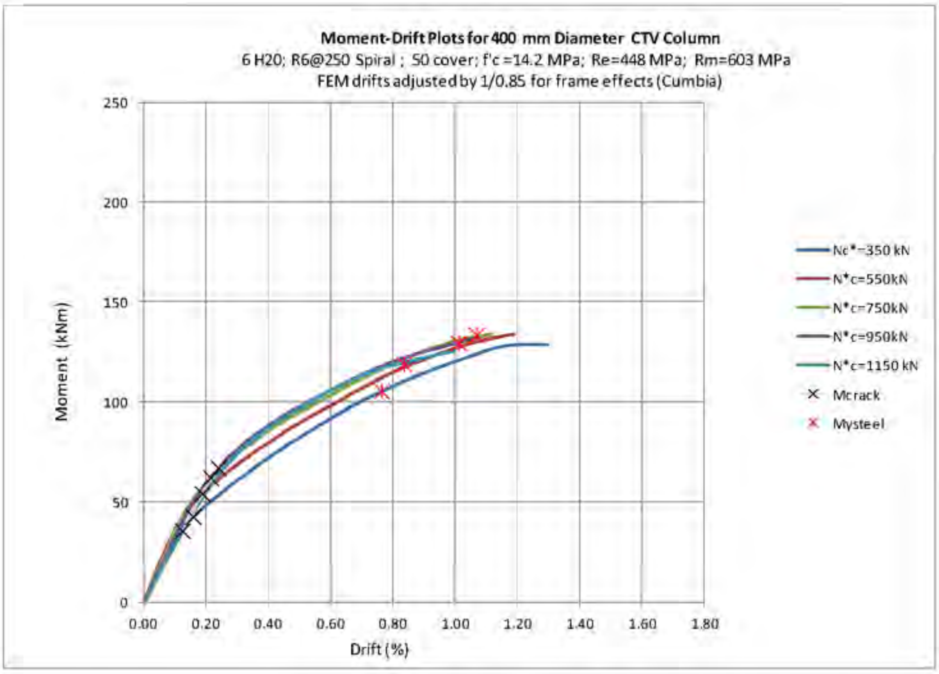
Moment-drift plots were developed using displacement and moment curvature relationships at varying axial actions calculated using Cumbia software (Montejo and Kowalsky 2007). The plots of the moment-drift curves for f'c =14.2 and f'c =27.5 MPa concrete are shown in Figure 159 up to a limiting concrete compression strain of 0.004. These curves show how the moment–drift relationship varies significantly with concrete strength and axial compression. The drifts to cracking moment, tensile yield of reinforcing steel and concrete yield at 0.002 strain are also shown.

The figures show fixed end moment drifts for columns on Line 1, 4 and F adjusted for frame effects by application of a 0.85 divisor. This divisor was determined by comparing the drifts of a plane frame model of Line F, using effective stiffness properties derived from moment curvature analyses for the assumed drift profile, with beam-column joints fixed against rotation or free to rotate.

The drift-compression plots shown in Figure 161 and Figure 162, were developed to show the variation of drift with compressive axial action, for the limiting conditions of steel tensile yield, concrete yield at 0.002 compression strain and concrete at 0.004 compression strain. A comparison of the curves also allows some assessment of the effect of a reduction of concrete strength on the drift capacities.

Drift demands on Line 1 column C/1 and Line F column F/2 were derived from a simplified ERSA model of the primary structure without masonry infill effects on Line A. This was the assumed design condition. The drift demands were computed for the K/SM=2.75 factored loading from NZS 4203:1984. The drifts were computed using the assumptions of cracked section properties of the North Core and South Wall described in Appendix E. The acceptance criteria of NZS 3101:1982 were then applied to identify if the seismic design and detailing requirements would have been triggered, and also if the drift demands satisfied the overall primary frame drift limits of NZS 4203:1984.

For the purposes of this check no adjustment was made to the ERSA point drifts to account for the possible development of inelastic behaviour in the South Wall under the K/SM=2.75 loading. This may have increased the drift demands.



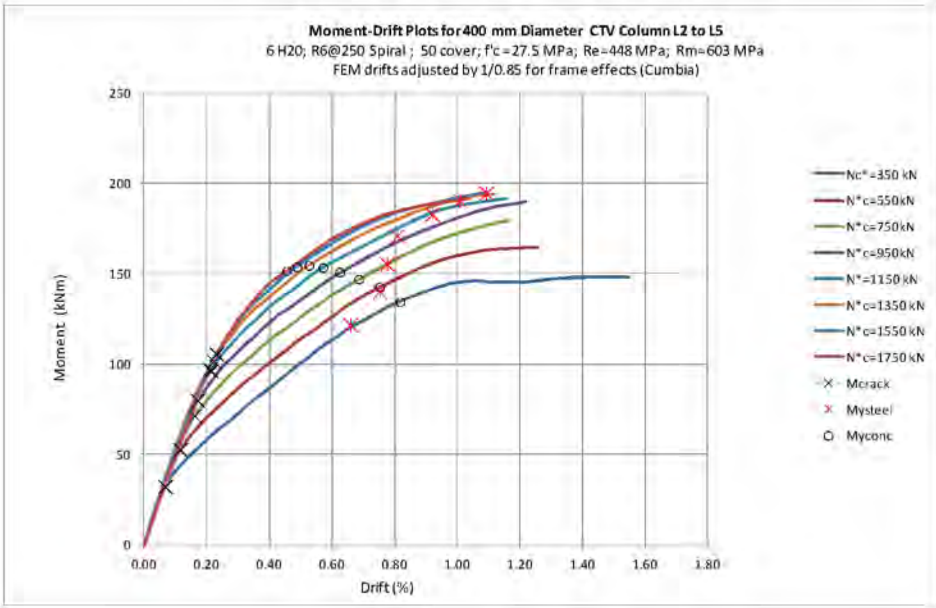


Figure 159 - Moment-Drift plots for 400 mm diameter CTV columns for f'c=14.2 and 27.5 MPa concrete, using Cumbia software for fixed end conditions adjusted for line 1, 4 and F frame effects. Concrete limiting strain was set at 0.004. This shows that yielding of the reinforcing steel starts at higher drifts as the axial compression action increases. Similarly the ability of the columns to drift more after starting to yield reduces as the axial compression action increases. Columns in the upper levels had lower axial compression actions compared to the lower level columns, and so were able to sustain more inelastic demand than those at lower levels. The crosses indicate the point at which yield of the extreme reinforcing steel bar occurs designated as the yield moment of the column. Due to the wide spacing of the spiral reinforcing, loss of concrete cover may have led to buckling of the H20 bars. For loads greater than 1550 kN the column drift capacity appears to reduce back along the upper drift curve back to squash capacity.

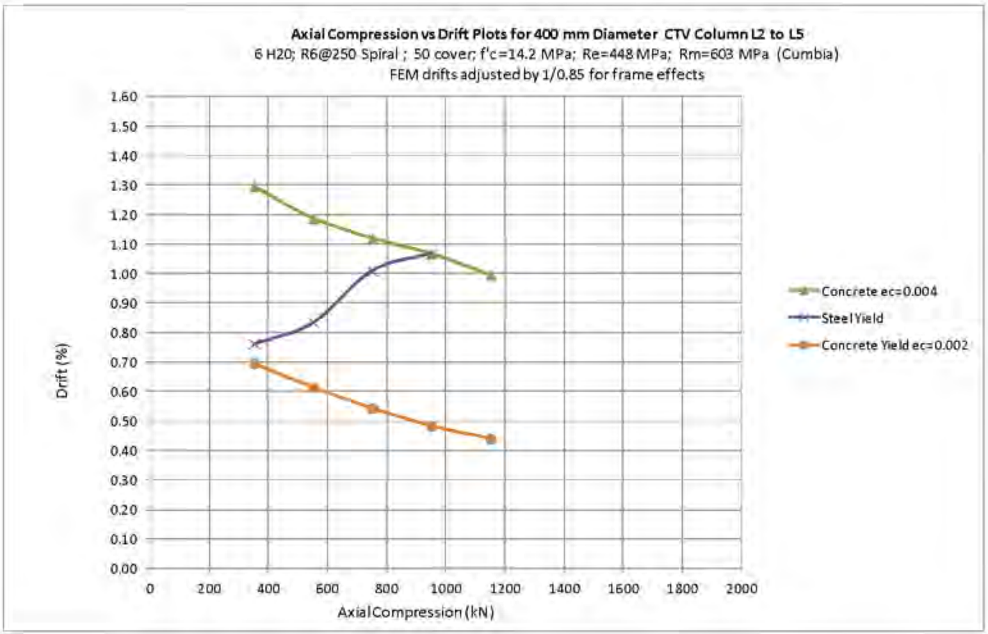


Figure 160 - Axial compression vs Drift for L2 to L5 columns on Lines 1, 4 and F (f'c=14.2 MPa).

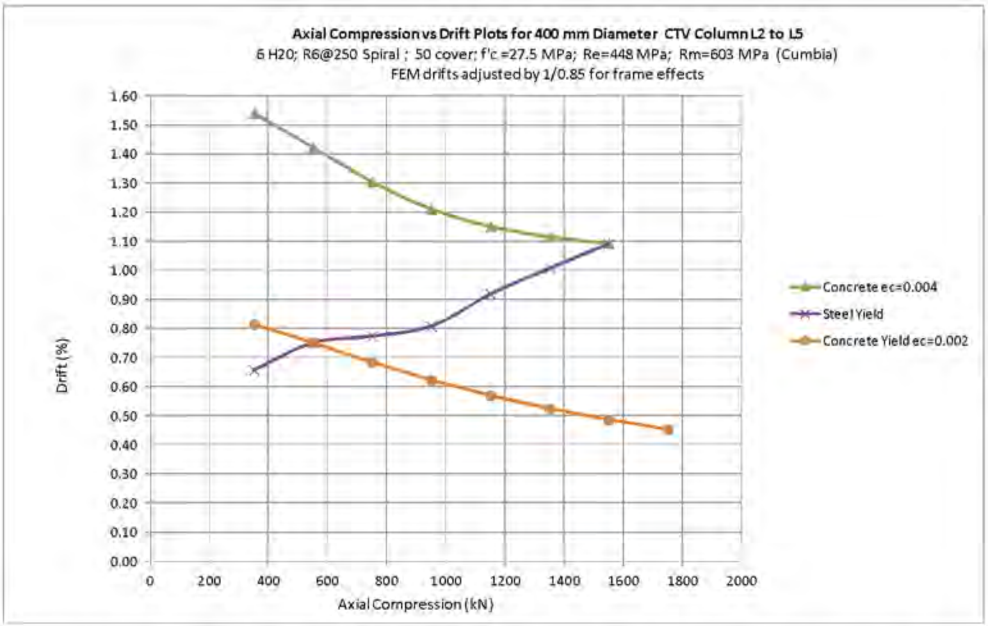


Figure 161 - Axial compression vs Drift for L2 to L5 columns on Lines 1, 4 and F (f'c= 27.5 MPa).

CHECK ON ADEQUACY OF NON-SEISMIC DETAILING IN COLUMNS

A check was made to determine the performance of the columns under the displacement demands of NZS 4203:1984 cl 3.8.1.1 satisfied the criteria of NZS 3101:1982 cl 3.5.14.3(a) for non-seismic detailing. The criteria required the columns to remain elastic when the ductile design spectra (S=1.0) drifts were scaled by a factor of K/SM=2.75.

If the columns could not remain elastic they were required to be designed and detailed to the seismic design provisions of NZS 3101:1982. For the purpose of this check the elastic performance limit was set as the minimum of tensile yield initiating in the vertical reinforcing steel or development of a concrete compressive strain of 0.002.

It appears to have been acceptable practice at the time for design engineers to check conformance with this criteria by determining if the more conservative dependable capacity of the Group 2 frame members were greater than the K/SM=2.75 drift demands.

The demands on the members were not only dependant on the drift imposed but also on the cracked section properties assumed for those members. The stiffer they were assumed to be, the greater the resulting demands for a given level of drift. The commentary (C3.5.5.1) recommended that:

> "Typically the moment of inertia of a beam section may be based on 50% of the moment of inertia of the gross concrete area, whereas for columns carrying significant axial compression, 100% of the corresponding moment of inertia may be assumed ....The allowances for the effects of cracking on stiffness must be consistent through the structure."

It appears to have been acceptable practice at the time for design engineers to use these recommendations of cracked section properties in their assessments. These would be stiffer than the cracked section properties derived in the moment – curvature analysis used for the development of the moment-drift and drift-compression curves in Figure 159 and Figure 161. As a consequence the elastic deformation limits would be lower and more conservative, making it more difficult to justify non-seismic detailing of the Group 2 columns.

The seismic design and detailing requirements for the Group 2 CTV Building columns in NZS 3101:1982 are interpreted as follows and as illustrated in Figure 162.

If the column could remain elastic at deformation induced by loads on the primary structure of not less than K/SM = 2.75 then non-seismic detailing was acceptable. The implication being that it was sufficient for it to achieve the K/SM=5 deformation without losing strength (Cl 3.5.14.3 a).

If the column did not remain elastic at deformation induced by loads on the primary structure of greater than K/SM = 2.75/2 but not greater than K/SM = 2.75, then it was required to be detailed using the limited ductility provisions of Chapter 14 in NZS 3101:1982. The implication being that it would then be capable of achieving the K/SM=5 drift without loss of strength (Cl 3.5.14.3 f).

If the column did not remain elastic at deformation induced by loads on the primary structure of less than K/SM = 2.75/2, then it was required to be detailed using the full ductility provisions of NZS 3101:1982. The implication being that it would then be capable of achieving the K/SM=5 drift without loss of strength (Cl 3.5.14.3 b).

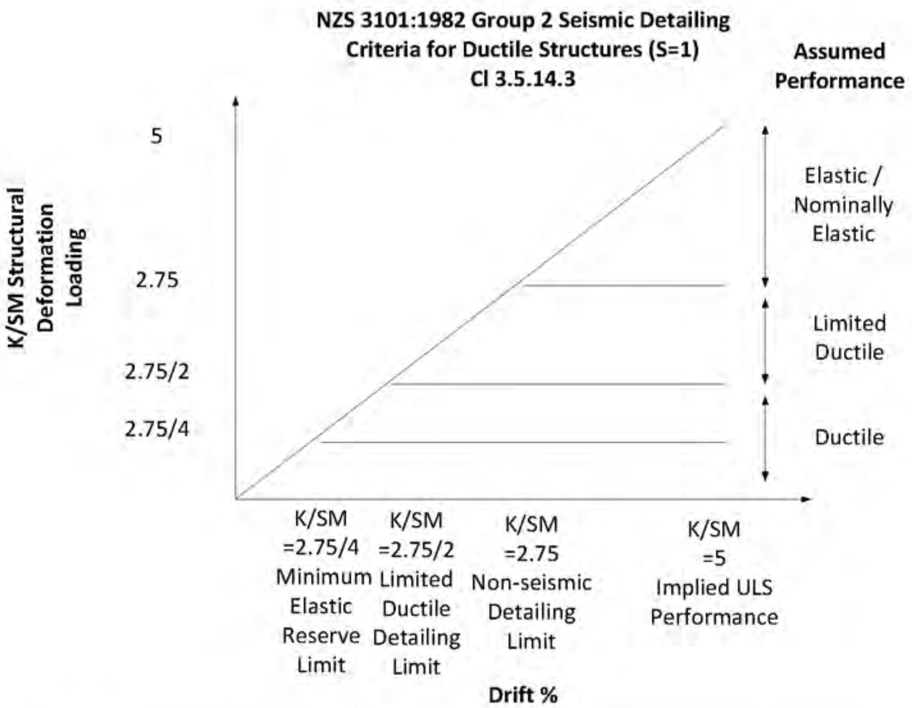


Figure 162 – Design and detailing limits from NZS 3101:1982 and implied performance for the CTV Building Group 2 columns.

The proviso was however in all cases that the column should have a minimum reserve of elastic performance and remain elastic at deformation induced on the primary structure by the K/SM = 2.75/4 factored loads (Cl 3.5.14.3 e).

It can be seen in Table 13 that the K/SM=2.75 drift demands on the Level 2 to 5 columns at C/1 exceeded their elastic deformation limits and would therefore have been required to have been designed and detailed using the seismic design provisions of NZS 3101:1982.

It can be seen in Table 14 that the K/SM=2.75 drift demands on the Level 5 column at F/2 exceeded its elastic deformation limit and would therefore have been required to have been designed and detailed using the seismic design provisions of NZS 3101:1982.

If adjustment was made for the effects of possible inelastic behaviour of the South Wall under the K/SM=2.75 loading it is likely that more of the F/2 columns would have exceeded their elastic deformation limits.

C/1 ERSA Inter-storey East-West Drifts
10% eccentricty of mass south and east of centre

	Column Drift Limit			No Line A Masonry Infill	
Level	**Axial Compression kN**	**Elastic Deformation Limit**	**Failure Limit ec=0.004**	**NZS 4203:1984 K/SM=2.75**	**NZS 4203:1984 ULS**
North -South Earthquake					
L5 - L6	336	0.82%	1.55%	0.29%	0.52%
L4	623	0.73%	1.37%	0.28%	0.52%
L3	906	0.64%	1.23%	0.26%	0.48%
L2	1189	0.58%	1.15%	0.21%	0.39%
L1	1478	0.50%	1.10%	0.13%	0.23%
East-West Earthquake					
L5 - L6	336	0.65%	1.55%	0.80%	1.46%
L4	623	0.73%	1.37%	0.79%	1.43%
L3	906	0.64%	1.23%	0.72%	1.31%
L2	1189	0.58%	1.15%	0.59%	1.08%
L1	1478	0.50%	1.10%	0.35%	0.63%

Table 13 - Displacements and inter-storey drifts along Column C/1 on Line 1 from ERSA for NZS 4203:1984 K/SM=2.75 and implied ULS at S=5. On this basis the columns on Line 1 would have triggered the requirement for the seismic design provisions of NZS 3101:1982 to be applied. The maximum drift of 0.80% at Level 5 is less than the primary frame drift limit of 0.83%.

F/2 ERSA Inter-storey North-South Drifts
10% eccentricty of mass south and east of centre

	Column Drift Limit			No Line A Masonry Infill	
Level	**Axial Compression kN**	**Elastic Deformation Limit**	**Failure Limit ec=0.004**	**NZS 4203:1984 K/SM=2.75**	**NZS 4203:1984 ULS**
North -South Earthquake					
L5 - L6	269	0.62%	1.58%	0.64%	1.16%
L4	513	0.73%	1.45%	0.64%	1.16%
L3	754	0.69%	1.30%	0.61%	1.12%
L2	995	0.61%	1.20%	0.56%	1.01%
L1	1245	0.55%	1.13%	0.42%	0.76%
East-West Earthquake					
L5 - L6	269	0.62%	1.58%	0.28%	0.51%
L4	513	0.73%	1.45%	0.28%	0.50%
L3	754	0.69%	1.30%	0.25%	0.46%
L2	995	0.61%	1.20%	0.20%	0.37%
L1	1245	0.55%	1.13%	0.13%	0.23%

Table 14 - Column F2 Seismic detailing limit checks to NZS 4203:1984. This shows that the Level 5 to 6 columns would have required the seismic design provisions of NZS 3101:1982 to be applied. The maximum drift of 0.64% at Level 5 is less than the primary frame drift limit of 0.83%.

ADEQUACY OF PRIMARY FRAME STIFFNESS TO NZS 4203:1984

The maximum C/1 east-west drift of 0.80% at Level 5 is less than the primary frame drift limit of 0.83%. This indicates that the structure may have satisfied the primary frame drift requirement.

The maximum F/2 north-south drift of 0.64% at Level 5 is less than the primary frame drift limit of 0.83%. This also indicates that the structure may have satisfied the primary frame drift requirement.

However the check was done using a simplified ERSA model with no adjustment made for the effects of inelastic behaviour of the South Wall occurring under the K/SM=2.75 loadings. Therefore it is possible that the structure may not have satisfied the primary frame drift criteria if such adjustments had been made.

The purpose of the primary frame drift limit was to ensure a minimum level of stiffness in the structure.

ADEQUACY OF DRIFT CAPACITY FOR 2010 STANDARDS

Ultimate limit state drifts were also calculated based on the demand from the ERSA model, neglecting p-delta effects, and multiplied by the drift modification factor of 1.24 from NZS1170.5 Table 7.1. This indicated a drift demand of 2.3%.

This is well in excess of the requirements of the standards in 1986 and indicates that the CTV Building may have had an average comparative drift capacity in the order of 40% to 50% of 2010 requirements.

According to the 2010 standards, the calculated 2.61% Ultimate Drifts along gridline 1 at levels 4 and 5, exceed the inter-storey deflection limit of 2.5% specified in NZS1170.5 (refer clause 7.5.1). This means the line 1 seismic resisting structure would have needed to be stiffened to comply with current standards.

COMPARATIVE DEMANDS OF EARTHQUAKES

To appreciate the relative demands of each event, the ERSA comparative drifts using the September Earthquake, and the December and February Aftershocks averaged maximum response spectra, assuming fully elastic response are shown in Table 15 to Table 17. These are shown for the cases with and without interference of the Line A masonry infill on the response. The drift profiles were derived from point drift maxima from the ERSA, These may be different to and will be less accurate than those derived from an inelastic analysis because of the simplifying assumption for this comparison of elastic behaviour of the structure to the loadings.

Based on a comparison of the drifts the February Aftershock appeared to cause an elastic response around 2.2 times that of the September Earthquake, which itself appeared to cause a response 2.0 times than of the December Aftershock.

C/1 ERSA Comparative Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre

	No Line A Masonry Infill			With Line A Masonry Infill		
Level	**SEP ULS**	**DEC ULS**	**FEB ULS**	**SEP ULS**	**DEC ULS**	**FEB ULS**
North -South Earthquake						
L5 - L6	0.57%	0.24%	1.28%	1.02%	0.59%	2.25%
L4	0.57%	0.24%	1.28%	1.00%	0.57%	2.21%
L3	0.52%	0.22%	1.18%	0.89%	0.48%	1.99%
L2	0.43%	0.18%	0.96%	0.74%	0.39%	1.64%
L1	0.25%	0.11%	0.56%	0.44%	0.24%	0.98%
East-West Earthquake						
L5 - L6	1.25%	0.55%	2.47%	0.86%	0.56%	1.87%
L4	1.23%	0.54%	2.42%	0.84%	0.54%	1.82%
L3	1.13%	0.49%	2.22%	0.73%	0.44%	1.59%
L2	0.93%	0.40%	1.82%	0.60%	0.36%	1.31%
L1	0.54%	0.24%	1.07%	0.36%	0.23%	0.80%

Table 15 – Line 1 column C/1 ERSA comparative east-west drift demands of the September Earthquake, and the December and February Aftershocks assuming fully elastic response.

D/2 ERSA Comparative Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre

	No Line A Masonry Infill			With Line A Masonry Infill		
Level	**SEP ULS**	**DEC ULS**	**FEB ULS**	**SEP ULS**	**DEC ULS**	**FEB ULS**
North -South Earthquake						
L5 - L6	0.42%	0.18%	0.93%	0.76%	0.45%	1.67%
L4	0.41%	0.17%	0.93%	0.74%	0.43%	1.64%
L3	0.38%	0.16%	0.85%	0.67%	0.37%	1.48%
L2	0.31%	0.13%	0.70%	0.55%	0.30%	1.22%
L1	0.18%	0.08%	0.40%	0.33%	0.19%	0.74%
East-West Earthquake						
L5 - L6	0.93%	0.41%	1.83%	0.65%	0.43%	1.40%
L4	0.91%	0.40%	1.79%	0.63%	0.41%	1.37%
L3	0.84%	0.36%	1.64%	0.55%	0.34%	1.20%
L2	0.01%	0.30%	1.35%	0.45%	0.28%	0.99%
L1	0.41%	0.18%	0.80%	0.28%	0.19%	0.61%

Table 16 – Line 2 column D/2 ERSA comparative east-west drift demands the September Earthquake, and the December and February Aftershocks assuming fully elastic response.

F/2 ERSA Comparative Inter-storey North-South Drifts
10% eccentricty of mass south and east of centre

	No Line A Masonry Infill			With Line A Masonry Infill		
Level	**SEP ULS**	**DEC ULS**	**FEB ULS**	**SEP ULS**	**DEC ULS**	**FEB ULS**
North -South Earthquake						
L5 - L6	1.27%	0.50%	3.09%	1.00%	0.42%	2.26%
L4	1.27%	0.50%	3.09%	1.00%	0.42%	2.26%
L3	1.22%	0.48%	2.97%	0.95%	0.41%	2.15%
L2	1.10%	0.44%	2.69%	0.84%	0.36%	1.89%
L1	0.83%	0.33%	2.02%	0.60%	0.27%	1.34%
East-West Earthquake						
L5 - L6	0.44%	0.20%	0.96%	0.71%	0.31%	1.61%
L4	0.43%	0.20%	0.95%	0.71%	0.31%	1.61%
L3	0.39%	0.18%	0.87%	0.68%	0.30%	1.53%
L2	0.32%	0.14%	0.72%	0.60%	0.27%	1.35%
L1	0.20%	0.10%	0.46%	0.43%	0.21%	0.96%

Table 17 – Line F column F/2 ERSA comparative north-south drift demands of the September Earthquake, and the December and February Aftershocks assuming fully elastic response.

EFFECT OF VERTICAL ACCELERATION

On the basis of the drift capacity reduction as column axial compression action increase shown in Figure 161 for concrete of f'c=27.5 MPa a reduction of drift of between 0.25% to 0.50% /1000 kN increase in axial demand may be justifiable .

It may be that at greater compression demands the reduction in drift capacity varies from this, however that has not been able to be established. This result needs to be considered in conjunction with the findings indicated by the NPA discussed in Appendix D.

EFFECT OF CONCRETE STRENGTH ON DRIFT CAPACITY

Based upon a comparison of the drift capacities shown in Figure 160 and Figure 161, it appears that a reduction in concrete strength from near the mean to the lower 5% strength measured during testing, (ie. a reduction of approximately 13 MPa), could lead to a reduction in column drift capacity of up to 15%.

SUMMARY

Columns at grids C/1 and F/2 do not appear to have satisfied the criteria of NZS 4203:1984 that would have allowed them to be detailed using the "non-seismic" provisions of the concrete structures standard NZS 3101:1982.

This was based on a check using a simplified ERSA analysis of the structure without adjustment for development of possible inelastic behaviour in the South Wall under the K/SM=2.75 loadings used for the check. Such effects may have increased the drift demands further.

The primary frames were found to have possibly provided adequate stiffness to limit the primary frame drifts of the structure, as required by NZS 4203:1984. However no adjustment was made for the effect of possible inelastic behaviour in the South Wall under the K/SM=2.75 loadings used for the check. This may have increased the drift demand beyond the code drift limit of 0.83%.

The effects of reduced concrete strength and increases of axial compression action from vertical acceleration on the columns appear likely to have reduced the column drift capacities.

APPENDIX G - DIAPHRAGM FAILURE ANALYSIS AT NORTH CORE

In order to investigate whether the floor diaphragm detached from the North Core before or after column collapse occurred, the in-plane bending capacity along two critical diaphragm failure sections was analysed.

Review of the collapse photos such as Figure 163and analysis indicated the following order of collapse:

- The slab on the south side of the Line 4 beam broke away due to loss of vertical support following column collapse on Line 3.
- The slab attached to the lift and stair well walls (D and D/E) then detached from the Drag Bars as column 4-D/E collapsed.

Analysis of the in-plane bending capacity of the diaphragm compared in-plane bending capacity along two critical failure sections shown in Figure 166 The first along the perimeter ABCD running from the edge of the Line C wall out 1200 mm to the ends of the slab saddle bars and along 11.35 m then back 1200mm the Line D/E wall. The second failure section EFGA ran from the attachment point of the Drag Bars at walls D/E and D and along the slab at Line 4 between Walls C and C/D.

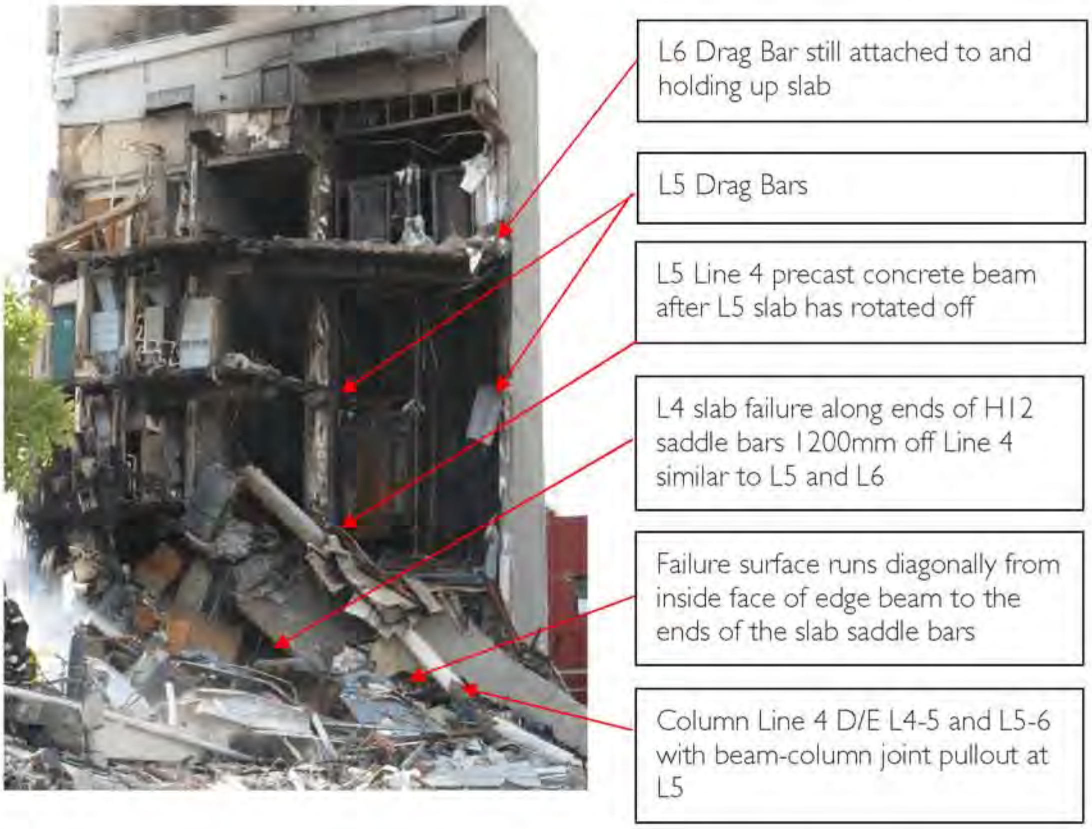


Figure 163 - Failure of slab adjacent to North Core.

The Drag Bars had been installed during remedial work to the building after construction. The copies of the sketches issued for their installation are shown in Figure 167 and Figure 168.

The in-plane bending capacity along ABCD was found to be greater than that along EFGA as shown in Figure 166. The in-plane bending capacity along ABCD included shear contributions of the AB and CD portions and the shear capacity of the with edge beam. The bending capacity along EFGA was limited by the tension capacity of the Drag Bar connections. The contribution of the profiled metal deck in tension was conservatively ignored but would have further increased the differential of strength between the ABCD and EFGA failure sections.

The diaphragm in-plane bending capacity at EFGA at the Drag Bars was the weakest link in terms of diaphragm attachment to the North Core. Failure of the slab was evident along line ABCD (but not through the perimeter beam) at L4 to L6 from a careful look at the photos (Figure 163and Figure 164).

For this to have occurred, column collapse along Line 2 and/or 3 would have occurred pulling the slab at the core downwards so that it failed in flexure and tension at the end of the saddle bars. This was consistent with the way the Level 3 and 4 slabs were found to have fallen, lying diagonally against the North Core after the collapse as shown in Figure 165.

The Line 4-D/E column adjacent to the North Core may then have been pulled down as the collapse progressed and the portions of slab immediately outside the lifts between walls D and D/E rotated downwards and pulled away from the Drag Bars.

The epoxied concrete anchors, that attached the Drag Bars to the slab, appeared to have held adequately in shear (as evidenced by the Level 6 slab which was still being held up by the Drag Bars). However as the slab portions rotated downwards the slab pried away from the anchors in tension, leaving the anchors at the north end of the Drag Bars vertical and those on the bent down outstand bent over by around 30 degrees.

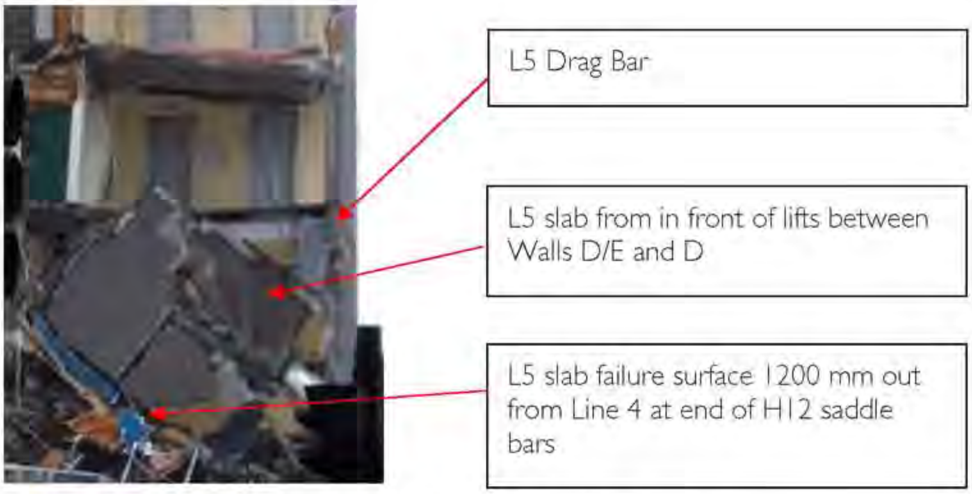


Figure 164 - Level 5 slab from in front of lifts shortly after the collapse.

Level	Diaphragm Bending Capacity at Core Walls (kNm)	
	Failure Section ABCD	Failure Section EFGA
Level 4	18737	9543
Level 5	18737	11365
Level 6	18737	12901

Table 18 - Diaphragm in-plane bending capacity at critical sections adjacent to North Core (Refer Figure 166 for identification of failure sections ABCD and EFGA)

FLOOR DIAPHRAGM CONNECTIONS TO THE NORTH CORE WALLS

The diaphragm connections to the North Core walls were required to be designed using the "Parts and Portions" provisions of the New Zealand Loadings Standard NZS 4203:1984.

These provisions did not make sufficient allowance for buildings such as this where significant inelastic displacement was expected in the primary seismic resisting frame.

In this case while both South Wall and the North Core walls were designed and detailed as fully ductile, the South Wall was able to yield and displace inelastically well before the North Core walls.

Initial ERSA using NZS 4203:1984 design loads with the floor diaphragm connected at Level 2 and 3 at Lines D and D/E indicated that these would be overstressed at low levels of seismic demand. However it was analysed further and found that the Line 1 and 5 shear walls could pick up additional shear to compensate, should those diaphragm connections to walls D and D/E at level 2 and 3 be lost.

This counters the view that lack of diaphragm Drag Bars to walls D and D/E at Levels 2 and 3 necessarily initiated the collapse.

The need for ties or Drag Bars to the shear walls on Line D and E was identified by an Independent Consulting Engineer during a pre-purchase review for a potential purchaser in early 1990. Correspondence from the Design Engineer to the Independent Consulting Engineer states:

> "The agreed maximum tie load is 300 kN per tie. We understand that this load would be reduced on lower floors, in accordance with the "Parts and Portions" section of NZS 4203:1984."

The documentation of the connection of the Drag Bar ties into the slab and walls obtained from the Design Engineer (Figure 167 and Figure 168), showed that the Drag Bar actions were calculated following the provisions of NZS 4203:1984. Bars were not designed or installed in Levels 2 and 3. This seems to have been deliberate and appeared to be based on the assumption that adequate shear capacity was provided at Walls C and C/D into the North Core at those levels to cope with diaphragm demands.

The authors assessment of the Drag Bar nominal capacities (ϕ=1.0) at level 4, 5, and 6 - are shown in Table 19.

Limit state capacities were calculated as the minimum of Drag Bar tensile yield; wall anchor shear, concrete crushing and pull-out; Drag Bar anchor shear, concrete crushing and pull-out.

Anchor capacities were calculated in accordance with the July 2011 edition of the FIB Design of Anchorages in Concrete guide (FIB 2011).

Wall	Level	Drag Bar kN	Wall Anchors kN	Slab Anchors kN	**Limit Capacity kN**
D	L4	698	302	420	**302**
D	L5	698	503	420	**420**
D	L6	698	603	630	**603**
D/E	L4	540	403	420	**403**
D/E	L5	540	503	558	**503**
D/E	L6	540	703	698	**540**

Table 19 - Diaphragm Drag Bar nominal capacities.

The slab diaphragm capacity itself was found to be less critical than the Wall D and D/E connections even ignoring the contribution of the profiled metal decking. The profiled metal decking would have been able to develop its tensile capacity in proportion to the level of development of the decking from the slab support to the critical location. Its contribution to shear capacity would also have been significant.

Along Line 1 and 2 the profile metal decking had pulled free of the beam lines during the collapse. This is consistent with the columns on that line settling and the slab being temporally held up along Line 1 and 4.

Figure 165 - North Core slabs leaning against the North Core showing that their collapse occurred after collapse of the Line 3 frame

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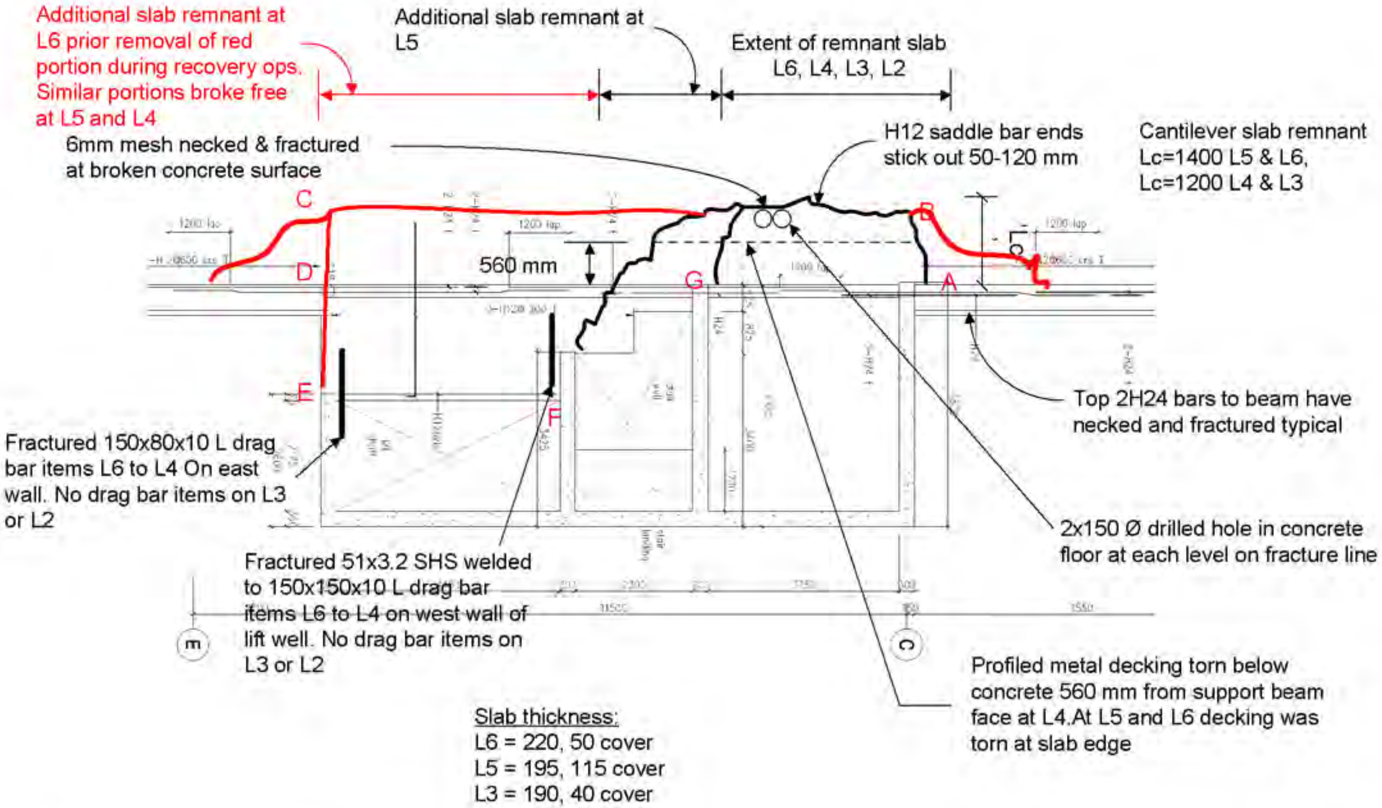


Figure 166 - North Core slab remnants after collapse based on site measurements in black and inferred by collapse photos in red..

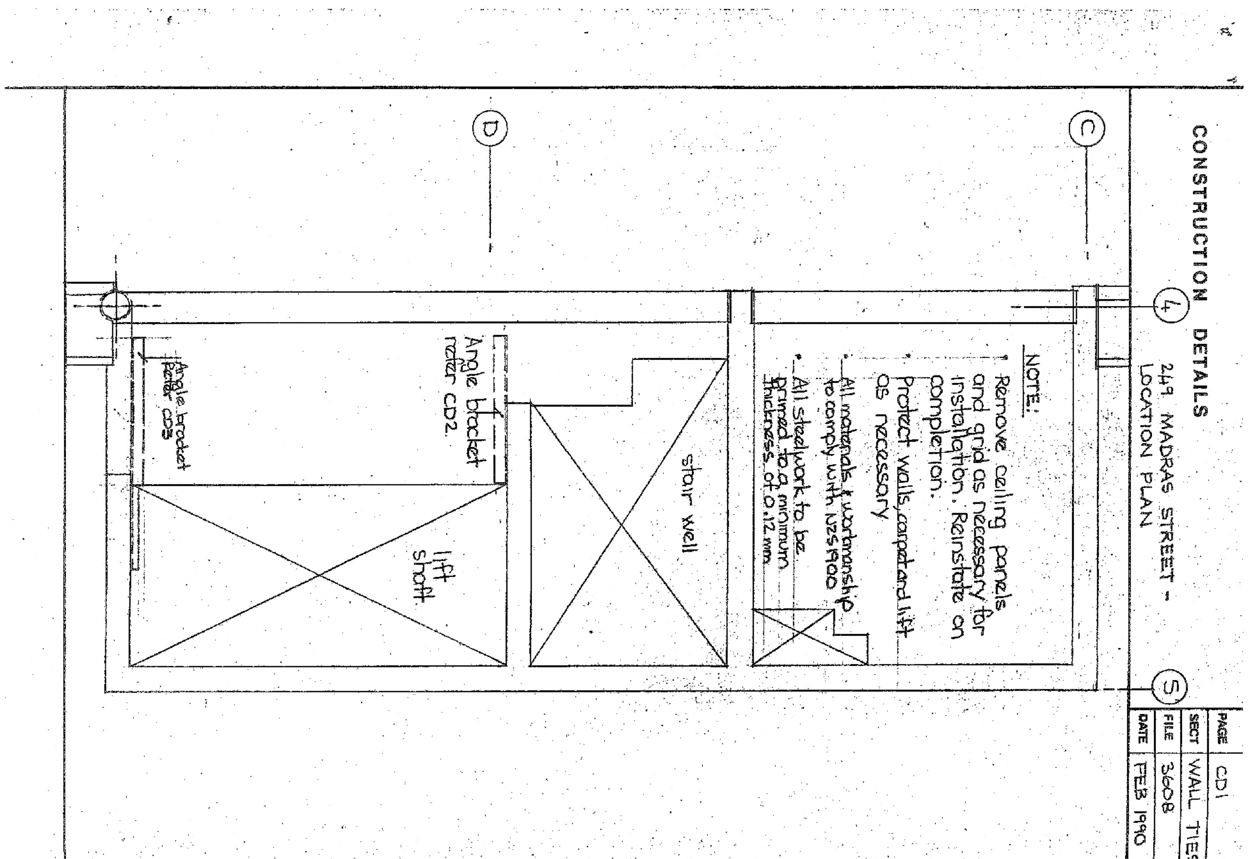
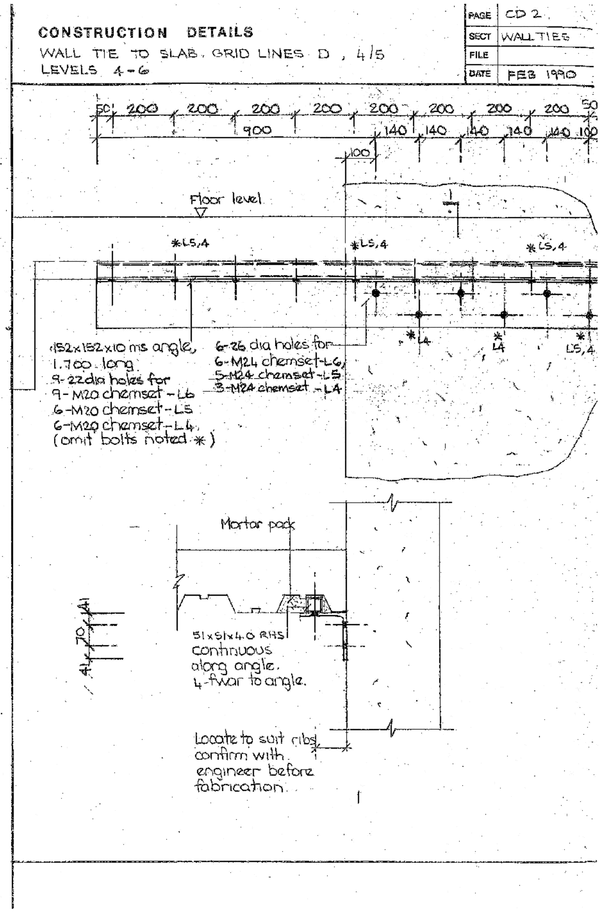


Figure 167 - Drag Bar locations on North Core walls



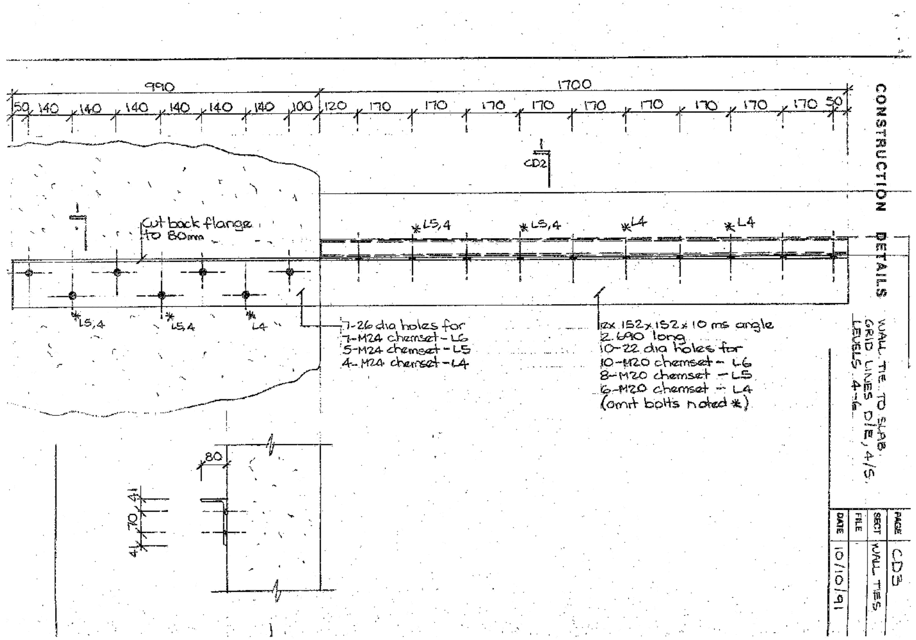


Figure 168 - Drag Bar details

APPENDIX H- GEOTECHNICAL REPORT SUMMARY

The general ground conditions at the site are described in Tonkin and Taylor's report as follows:

"The top four metres of the soil profile appear very consistent over the whole site, with silt (moist, firm) generally down to 1.5 m depth, overlying silty fine to medium sand. The water level is within this sand.

The geotechnical report of 1986 interpreted site conditions to differ below this level as follows:

- Over the major portion of the site, a thick dense gravel layer of 5 to 6 m thickness is present, overlying a deep layer of dense sand.
- For the remainder of the site, over the NE quadrant, the gravel is not present and is replaced by more sand and silt.

The 1986 report pointed out that:

> "... the transition between the gravel and soft sediments overlying the sand .... is quite abrupt and crosses the north-east corner of the site."

The appended Geotechnical Advice by Tonkin and Taylor concluded:

> "The geotechnical investigation carried out (by others) in 1986 was typical of the time and appropriate for the expected development. The report contained recommendations for further investigation. A modern investigation would now likely involve more deeper boreholes with more sampling and SPT's. Cone Penetration Tests would offer the opportunity of mapping the "transition" between gravel/no-gravel areas and also quantitative data for liquefaction analysis. Shear wave measurements would enable assessment of dynamic response parameters for dynamic analyses."

Liquefaction was not mentioned in the 1986 geotechnical report though the potential for liquefaction in Christchurch was well known at the time. Some of the soils at depth could have been subject to liquefaction or strength loss.

The type of foundations employed for the CTV building was typical for the size of the building and the Christchurch CBD. Provided liquefaction was not an issue, the shallow spread footings would seem appropriate and design recommendations were conservative for static conditions...."

One area of localised surface water or liquefaction was reported on the west side of the adjacent empty site to the west of CTV adjacent but this may have been due to the fire fighting that occurred. Otherwise there have been no reports of obvious liquefaction in the immediate vicinity of the CTV building.

On the subject of liquefaction, from Tonkin and Taylor's geotechnical review; "In summary, a thin layer, between water level at 2.5 – 3 m depth and gravel at 3.5 to 4 m depth, may have liquefied during and following the February earthquake. At the NE quadrant, this may have extended deeper. The limited thickness of the layer and the confining effect of the larger footings would mean complete bearing future would be unlikely, but "yield" with resulting settlement and differential settlement could have occurred.

In order to carry out a dynamic analysis of the CTV building for earthquake loading, the structural analysis required representation of the soil-foundation interaction as "subgrade reaction" stiffnesses. Tonkin and Taylor carried out computations using the Barkan formulae to give probable lower bound soil stiffness parameters, most likely parameters and probable upper bound parameters for use in the structural analyses that were carried out for this investigation.

Seismic ground motions at the CTV site were deduced from strong-motion recordings surrounding the CBD. The five stations of interest were:

Botanical Gardens: CGBS

Cathedral College: CCCC

Christchurch Hospital: CHHC

Rest Home Colombo Street: REHS

Page Road Pumping Station: PRPC

The last two of these (REHS and PRPC) showed significantly higher amplification than the others, both with respect to Peak Ground Accelerations (PGA) and spectral accelerations.

A borehole (BH 103) drilled for the Department of Building and Housing (DBH) at the REHS site logged significant thickness of "very soft organic silt" and "very soft peat". The PRPC station is located in a known liquefaction zone, with a nearby borehole (ECAN – M35/5124) logging sand to 27m depth, overlying sands and gravels.

The other three stations (CGBS, CCCC, CHHC) were all expected to have generally similar profiles of variable inter-bedded silts, silty and gravelly sands, overlying dense sands.

For this reason Tonkin and Taylor considered the REHS and PPPC records should be disregarded and the CTV site response should be assumed as similar to the average of the other three stations.

APPENDIX I - DESIGN AND CONSTRUCTION STANDARDS AND SPECIFICATION CLAUSES

A selection, but not exhaustive listing of relevant design and construction clauses, from Standards, Specifications and the Building Permit, referred to in the text are listed for the readers' convenience as follows:

PLAN AND VERTICAL IRREGULARITY

Plan and vertical irregularity criteria in the General Structural Design and Design Loadings Standard NZS 4203:1984 are as follows:

Cl. 1.4.2 "…the deflections of the structure as a whole, and any of its parts, shall not be such as to impair strength or serviceability of the structure."

Cl. 3.1 "The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically around the centre of mass of the building."

C3.1.1 "….Geometrically dissimilar resisting elements are unlikely to develop plastic hinges simultaneously, and ductility demands may also be increased by torsional effects."

Cl. 3.4.7.1(c) "For irregular structures more than 4 storeys high, horizontal torsional effects shall be taken into account by 3-D modal analysis of cl 3.5.2.2.2." (ie ERSA)

C3.4.7.1 "It should also be remembered that in torsional situations energy dissipation cannot usually be distributed evenly among resisting elements…. Structures of moderate eccentricity are those for which the torsional component of shear load in an element most unfavourably affected does not exceed three quarters of the lateral translational component of shear load".

INTER-STOREY DRIFT LIMITS

Drift limit criteria in NZS 4203:1984 were as follows:

Cl. 3.8.1.1 "Computed inter-storey deflections shall be those resulting from the application of the horizontal actions specified in section 3.4 or 3.5 and multiplied by the factor K/SM appropriate to the structural type and material, … and K=2.2 for the method of section 3.5 (ERSA)".

Cl. 3.8.1.2 "Computed deformations shall neglect foundation rotations."

Cl. 3.8.3.1" Inter-storey deflections computed in accordance with 3.8.1 between two successive floors shall not exceed …. 0.010 times the zone factor … where the zone is: 5/6 for seismic zone B …"

SEPARATION OF SECONDARY STRUCTURAL ELEMENTS

Separation of elements criteria in NZS 4203:1984 were as follows:

Cl. 3.8.4.1(a) "…infillings… (cl 3.8.4.1(b)) shall be so separated from the structure that there is no impact when the structure deforms to twice the extent computed by clause 3.8.1."

Cl. 3.8.4.1 (b) "Pre-cast concrete claddings"… (cl3.8.4.2 (b)) "shall be separated so that there is no impact when the structure deforms to the computed deformations in cl 3.8.1"

DESIGN OF REINFORCED CONCRETE SECONDARY ELEMENTS

The requirements for the design of secondary structural elements by the Code of Practice for the Design of Concrete Structures NZS 3101:1982 were as follows:

Designation of Group 1 and 2 Secondary Elements

Cl. 3.5.14.1 "Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 by virtue of their detailed separations are not subjected to loading induced by the deformation of the supporting primary elements or secondary elements of Group 2.

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to … loadings induced by deformation of the primary elements."

Group 1 Separated Elements

Cl. 3.5.14.2 "Group 1 elements shall be detailed for separation to accommodate deformations $\nu\Delta$ …. Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, … For elements of Group 1:

…(c) ….Fixings for precast units shall be designed and detailed in accordance with 3.5.15."

Cl. 3.5.15.1"When seismic deflection of the structure results in relative movement between a precast element and the points on the structure to which it is fixed, the fixings shall be designed to give clearance for the relative movements at these fixing points, corresponding to the seismic deflection computed in NZS 4203."

Cl. 3.5.15.2 "In buildings where the relative movements at the fixing points, computed in accordance with 3.5.15.1, are provided for by the capacity of the steel fittings for ductile deformation, and the relative movements do not require deflections in the fixings in excess of twice their yield deflection, the clearances required by 3.5.15.1 need not be provided."

Cl. 3.5.15.3 "For exterior elements and elements adjacent to any means of egress, the fixings, together with their anchorages shall be designed to deform in a ductile manner under movements exceeding the clearances required by 3.5.15.1.

Group 2 Non-separated Secondary Elements

Cl. 3.5.14.3 "Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\nu\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour.

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\nu\Delta$,...

(d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation, $\nu\Delta$ specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation.

(e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one quarter of the amplified deformation, $\nu\Delta$, of the primary elements, as specified in NZS 4203.

(f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 $\nu\Delta$ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise the additional seismic requirements of other sections shall apply."

CONSTRUCTION MONITORING AND INSPECTION REQUIREMENTS

It is likely that the Council by-laws required construction monitoring and inspection relevant to the CTV Building construction to be as follows:

Building Permit Conditions (Application No. 1747)

"Item 2 The Engineer responsible for the structural design (including the foundation system) confirming in writing that the intent of his design has been complied with before the building is occupied."

Code of Practice for the Design of Concrete Structures NZS3101:1982

Section 1.1 states that "It is only applicable to structures and parts of structures complying with the materials and workmanship requirements of NZS 3109".

Specification for Concrete Construction NZS 3109: 1980

Cl. 1.3.1 "All structural concrete shall be inspected by the person responsible for the design or by a competent representative nominated or approved by him. Such inspection shall establish that the design is being interpreted correctly and that the works are being carried out generally in accordance with the standards specified."

Cl. 5.6.3 "Types of joint. Construction joints shall be one of the following basic types:

….Type B construction joints shall be made at locations indicated on the drawings where it is necessary to develop shear friction across the joint. The surface of cast concrete shall be prepared by one of the methods specified in clause 5.6.2 the extent of treatment shall be such as to produce a roughened or broken surface to a depth of approximately 3 mm above and below the average level."

Cl.6.2.1 "… Concrete used in construction shall be either made on the site, or supplied ready mixed, or supplied in the form of precast products. Site mixed concrete production shall comply with NZS 3104 or NZS 3108 as appropriate. Ready mixed concrete and concrete used in the production of precast products off the site shall comply with NZS 3104."

Cl. 6.10.1"General. Prior to commencement of the supplying of concrete, the constructor shall produce evidence to the satisfaction of the engineer supervisor that the concrete mixes proposed for the project are adequately designed and that the production standards nominated can be achieved consistently."

Cl. 6.10.3.2 "Mix design. Evidence shall be provided to the satisfaction of the engineer supervisor that each concrete mix proposed has a target mean strength in compliance with the requirements of table 7 for the appropriate plant grading and specified strength."

Cl.6.11.1 " When the constructor wishes to change, in a manner likely to reduce its mean strength, a mix design which the engineer supervisor has approved as specified in 6.9.2 or altered as provided in 9.5.6.3, the engineer supervisor's approval shall first be obtained…"

Cl. 9.1 Tests shall be carried out during construction to check the compliance of the concrete with this specification… Proposals for location of sampling and frequency of testing shall be submitted to and subject to the approval of the engineer supervisor."

Specification for Concrete Production- High Grade and Special Grade NZS 3104: 1983Cl. 102 Definitions

“Engineer Supervisor means the professional engineer (or architect), his deputy, or authorized representative, nominated on behalf of the owner to supervise the works to which concrete is being supplied.”

“Engineer to the Plant means the engineer experienced in quality control of concrete production, and in mix design, nominated by the concrete producer to assume responsibility for mix designs and for the standard of production…”

Cl. 211.3 Availability of (Mixing) Records

“The records shall be available for inspection on request by the engineer supervisor.”

s being supplied.”

“Engineer to the Plant means the engineer experienced in quality control of concrete production, and in mix design, nominated by the concrete producer to assume responsibility for mix designs and for the standard of production…”

Cl. 211.3 Availability of (Mixing) Records

“The records shall be available for inspection on request by the engineer supervisor.”

APPENDIX J - DRAWINGS AND SPECIFICATION

Portions of structural and architectural drawings prepared by DENG and ARCH are shown to aid with interpretation of the report. (Portions are included with permission of DENG and ARCH).

A3 versions of some of the drawings are presented in an attachment to this report in Appendix L.

DRAWINGS

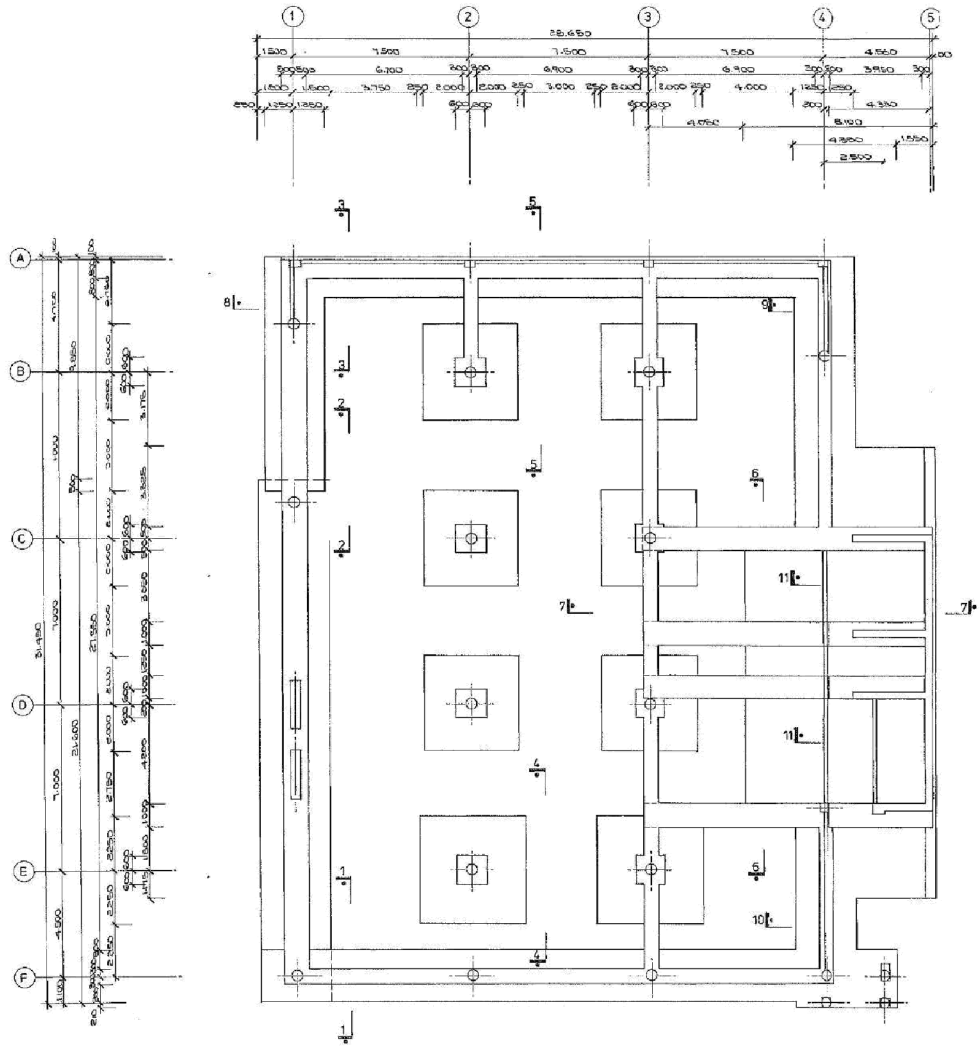


Figure 169 -Foundation Layout (Extract from DENG Dwg S2)

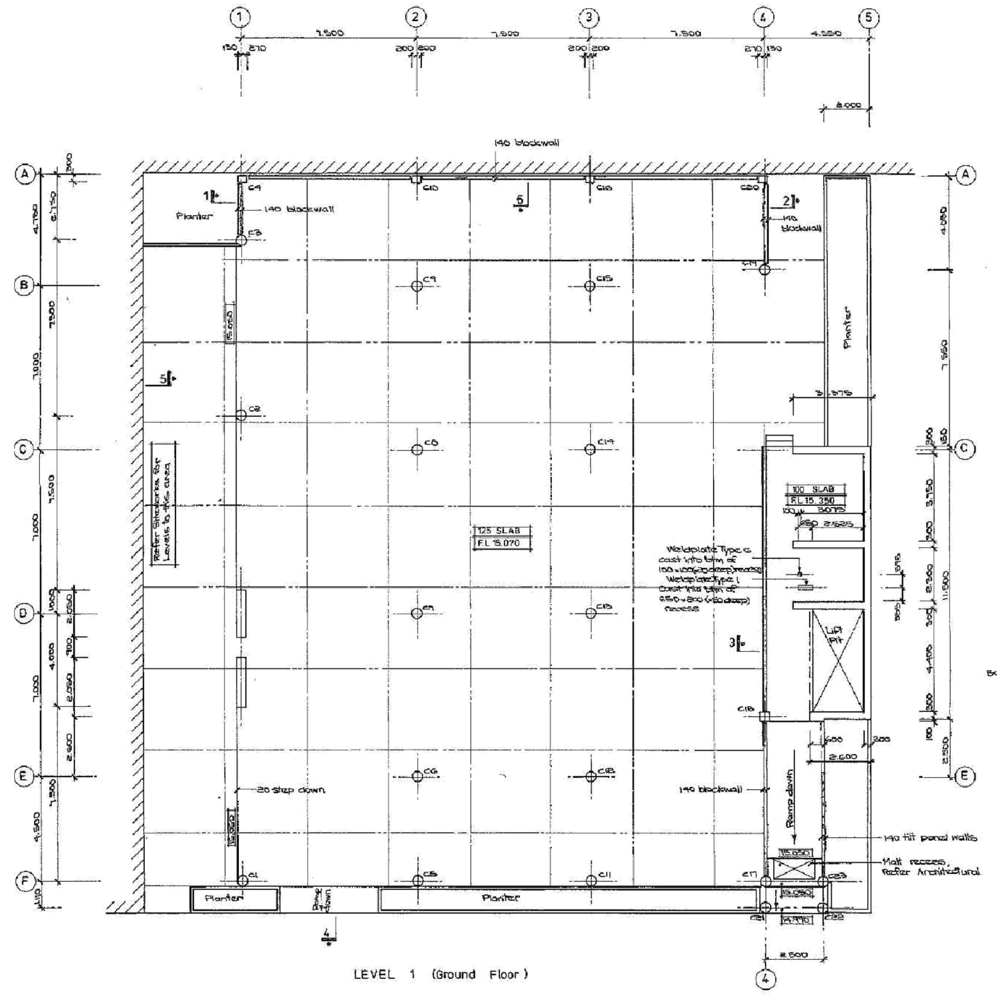


Figure 170 -Level 1 ground floor slab layout (extract DENG Dwg S9)

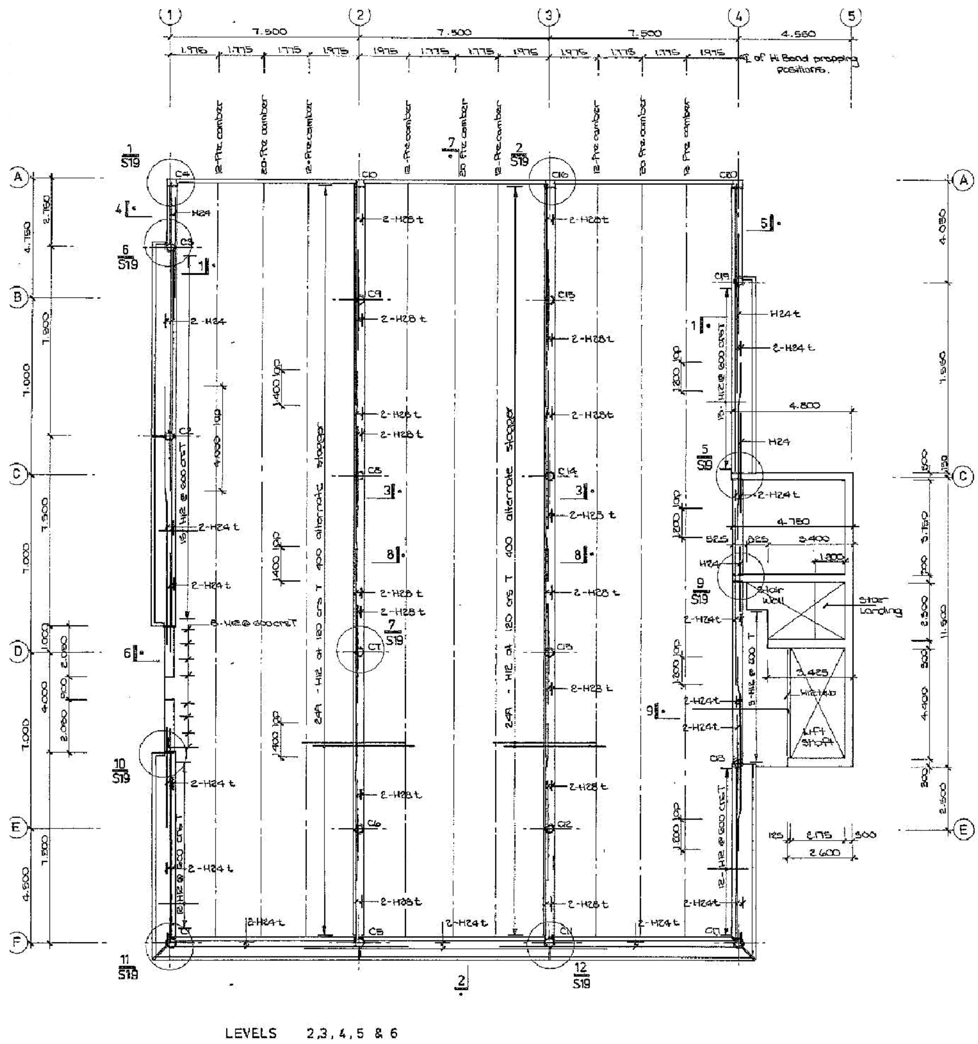


Figure 171 -Level 2 to 6 Floor Layout (Extract from DENG Dwg S15)

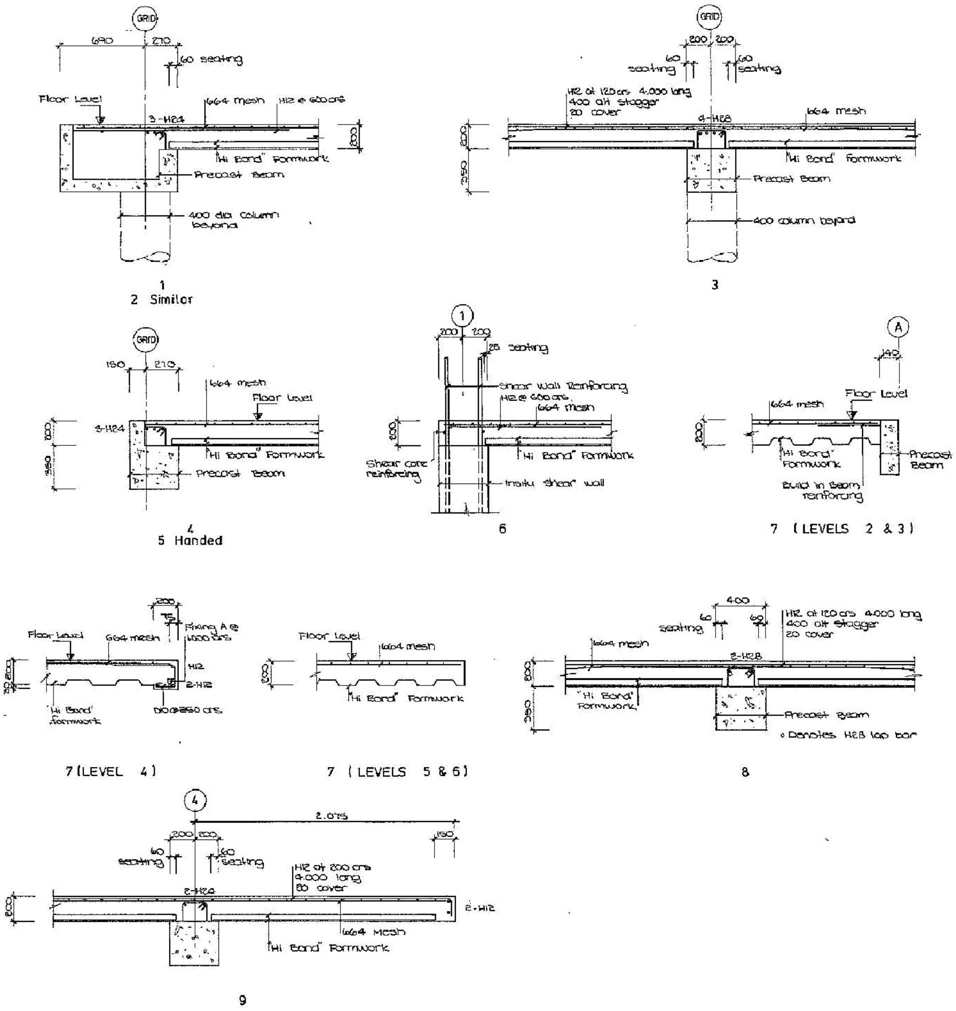


Figure 172 -Level 2 to 6 floor slab details (Extract from DENG Dwg S15)

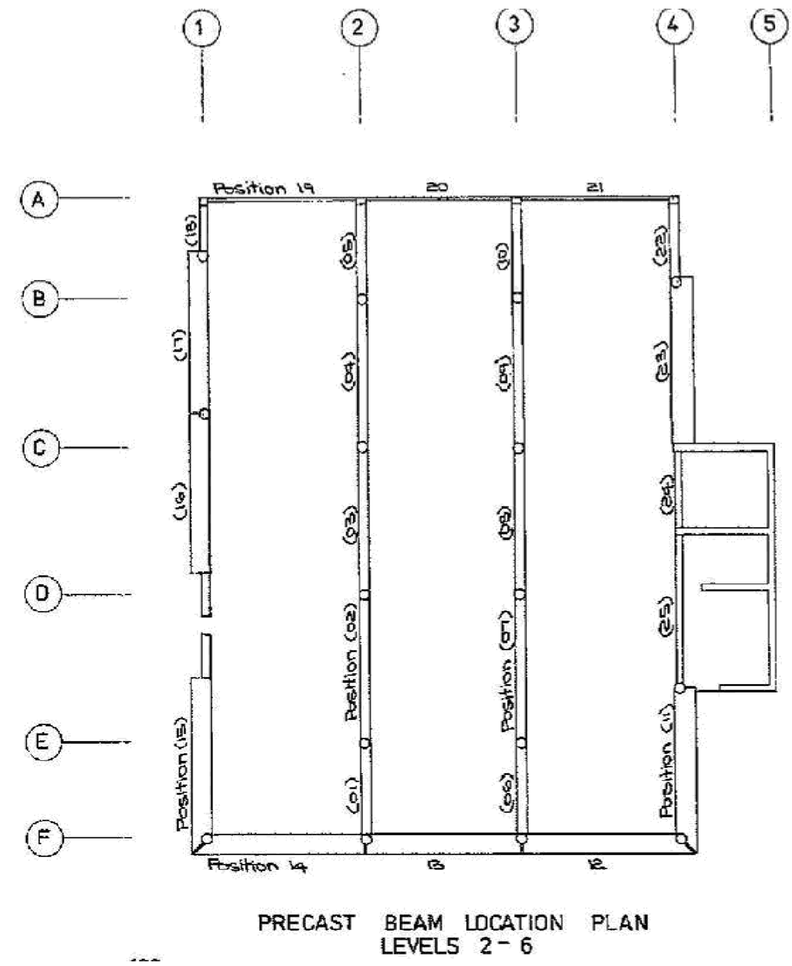


Figure 173 -Precast beam layout drawings (Extract DENG Dwg S18

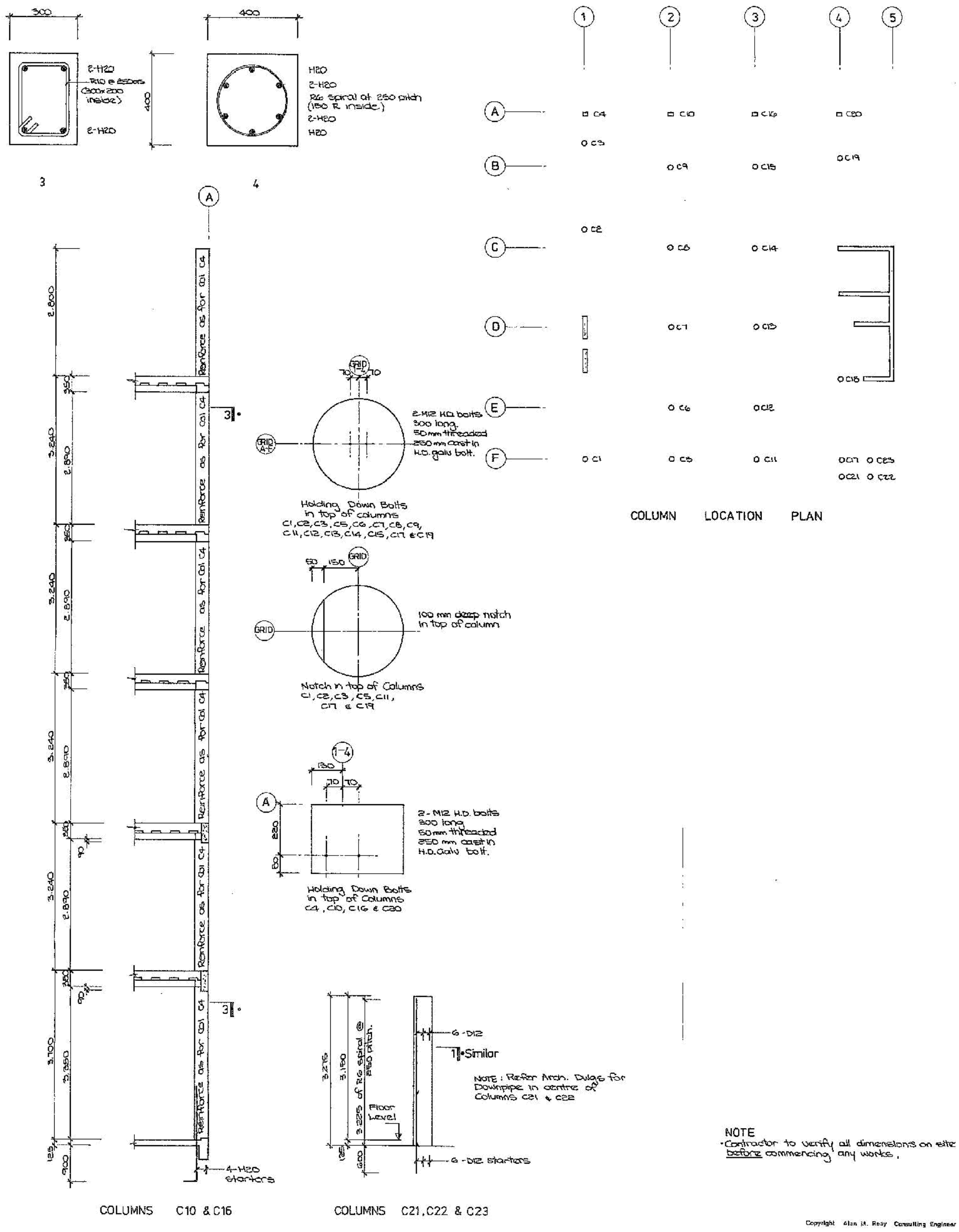


Figure 174 -Columns (Extract DENG Dwg S14)

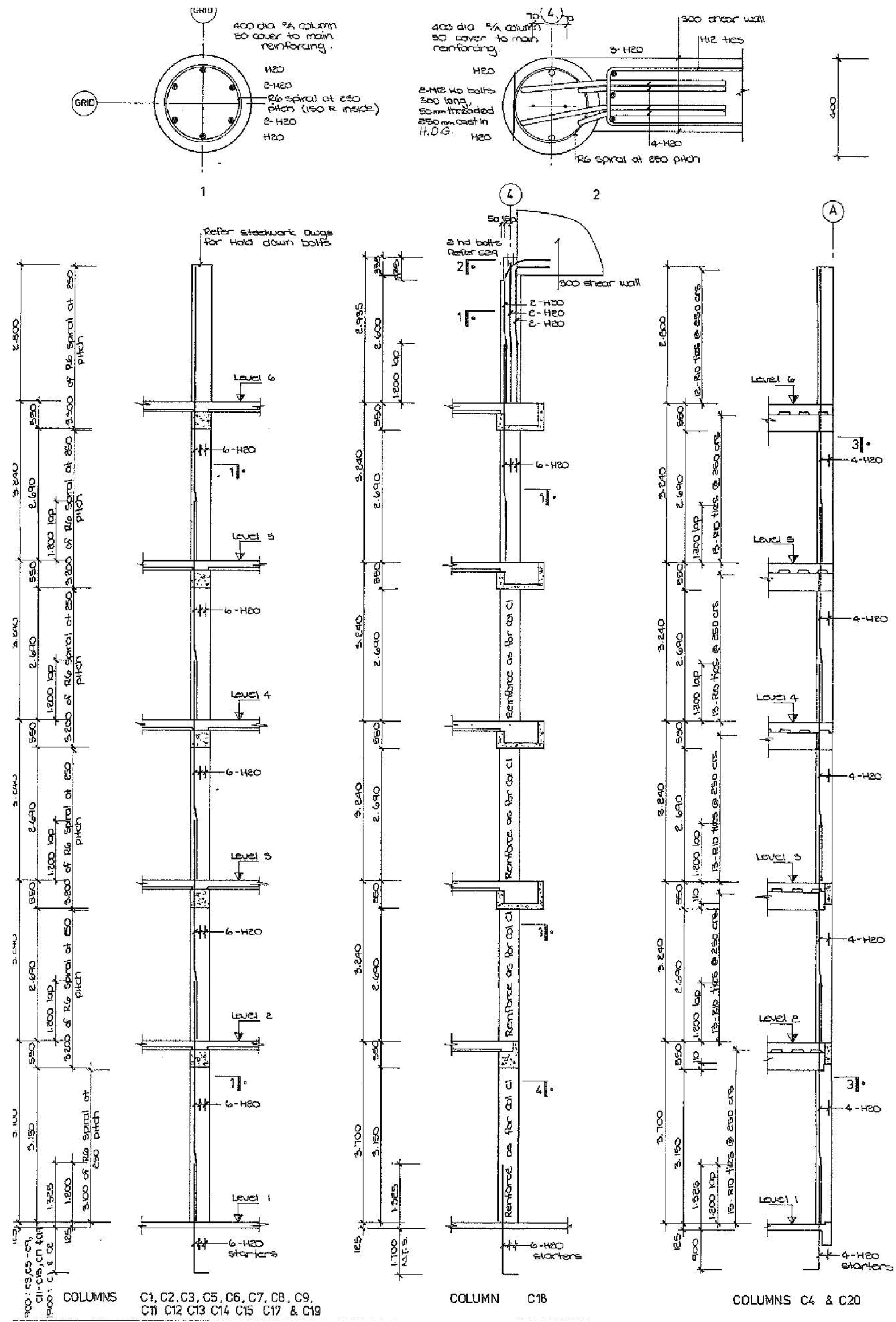


Figure 175 -Columns (Extract DENG Dwg S14)

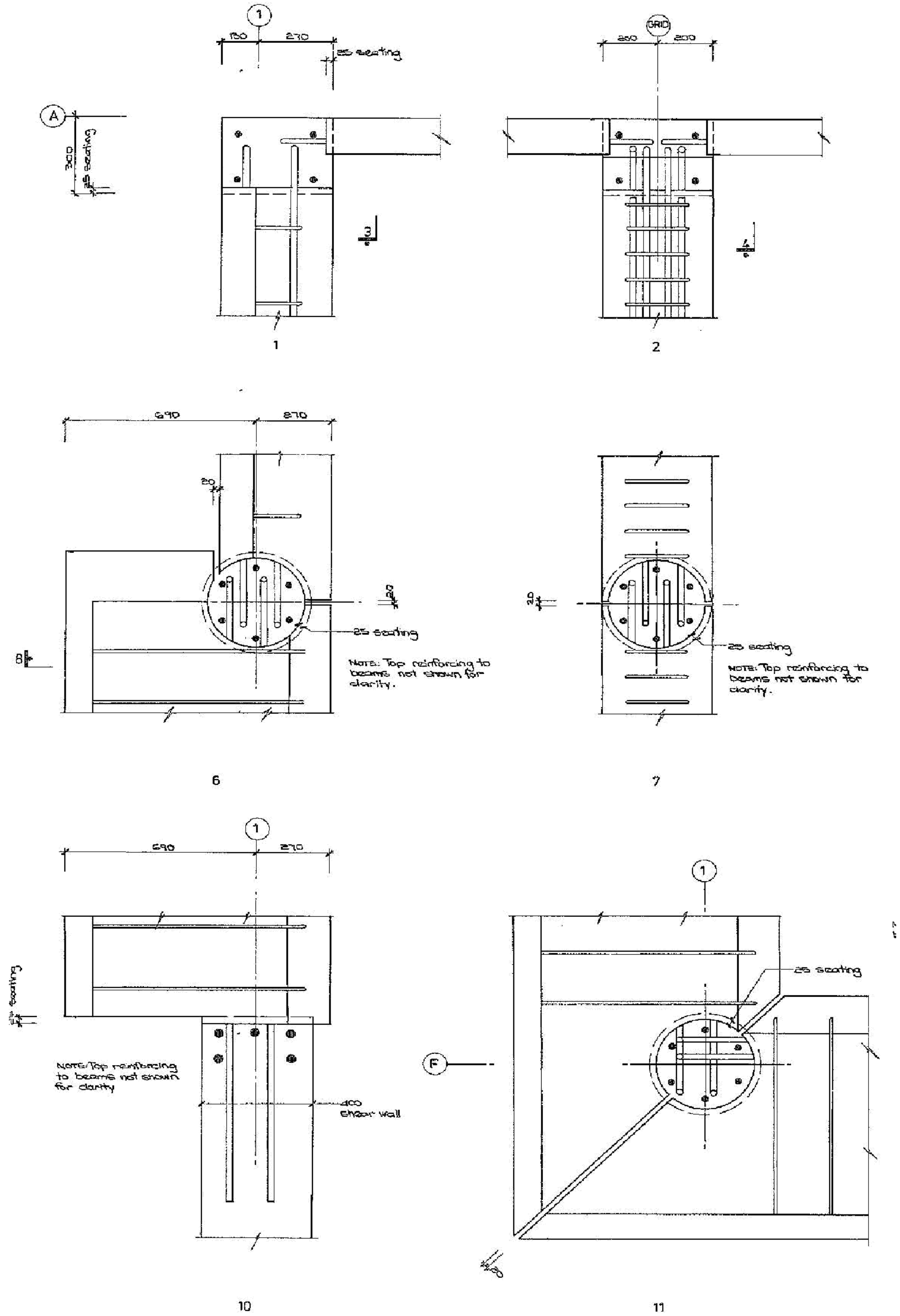


Figure 176 -Beam-Column Joints (Extract DENG Dwg S19)

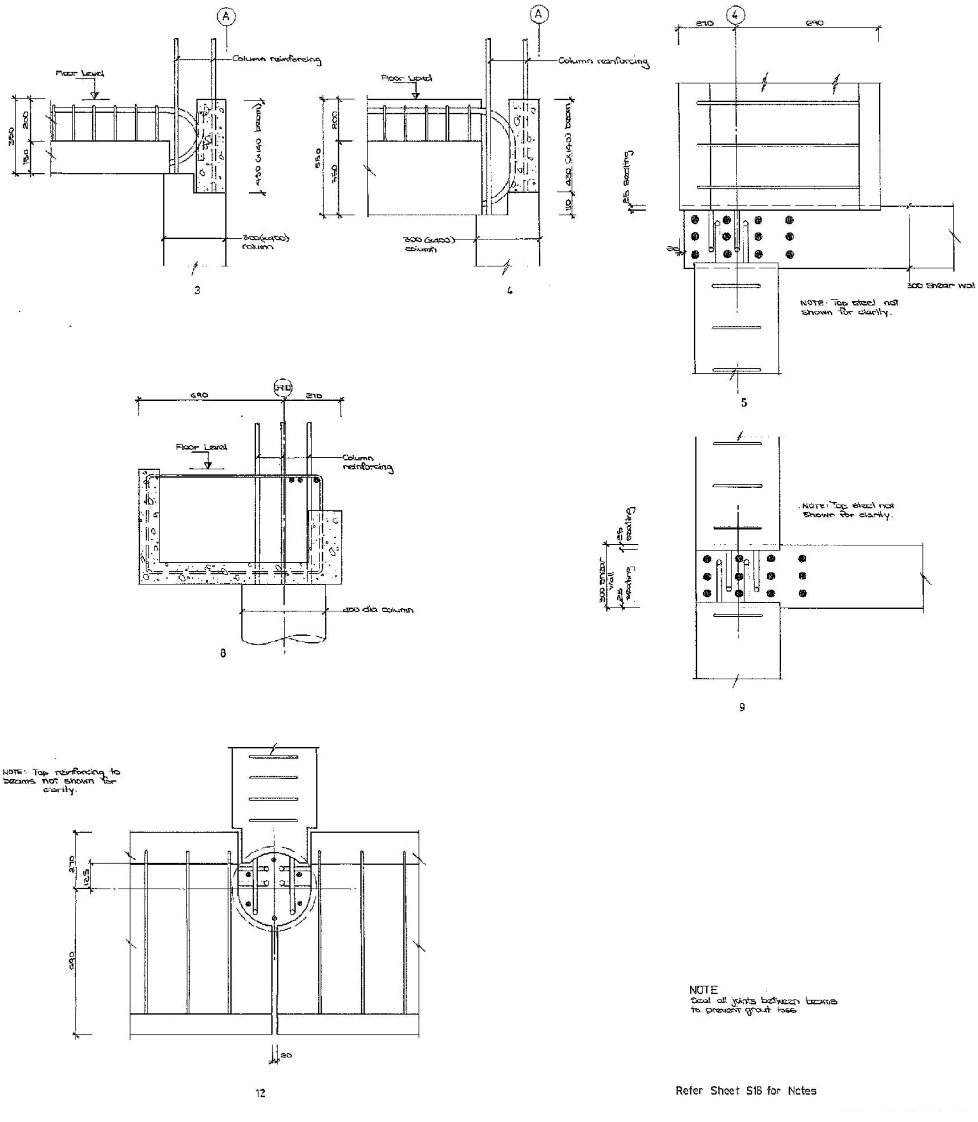
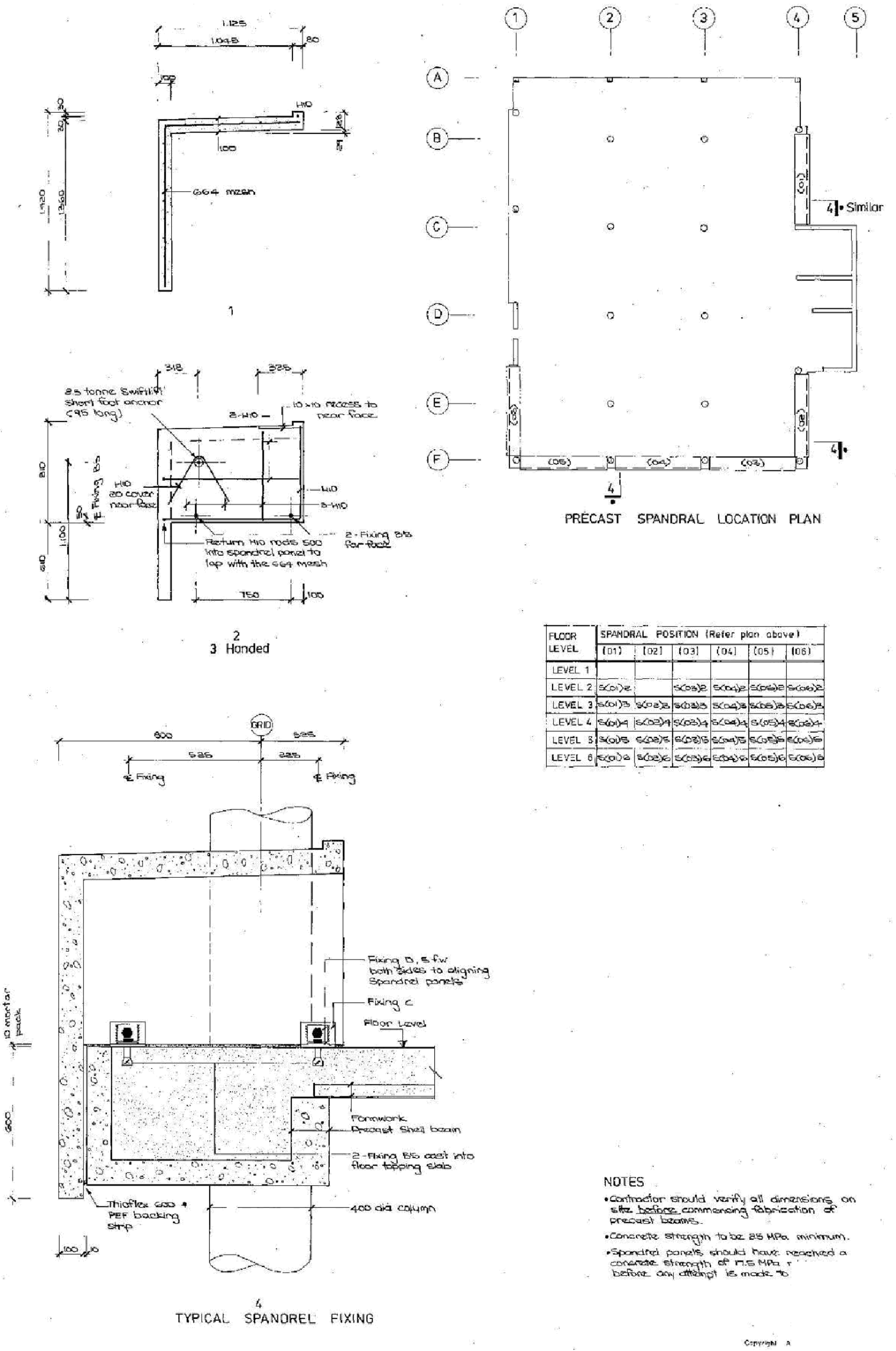


Figure 177 -Beam-Column Joints (Extract DENG Dwg S19)



FLOOR LEVEL	SPANDRAL POSITION (Refer plan above)					
	(01)	(02)	(03)	(04)	(05)	(06)
LEVEL 1						
LEVEL 2	S(01)2		S(03)2	S(04)2	S(05)2	S(06)2
LEVEL 3	S(01)3	S(02)3	S(03)3	S(04)3	S(05)3	S(06)3
LEVEL 4	S(01)4	S(02)4	S(03)4	S(04)4	S(05)4	S(06)4
LEVEL 5	S(01)5	S(02)5	S(03)5	S(04)5	S(05)5	S(06)5
LEVEL 6	S(01)6	S(02)6	S(03)6	S(04)6	S(05)6	S(06)6

Figure 178 -Pre-cast spandrel panels (Extract from DENG Drawing S25)

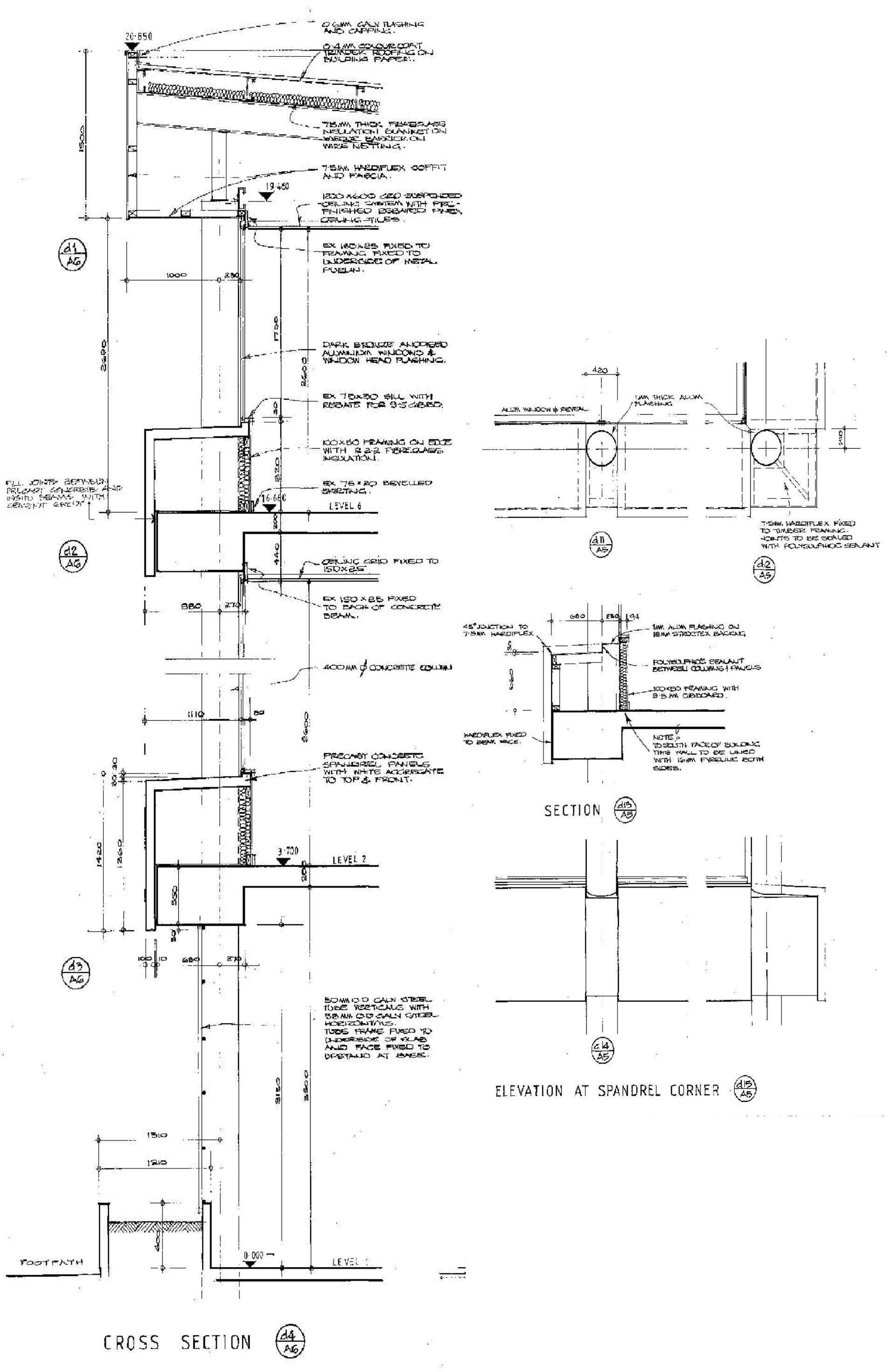


Figure 179 -Spandrel Panel Details at 400 mm Diameter Columns (Extract ARCH Dwg A7)

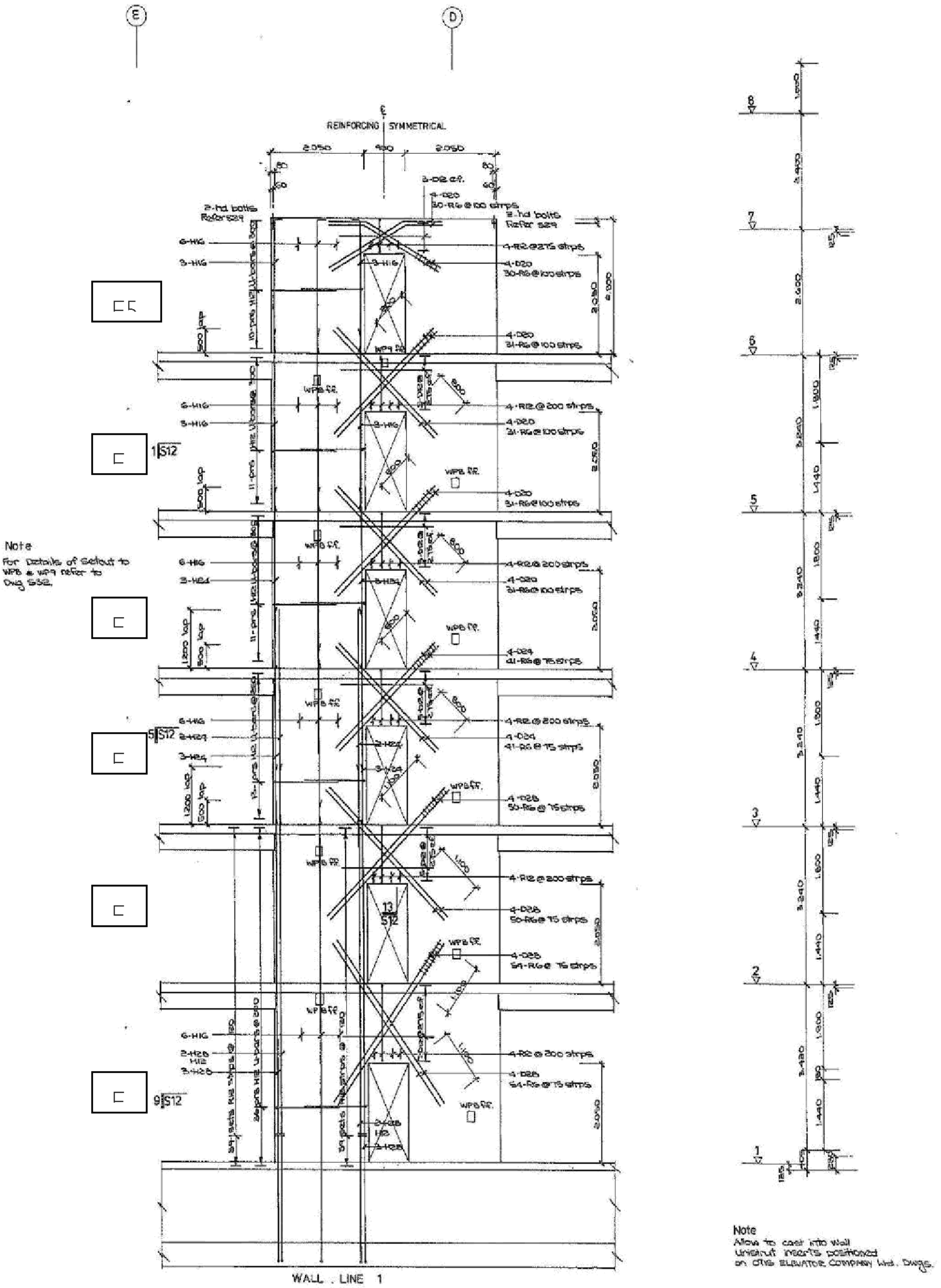


Figure 180 -Line 1 South Wall with Items E1 to E5A identified (Extract from DENG Dwg S10)

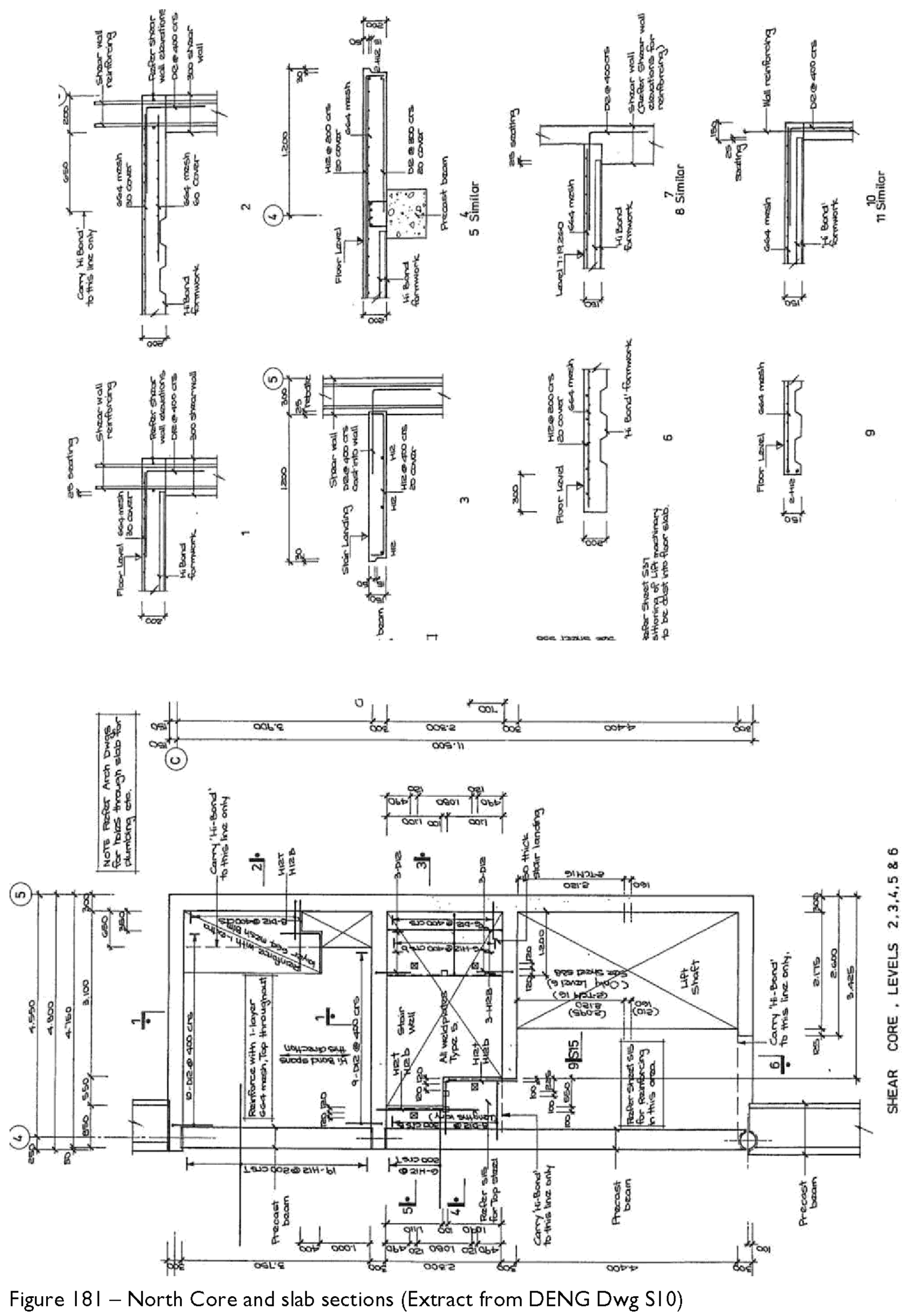


Figure 181 – North Core and slab sections (Extract from DENG Dwg S10)

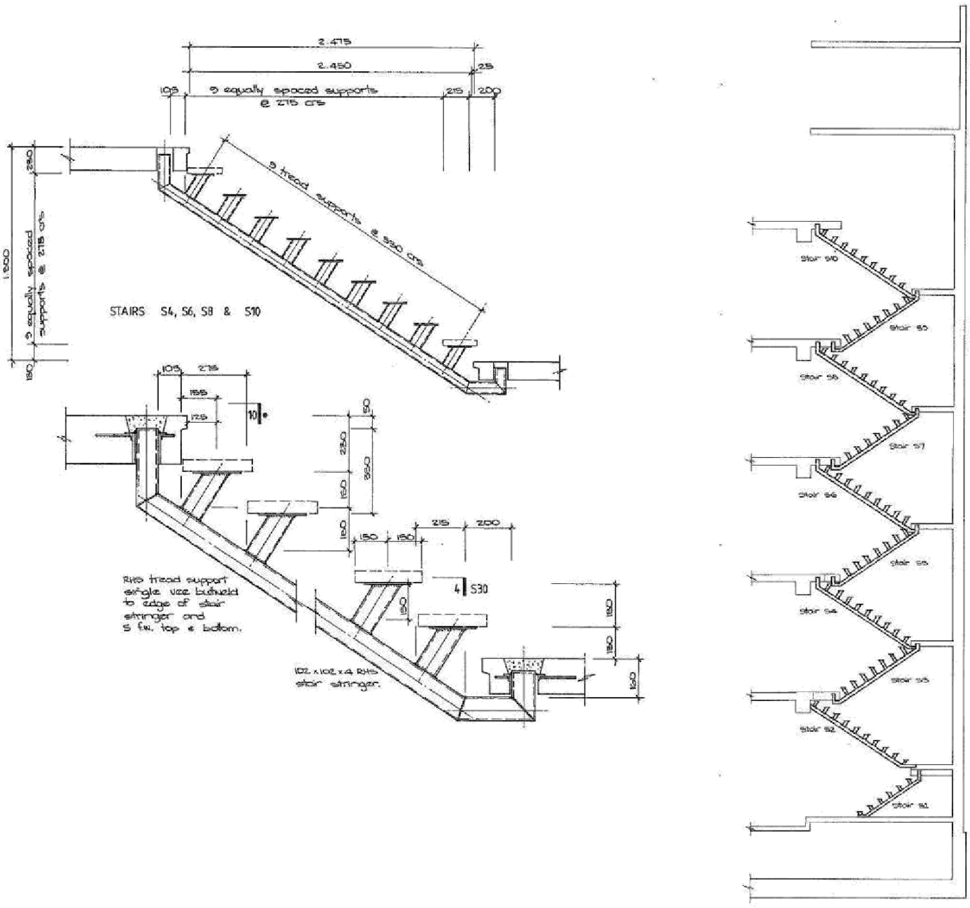


Figure 182 -Line 4 to 5 Stairs and detail of Stair S8 Level 4 to 5 (extract from DENG Dwg S31)

CONCRETE AND REINFORCING STEEL SPECIFICATION

2503

2. CONCRETE & REINFORCING STEELWORK.

2.1 GENERAL
Refer to the General and Special Conditions of Contract Clauses which shall apply to all work in this section of the Specification.

2.2 SCOPE
This section of the specification includes the supply, forming and casting of all cast-in-place, plain and reinforced concrete including all items necessary to complete the work indicated on the drawings and not specifically described elsewhere in this Specification. This section of the Specification includes the supply, erection, reinforcing and casting of the components of the approved proprietary floor system specified in Clause 2.16 of this Specification.
This section of the Specification includes the erection of all precast concrete. The PRECAST CONCRETE section includes manufacture of precast concrete units as detailed and delivery to the site if necessary.

2.3 MATERIALS AND WORKMANSHIP
The Contractor shall comply with all requirements of NZS 3109:1980 except where specified otherwise herein or instructed otherwise by the Engineer. A copy of this standard shall be kept on the site and relevant parts read with the following clauses of the Specification.

2.4 CONCRETE
Site concrete and concrete required to make good excavations shall be 10 MPa at 28 days or better. All other conrete shall be SPECIAL ro HIGH GRADE, from an approved ready-mix plant, and as defined in NZS 3109: Clause 6.2 and of the following strengths:-

Foundation beams and pads	20 MPa
Columns at Level 1	35 MPa
Columns at Level 2	30 MPa
Columns at Level 3	25 MPa
All other structural concrete including floors and walls	25 MPa

The maximum aggregate size shall be 19mm.

2.5 CONCRETE TESTS
The ready-mix supplier shall make control tests in accordance with NZS 3104, and shall pay the costs of such tests. Tests shall be made either at the ready-mix plant or at the site, except that if the Engineer specifically calls for tests at the site as a result of any dissatisfaction with the plant testing procedure, these shall be done by the ready-mix supplier.

2. cont'd... 2503

2.6 REINFORCEMENT

All reinforcement shall comply with NZS 3402 (1973). Bars prefixed with a 'D' on the drawings shall be deformed Grade 275 steel. Bars prefixed with a 'R' on the drawings shall be plain Grade 275 steel. Bars prefixed with an 'H' on the drawings shall be deformed Grade 380 steel. Mesh shall be hard drawn steel wire fabric to NZS 3422 (1972). All reinforcement and workmanship shall conform to the requirements of NZS 3109:1980.

2.7 FAIRFACE FINISHES

All concrete surfaces that will be visible in the finished job, or covered with paint, Enduit plaster, or tiles, shall be finished fairface. All concrete required to have a fairface finish shall be cast to a high standard using accurately constructed form work and to a high standard of workmanship. In addition to surface tolerances specified ~~below~~, the finished surface shall conform for blowholes with illustration 4 in the NZ Standard NZS 3114:1980 "Specification for Concrete Surface Finishes." Refer to the Architect's drawings for the finish required on concrete surfaces.

2.8 SLAB FINISH

Except as specified below, all slabs have a steel trowelled finish. Screed off and lightly wood float. Finish slabs with approved power floating and compacting machines to leave a dense, level surface which does not vary more than 6mm from a 3 metre straight edge, and not more than ± 15mm from true level.

2.9 SITE CONCRETE

Form and cast 50mm site concrete beneath main foundations and elsewhere as necessary to provide a clean, dry working platform. Ensure ground surface is clean and dry and there is no evidence of soft spots.

2.10 FOUNDATIONS

Form and cast main foundation beams as detailed. It is envisaged that the beams will be cast in stages with construction joints. Allow to scrabble or green cut the faces of these joints. The exact location and details of all construction joints are to be agreed with the Engineer before pouring concrete.

2.11 LIFT PIT

Form and cast lift pit walls and floor with sump as detailed. Build in PVC 140mm HYDROFOIL waterstop or similar to all construction joints in floor and walls. Waterproof the concrete with SIKA Plastocrete-N-Waterproofer or approved equivalent.

2 cont'd... 2503

2.12 GROUND FLOOR SLAB
Form and cast ground floor slab on damp proof course on compacted hardfill. Cast in strips and sawcut into panels where agreed by the Engineer on site. The maximum spacing of sawcuts or construction joints shall not exceed 3.75 metres.

2.13 PROPPING OF PRECAST BEAMS
Precast beams shall be propped to support the dead weight of the beam until the floor concrete has reached 20 MPa.

2.14 CHASES, HOLES AND NIBS
Form all chases, holes, upstands and nibs as shown on the drawings or required by other trades. Chases and holes shall be accurately positioned and formed at the time of casting the concrete.
Set concrete shall not be hacked unless specific approval is obtained from the Engineer.

2.15 BUILDING IN
As the work proceeds, build in all necessary bolts and other fixings. The Concretor shall ascertain from all other sub-contractors all particulars relating to their work with regard to order of its execution and details of all such provisions of fixings sleeves, chases, holes, etc., and of all necessary items to be built into concrete and shall ensure that all such items are provided for and/or positioned.

No claim will be recognized or allowed for at extra cost of cutting away or drilling concrete work already executed in consequence or any neglect of the Contractor to ascertain these particulars and make the necessary provision beforehand.

2.16 FLOOR SLABS
Concrete floors have been detailed to use the 'DIMOND HI-BOND H.S.' composite steel/concrete floor system. This has a profiled metal deck of 54mm overall depth, made from G500 steel, 0.75mm thick.

The floor shall be handled, laid, and fixed in accordance with the manufacturer's written "laying instructions".
Provide temporary propping to floors as shown on the drawings, with an upward camber to the propping lines as detailed. Floors shall be constructed of a uniform thickness, so that slab surfaces as constructed shall follow the cambered profile of the floor decking.
Propping shall extend over at least three levels at all times, to distribute the weight of the floor being poured into three lower floors, and to support mobile scaffolds being used to erect precast floor beams.

2503

3. PRECAST CONCRETE

3.1 GENERAL
Refer to the General and Special Conditions of Contract clauses which shall apply to all work in this section of the Specification.

3.2 SCOPE
This section of the specification includes the manufacture and supply on site of the following precast units:-

1. Precast beams
2. Precast wall panels

The work includes the fabrication and supply of all structural steel fittings to be built into the units as detailed on the drawings.

3.3 MATERIALS AND WORKMANSHIP
All formwork, concrete and concreting and finishing shall be in accordance with the relevant clauses of Concrete and Reinforcing Steelwork Specification except where noted otherwise in this section.

3.4 CONCRETE
All concrete shall be HIGH or SPECIAL GRADE complying with NZS 3109 Clause 6.2.Concrete for all precast work shall be 25 MPa at 28 days with 18mm maximum size aggregate.

3.5 TOLERANCES
All precast units shall be manufactured to the following tolerances unless stated otherwise on the drawings:

- Length	± 6 mm
- Cross Section	± 3 mm
- Squareness (of cross section and ends)	± 3 mm
- Twist (dimensions from plane containing the other three corners	± 3 mm
- Built in Items	± 5 mm

The above tolerances are given as a guide. Their application in any particular case shall be subject to interpretation by the Engineer.

3.6 FINISHES
All precast concrete exposed in the finished building shall be cast to a high standard using accurately constructed formwork and a high standard of workmanship. Precast items that do not meet the required standard to the satisfaction of the Engineer will be rejected. Formwork shall be such as to produce a high quality fair face finish on all exposed surfaces. Formwork shall be made from sheet steel or dressed plywood treated with a polyurethane finish to a high quality smooth surface, or similar.

3. cont'd... 2503

In general finished surfaces shall be smooth and formed with moulds or by careful trowelling. Surfaces shall be free from honeycombing, grout loss, excessive air holes or other imperfections. Arrises shall be straight clean and sharp and free from spalling or damage. All exposed surfaces shall have a similar appearance and standard of finish. Surfaces finished by trowelling shall be finished to the same standard and uniformly match surfaces against formwork.
Formwork shall be sealed at all corners, joins and inserts to prevent all grout loss.
All surfaces against which concrete is later to be cast shall be left roughened by brooming the poured face while the concrete is still plastic. Clean surfaces thoroughly from all laitance and loose concrete.

3.7 HANDLING
A high standard of finish is required and handling shall be such as to prevent any damage to units.
Approved lifting devices or hooks shall be provided in all precast units and these shall be made available to the Contractor for erection purposes and removed cleanly after use. Units shall be handled only by the hooks or devices provided. They shall be loaded and transported so that no forces are applied in excess of those occurring during normal lifting. Twisting forces shall not be permitted to occur. Units shall be strapped and secured to prevent movement or damage during transportation.

Details of lifting hooks and devices, and their positions, shall be submitted to the Engineer for approval before manufacture commences. Care shall be exercised at all times, that hooks or devices suffer no bending or other damage. Lifting hooks or devices set permanently in the units shall have a safety factor of at least 4 and for repetitive use shall have a safety factor of at least 6.

3.8 STACKING
Units shall be stacked on timber dunnage and suitable soft packing placed under the lifting points. Stacking shall at all times be such as to minimise the effects of creep and to avoid undue distortion of units.
Stacking of units shall be carried out on an area capable of withstanding the bearing pressures involved and in such a way that damage to units, lifting hooks, and to other embedded fixtures and to other units shall not occur.

3.9 MARKING
Mark all units with a mark number, orientation in finished job, and date of casting. The marking shall not be permitted to affect the fairface finish.

3.10 INSPECTION
The Engineer or his representative will inspect the precast units at all stages of manufacture to ensure conformity with this specification. Units which do not conform to the required tolerances, which shown grout leakage, which have been damaged, or which are otherwise defective shall be liable to rejection and may be used in the structure only at the Engineer's discretion.

3. cont'd... 2503

No repair work shall be done without specific instruction from the Engineer.

3.11 BUILDING IN

Supply and fix all lifting bolts, cast in sockets, timber grounds and other fixings as shown on the drawings or as required for the proper erection of the units in the finished work.

3.12 PRECAST SHELL BEAMS

Form and cast the beams as detailed including all reinforcing starters, structural steel fixings, holes for services, rebates, etc, as detailed.
The beams have been detailed to minimise their weight and hence crane capacity. The surface of the beams inside the stirrups shall be roughened to ensure good bond to the infill concrete. Outside the stirrups the surface shall be straight and level to receive the proprietary floor system.

Sides and soffits shall be finished as clause 3.6 where exposed in the completed building, otherwise to a reasonable fairface finish.

Figure183 Extract from DENG Pre-cast Concrete Specification

APPENDIX K – SEPTEMBER EARTHQUAKE DAMAGE REPORT

This report was prepared for the Building Owners by the Owner's Inspecting Engineer and is published with permission of the OIE. This report is referenced in Chapter 4.

249 Madras Street

Damage Report

4 September 2010 Earthquake

Christchurch

6 October 2010

249 Madras Street
Earthquake Damage Report

This report has been prepared for . No liability is accepted by this company or any employee or sub-consultant of this company with respect to its use by any other parties.

This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval to fulfil a legal requirement.

Quality Assurance Statement		
Task	Responsibility	Signature
Project Manager:		
Prepared by:		
Reviewed by:		
Approved for Issue by:		

Revision Schedule					
Rev. No	Date	Description	Prepared by	Reviewed by	Approved by

Prepared by:

249 Madras Street
Earthquake Damage Report

249 Madras Street
Earthquake Damage Report

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249 Madras Street
Earthquake Damage Report

INTRODUCTION

Following a telephone discussion with , Building Manager, on 24 September 2010, was invited to inspect the building at 249 Madras Street and to report on damage sustained during the 4 September Christchurch earthquake and subsequent aftershocks.

INSPECTION

The inspection was carried out between 10:00am and 2:00pm on Wednesday 29 September in the company of and from . from CTV was also present during the inspection of the ground and first floors occupied by CTV.

During the inspection, external walls were viewed from the ground with the exception of the west wall which is not accessible. Internal surfaces of walls in most rooms were viewed. In a few locations, ceiling tiles were lifted to view underside of floors and wall/column beam connections.

Some structural components are sealed behind fixed linings. These linings were not removed. We did not go inside the two car lift shaft.

BUILDING CONSTRUCTION DETAILS

We have not sighted any structural drawings for the building. I understand that the Building Manager was unable to obtain drawings and Council records are currently unavailable following earthquake damage to their archive systems.

We did obtain a copy of a layout plan for the ground and first floors from CTV.

From these limited drawings and from our inspection we believe that the building consists of the following structural systems. Photo 1 Appendix 1 shows the South elevation of the building.

The building is rectangular in shape measuring overall approximately 30.5m in the east-west direction and 26.0m in the north-south direction. It is five storeys high with a lift machine room and tank room at roof level.

The two car lift shaft, stairwell and bathrooms project from the north side of the building about half way along the north wall. A concrete shear wall extends across the north side of these facilities. Finger walls project at right angles to the north side wall at each end and between the facilities; four finger walls in total. On the south side of the building, opposite the north side shear wall, there is a further concrete shear wall in the plane of the south wall. We believe that these walls form the principal lateral load carrying systems for the building.

The remainder of the structure consists of gravity columns (mainly circular in section), perimeter beams and internal beams running in the east-west direction only at all floors. Beams and columns are all of concrete construction. Floors are of steel tray deck with concrete topping construction. Precast concrete spandrel panels are attached to the perimeter beams and weather proof the building up to window sill level.

We have no information regarding the foundations of the building but assume they consist of a combination of concrete strip and pad type footings.

SEISMIC PERFORMANCE OF THE BUILDING

Initial reports indicate that the 4 September 2010 earthquake produced ground accelerations in Christchurch similar to those required for current design of new buildings. The building at 249 Madras Street was, we understand, designed and constructed in the 1980's. It is likely that the code required design loads at the time were similar to or lower than current requirements.

Accepted design practice requires that buildings remain standing after the 'design earthquake" but it is expected that some damage would be inflicted. The building at 249 Madras Street does exhibit considerable damage with regard to linings and finishings. There is also some minor structural damage, but there are no obvious structural failures. In that respect we believe that the building has performed reasonably well.

We have not attempted for the purpose of this report to investigate or recommend restoration systems. However, diagonal shear cracking and cracking of construction joints has occurred in the shear walls, as reported below. We believe that there has been no yielding of the reinforcement in these walls and that structurally their integrity is still sound. However we would recommend repair of those cracks with a width of more than 0.2mm with epoxy injection. The damaged linings and finishings should also be repaired.

We comment on the various types of damaged observed as follows.

South Elevation Shear Wall

This wall is in fact what is termed a coupled shear wall. It has door holes in the middle of the wall at each storey providing access to the external fire escape. Beams across the door heads couple the walls, each side of the doors, together. The exterior of this wall is coated with a plaster splash coat. The rough texture of the finish on the wall makes it difficult to detect any cracking on the outside face, but there is one diagonal crack visible on the outside ground storey just below the fire escape landing. Photo 2.

At ground storey, the inside of the wall is strapped and lined with plaster board. The plaster board contains some significant cracks. However, the limited portion of the structural wall itself, visible above the ceiling tiles, showed no obvious cracking.

On the first storey, the inside of the structural wall is finished with a thin skim coat of gypsum plaster painted a light colour. Some diagonal cracks can be clearly seen in the gypsum plaster and measure up to approximately 0.2mm in width.

No cracking was observed in the gypsum plaster lining of this wall at levels above the second floor. It seems likely that cracking is present in the ground storey portion of the wall, similar to that of the first storey. We would expect that any cracks present are relatively fine and similar in width to those on the first storey. We recommend that the internal ground storey strapping and plaster board lining be removed to view the structure behind. The lining is damaged and would have to be replaced anyway. Cracks greater than 0.2mm in width should be repaired with epoxy injection. The external surface of the wall should be protected against the ingress of water in any fine cracks with the application of a silicon sealer or similar.

North Side Shear Walls

The north side shear wall and its adjoining finger walls exhibit some minor structural damage. There are some diagonal shear cracks in the walls surrounding the bathrooms and stairwell in the storeys below the second floor level measuring mostly in the order of 0.2mm in width but with three measuring possibly as much as 0.3mm in width. At higher levels there are a few finer cracks.

As visible in the stair well, there are construction joints in the walls immediately below and above each floor level. This is a normal construction practice. At almost all floor levels, cracking has occurred along part of the length of these construction joints and these cracks measure generally in the order of 0.2mm in width but with a few up to possibly 0.35mm in width. Photo 3. Again the cracks larger than 0.2mm in width should be repaired with epoxy injection and the external surface weather proofed.

At the north west corner of the north side shear wall at ground storey, a crack in the concrete is visible. We do not believe that this is earthquake damage. It is our opinion that the concrete cover thickness to the reinforcement has been inadequate here and the reinforcement has corroded. The oxidation of the steel makes it expand and this has fractured the concrete. This is not a major concern but it should be treated and repaired.

Columns, Beams and Spandrel Panels

As stated above, we believe that the columns and beams provide gravity support only and have not been designed to resist lateral loads. However, they do have some stiffness and when the building moves in an earthquake and they do attract some load. Generally we observed very little damage to beams and columns. However there are a few exceptions. The north-east corner column immediately above the third floor spandrel exhibits some minor cracking which is very fine and in our opinion requires no treatment. At the top storey, the first column west of the north-east corner of the building also exhibits some cracking. The appearance of the cracking is accentuated because the paint has chipped off at the cracks (photos 4). One of the south side columns at the top storey also exhibits some fine cracking. We recommend that the cracks in these upper storey columns be injected with epoxy resin.

The first floor beam on the north face of the building in the span between the north-east corner of the building and the adjacent column to the west has some fine diagonal cracking (Photo 5). We recommend that this crack be injected with epoxy resin. We did not see any signs of distress in beam column joints.

The precast concrete spandrel panels appear to have sustained very little damage. However, each side of the south side shear wall, the ends of the spandrels have been plastered. This plaster is spalling off at most levels as a result of differential movement caused by the earthquake. It is a hazard to people below. It should be removed, the concrete surface properly prepared and a strong bonding epoxy plaster re-applied. (Photo 6).

At the fifth floor level, the end of the spandrel panel on the north elevation adjacent to the lift lobby is showing signs of corrosion of the reinforcement. This can be seen out the lift lobby window. This is not a structural problem and has not been caused by the earthquake but it should be treated.

Flooring

As described above, the floor construction consists of a composite concrete topping and steel tray deck system spanning north to south between concrete beams. These floor systems are relatively light weight and flexible and it is common for them to exhibit some deflection. At most of the floors in the building at 249 Madras Street, it is possible to detect high points in the floor over the support beams and sags in between. This is not caused by the earthquake and is a fairly normal and acceptable effect of this type of construction.

In the limited number of locations where we removed ceiling tiles and observed the floor to beam connections we did not see any signs of distress. (Photo 7).

Non-Load Bearing Concrete Block Walls

At the west end of the building in the garage at ground storey there are concrete block infill panels between the structural columns. These block infill panels are separated by a flexible sealant from the columns. They do not appear to have suffered any damage.

At the next level up on the west end wall, the interior is timber framed and plaster board lined. It is not possible to view the exterior cladding because of the close proximity of the adjoining building. However, we assume that there is a similar concrete block wall, also separated from the structural columns. In the north-west corner of the building, the internal lining has been damaged by movement of the building. There is a gap between the internal framing/lining on the west wall and the north-west corner column. It is possible to see daylight through this gap. We assume that the sealant in the outer concrete block wall to concrete column joint has fallen out. This needs further investigation and repair.

At ground storey, there is a concrete block wall parallel to the north side shear wall but on the opposite side of the stair well. This wall has a thin gypsum plaster coating in the stair well. At the top of the wall the plaster coating has been peeled off. It appears that its was touching one of the stair well structural walls and the differential movement has damaged the plaster. There may also be some minor cracking of the top block course which should be repaired. However, this is not a structural component and does not contribute to the integrity of the building. Photo 8.

Internal Framing and Linings

At numerous locations at all levels, there is damage to internal framing and linings. Commonly, internal walls and their linings have been finished hard against structural walls and columns. With movement of the building during the earthquake(s), the structural components have applied in-plane loads to the stiff plaster board lined walls. There are many instances where the plaster board linings have been damaged where they adjoin the structural components. Sometimes, the plaster board has buckled some distance away from the structural wall or column. Photos 9 and 10. There are also numerous instances of plaster board cracking over door heads and under windows and elsewhere. Photos 11 and 12. Ceiling covings and skirting boards have also been damaged. At the south end of one internal north south wall on the second floor, the partition wall has racked sufficient for the double doors contained in the wall to be binding. Photo 13.

Where ceiling linings adjoin concrete columns, the plaster linings have been damaged. In some cases the rails for the suspended tiled ceilings have been buckled. Photo 14.

It would appear that partition walls running north-south have been damaged worse than others. There is some anecdotal evidence that the earthquake accelerations were higher in this direction. It also appears that the damage to partitions is worse on the second and third floors. This may be a result of the response of the building to the magnitude and frequency of the earthquake shaking.

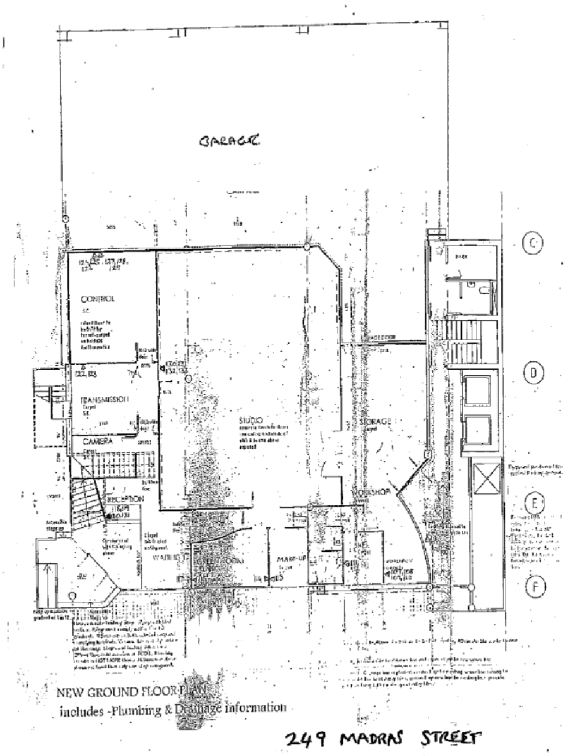
Windows

A few windows, particularly in the east elevation have been broken. Photo 15. This may also reflect greater movement of the building in the north-south direction. There are no windows in the opposite west wall but the damage to interior linings on that west wall is significant.

Rubber seals around a first floor window have also fallen out.

APPENDIX I

FLOOR PLANS AT 249 MADRAS STREET



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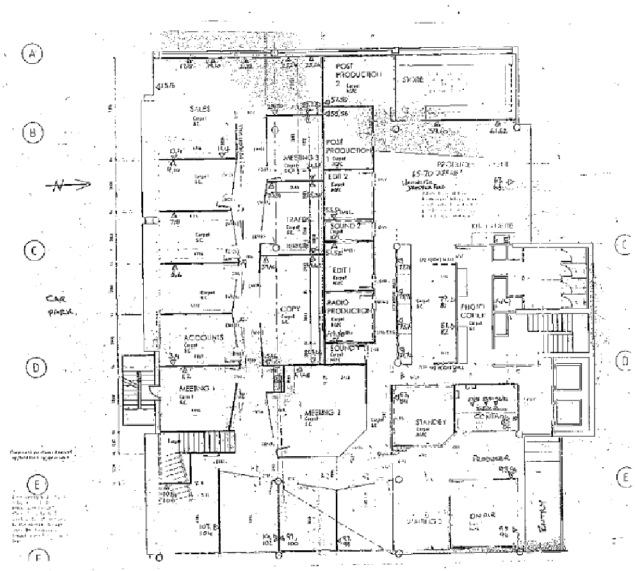

249 MADRAS STREET
FIRST FLOOR PLAN

249 Madras Street
Earthquake Damage Report

APPENDIX 2

PHOTOS OF DAMAGE AT 249 MADRAS STREET

FOLLOWING 4 SEPTEMBER 2010 EARTHQUAKE AND AFTERSHOCKS

Photo 1 South Elevation of Building

Photo 2 Cracking in south shear wall (not visible in photo) under fire escape landing

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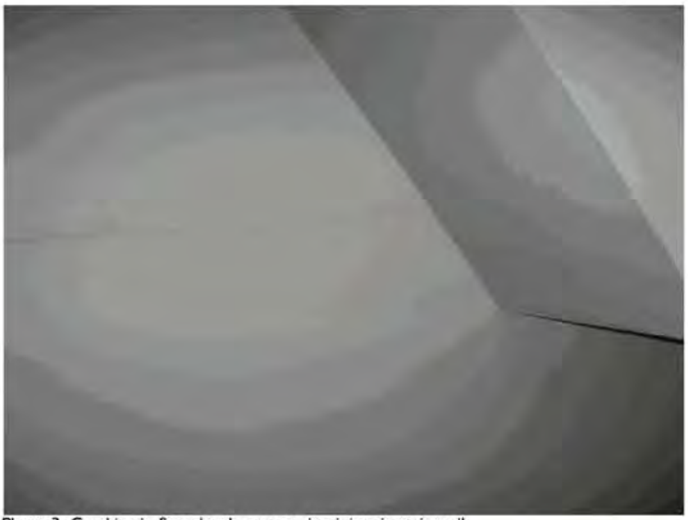

Photo 3 Cracking in floor level construction joints in stair well

Photo 4 Cracking in top storey column adjacent to lift lobby.

Photo 5 Cracking in first floor beam north elevation over entry.

Photo 6 Spalling of plaster off ends of spandrel panels.

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Photo 7 Internal bem column joint under first floor – no evidence of damage.

Photo 8 Spalling gypsum plaster off non-load bearing concrete block wall in stairwell.

Photo 9 Partition lining damaged at junction with concrete column.

Photo 10 Partition lining damaged at junction with concrete beam.

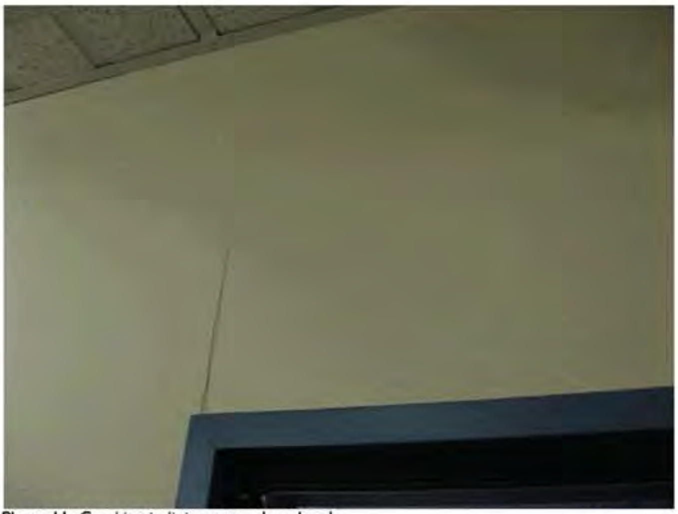

Photo 11 Cracking in linings over door head

Photo 12 Crackling in linings under window.

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Photo 13 Racked second floor partition wall and binding double doors.

Photo 14 Cracked wall lining and damaged ceiling coving.

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Photo 15 Broken window.

APPENDIX L – A3 DRAWINGS

This Appendix is published as a separate volume containing A3 drawings of the structure.