Christchurch Earthquake

CBD Building Performance Technical Investigation

Report on the Structural Performance of the Hotel Grand Chancellor in the Earthquake of 22 February 2011

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For:
The Department of Building & Housing

Final: 26 September 2011

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Executive Summary

In the short but violent Lyttelton aftershock of 22 February 2011, the Christchurch Hotel Grand Chancellor building suffered major structural damage. The extent of damage suffered by the building was significantly increased by the collapse of a key supporting shear wall which failed in a brittle manner.

The building survived the 4 September 2010 earthquake and the 26 December 2010 aftershock events without apparent significant structural damage and was fully in use when the February event occurred. During the approximate 12 seconds of intense shaking that occurred at 12.51pm on 22 February, the building suffered a major structural failure with the brittle rupture of a shear wall in the south-east corner of the building. This shear wall had supported vertically approximately one-eighth of building's mass and was also expected to carry a portion of lateral earthquake loads.

As a result of the wall failure, the south-east corner of the building dropped by approximately 800mm and deflected horizontally approximately 1300mm at the top of the building.

There was sufficient redundancy and resilience within the overall structure to redistribute the loads from the failing element and to halt the collapse.

This major movement induced other damage including column failure at the underside of the podium, beam yielding, stair collapse and precast-panel dislodgement. The collapse of the stairs, in particular, was dependant on the wall failure. Other more minor structural damage was consistent with what may have been expected in a well-performing reinforced concrete structure in a seismic event of this nature.

The investigation found that, for the most part, the structural design appeared to be compliant with the codes of its day. However, for the failed wall D5-6 it does appear that there were some items of non-compliance that most likely contributed to the failure. The magnitude of possible axial loads was under-estimated and the wall lacked the confining reinforcing needed to provide the ductility required to withstand the extreme actions that resulted from the February 2011 aftershock. In addition the assessed response of the building to this shaking exceeded the actions stipulated by both the current and contemporary loadings codes for a building of this type, structural period (of vibration) and importance.

1.0 Introduction

This report was commissioned by the Department of Building and Housing (DBH) in response to the Government's request for an investigation into the performance of significant building structures during the 22 February 2011, Lyttelton aftershock.

The Hotel Grand Chancellor is a high-rise reinforced concrete structure located at 161 Cashel Street in the Christchurch central business district. Prior to 22 February the building housed 15 storeys of premium quality hotel accommodation above 12 half-floors of a public parking facility.

The report has been prepared by Dunning Thornton Consultants and reviewed by an expert panel appointed by DBH. It examines the reasons for collapse and the general structural performance of the Hotel Grand Chancellor and compares that with contemporary and current design and construction expectations.

Hotel Grand Chancellor: Pre-September Earthquake
(source: C Lund & Son Ltd website)
Fig.1

2.0 Objective and Scope

The purpose of this technical investigation into the performance of buildings in the Christchurch CBD during the 22 February aftershock, is to establish and report on:

- *the original design and construction of the buildings;*
- *the impact of any alterations to the buildings;*
- *how the buildings performed in the 4 September 2010 earthquake, and the Boxing Day aftershock, in particular the impact on the buildings;*
- *what assessments, including the issuing of green stickers and any further structural assessments, were made about the buildings' stability/safety following the 4 September 2010 earthquake, and the Boxing Day aftershock; and*
- *why these buildings collapsed or suffered serious damage.*

The investigation will take into consideration:

- *the design codes, construction methods, and building controls in force at the time the buildings were designed and constructed and changes over time as they applied to these buildings;*
- *knowledge that a competent structural / geotechnical engineer could reasonably be expected to have of the seismic hazard and ground conditions when these buildings were designed;*
- *changes over time to knowledge in these areas; and*
- *any policies or requirements of any agency to upgrade the structural performance of the buildings.*

The investigation will use records of building design and construction, and will also obtain and invite evidence in the form of photographs, video recordings and first-hand accounts of the state or the performance, of the buildings prior to, during, and after the 22 February 2011 aftershock.

Matters outside the scope of the investigation

The investigation and report is to establish, where possible, the cause or causes of building failures. It is not intended to address issues of culpability or liability arising from the collapse of the building. These matters are outside the scope of the investigation.

3.0 Approach / Methodology

The methodology/approach undertaken to arrive at the conclusions derived in this report has involved information gathering, onsite observations and investigations, materials testing, numerical analysis, postulation and review.

Information available has included:

- Original structural drawings, design calculations and specifications.
- Construction monitoring reports from the original construction.
- Christchurch City Council property records.
- Reports following the September earthquake.
- Post-February survey records of ground levels.
- Post-February concrete strength test reports for selected shear walls.

A list of the documentation that was obtained is included in Appendix A.

Analysis has included:

- Derivation of seismic weight of the structure.
- 3D Static and limited dynamic analysis of the structure.
- Determination of seismic loads from contemporary and current codes.
- Comparison with actual actions experienced on 22 February, as derived from the recorded spectra.
- Some detailed analysis of the critical wall element.

4.0 Description and History

4.1. General

[Refer to Appendix B for annotated extracts from the original structural drawings, and to Appendix C for photographs]

The Hotel Grand Chancellor complex occupies a property on the north side of Cashel Street at number 161. An adjacent carpark was designed and constructed in the mid 1970's. This building, though structurally separate, shares vehicle access ramps with the hotel and appears from the street to be structurally contiguous with the podium of the hotel. *[Refer App. B page 2]*

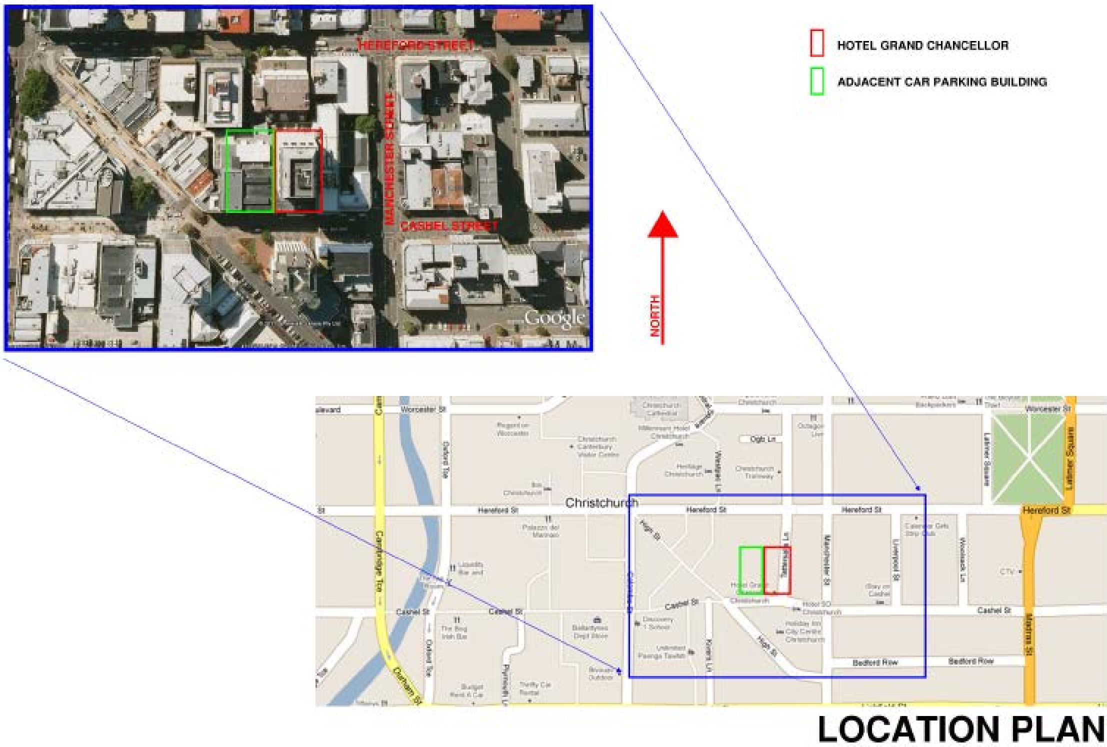


Fig. 2

The hotel itself has a tower with 15 levels of accommodation above 12 half-levels of carparking (equivalent to 6 full floors) and ground floor reception *[Refer App. B page 3].* The tower has plan dimensions of approximately 33m x 24m (with the shorter dimension parallel to Cashel Street) and is set back from Cashel Street by 17m. The set-back is occupied by a podium, to the height of the carpark, and by the entrance atrium. On the north side the tower is set back 6.5m from the boundary, a space utilized in the lower levels by suspended vehicle ramps. Within the ground floor space a right-of-way exists along the eastern boundary, occupied by Tattersalls Lane.

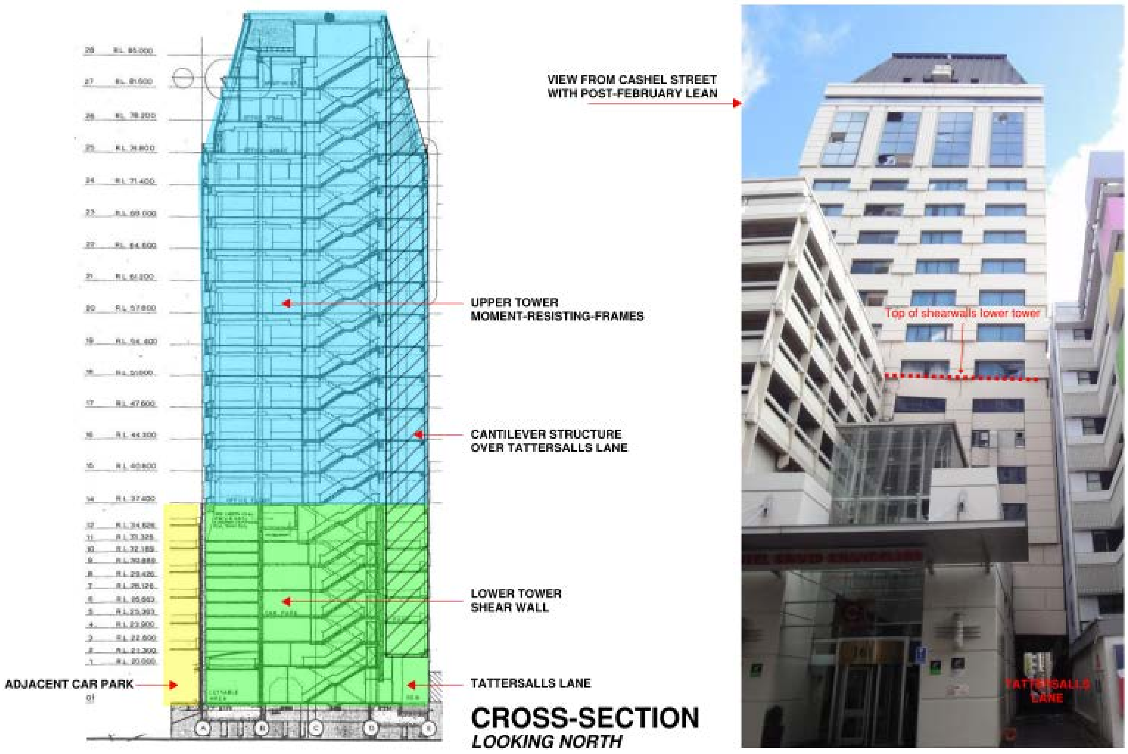


Fig.3

The hotel has conference facilities positioned mainly on top of the adjacent carpark building but with lift access and lobby at level 14 of the hotel tower.

The tower building was constructed between 1985 and 1988 with a number of building permits issued between 1985 and 1987. The tower was originally intended as office accommodation and then for a while was promoted as a possible hotel and casino. It was completed as a hotel with conference facilities.

The initial design was advanced on the premise that foundations, columns and walls could be constructed along (and within) the eastern side of the Tattersalls Lane right-of-way. Construction was reasonably well advanced in the western half of the site before legal action effectively prevented construction of any structure within the right-of-way. This reduced the footprint width of the building from 24m to around 19m and required a structural redesign. *[Refer App. B page 4].* It also added to the structural irregularity. *[Refer 4.2]*

4.2. Description of Structure

[Refer to Appendix B for annotated extracts from the original structural drawings and to Appendix C for photographs]

The Hotel Grand Chancellor is a reinforced concrete structure with both vertical and horizontal irregularity. The vertical irregularity arises from fact that the upper tower relies on frame action (moment-resisting reinforced concrete frames) for its seismic resistance while the lower tower relies on reinforced concrete shear walls. The two structural forms inherently have different stiffnesses (displacement under a given load) and, if not linked, would respond differently when subjected to seismic shaking.

The horizontal irregularity arises from the cantilever bay, between grids D and E, over Tattersalls Lane. The centre of rigidity is somewhat to the west of the centre of mass, for both the upper tower and the lower tower.

4.2.1. Lower Tower

Foundations consist of large pile caps/rafts supported on multiple, driven, bulb (Franki) piles. The specification indicates that the piles would be driven through sands and onto gravels at approximately 6.5m. The piles were to be driven on a performance basis (i.e. it was the contractor's responsibility to prove the load capacity of the piles). Pile records indicate that actual founding depths varied from 5m to 13m below ground level but typically in the 6m to 8m range.

Geotechnical investigations carried out at the time of design included two borelogs on the western side of the site and three Dutch Cone Penetrometer tests carried out within the centre of the site. A copy of this geotechnical information is included in Appendix G.

From ground floor to level 14 (half-level car parking floors equivalent to 7 full floors) the structure consists of insitu flat-slab concrete floors with insitu, reinforced concrete, cantilever shear walls (not coupled). These walls are arranged irregularly in plan, accentuated by the right-of-way set-back.The wall that failed lies on grid D, between grids 5 and 6 (Wall D5-6). *[Refer App. B pages 5-9 & 12]*

The eastern bay is supported by an unusual structural arrangement consisting of deep transfer beams cantilevered over the right-of-way between levels 12 and 14 to support a series of tension hangers which, in turn, support a long deep transfer beam along the eastern boundary above the first floor. *[Refer App. B page 13]* Interspaced with the hangers are column/struts supported by the long beam and which, together with the hangers, support the perimeter beams on the eastern, boundary side of the tower (grid E).

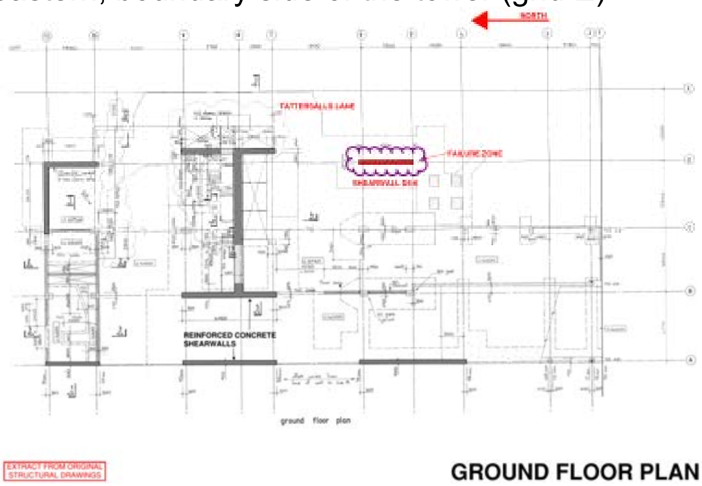


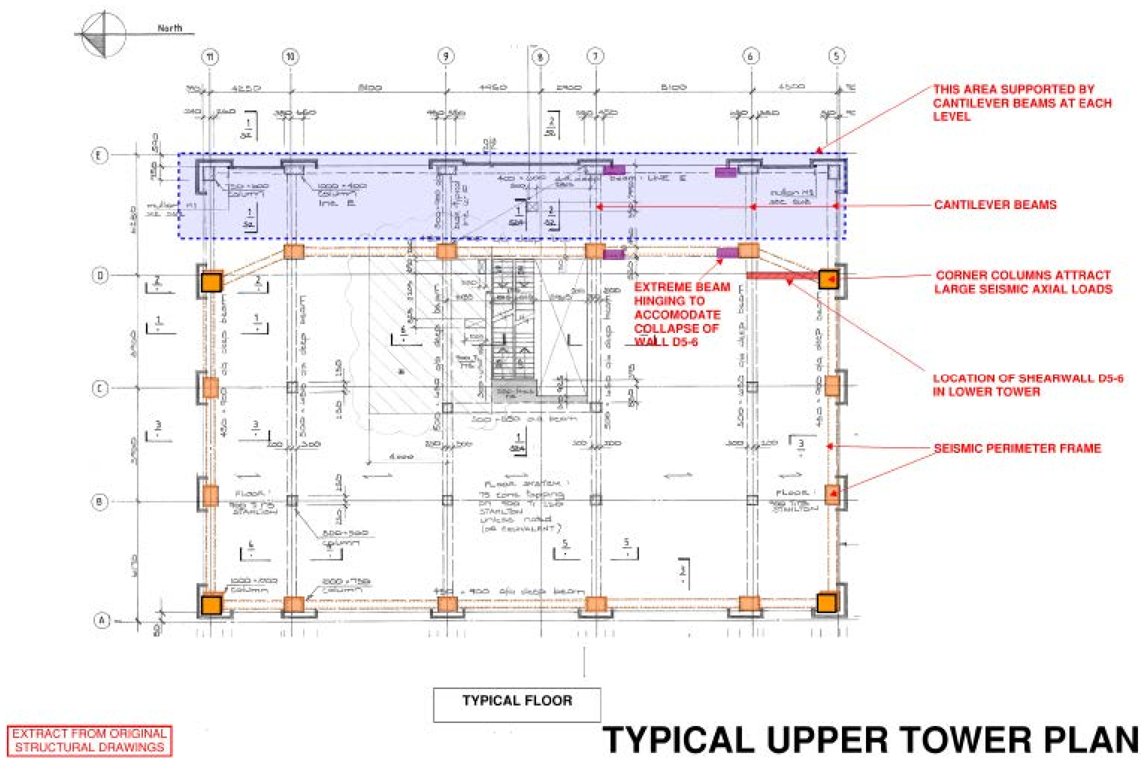
Fig. 4

Of note are the deep cantilever transfer beams that lie on grids 5 and 6. These beams which are part of the eastern bay hanging system are both supported at the fulcrum of their cantilevers by the critical wall D5-6. The beams are each a full floor height and are tied into the concrete floor diaphragms at levels 12 and 14. *[Refer App. B pages 9 & 15]* The fact that they are tied into the floors means that they will attract in-plane shear loading as the building experiences inter-storey drift (relative displacement between floors, which will occur during seismic motion in an east-west orientation).

4.2.2 Upper Tower

At level 14 a vertical irregularity occurs as the shear walls stop, and from levels 14 to 28 the structure has a perimeter seismic frame (off-set one grid on the eastern side). *[Refer App. B page 10]* These upper floors are a proprietary, precast-prestressed rib and timber-infill system with insitu topping. This flooring is supported on the seismic frames and on additional frames (beams and columns) not specifically designed as primary seismic resisting elements. In the upper structure, beams at each numerical grid cantilever over Tattersalls Lane at each floor level.

There is an apparently purposely designed vertical separation at level 14 along the eastern boundary line – grid E. This means that the vertical loads accumulating along grid E are not transferred directly down onto the strut/hanger system that exists in the lower structure.



TYPICAL UPPER TOWER PLAN

Fig.5

However, the loads from the eastern bay, between grids D and E, do find a load path to wall D5-6 via the upper columns on grid D. In particular, the columns at grid D5 and D6 are supported directly on the wall D5-6. *[Refer App. B page 15]*

The seismic perimeter frame lies on grids A, D, 5 and 11. The internal columns of seismic frames do not typically carry additional axial (vertical) loads induced by seismic actions however the end columns of seismic frames can attract large seismic axial loads in addition to their normal gravity loads. Column D5 is an end column for the frames on both grid D and 5 which means that it can attract bi-directional seismic axial loading. These loads feed directly onto the critical wall D5-6.

4.2.3 Wall D5-6

Wall D5-6 is a doubly reinforced (two layers of reinforcing in each direction, horizontal and vertical) concrete cantilever shear wall that extends from the pilecap at ground floor to level 14. Typically its clear height between floors is approximately 2.4m but between ground and first floor its clear height is approximately 5.1m. *[Refer App. B page 12]*

The wall has plan dimensions of 4.9m x 0.4m. It is the clear height divided by the width of the wall which defines its slenderness.

The specified concrete strength for the shear walls was 35MPa. Post-February core sample test results taken from other walls of the same thickness at the base of the building and from the D5-6 wall above the ground level were consistent with the specified concrete strength. *[Refer App. E]*

The wall is relatively lightly reinforced (0.45% vertical) and has only nominal confinement reinforcing, ties and links, at each end of the wall. *[Refer App. B page 12]*

The wall has the potential to attract high axial (vertical) loads resulting from:

- Gravity loads from a contributing area of approximately 100m^2 x 21 floors.
- Bi-directional seismic frame action (overstrength beam shears).
- Induced loads resulting from the shear loads attracted by the cantilever transfer beams between levels 12 and 14.
- Vertical seismic accelerations.

[Refer App. F page 7 for graphical representation]

The wall also has the potential to attract moments and shear loads (in-plane and out-of-plane) in proportion to its stiffness and the relative displacement of the floors that it is connected to.

Wall D5-6 naturally attracts extreme vertical actions compared with other shear walls in the building. There are three other, similar walls that also support columns subject to bi-directional axial actions;

- The wall at D10-11 supports a similar floor area to D5-6 but it has a return wall to brace the highly loaded end. Its maximum unsupported height is also only 3.6m compared to 5.1m and only one of the transfer beams it carries is full depth between storeys.
- The wall at A10-11 supports only one quarter of the area that D5-6 supports, has a lower height and is not affected by the transfer beam action.
- The wall at A4-6 also supports only one quarter of the area that D5-6 supports and is not affected by the transfer beam action. This wall is also twice the length of D5-6.

4.3 Stairs

The tower building has two egress stairs arranged in a back-to-back scissor alignment from the ground floor to the top floor. The stair flights each span floor-to-floor and are precast concrete with a throat thickness of 200mm in the lower tower and 300mm in the upper floor. They are supported by but not rigidly fixed into the concrete floors via steel stubs that cantilever out of the ends of the precast flights (top and bottom). These stubs sit in recesses formed in the edges of the floor landings. The connections were detailed with provisions for approximately 70mm *lengthening* of the inter-storey diagonal length (with a nominal factor of safety), as a result of interstorey drift, but with minimal provision for diagonal *shortening* resulting from inter-storey drift. *[Refer App. B pages 18 & 19]*

4.4 Precast Cladding Panels

The facades of the Grand Chancellor are clad with precast concrete cladding panels fixed to the perimeter columns and beams, with detailing typically allowing for beam hinging and frame drifts.

4.5 Post-Construction Alterations

No evidence of any significant structural alterations following the completion of the building has become apparent during the course of the investigation.

4.6 Effects of Time

No evidence of structural issues causing concern or requiring maintenance, during the occupancy of the building prior to the 4 September 2010 earthquake, has become apparent during the course of the investigations. Refer to section 5.3 for a description of damage recorded after the 4 September earthquake.

4.7 Design Standards

The principal relevant design standards current at the time of the Hotel Grand Chancellor's design were:

- *NZS4203:1984 – Code of Practice for General Structural Design & Design Loadings for Buildings*
- *NZS3101:1982 – Concrete Structures Standard – Code of Practice for The Design of Concrete Structures*

5 Earthquake Effects on Site and Building

5.1 Response Spectra

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

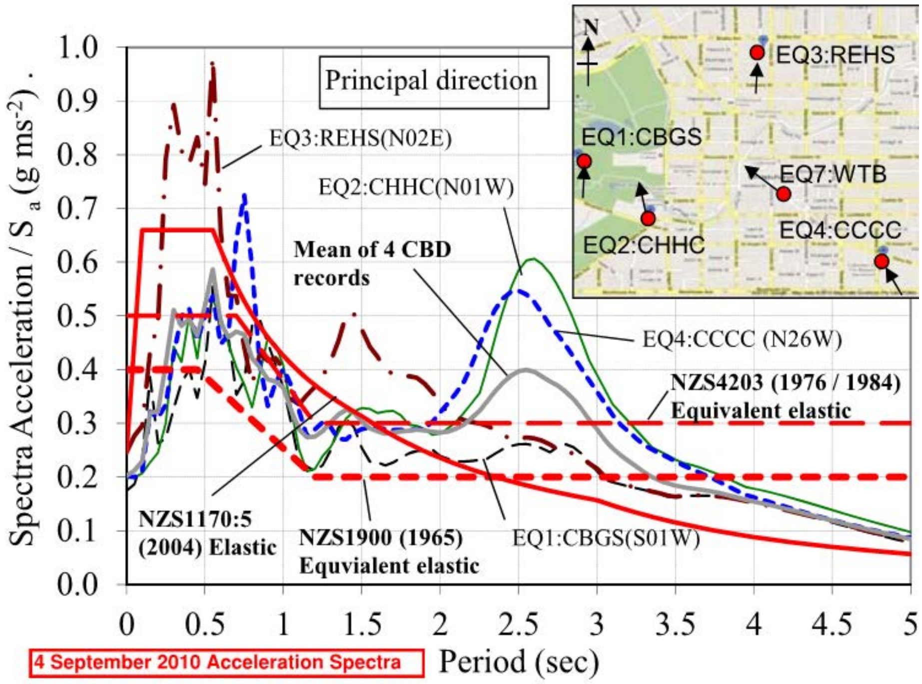


Fig. 6(a)

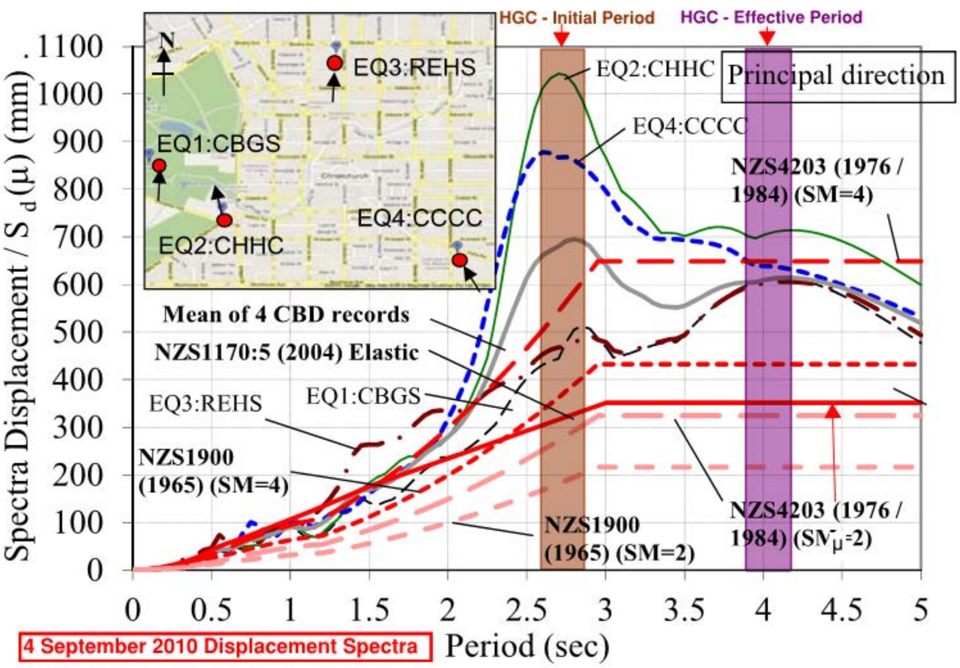


Fig. 6(b)

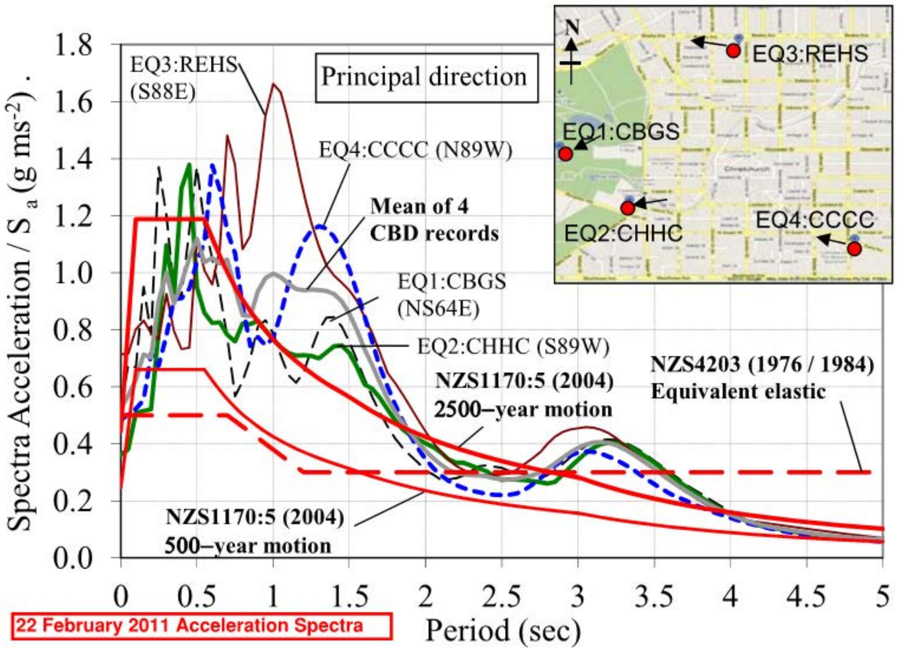


Fig.6 (c)

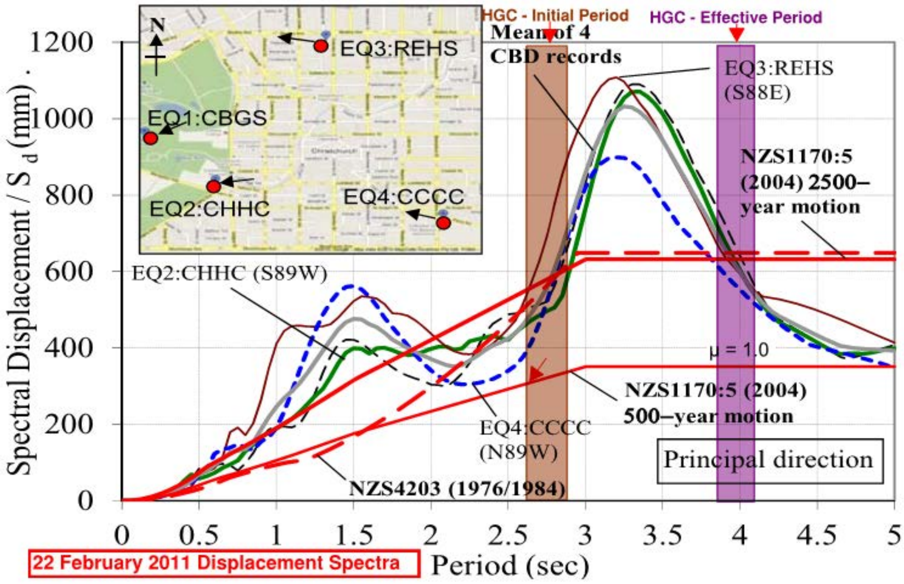


Fig. 6(d)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

This can be summarised in the following graph:

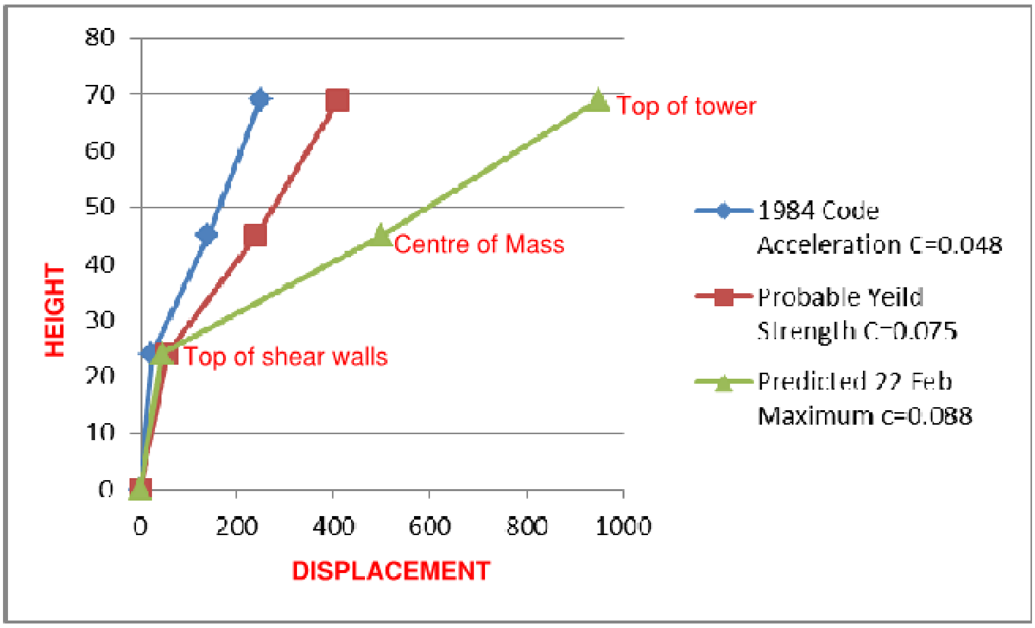


Fig.7

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

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

5.2 Liquefaction and Foundation Issues

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

5.3 Damage Prior to the February Event

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

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

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

- hairline cracks in seismic frames
- movement of seismic joint between tower and carpark
- outwards movement of one precast cladding panel attributed to missing bolts
- various areas of cracked plaster board.

Details of this damage are included in Appendix D in extracts from reports prepared following the September earthquake.

There is no suggestion that significant structural damage occurred either in September or December. There is also little or no evidence to suggest that significant problems could have been identified as a result of the earlier events.

The possible exception to this was the damage at a number of stair landings. This was interpreted as a “failure to slide” rather than a minor compression failure resulting from a lack of provision for stair flight shortening as a result of inter-storey drift. In any event, subsequent analysis and observations suggest that the resulting stair failure occurred only as a consequence of the failure of the shear wall D5-6.

The reported hairline cracking in the seismic frames suggests that the upper tower did yield but without any significant ductility demand. The extent of plasterboard damage is consistent with this, suggesting drift associated with ductility demands between 1 and 2.

5.4 Pounding

The interaction between adjacent buildings that have insufficient seismic gaps to allow for relative differential movement is referred to as pounding. As reported in 5.3, movement of the seismic joint between the hotel tower and the adjacent carpark building was recorded following the September event. Refer to section 6.15 for a description of pounding damage resulting from the February aftershock.

6 Failure/Damage Description

This section describes the structural damage that occurred as a result of the 22 February aftershock that has been observed during the investigation. Observations and subsequent analysis suggest that the bulk of the severe damage occurred as a consequence of the failure of a critical element within the structure, namely the D5-6 shear wall. The reasons for the failure of this wall are described at length in section 8.

6.1 Shear Wall D5-6

This wall failed between ground and first floor and effectively shortened by approximately 800mm. As a result, the tower structure developed a lean towards the south-east corner. The failure took the form of a brittle diagonal failure transversely across the wall that appears to have originated at the top of a vertical reinforcing splice located 700mm above the ground floor. The wall dropped, sliding off the diagonal crack, moving towards the west. There was evidence of a corresponding lateral hinge at the underside of the first floor. *[Refer App. B page 12 & App. C Photographs 6-10]*

Photo 6 - Shearwall D5-6 - Base Failure

Fig. 8

6.2 Columns B5 and B6 (also C5 & C6)

These columns are located directly to the west of wall D5-6 and are connected to the western end of the cantilever transfer beams at the underside of level 12.

As the wall D5-6 dropped the adjacent columns at B5 and B6 were subjected to very large axial loads combined with large moments arising from the lateral displacement induced by the rotation of the cantilever transfer beams, as their fulcrum point subsided. At this location the columns had moderate confining reinforcing but were not detailed for hinging. The columns yielded under flexural/axial actions and shortened also by around 500mm *[Refer App. B page 15 & App. C Photographs 20-22]*

6.3 Upper Tower Beams D6-7 and E6-7

As the upper columns at D5 and D6 dropped, following the shear wall below, these upper tower beams experienced major displacements, well in excess of their expected ductility demands. The beams formed large-rotation hinges at the faces of the columns but the reinforcing did not fracture. *[Refer App. B pages 11 & App. C Photographs 17-19]*

6.4 Lower Tower Beams E6-7

Similar to the beams above, these beams were forced to form large-rotation hinges at the face of the tension hanger at E6 and the suspended strut-column at E7. The adjoining tension hanger at E8 experienced a major increase in tension load but survived intact. *[Refer App. B page 13 & App. C Photographs 14]*

6.5 Cantilever Transfer Beam 8D-E

Parallel to the level 12-14 cantilever transfer beams on grids 5 and 6 is a further cantilever beam on grid 8. This beam is also a full floor-to-floor depth between levels 12 and 14 and is an extension of the main spine shear wall on grid 8. This beam supports the hanging column at E8 which, as described in 6.4, experienced a major increase in load as the south-eastern corner of the tower dropped. As the hanger load increased a lap failure initiated in the lapped beam stirrups and the bars slipped by up to 80mm. This mechanism appears to have come close to collapse. *[Refer App. B page 14 & App. C Photographs 15 & 16]*

6.6 Precast Panel Connections

At the locations where the south-east corner underwent major distortion, the precast façade suffered significant distress. While a number of panels ruptured and panel connections were broken, they generally displayed a high level of robustness against total dislodgement.

6.7 Level 2 Slab South-East Corner

Between grids D and E there is a non-structural wall that separates the hotel lobby from Tattersalls Lane. This wall is parallel to the shear wall D5-6 and lies approximately midway between D and E. As the corner subsided, there was sufficient vertical load carrying capacity in the non-structural wall to break the back of the level 2 slab. *[Refer App. B page 7 & App. C Photograph 13]*

6.8 Level 14 Diaphragm Slab

At the underside of the level 14 slab, where the cantilever transfer beam on grid 6 connects to the slab, a shear failure has initiated along the north side of the beam. *[Refer App. B page 15]*

6.9 Carpark Shear Walls Adjacent to Grid A

Along the western side of the Grand Chancellor there are three shear walls along grid A, in the lower tower, abutting the carpark structure. The carpark structure itself has parallel walls in these locations separated by a seismic gap, filled with polystyrene.

Between levels 10 and 12 and between levels 12 and 14 mid-height, flexural cracks are visible in the carpark wall between grids 4 to 6, that is the wall that it in the same east-west line as the D5-6 wall and the cantilever transfer beams on grids 5 and 6.

6.10 Stairs

As the tower lurched towards the east, all but the upper most southern-side stair flights catastrophically collapsed down into the stairwell. The momentum of the collapsing stairs also took-out the three upper stair flights in the carpark levels of the tower. *[Refer App. B pages 18 & 19 & App. C Photographs 23-25]*

6.11 Upper Tower Perimeter Seismic Frames

The upper tower seismic frames are typically enclosed by architectural linings. During the investigations a number of elements were exposed, particularly around the beam-column joint regions. In all locations, beam cracking consistent with the onset of hinging was apparent. While the degree of cracking varied it was generally consistent with a ductility demand in the range of 2 to 4, but with few cycles. Many joints had “D-Bars” to force the hinge off the column face, which occurred successfully. The degree of hinging is seen as unrelated to the critical collapse, and indeed the good performance of these mechanisms contributed significantly to the building remaining upright after the wall failure.

6.12 Upper Tower Grid E Frame

The upper tower frame along grid E is supported on the beams that cantilever across the right-of-way at each level. As such the columns on grid E do not carry axial load. It is apparent from on-site observations that all of the upper tower floors to the east of grid D (that is the cantilevering eastern bay) have a residual deflection that appears unrelated to the critical collapse.

6.13 Upper Tower Floors

Floor linings in the upper floors were removed in several locations to check for signs of floor plate elongation resulting from beam hinging and resultant frame dilation. Relatively fine floor cracks were observed in the corner bays of the frame, reinforcing the observation that a low number of cycles had occurred during the short duration of high accelerations. This issue was unrelated to the critical collapse.

6.14 Ground Level Shear Walls

Hairline flexural cracking is visible in a number of the ground floor shear walls. This is consistent with the level of shaking experienced by the building structure and the low ductility demand on the shear walls. This issue was unrelated to the critical collapse.

6.15 West Side Pounding

Following the 22 February event, there were media reports that pounding was the cause of the Grand Chancellor failure. While the damage to the southern (Cashel Street) façade has an appearance that could, at first glance, be interpreted as resulting from pounding, this was in fact vertical displacement damage caused as the D5-6 wall failed. *[Refer App. C Photograph 5]*

There is no sign of significant building-to-building interaction on the north or east sides.

There is some significant but local damage on the west side where the carpark conference centre roof was inappropriately connected across the seismic gap at one location.

There is also some minor flexural cracking in the adjacent carparking building in the adjoining walls that may be attributed to pounding as the hotel building suffered its partial collapse. This is described in section 8.3

Where the conference centre roof abuts the tower structure it is apparent that a precast panel connection relied on the adjoining structure for vertical support. This has resulted in the dislodgement of a precast panel. The extent of this damage may have been accentuated by the critical wall collapse.

7 Failure/Collapse Mechanism

During intense shaking that occurred at 12.51pm on 22 February, the building's seismic resisting structure was pushed to its yield point and beyond its elastic limit reaching a ductility demand of approximately 3.5. The strong, concurrent ground motions, particularly the east-west component, had the ability to generate high axial loads in the shear wall D5-6. It is probable that vertical accelerations added to the axial loads. Under these conditions and with the confinement provided the wall had only limited available ductility and failed in a brittle and abrupt manner.

At that moment most of the south-east corner was momentarily unsupported and began to fall. Load transfer occurred to re-distribute the unsupported forces. The transfer beams on grids 5 and 6 attempted to cantilever further and to transfer the loads to the columns at 5C and 5D. This had the effect of overloading the columns at 5C and 5D until they also yielded, forming flexural/axial hinges. There was insufficient confinement capacity for this unexpected load and they too failed in a brittle manner, effectively shortening the column. They did however maintain a vital, axial load capacity.

At the same time as the lower tower D5-6 wall was displacing vertically, the upper tower seismic frame on line D was subjected to major differential displacement between grids 6 and 7. This caused the formation of major hinges at the column faces at D6 and D7, all the way up the upper tower. Although the beams effectively rotated beyond any reasonable ductility demand, they maintained an effective load transfer mechanism for vertical load into the corner of the large core shear wall.

A similar mechanism occurred in the grid E frame although it was not detailed for primary seismic actions. This frame transferred significant additional axial load onto the grid 8 transfer beam, causing it to yield and come close to failure.

The seismic frame on grid 5 was not able to act in the same way as the frame on grid D because its support was undermined by the failure of the grid C (and grid B) columns. This frame effectively rotated about grid A.

Finally, the wall D5-6 regained same axial capacity as it came to rest on the pilecap. Within the mechanisms described above there was sufficient redundancy and resilience to redistribute the loads from the failing element and to halt the collapse of the tower as a whole.

8 Evaluation and Analysis

Structural analysis and evaluation has been carried out for the main building structure. Emphasis during the evaluation has been placed on the structural elements that exhibited failure.

8.1 Shear Wall D5-6 Failure

Initial inspections and assessment of the building's form following the earthquake suggested that the initiation of the major structural failure commenced with the failure and subsequent shortening of the D5-6 shear wall. As discussed in section 7.1 the failure is a transverse-diagonal, brittle rupture that obviously occurred abruptly and suddenly with little sign of progressive flexural yielding or concrete crushing. Subsequent assessment and analysis suggests that the failure occurred when the wall was subjected to extremely high axial compressive loads with little available ductility or confinement outside of the short end zones, at each end of the wall. With displacements requiring a curvature ductility at the base of the wall, of which there was little available and no effective confinement, and a tendency towards buckling due to the slenderness ratio being less than recommended, the wall failed abruptly, out-of-plane. The factors affecting this assessment are as follows:

- The D5-6 shear-wall attracts a large contributing area of gravity load due to the building's irregular geometry that results from the cantilever over the Tattersalls Lane right-of-way.
- The column above the shear wall at grid D5 is the end column for the seismic perimeter frames on both grid D and grid 5 – i.e. it is potentially subject to biaxial bending and, more relevantly, high over-strength beam shears, resulting in high, seismically induced axial loads from two directions concurrently.
- The seismic spectra for the February aftershock indicate strong motion particularly in the east-west and also north-south directions. Thus the structure, with its grids arranged in a North-South and East-West alignment will likely have experienced concurrent actions.
- When subject to inter-storey drift in the East-West direction the storey-height cantilever transfer beams on grids 5 and 6 will have been subject to induced horizontal shear (when displacement compatibility is considered) and this in turn will have induced further axial loads into the D5-6 shear wall at both D5 and D6.
- The 22 February event spectra indicate high vertical accelerations and it is probable that these also will have added to the axial loads
- Under a compression cycle (i.e. biaxial compression at column D5) the wall would be very unlikely to yield in flexure (bending) due to interaction of the high axial load. This means that, while the upper tower was yielding and utilising available ductility with increasing displacement, the in-plane moment in D5-6 could keep increasing until a significant portion of the wall length (in excess of 50%) was beyond the neutral axis (i.e. in the compression block).
- Under these conditions, that is when a significant length of the wall is at the ultimate allowable concrete strain and is unconfined, any additional strain arising from out-of-plane actions could propagate abrupt, brittle failure.
- Comparison with NZS 3101:1982 suggests that the D5-6 shear wall exceeded the recommended slenderness ratio, which leads to the deduction that the tendency to buckle may have exacerbated the propensity to failure in the highly loaded, unconfined sections of wall.
- Review of the construction drawings indicate a relatively low level of confinement reinforcing at the base of the wall. Confinement hoops were limited to the four 'primary' vertical bars at each end of the wall. The amount of primary reinforcing was low because compression-flexural interaction suggested that only nominal flexural reinforcing was required. As the design contemplated that only nominal confinement was required, this (combined with the small area of flexural reinforcement) resulted in only a small portion of the wall having confinement reinforcing. Other shear walls within the building that had smaller axial loads had greater confinement.
- Calculations suggest that the wall flexural reinforcing may have initially yielded under tension/moment interaction (Tension generated from the biaxial frame action and the transfer beam action and probably vertical accelerations).

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

Calculations supporting these assessments are summarised in Appendix F.

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

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

A concrete column loaded in compression will naturally shorten and as a consequence, expand its girth. This redistribution of volume can result in internal tensile stresses, particularly if one end of the column is constrained from expansion. Confining links and hoops within a column or wall effectively restrains the expansion and forces the concrete into transverse compression making it more resistant to tensile forces. Reinforcing links that pass through potential tension cracks can also directly resist the tensile forces.

Wall D5-6 had extremely high insitu axial gravity loads and during east-west ground motions attracted additional axial loadings. During north-south ground motions, flexural actions concentrated these axial loads into a 'stress-block' at one end of the wall. In such conditions, unconfined concrete is very likely to suffer brittle compressive failure. Such failures have been observed all around the world following earthquakes and have been reproduced in laboratory testing.

NZS3101:1982 – The Code of Practice for The Design of Concrete Structures – the current standard at the time that the Hotel Grand Chancellor was designed, required that the stress-block (neutral axis depth) at the base of a shear wall be confined when the length of the stress-block exceeded a certain proportion of the wall length. This condition was not satisfied under the loadings specified in NZS4203:1984 – The Code of Practice for General Structural Design & Design Loadings for Buildings – the current loadings code at the time that the Hotel Grand Chancellor was designed, when the many possible contributing axial loads are combined.

An alternative way to consider the failure is to recognise that, as the compressive stresses on unconfined concrete increase, the available curvature ductility from the critical wall section decreases. This is illustrated in the moment-curvature diagrams contained in Appendix F.

While in-plane flexural actions will concentrate the high axial loads at one end of the wall with uniform transverse distribution, out-of-plane (transverse) displacements will result in non-uniform (eccentric) stress (and strain) across the compression zone. This will cause crushing, initiating at one side of the wall, probably at a locally weak spot, in this case the top of the reinforcing laps. The crushing or spalling then increases the effective eccentricity on the spalled section leading to a progressive and abrupt failure, of the form observed at wall D5-6.

Underlying seismic design practice is the requirement for critical structural elements to have sufficient robustness and resilience that will enable them to perform in a non-brittle manner when actions, anticipated by design, are exceeded. It is apparent that this was not achieved for wall D5-6.

8.2 Stair Flights Collapse

Analysis suggests a maximum inter-storey drift at first yield of 0.5% under NZS4203:1984 loadings. Multiplying this by the implied ductility factor (K/SM = 3.44) gives an ultimate drift (lateral displacement) of 1.7% of the height, or approximately 60mm per floor. Examination of the displacement response spectra from the February aftershock suggests drifts of around 1.9% or 65mm/floor. The stair detailing provided for a horizontal spreading of the supports of 70-80mm (each end) but effectively for no or minimal shortening. As there was little or no evidence of compression damage to the stair units themselves (in the remaining flights) this suggests that the damage evident at the supporting floor landings may have occurred during compression cycles. Some evidence of this was recorded following the September event *[refer to Appendix D page D2]*. The damage is also visible in the remaining stair landings. This in effect means that if the building structure had performed adequately the stairs would have been unlikely to collapse.

NZS4203:1984, the loading code current when the Hotel Grand Chancellor was designed, required that elements, such as stairways, that are capable of altering the intended structural behaviour of the building to a significant degree, be separated to avoid impact. While adequate separation to avoid impact was not provided it is apparent that the stair actions did not significantly affect the behaviour of the building.

Post February, permanent (plastic) upper tower displacements have been measured at 700mm in the North-East corner and 1300mm in the South-East corner. At the location of the stairwell (approximately midway between these two points) the permanent displacement will be around 1000mm. During the 22 February aftershock there would have been additional elastic deflection (which would have rebounded at the completion of the shaking) of 250mm. This then would have resulted in a net tower displacement of around 1250mm at the location of the stairs, immediately following the failure of wall D5-6.

A total tower deflection of around 1250mm implies an average displacement per floor of 90mm with a likely variation of perhaps 20mm (range of 70-110mm). The stair landing seating detail *[Refer App. B page 19]* shows a gross allowance for movement (net lengthening) of 100mm at each end of the stair. Making due allowance for construction tolerance, minimum seating and a nominal factor of safety would reduce the safe movement to 70mm (at each end), less than the measured and calculated potential movement of 110mm. While it is possible that the movement could have been shared at the two ends of each stair flight it is more probable that once movement started it would have continued to occur all at one end. It would only have taken the collapse of one flight near the top of the building to instigate a progressive failure all the way down the stairwell.

The conclusion may be drawn that the stair collapse resulted from the wall failure rather than from inadequate stair seating alone.

8.3 Carpark Shear Walls Adjacent to Grid A

The flexural cracking visible at the mid-height of the adjacent building's shear walls may be explained by the failure mechanism of the D5-6 wall. As the wall failed and dropped, the cantilever transfer beams rotated about the columns on line C and shunted the level 12 diaphragm slab towards the west and the carpark building. This displacement transmitted a force, via the polystyrene in the seismic gap, as a distributed load onto the adjacent wall, sufficient to cause out-of-plane yielding.

8.4 Upper Tower Grid E Frame

The residual deflection in the grid E frame appears to have resulted from seismic frame action within the grid E "gravity" frame. Although not intended to act as a seismic frame, compatibility induced actions have resulted. Seismic actions from north-south displacements have induced axial loads in the end columns of the frame. These in turn have caused yield level deflections in the cantilever beams (noticeable at the northern end of the frame.) However, the uplift actions of the grid E frame have not been sufficient to reverse the deflections as the uplift actions are counteracted by the gravity cantilever moment. This has caused a ratcheting down at the ends of the frame.

9 Conclusions

The Grand Chancellor appears to have been generally well designed. The upper tower seismic frames, with offset beam hinge locations were state-of-the-art for the time of its design and appeared to perform well. The shear walls typically also appeared to perform well, as did the precast concrete façade panels. However, the structure contained a critical structural vulnerability resulting from the fact that the capacity of the D5-6 shear wall could be exceeded by the demand actions (that could be expected during code-level shaking) to the extent that a brittle and abrupt failure could occur. The 22 February aftershock induced actions within the wall that exceeded its capacity and caused failure and partial collapse. Some redundancy and resilience within other areas of the structure, which provided alternative load paths, prevented an on-going building collapse.

The factors that contributed to the critical vulnerability are as follows:

- The plan irregularity, partially caused by a late planning change which excluded structure from the Tattersalls Lane right-of-way, resulted in a disproportionately large contributing area being supported by the D5-6 wall and a horizontal irregularity.
- Vertical irregularity arising from a framed structure atop a shear wall-podium with transfer beams at the interface.

- Extremely high axial (vertical) wall actions arising from a combination of:
    - Gravity (Dead plus Imposed) loads.
    - Axial loads resulting from biaxial over-strength shears from the frames above.
    - Actions resulting from in-plane actions of storey-high cantilever transfer beams.
    - Vertical earthquake accelerations.
- Wall slenderness ratio did not meet code requirements, for the levels of axial load.
- Insufficient confinement at the base of the wall, in respect to code.
- Insufficient available ductility in the critical member (Wall D5-6) relative to the demands experienced during the February aftershock.
- Lapping (unconfined) in high compression zone/hinge zone.
- Code defined actions exceeded by the February earthquake.

Of all these factors, the low level of confinement at the base of the wall is probably the most significant in leading to failure. The extremely high actual and potential axial loads required that the wall be confined like a column subject to high axial loads. *[Refer to Appendix B, page 12 for a drawing showing the confinement that was provided and the code required confinement, when higher axial loads are considered]*

Other areas of major damage, including the stair failure and the grid 8 transfer beam lap failure, were consequential to the wall failure.

No construction related issues that may have contributed to the failure have been identified.

10 Discussion

10.1 Response to 22 February Event

A review of the original structural calculations suggests that it was intended for the building was to achieve a *Required* or *Dependable* strength of 0.048g for the frames and typically 0.06g for the walls of the Hotel Grand Chancellor structure. These values are design base-shear coefficients, compliant with NZS4203:1984. Actual Probable strengths are likely to be around 0.08g to 0.1g. When a ductility factor of 4 (implied by the 1986 codes) is applied this would suggest that the building should perform satisfactorily at spectral accelerations up to 0.3 to 0.4g.

The recorded spectral accelerations around the Christchurch CBD in February were between 0.3 and 0.4g for a building with a period equivalent to that of the Hotel Grand Chancellor *(refer to spectra in Fig 6(c)).* This suggests that the demand actions imposed by the February event marginally exceeded the actions required by the contemporary standards. However the duration was shorter than anticipated by design codes. A reasonable deduction is that the February shaking was approximately equivalent to a ‘code event’ for this structure.

10.2 Did the Structure Comply With the Codes of the Day?

As discussed in section 9.0, in most respects the structure appears to have complied with the codes and standards that were applicable when the structure was designed. However, in respect of Wall D5-6 the confinement and slenderness requirements were not achieved when all the potential axial loads are considered.

10.3 Would the Failure Have Occurred In A ‘Code’ Event?

As discussed in 10.1, the building’s response to the shaking in the February event was of a similar acceleration to what might be expected during a 500 year return-period event, as prescribed by NZS4203:1982 – a ‘code’ event. In addition the recorded vertical accelerations were large and the response of the structure to these may have exceeded code expectations.

Therefore it can be said that it is possible that the wall may have failed in a ‘code’ event. However a ‘code’ event is really only described by the magnitude of peak ground acceleration and without consideration of the direction of shaking (horizontal and vertical) or of the duration of the motion. Accordingly it is also possible that the building could have survived a ‘code’ event without wall failure.

In order to generate the extreme axial actions on the wall, strong motions in the east-west directions were required to mobilize the loads induced by the transfer beams. In addition, concurrent north-south actions were also required to maximize the axial loads and to induce the large compression stress-block at the base of the wall.

If the base of the wall had been more rigorously confined there is a reasonable likelihood that the wall would have survived without failure. If the shaking had been of longer duration then even a confined wall may have failed because loss of the cover concrete would have left the wall quite slender and vulnerable to buckling.

10.4 Would the Building Have Collapsed in a NZS1170.5 Defined Event?

The design basis earthquake as defined by NZS1170.5 is similar to, but a little smaller than, an event defined by NZS4203:1984, for a building having a period equivalent to that of the Hotel Grand Chancellor. *(refer to spectra in Fig 6(c)).* Therefore, there is a likelihood of possible collapse during NZS1170.5 defined actions. A relevant issue is that the D5-6 wall did not have sufficient robustness to cope with an event larger than that defined by the Standard. This was exposed on 22 February 2011.

10.5 What was the %NBS on 21 February?

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

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

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

11 Recommendations

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

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

connected to more than one floor but which are not intended to act as shear walls

- ***Design Rigour for Displacement Induced Actions.***
  Designers generally have tended to separate seismically resisting elements from 'gravity-only' frames and other elements of so-called secondary structure. However not enough attention has always been paid to ensure that the secondary elements can adequately withstand the induced displacements that may occur during seismic actions. Non-modelled elements should perhaps be detailed to withstand 4% displacement. Modelled elements should be detailed to withstand a minimum of 2.5% displacement. – DBH should promote an increase in design awareness relating to displacement induced actions
- ***Frames Supported on Cantilevers.***
  Although this is not a common arrangement, caution needs to be taken when supporting a moment resisting frame on cantilever beams as effective ratcheting can lead to unexpected deflections. – DBH should consider a design advisory relating to ratcheting action of cantilevered beams and frames.

APPENDICES

A. List of Reference Material

B. Selected annotated original documentation

C. Photographs

D. Post September Inspection Reports

E. Concrete test results

F. Evaluation & Analysis

G. Geotechnical Information

APPENDIX A

List of Reference Material

Original Structural Drawings

Original Structural Calculations

Original Structural Specification

Original Structural Site Reports

Original Structural Construction Sketches

Original Structural Design Certificates

Original Construction Photographs

CCC Property File Data

Engineering & maintenance reports following the September Earthquake (refer Appendix D)

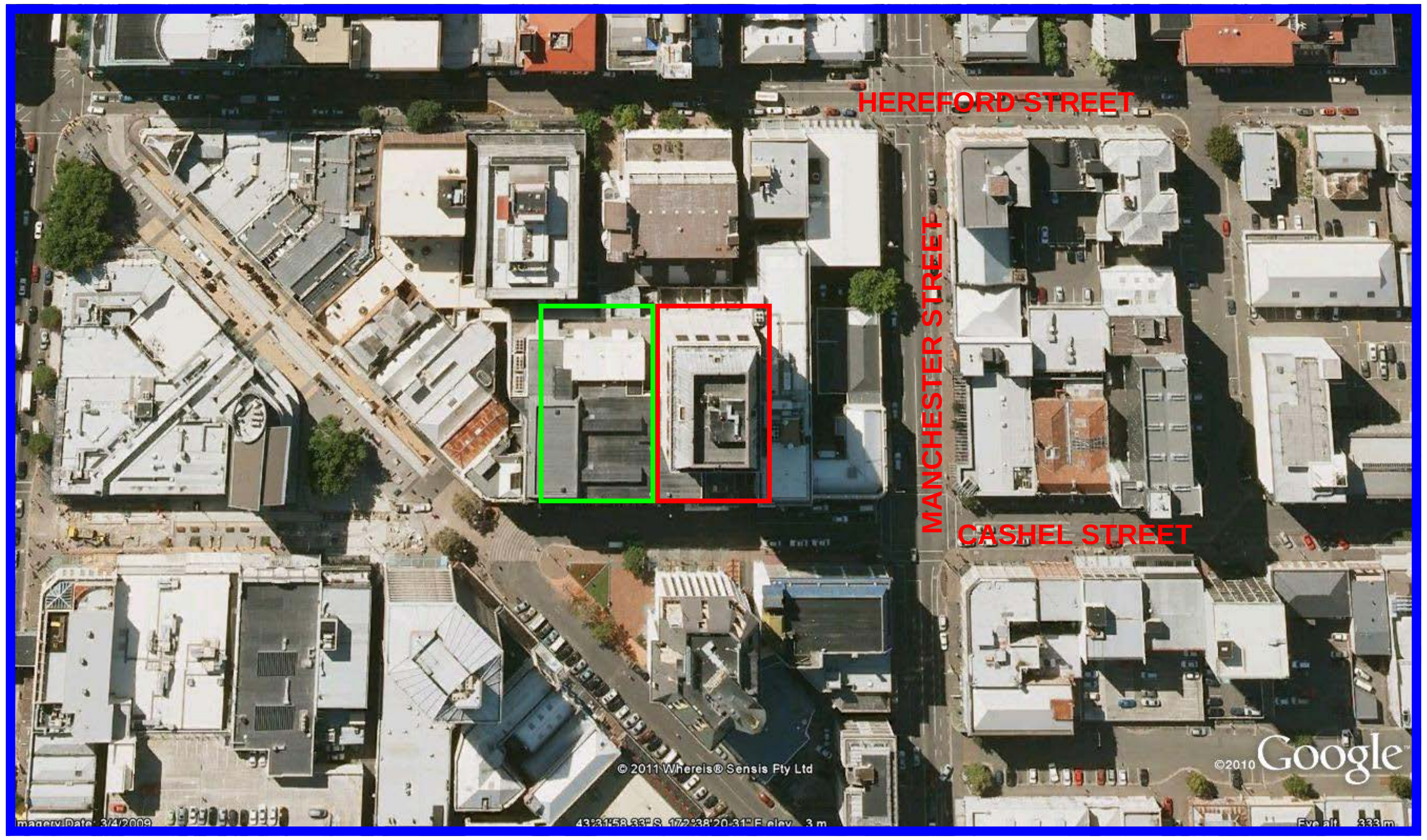
Post September Building Safely Evaluation Level 1

Contextual Report : General Performance of Buildings in Christchurch CBD
Dr. Weng Yuen Kam & Ass. Prof. Stefano Pampanin

Codes
NZS4203:1984 - General Structural Design & Design Loadings for Buildings
NZS3101:1982 – Code of Practice for The Design of Concrete Structures
NZS1170.5:2004 – Structural Design Actions Earthquake Actions - NZ
NZS3101:2006 – Concrete Structures Standard

APPENDIX B

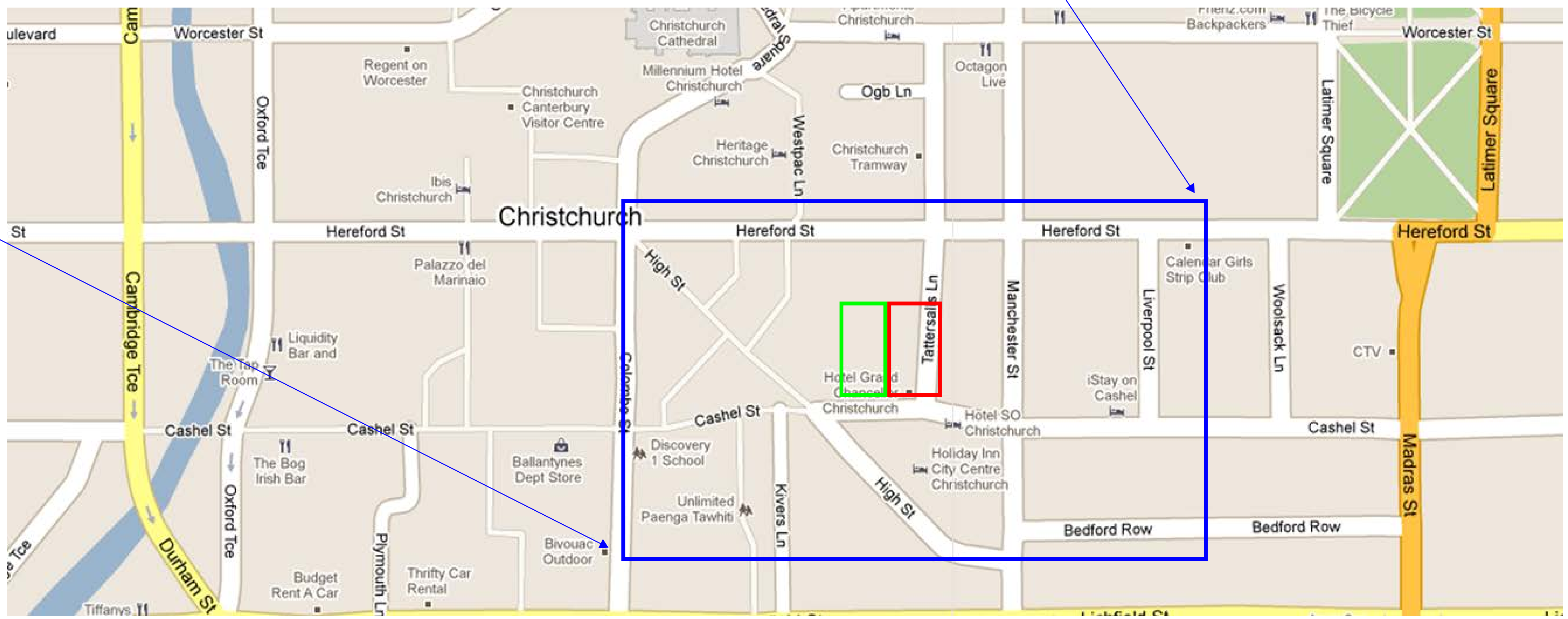
Selected Annotated Original Documentation



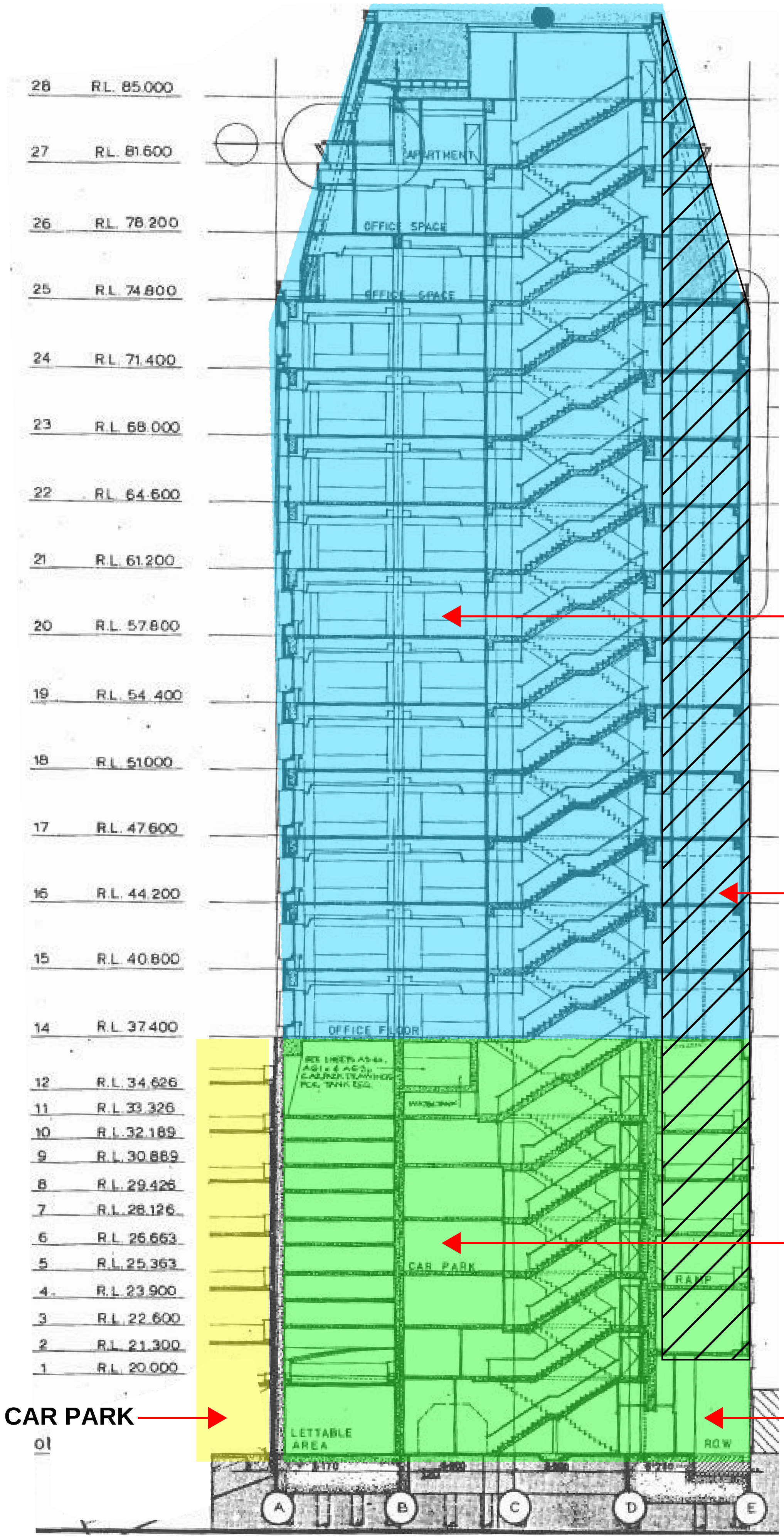
HOTEL GRAND CHANCELLOR

ADJACENT CAR PARKING BUILDING

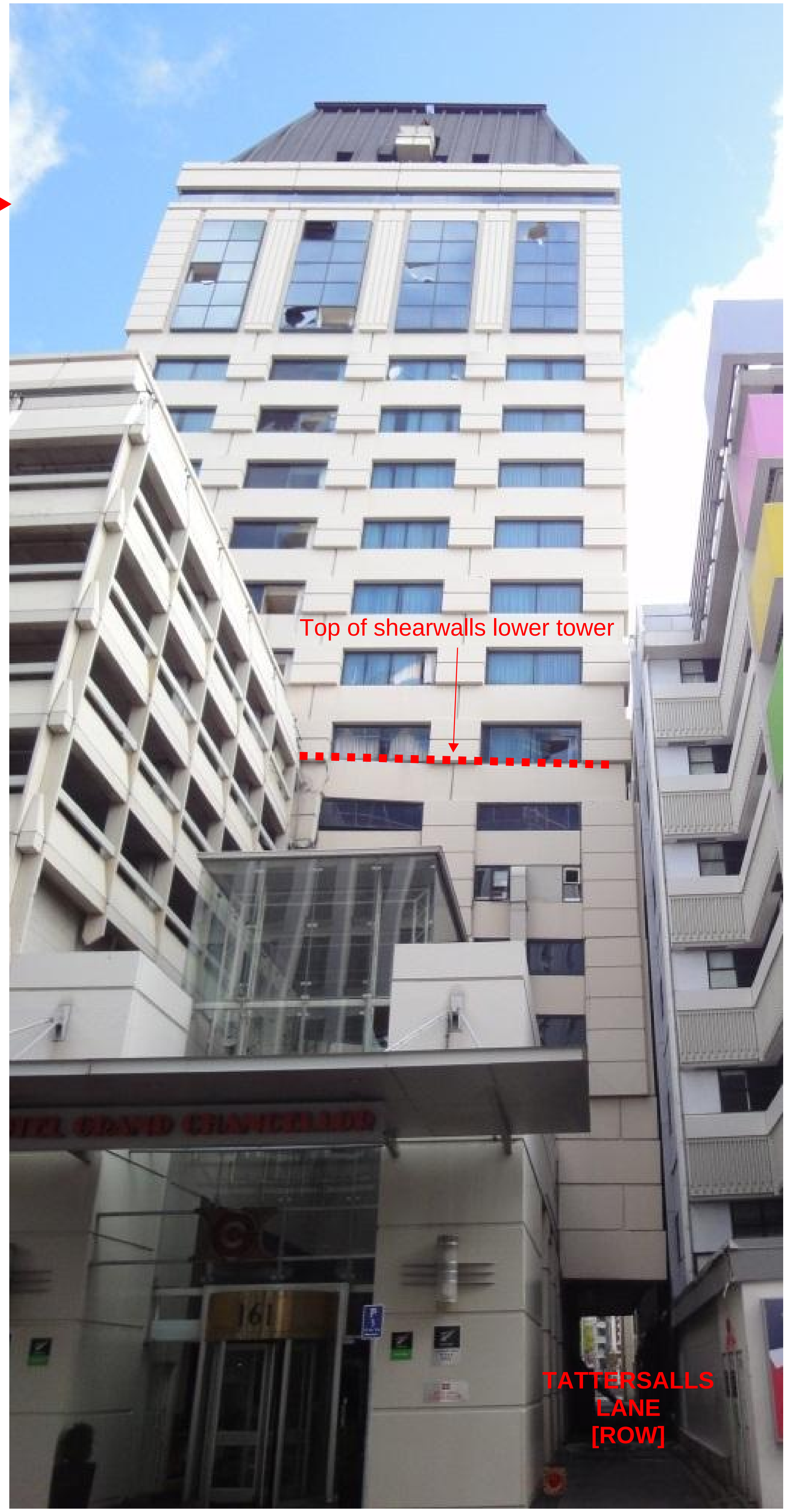
NORTH



LOCATION PLAN

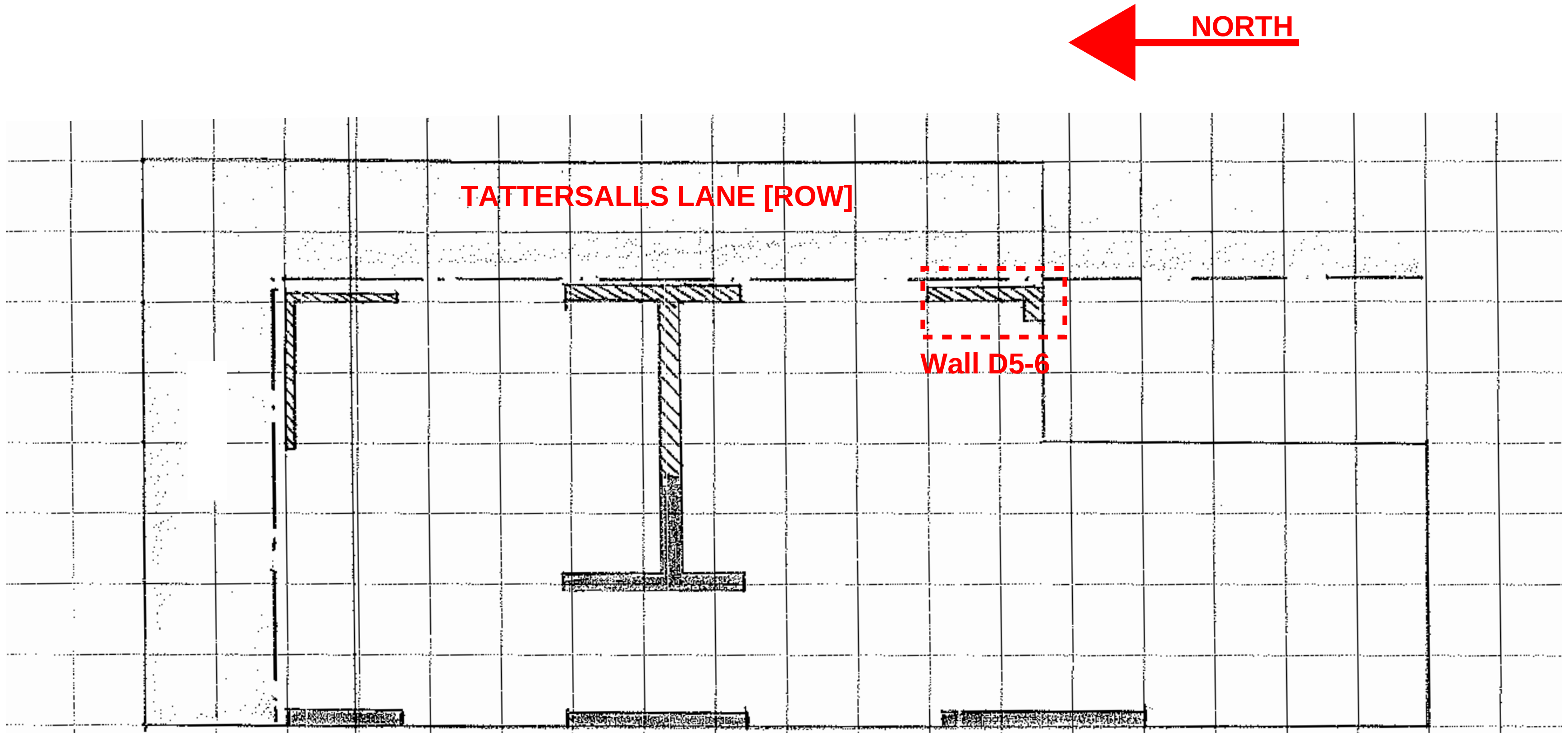


VIEW FROM CASHEL STREET
WITH POST-FEBRUARY LEAN

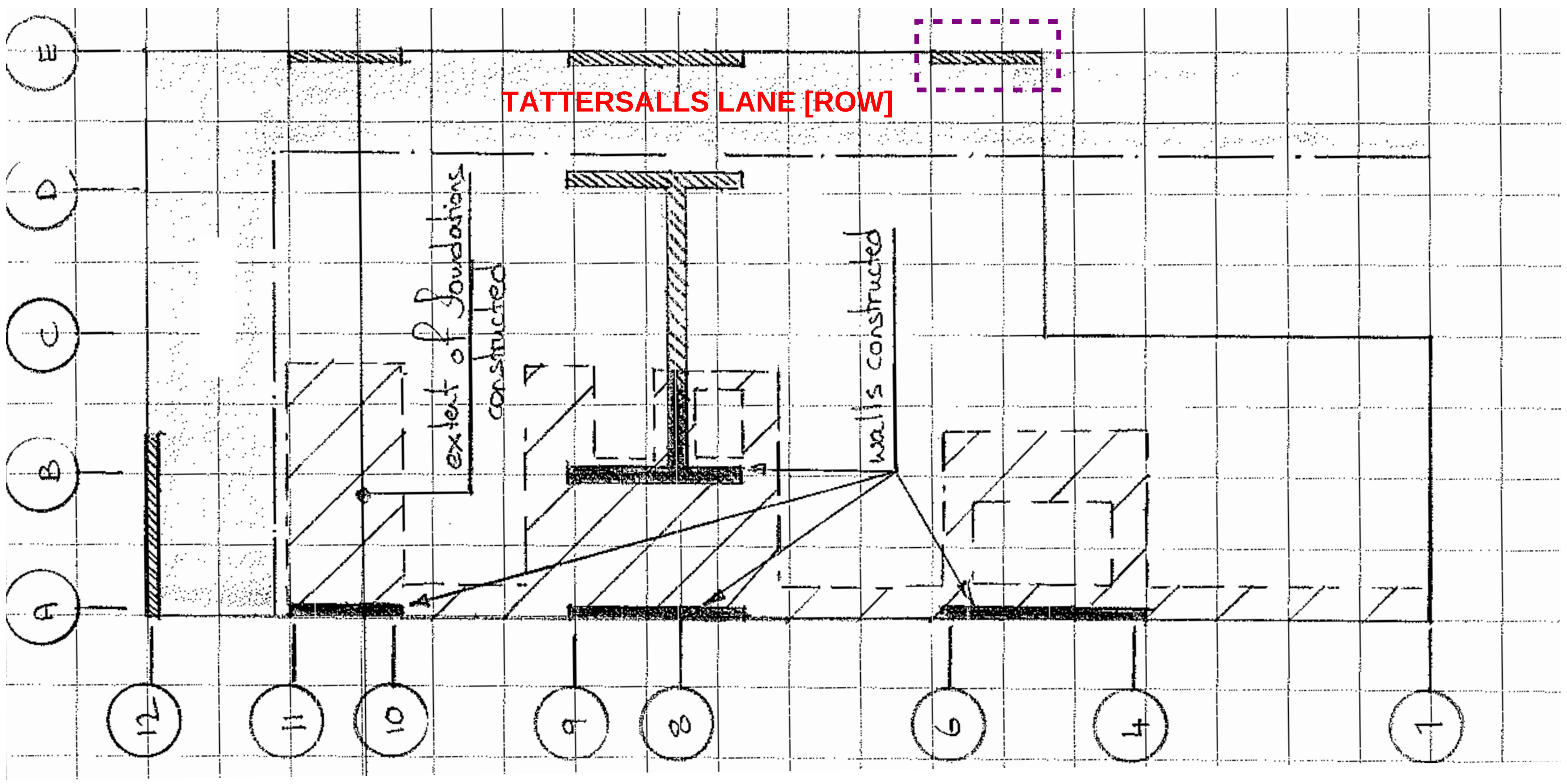


CROSS-SECTION

LOOKING NORTH



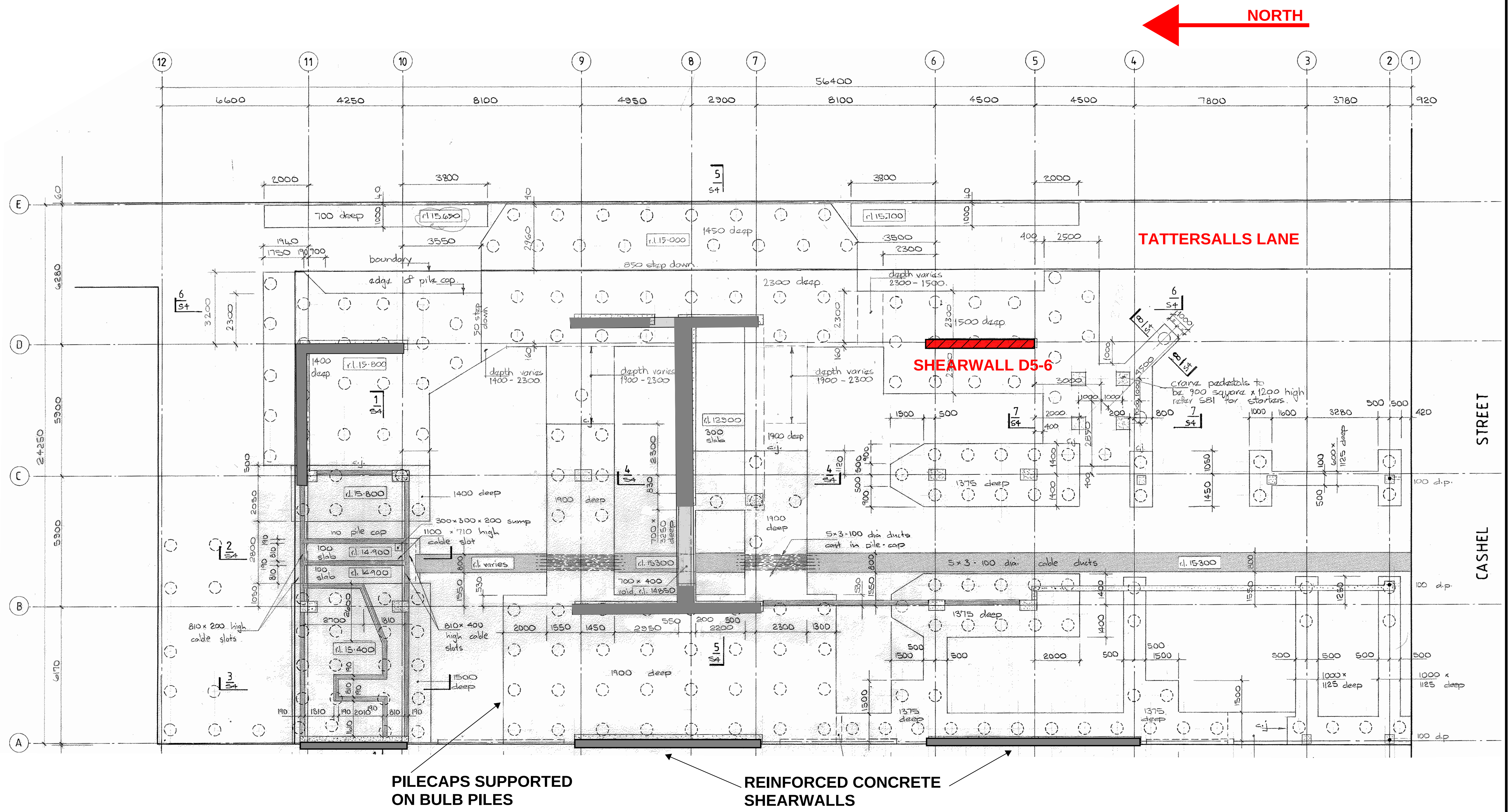
STRUCTURE AS MODIFIED TO SUIT RIGHT-OF-WAY



STRUCTURE AS ORIGINALLY CONCEIVED

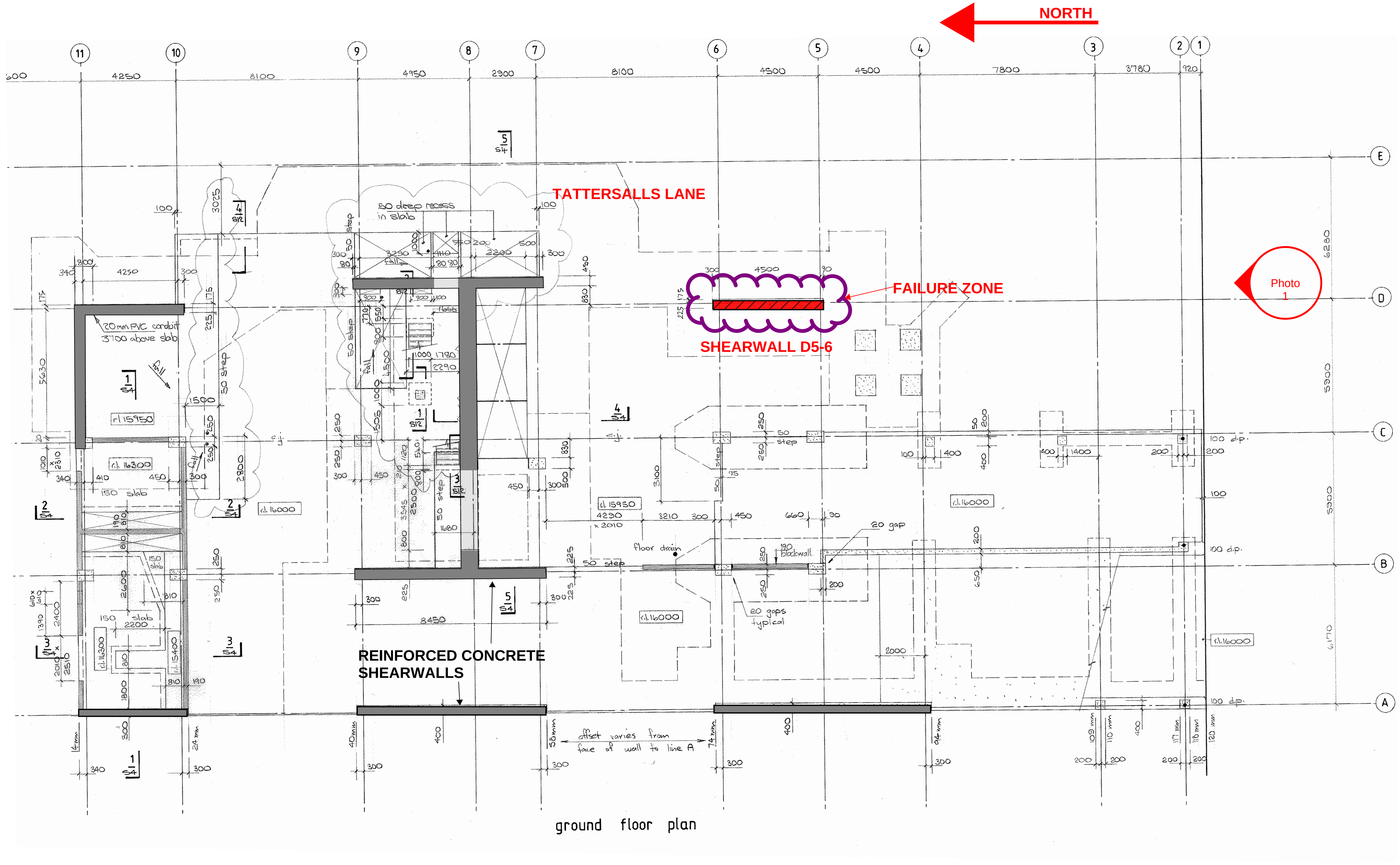
EXTRACT FROM ORIGINAL STRUCTURAL CALCULATIONS

STRUCTURAL CONCEPT



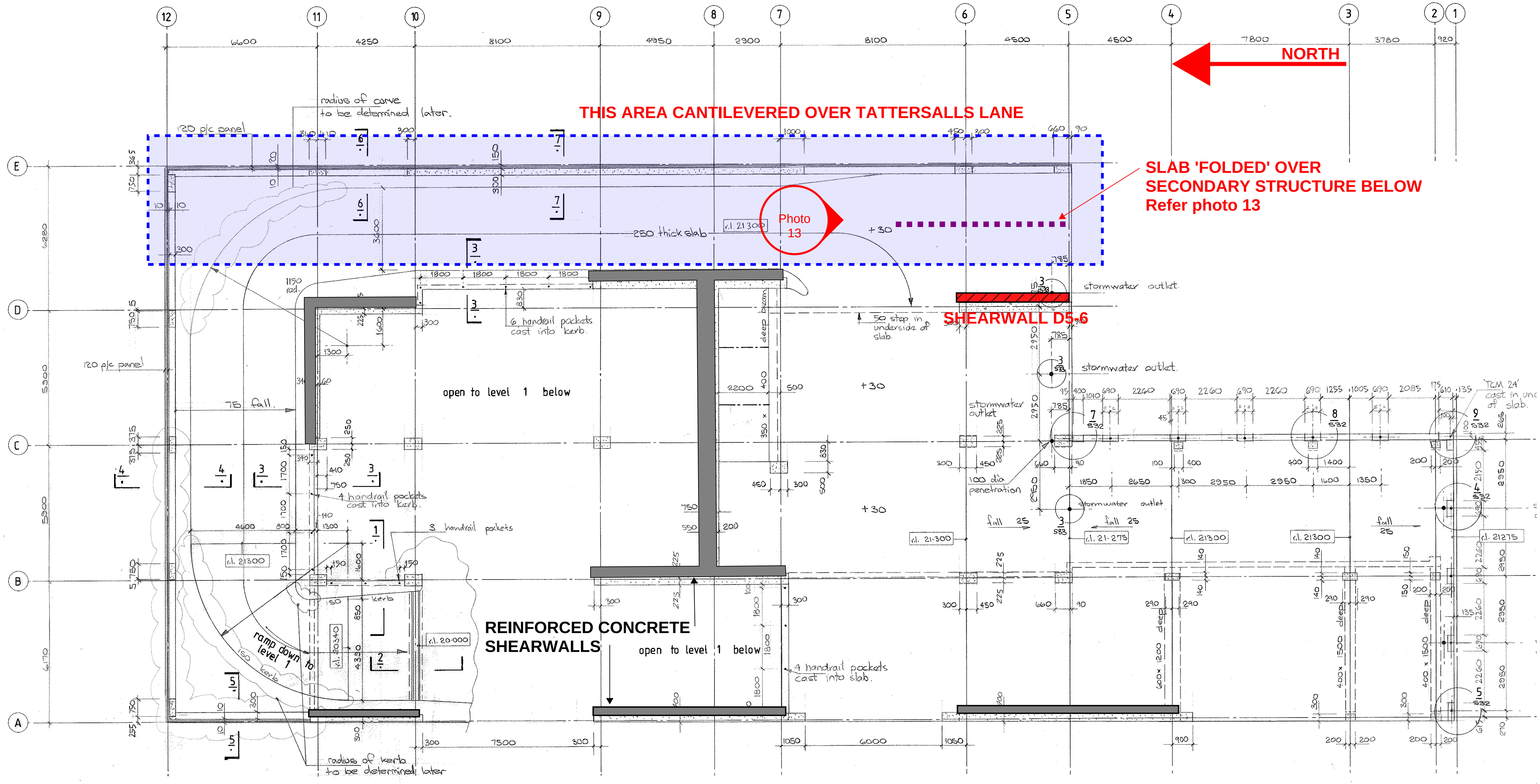
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

FOUNDATION PLAN



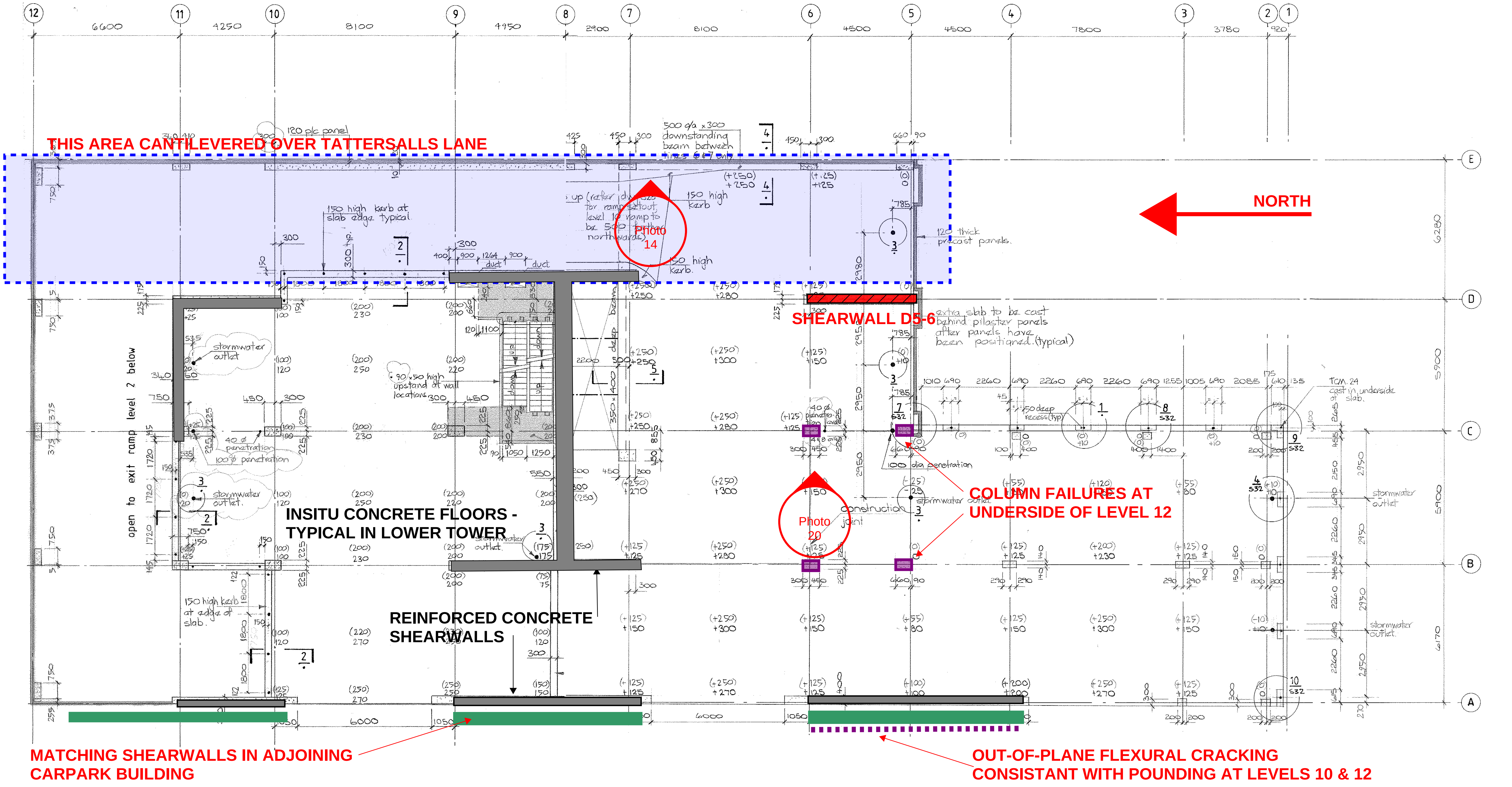
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

GROUND FLOOR PLAN



EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

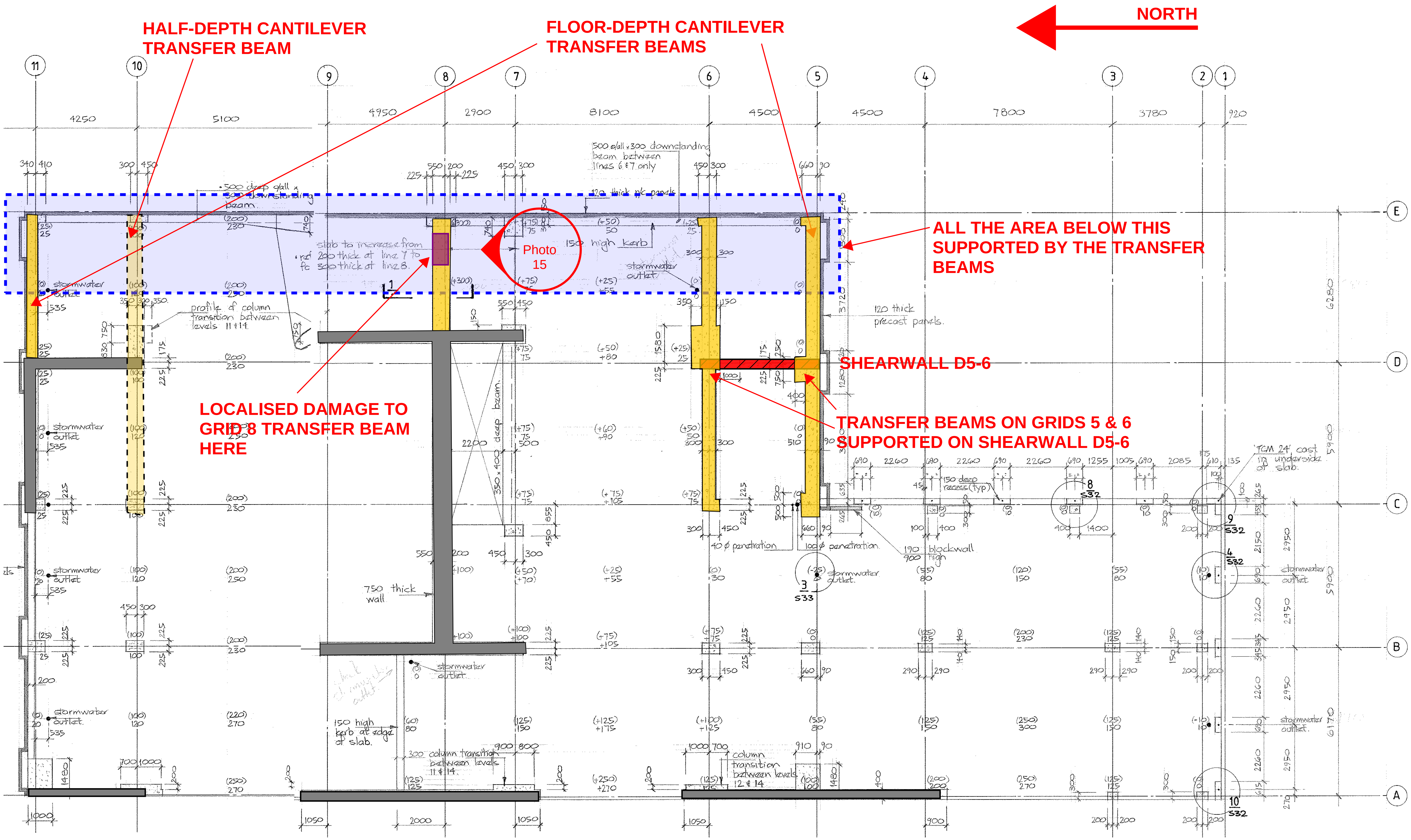
LEVEL 2



EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

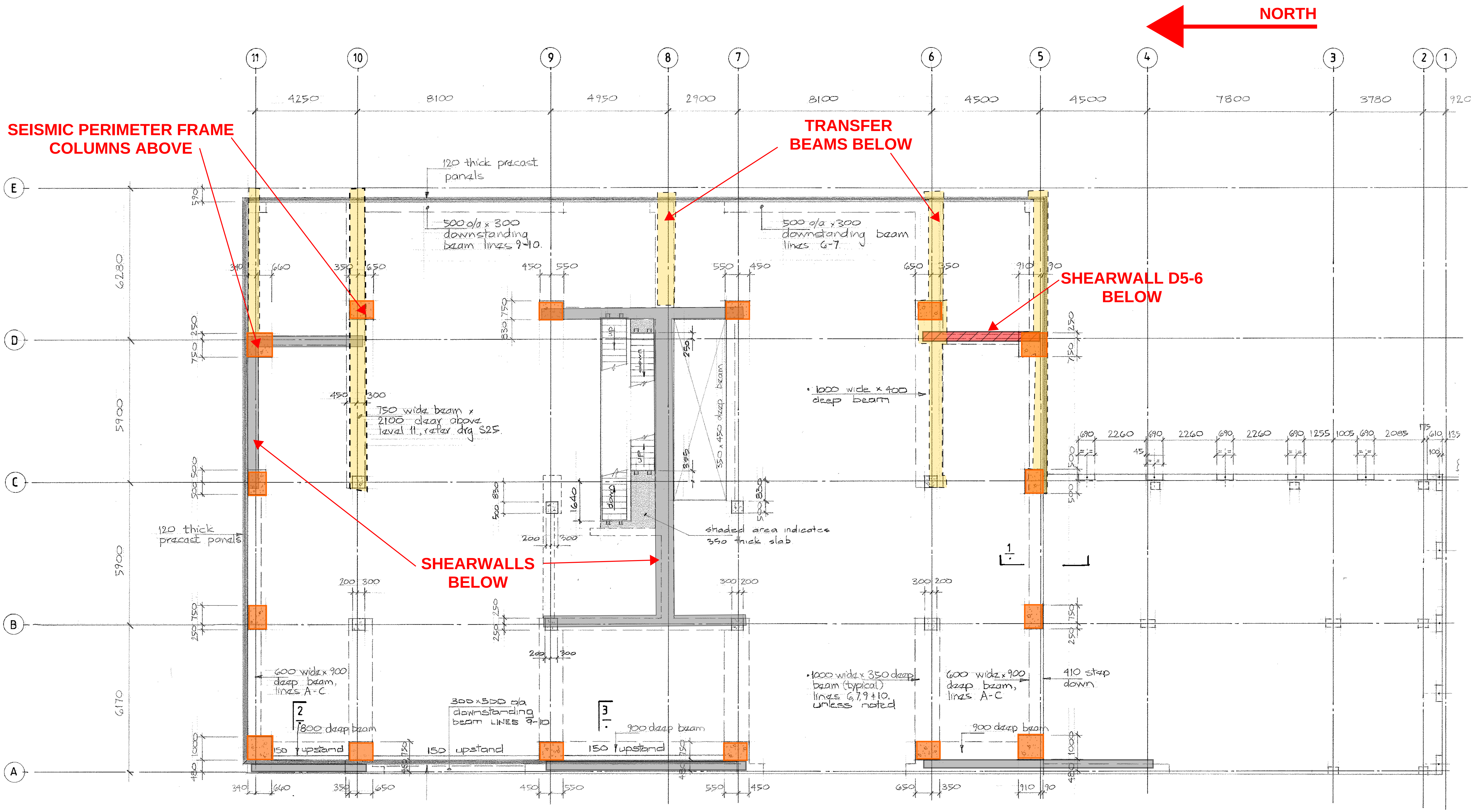
LEVELS 3 TO 8 SIMILAR

LEVELS 9 & 10



EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

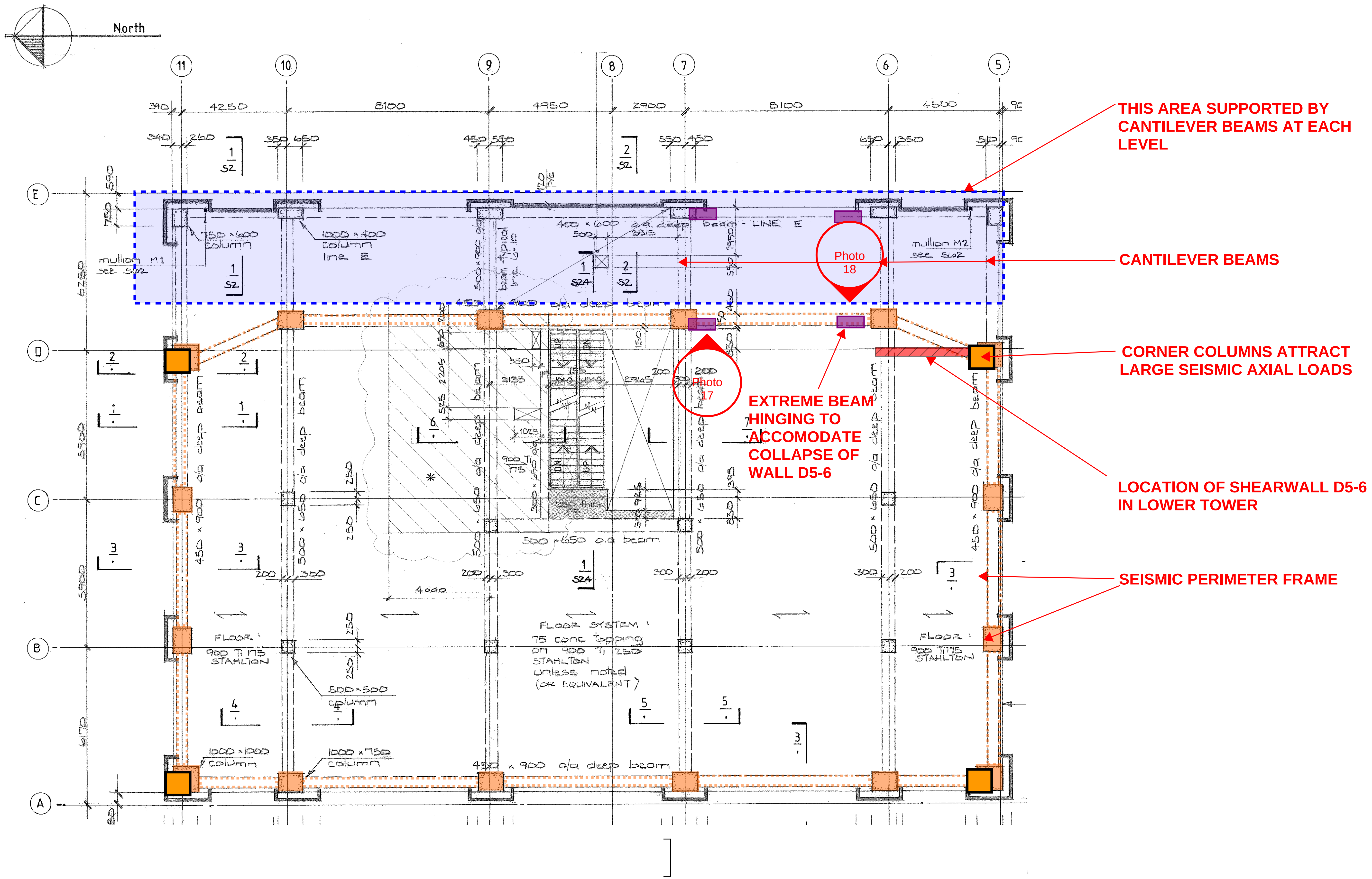
LEVELS 11 & 12



NOTE: LEVEL 14 IS THE TRANSFER FLOOR WHERE THE SEISMIC RESISTING SYSTEMS CHANGE FROM SHEARWALLS BELOW TO MOMENT RESISTING FRAMES ABOVE

EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

LEVEL 14

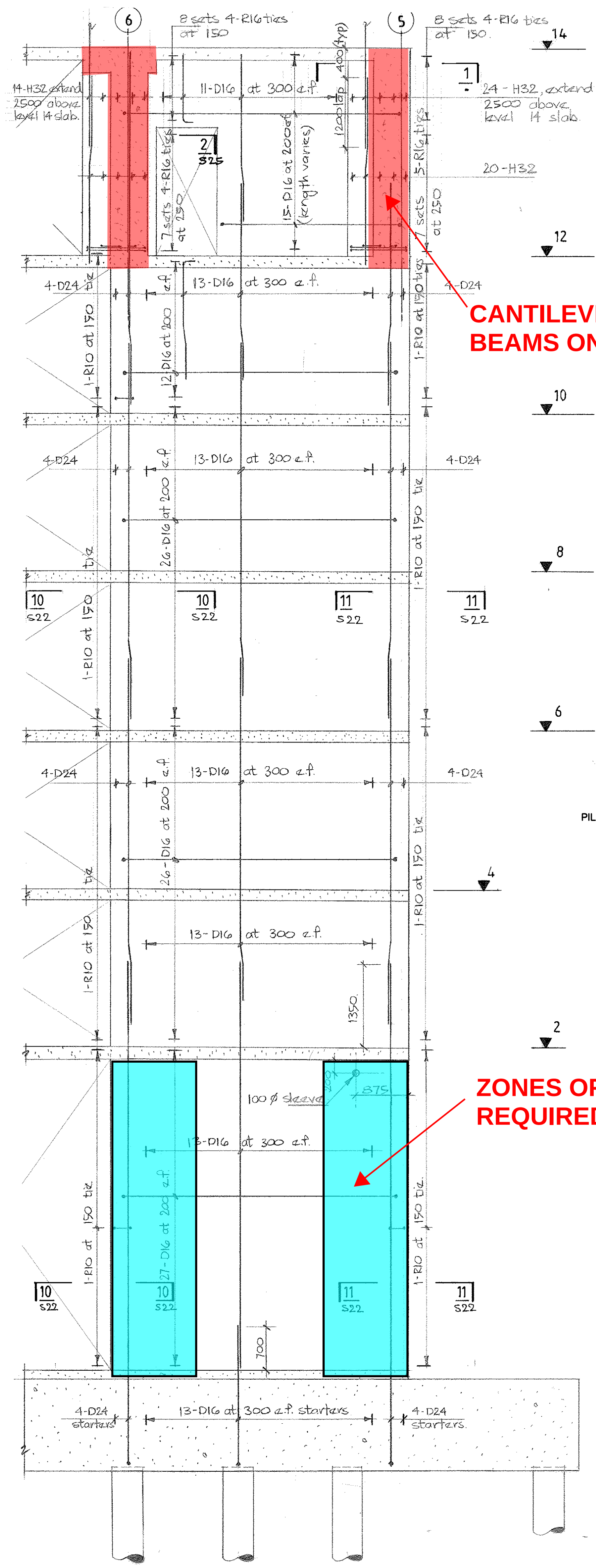


TYPICAL FLOOR

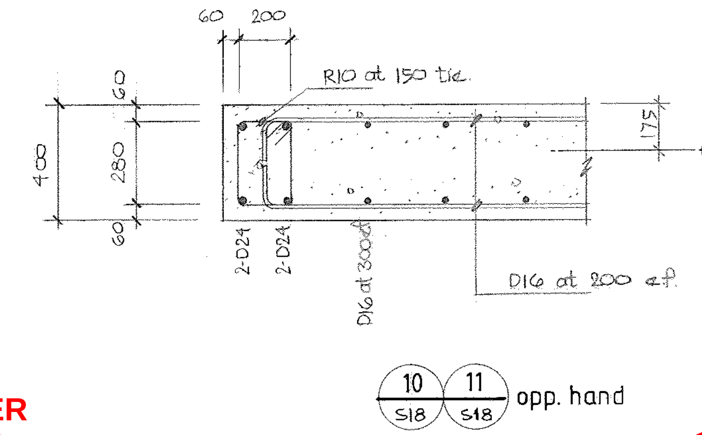
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

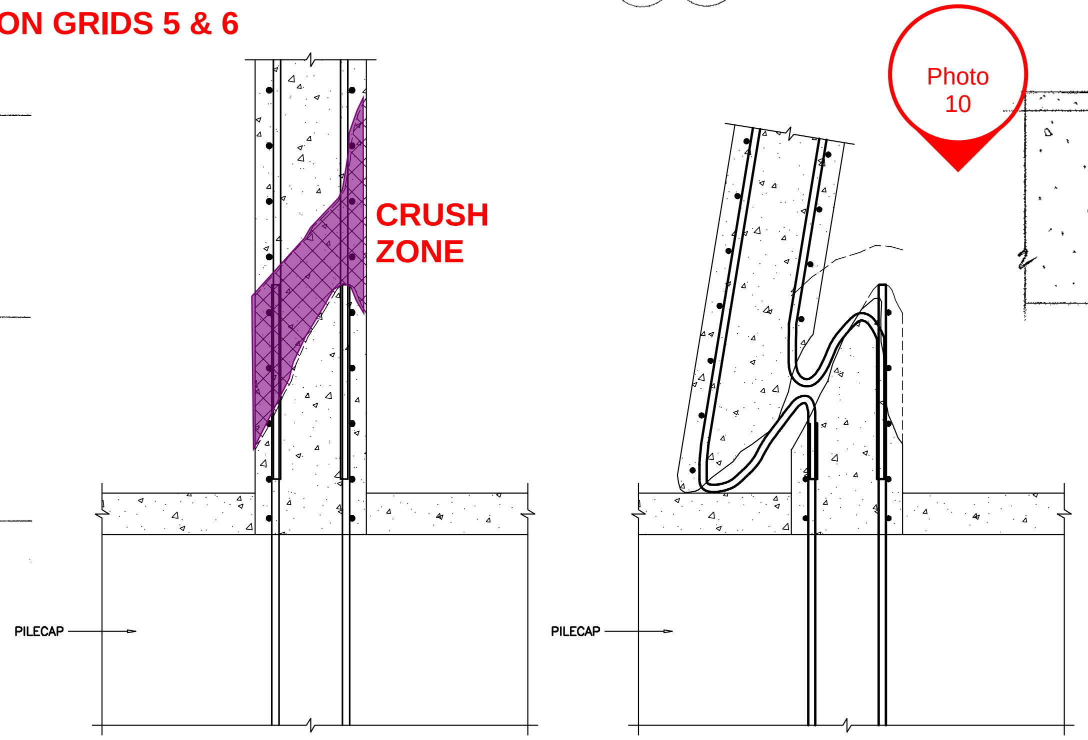
TYPICAL UPPER TOWER PLAN

EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

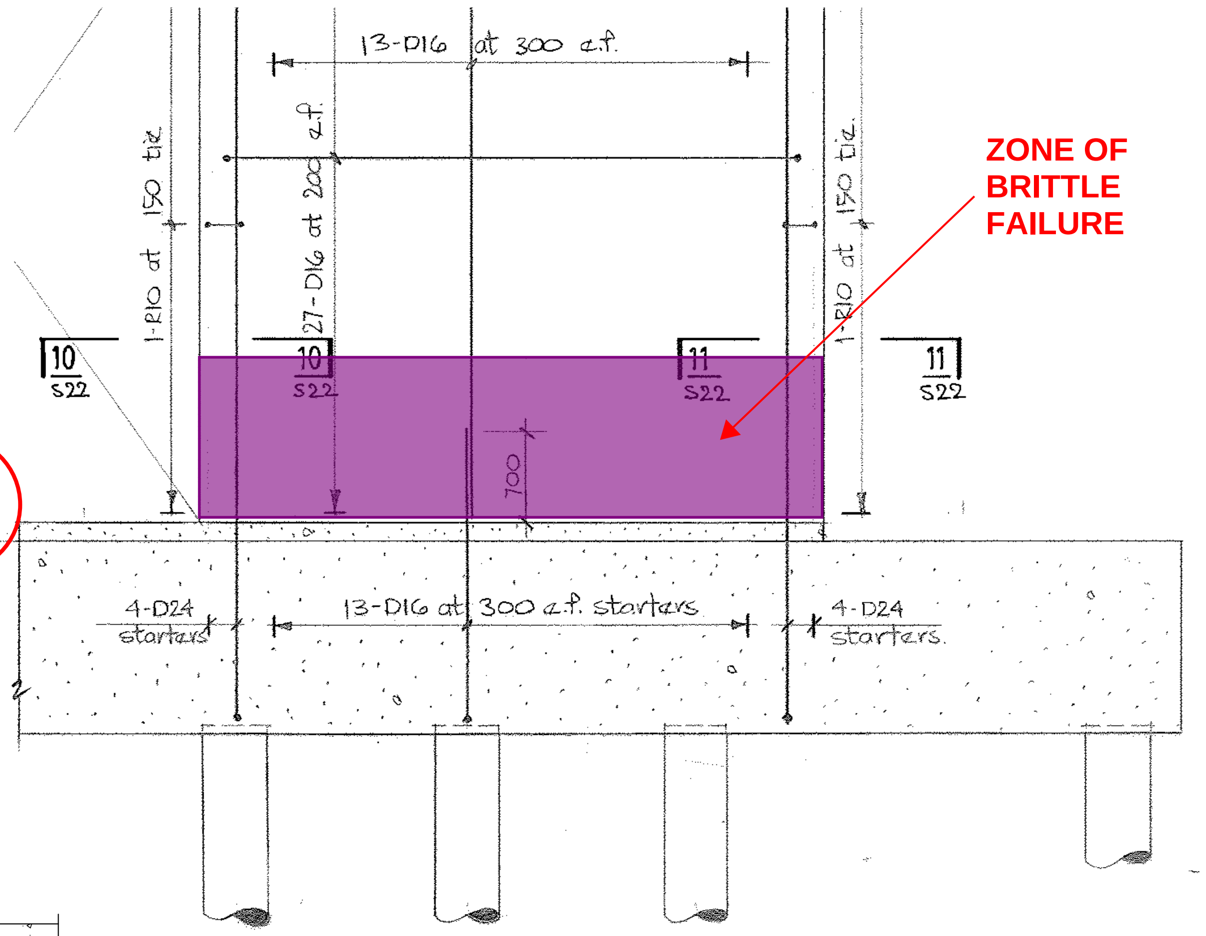


WALL D5-6 ELEVATION



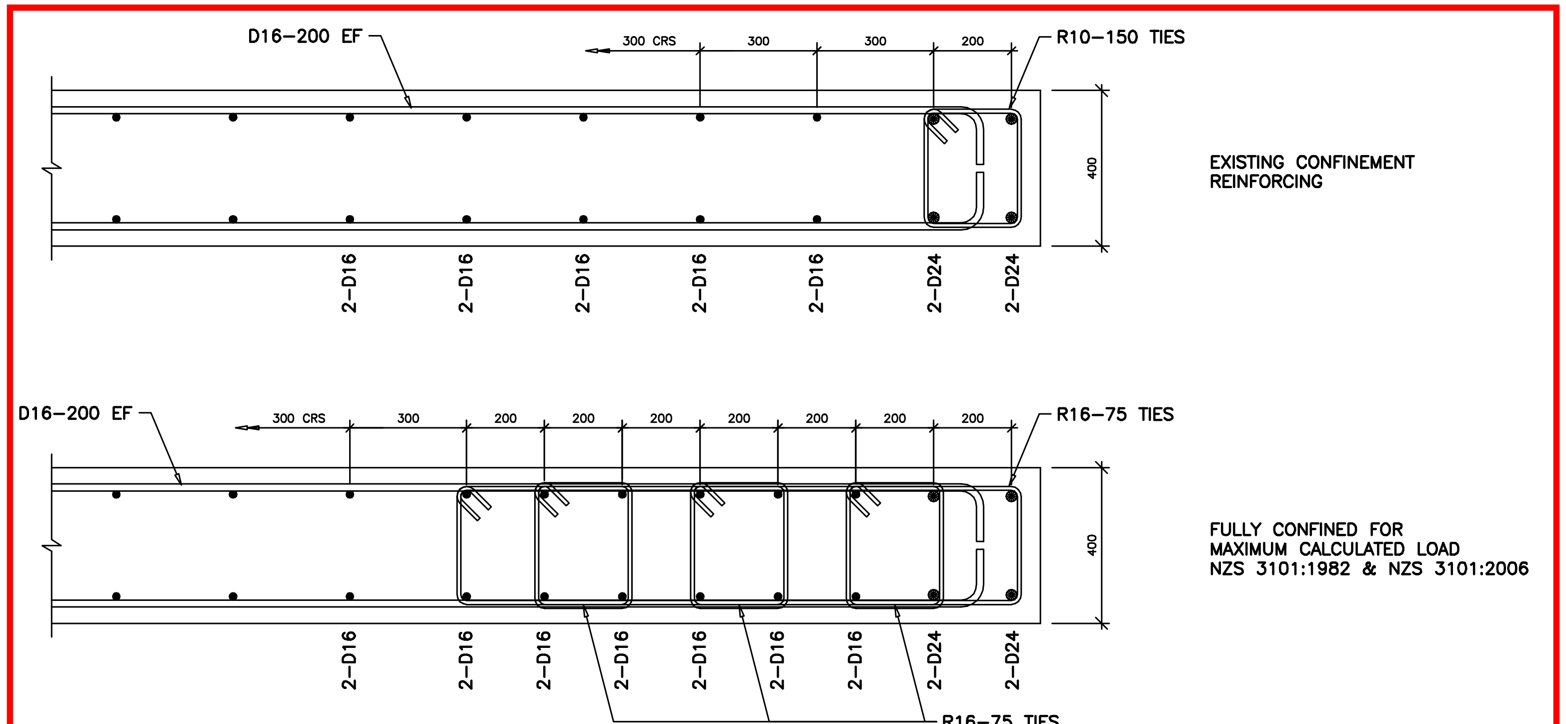


CROSS-SECTION SHOWING FAILURE MODE

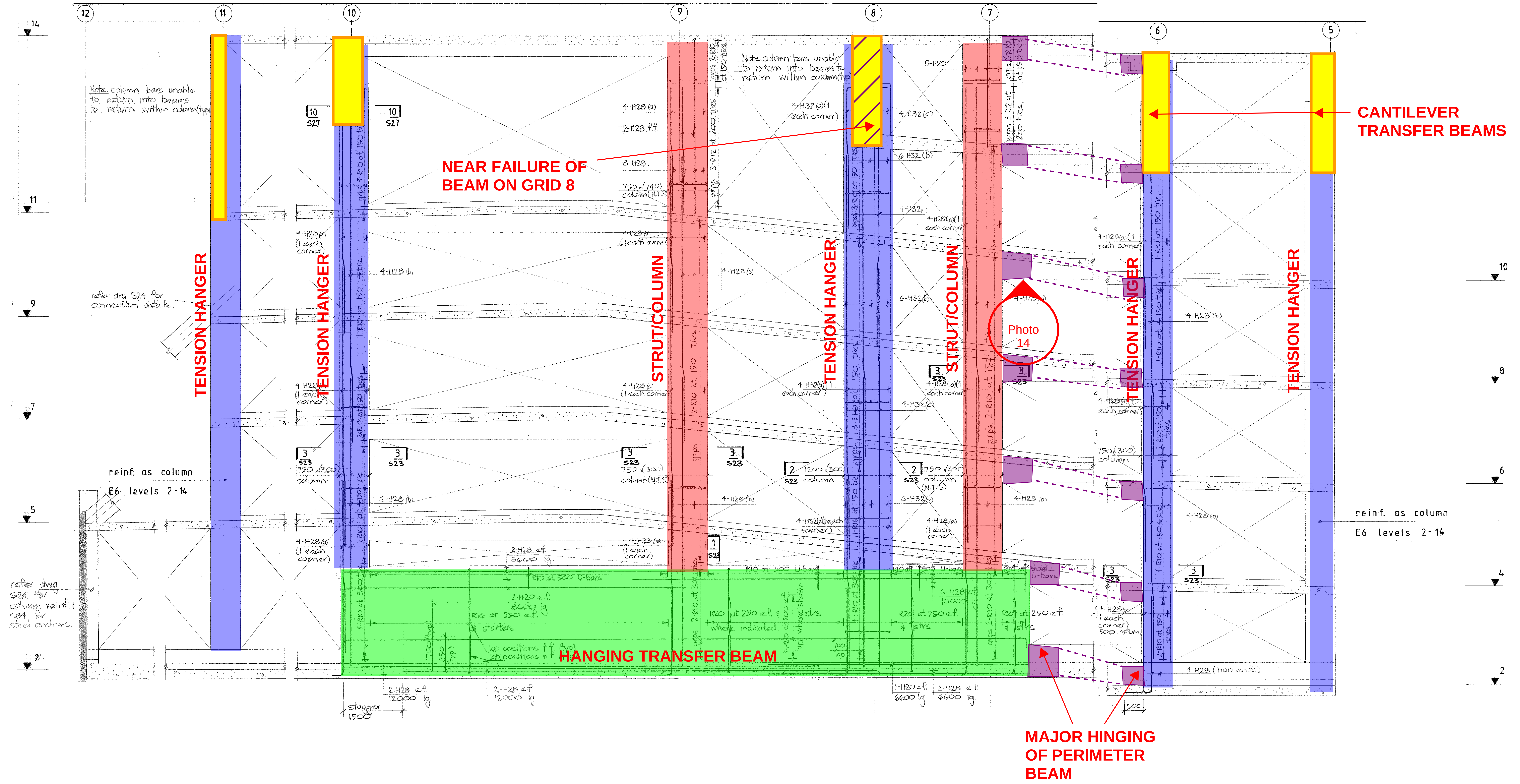


BASE OF WALL DETAIL

WALL D5-6

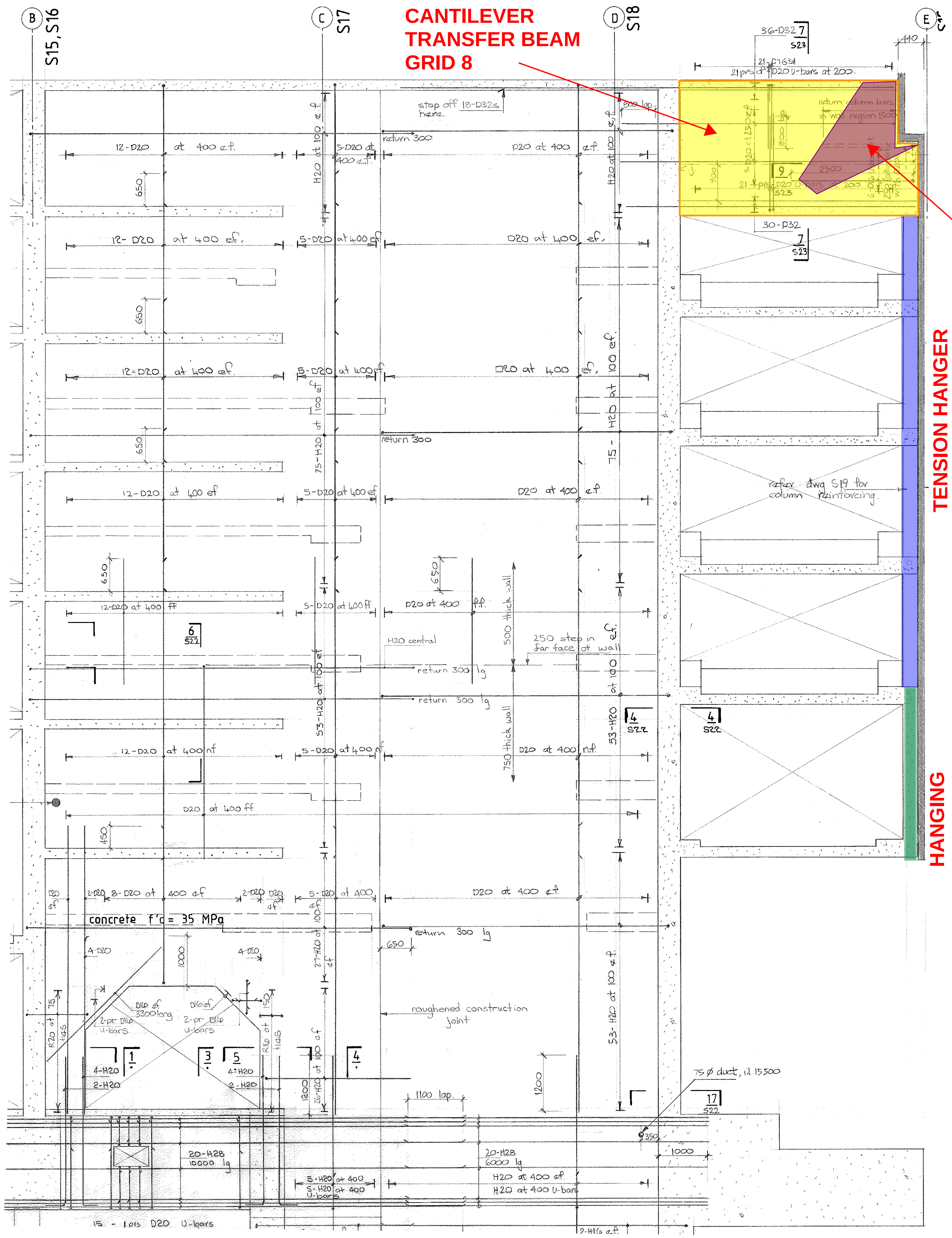


CONFINEMENT REQUIREMENTS

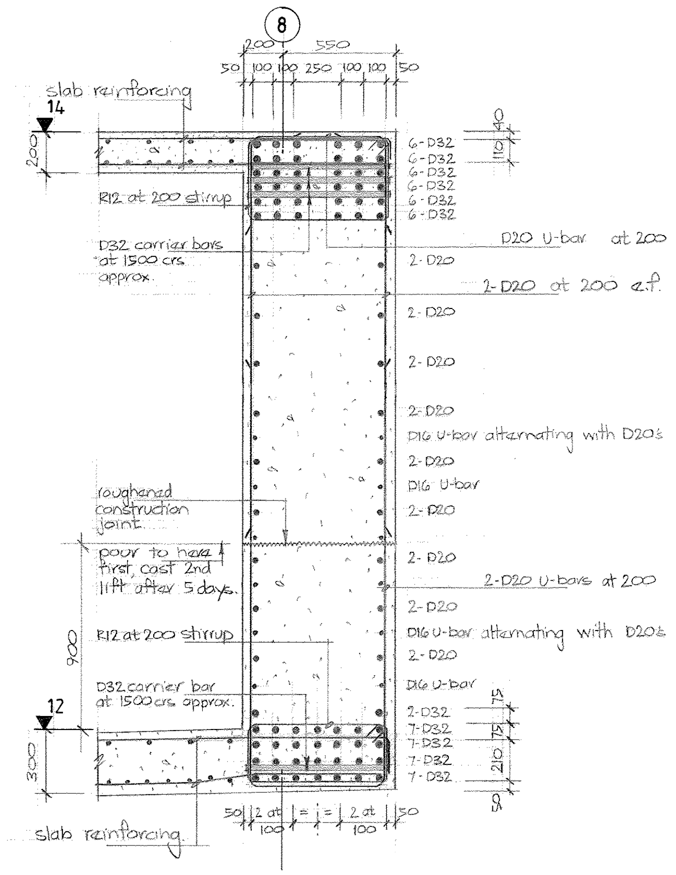


EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

HANGING WALL LINE E



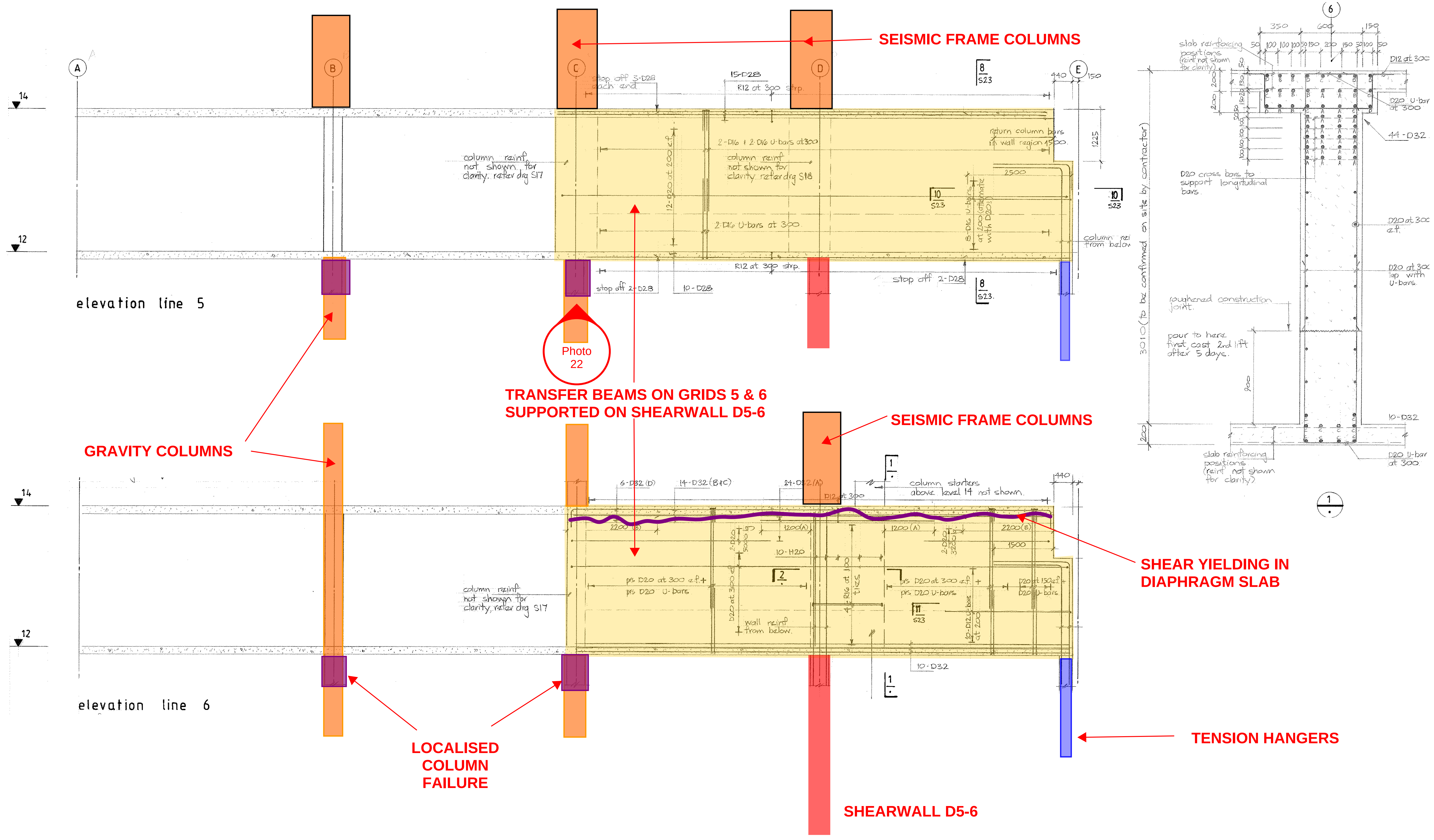
LAP FAILURE



EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

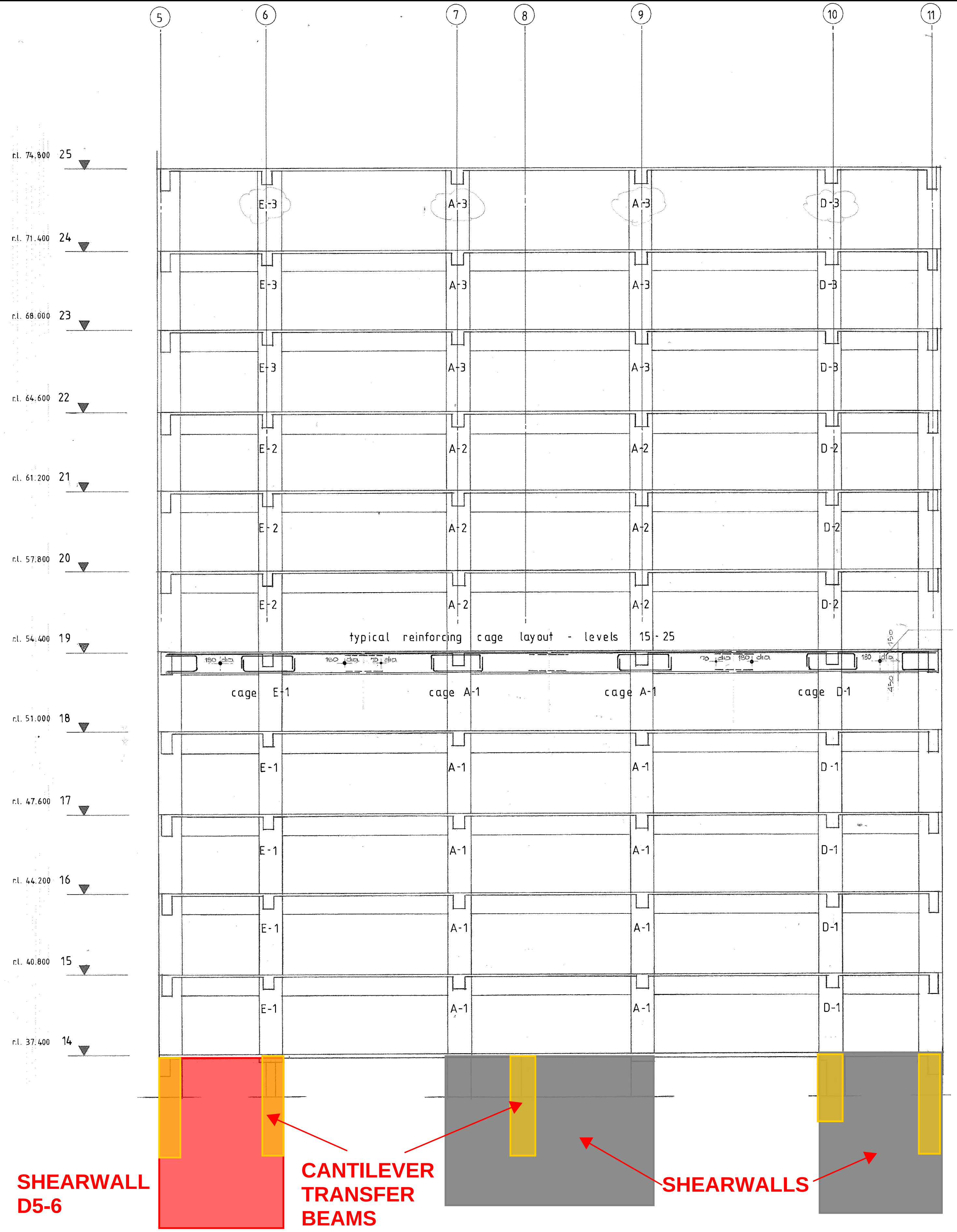
ELEVATION GRID 8

TRANSFER BEAM GRID 8



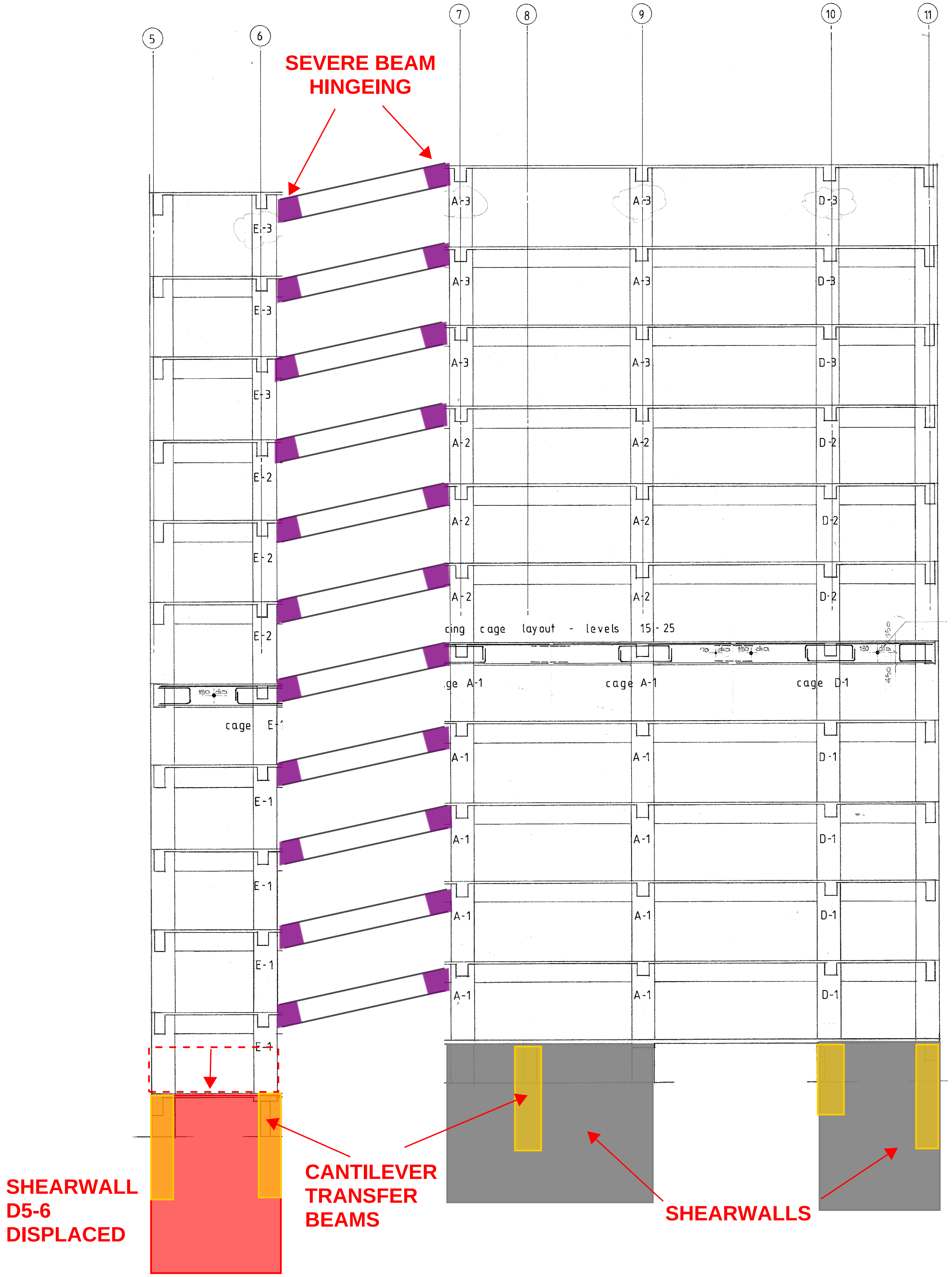
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

TRANSFER BEAMS GRIDS 5 & 6



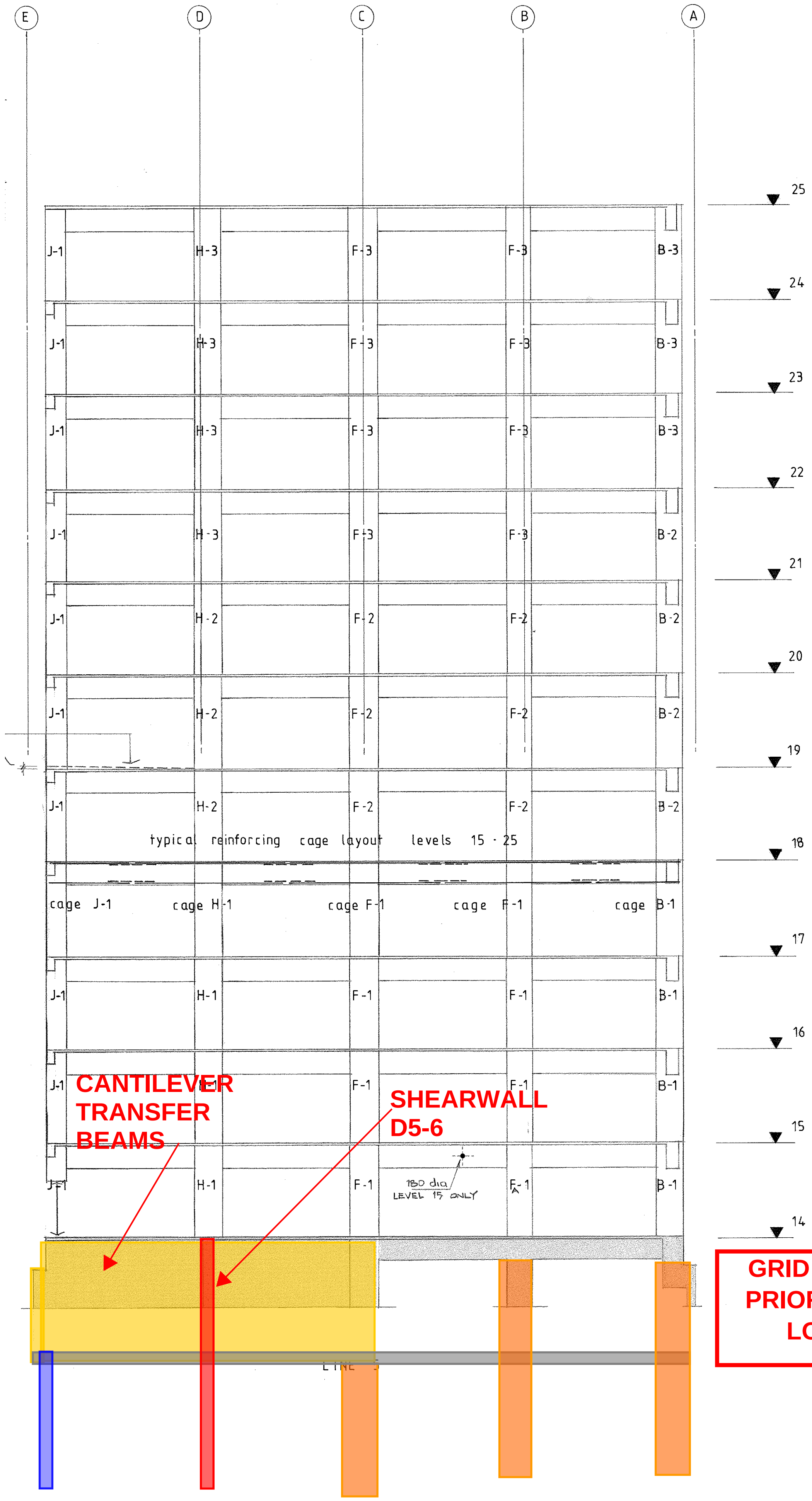
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

GRID D SEISMIC FRAME PRIOR TO 22 FEBRUARY



GRID D SEISMIC FRAME POST 22 FEBRUARY

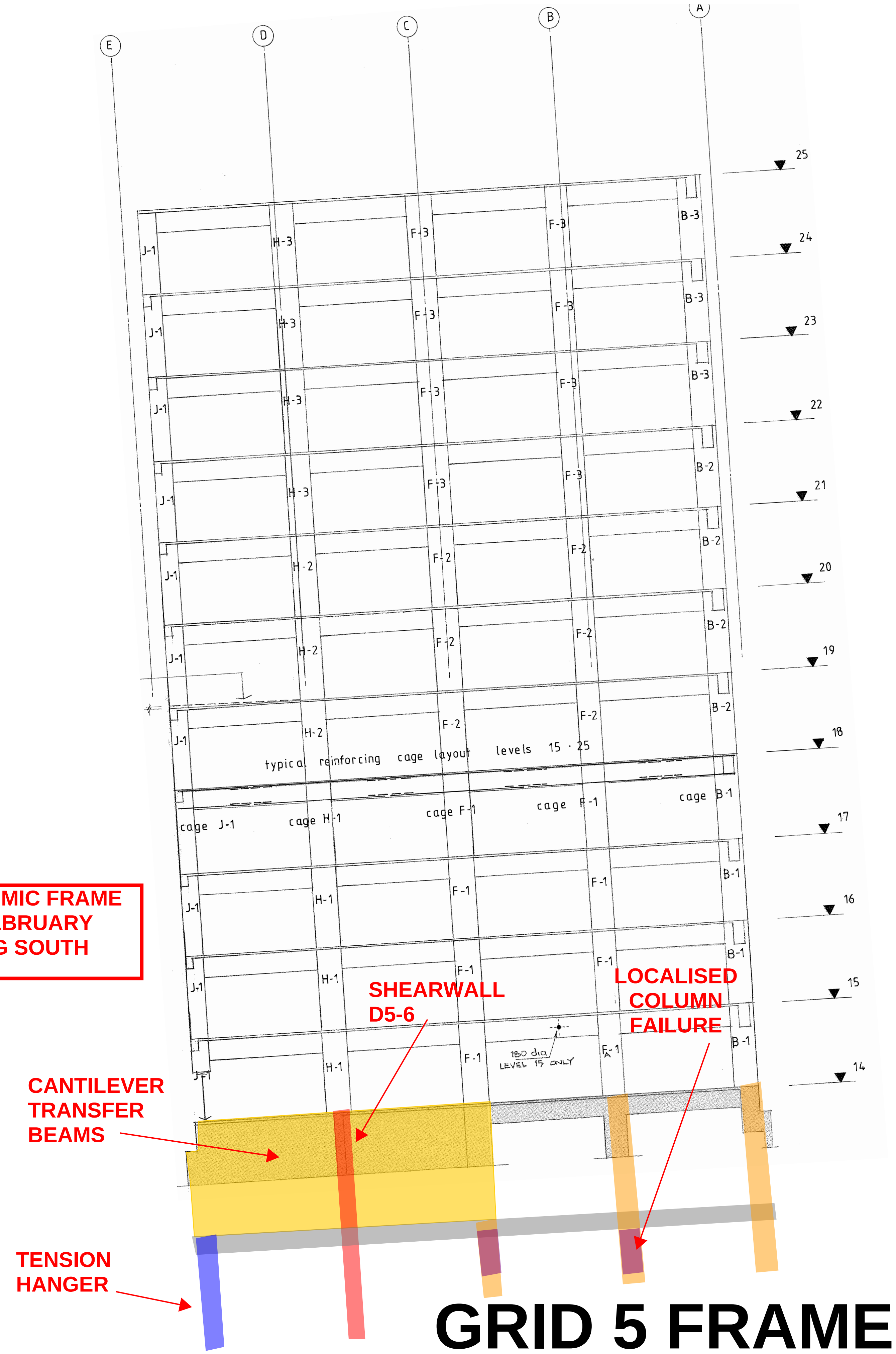
GRID D FRAME



GRID 5 SEISMIC FRAME PRIOR TO 22 FEBRUARY LOOKING SOUTH

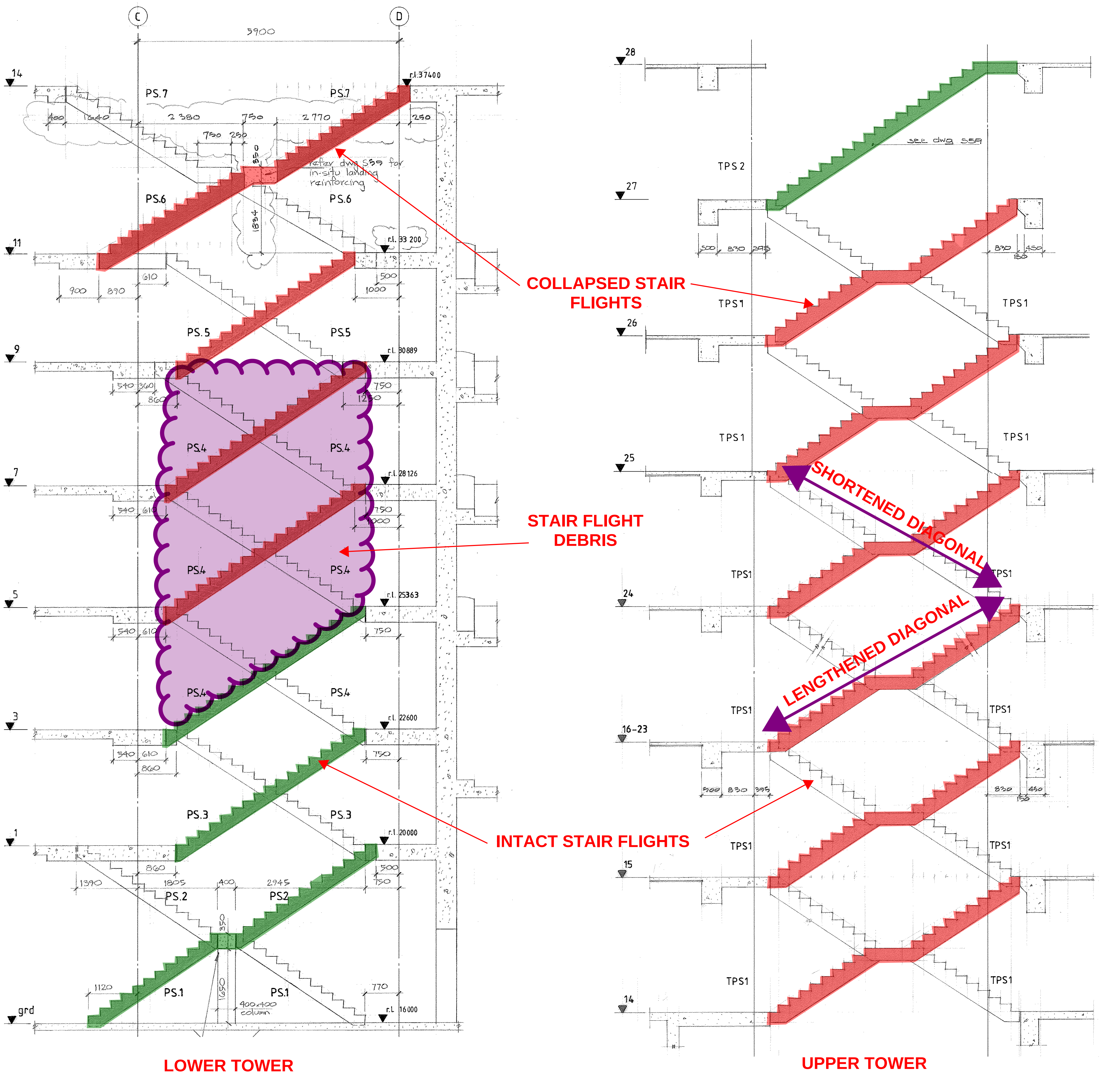
EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

GRID 5 SEISMIC FRAME POST FEBRUARY LOOKING SOUTH

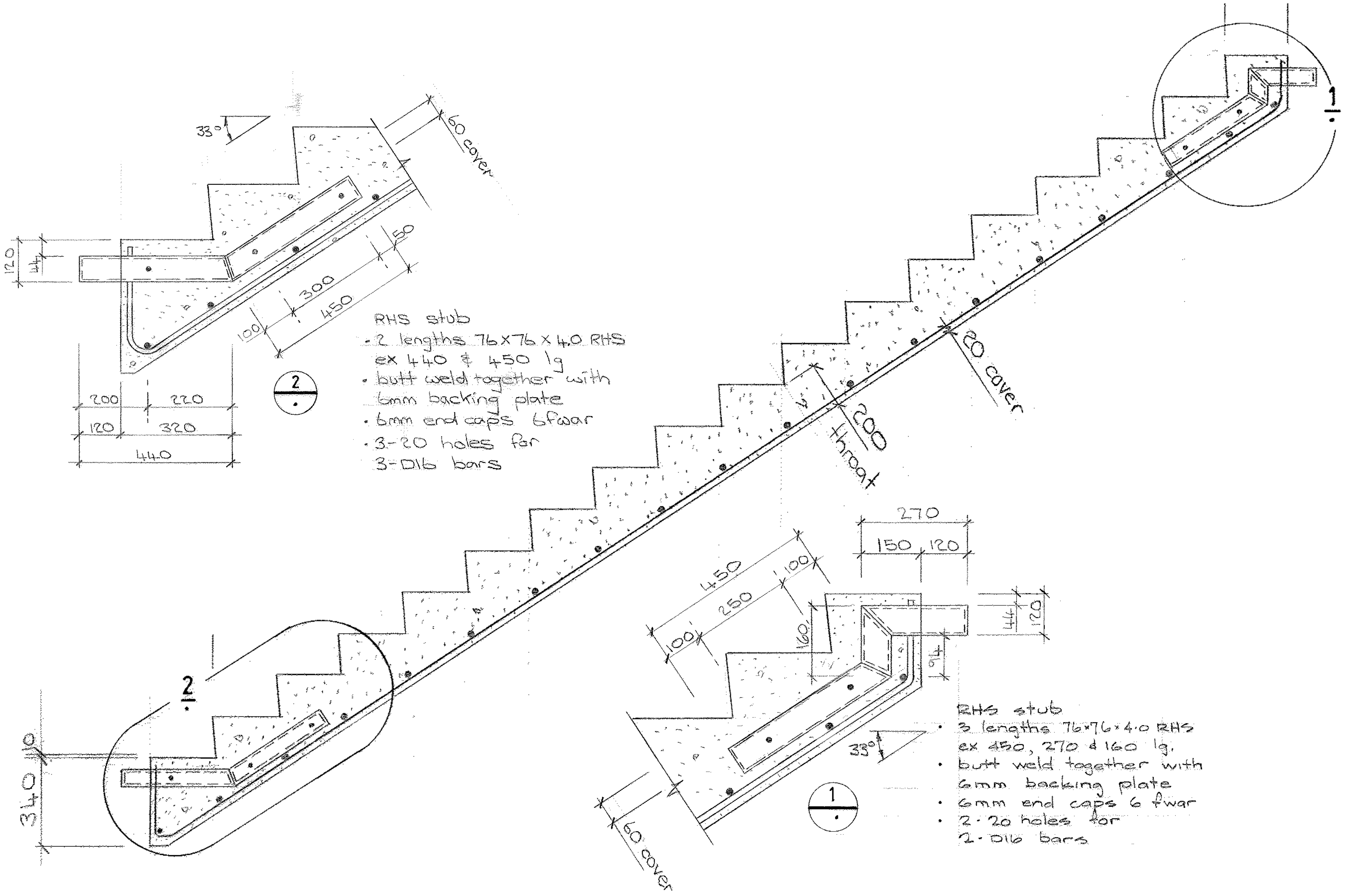


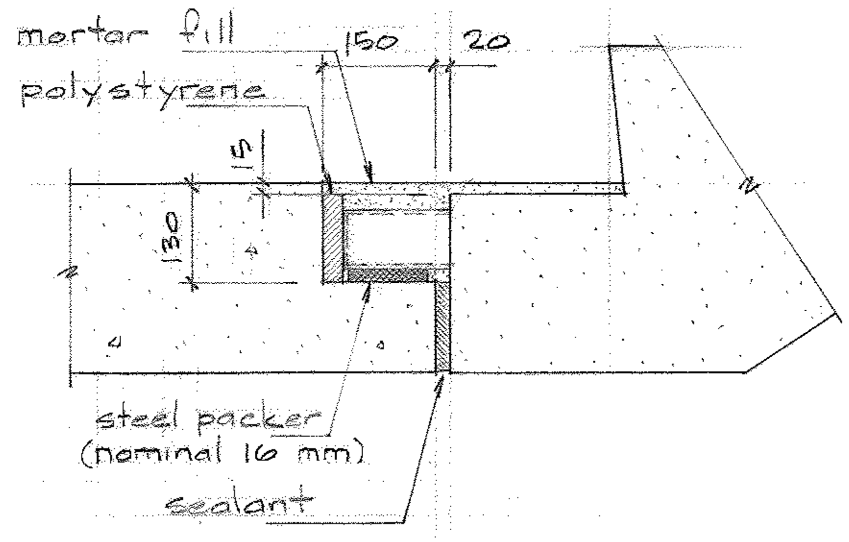
GRID 5 FRAME

EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

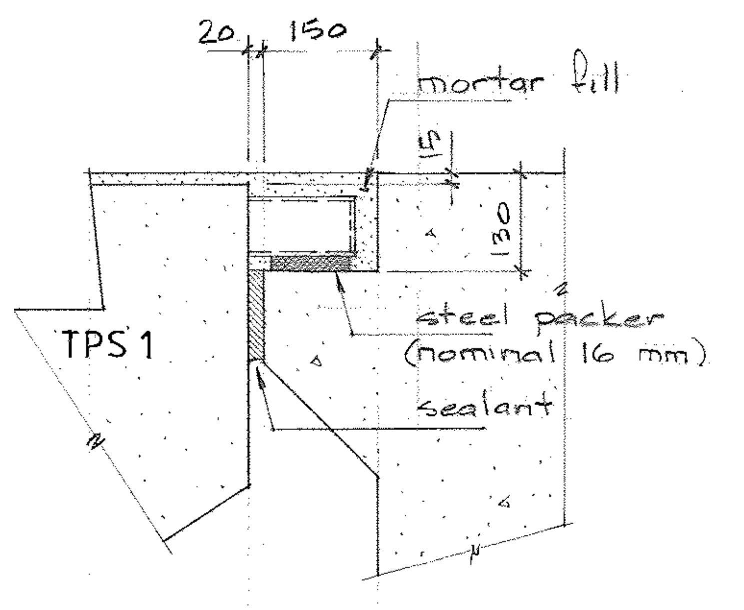


STAIR SECTIONS

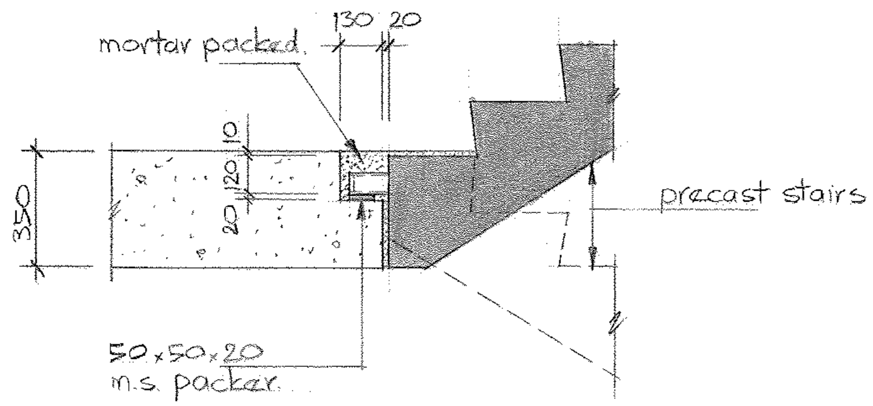




bottom fixing TPS1
TPS2



top fixing TPS1



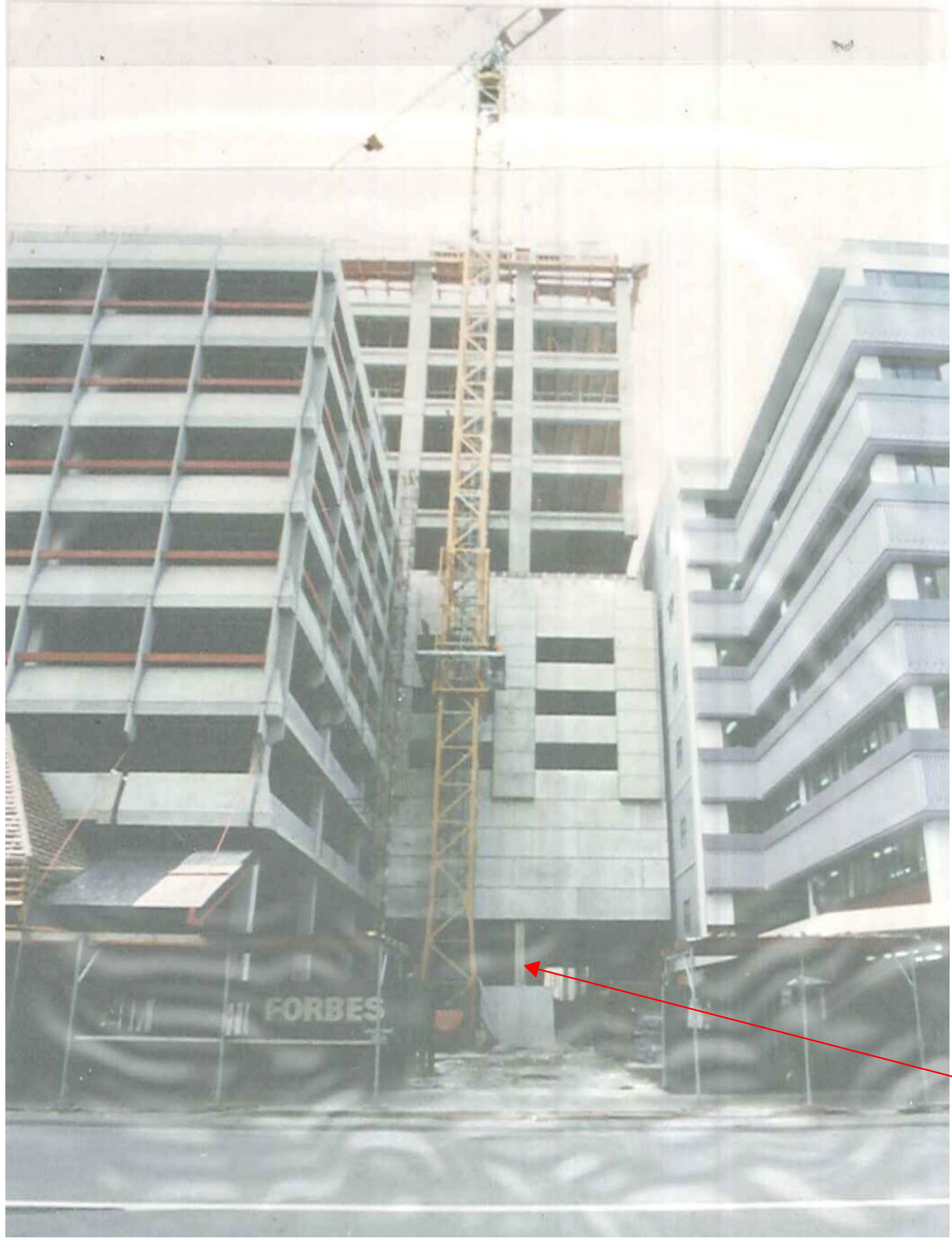
typical stair fixing
at floor levels

EXTRACT FROM ORIGINAL STRUCTURAL DRAWINGS

TYPICAL PRECAST STAIR DETAILS

APPENDIX C

PHOTOS



Cantilever
Transfer beam
on Grid 5

Shearwall D5-6

Photo 1 - Southern Elevation - During Construction

Separation between upper
and lower frames on grid E

Photo 2 - Southern Elevation - During Construction

Photo 3 - Southern Elevation - Post February

Photo 4 - Eastern Elevation - Post February

Photo 5 - Damage at junction between podium and tower - not related to pounding

Photo 6 - Shearwall D5-6 - Base Failure

Photo 7 - Shearwall D5-6 - Hingeing at top of ground floor

Photo 8 - Shearwall D5-6 - Hingeing at top of ground floor

Photo 9 - Shearwall D5-6 Failure - End View

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

Photo 11 - Similar Shearwall Failure
Not the Hotel Grand Chancellor

Photo 12 - Similar Shearwall Failure
Not the Hotel Grand Chancellor

Photo 13 - Folded slab at level 2

Photo 14 - Hingeing in grid E beams 6-7

Photo 15 - Near-lap failure grid 8 cantilever transfer beam

Photo 16 - Near-lap failure grid 8 cantilever transfer beam

Photo 17 - Hingeing in grid D tower beams

Photo 18 - Hingeing in grid D tower beams

Photo 19 - Hingeing in grid D tower beams

Photo 20 - Crushed columns at level 10, lines 5 & 6

Photo 21 - Crushed columns at level 10, lines 5 & 6

Photo 22 - Crushed columns at level 10, lines 5 & 6

Photo 23 - Top of Intact stair flight


Photo 24 - Highest surviving stair, supporting debris


Photo 25 - Stair Debris

APPENDIX D

POST-SEPTEMBER INSPECTION REPORTS

[EXTRACTS]

EXTRACT FROM LETTER DATED 28 SEPT 2010
From inspecting Engineer to Building Owner

The hotel is predominately a concrete frame structure with interspan concrete floors. There is an adjoining car park with seismic separation.

The following structural observations and inspections were made;

- No damage was observed to the surface of the roof or within the roof plantroom.
- A concrete beam column joint was observed from level 26. Access was from the laundry cupboard. A 45 degree hairline crack in the beam was observed indicating the beam has been stressed next to the support but no structural damage was observed and no repair is required.
- A concrete beam column joint was observed from level 18. Access was from the laundry cupboard. A hairline crack was observed similar to the floor 26.
- A concrete beam column joint was observed from level 17. Access was via room 1715. Three hairline cracks were evident indicating stressing but no structural damage was observed and no repair is required.
- The stairs construction is precast stair units with insitu concrete landings. The stairs appear to have been have been detailed to be fixed at the top with a sliding support at the base. An uneven landing surface under the floor lining was exposed to reveal that the concrete cover to the landing had spalled away from the top of the stair unit connection, This is considered to be because the stair unit had not slid sufficiently at the base. This defect was observed to be typical at a number of landings. The floor linings will need to be lifted at every level to confirm the extent of the damage and a concrete patch repair undertaken.
- The base of a single cladding panel to the carpark had moved outward in the absence of bolts on the base connection. This panel was being moved back and bolted into place.
- The seismic joint between the carpark and hotel structure was observed and had worked as expected with no structural damage noted. However there was some superficial damage to flashing plates and cracks in bituminous the ramp which need repair and replacement.

In addition to the structural observations above the following damage was noted

- Gib board in the stairwell was cracked at a number of levels both through the gib board and along joints. This will need re-stopping of cracked joints and replacement of gib where the sheet is cracked
- Fire sprinkler heads have moved, typically popping up through the gib pipe penetration. This will need to be mended by a sprinkler installation team.
- Gib board in the rooms was observed to be badly damaged in some rooms requiring re-stopping of joints and in some locations replacement of the board.
- Around the lobby entrance to the lifts some tiling had come loose which will need to be re-grouted. This is considered to be a falling hazard.
- Front lobby window to seals were loose and need to be reinstated.
- Cracking to gib board primarily to the beam column joint locations in the lobby. Cracking needs to be locally repaired
- Doors that catch against the frames will need to be re-hung and potentially the frame adjusted.

- The upper level of the carpark showed some minor spalling of the cover concrete in the ramp area. This was not considered to be the result of earthquake movement but rather due to water increase causing rusting of the concrete resulting in expansion and cracking of the cover concrete. Loose material should be removed, the reinforcement wire brushed free of rust and a concrete patch repair made.
- Concrete infill on carpark level 5 adjacent to the seismic joint had crumbled. This could be replaced with timber or more concrete
- A small concrete infill along the line of the seismic joint had been cracked and was loose. These loose bit needs to be removed as they are a falling hazard. Any exposed reinforcement should be painted and flashing should be placed over damaged area for durability.
- Some cracked windows and movement cracking in sealants and seals around windows.

Based on the above observed damage there is no concern for the structural stability or strength of the structure, which appears to have performed well under the earthquake.

The following recommendations are made

- Repairs to be undertaken as noted above. Repair to brittle elements such as gib and decorating repairs should not be undertaken until aftershocks have ceased and the hotel structure has ‘settled’ releasing any stresses due to the significant movement that has occurred. This could be ongoing for some weeks and repair before this could result in new cracking.
- Inspection of the outside of the building be undertaken to record and repair defects, specifically window seals and cracked panes of glass
- Inspection of the inside of the lift shaft to confirm structure is ok and there is no compromise of the fire separation between floors.
- There was significant gib damage to the conference floor at one beam column joint. This will need repair but should be inspected by an Engineer once the gib has been removed.
- The connection points of all concrete panels in the car park should be checked for any signs of hairline cracks or damage emanating from the bolt fixings
- In assessment of the damage it should be noted that there could be some ongoing movement and the extent of the damage may increase over the next few weeks

The above information is provided based on checking and observation of areas where access was available and advice is provided based on these observations.

EXTRACT FROM LETTER DATED 26 OCT 2010
From inspecting Engineer to Building Owner

This inspection took place on 1 October 2010. Gib lining had been removed from either side of the wall adjacent to the sliding doors to enable visible inspection by torchlight. It is noted that the location of the inspection coincides with change in floor area from the lower floors to the tower.

Primary structure observed was 2 steel columns, and concrete panels which were understood to be attached to the concrete frame of the tower. There was no damage noted of to the structure of the building and the gib cracking is thought to be due to the movement between the steel and concrete frames. The waterproofing detailing should be checked in this area as there may have been some movement of connections.

It was noted that there was some lateral displacement of the hangers and guide rail to the folding partition. These doors and others similar should be monitored for any crabbing or stiffness that is occurring as the lateral loads placed on the fixings may have caused some distortion of the hangers.

EXTRACT FROM LETTER DATED 1 FEB 2011
From inspecting Engineer to Building Owner

Summary of works:
A detailed inspection was carried out by industrial abseilers. The inspection was visual, with tap testing and immediate removal of dangerous structures as required.

Photographs are supplied of damaged areas and marked on elevation drawings.

Four types of damage where noted:

1. Cracking of the sealant/expansion joints. We estimate that around 400m + of sealant needs to be replaced.

2. Concrete cracks. There is a range of severity in the cracking with the worst being around the car park area and at level 16.

3. Broken windows. Most of these have been temporarily repaired and you will be aware of them already.

4. Separation of the corners of some window frames.

The true extent and complexity of the damage may not be apparent till the damaged areas are fully opened up, and further aftershocks might add to the damage. But overall the exterior of the Grand Chancellor has stood up well.

This damage is similar to the damage we are currently repairing on several commercial buildings in the city and we are capable of carrying out any or all of the external repairs via rope access.

We are extremely happy with the standard to which this work has been carried out.

There are no health and safety issues to report.

APPENDIX E

CONCRETE TEST RESULTS

CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	Concrete Condition Assessment
Location :	Grand Chancellor Hotel
Client :	Dunning Thornton Consultants Limited
Contractor :	John Jones Steel Limited
Sampled by :	Concut Limited
Date sampled :	8 June 2011
Sampling method :	Rotary Coring Drill
Sample description :	Nominal 100mm Diameter Concrete Cores
Sample condition :	Dry as received
Date cored :	8 June 2011
Source of concrete :	Not advised
Grade of concrete :	Not advised
Design strength :	Not advised
Actual slump :	Not advised
Date laid :	Not advised

Project No :	6-LABS0/CASH
Lab Ref No :	5921
Client Ref No :	Adam Thornton

Test Results

Lab reference no		166/1	166/2	166/3	166/4	166/5	166/6	166/7
Client reference no		7 & 8 Grid B	7 & 8 Grid B	7 & 8 Grid B	Stairwell	Shear wall Level 2-1	Shear wall Level 2-2	Spare
Date tested		14 June 2011						
Age	(days)	Not Advised						
Size & position of any reinforcement		Nil Steel	Nil Steel	Nil Steel	Rebar	Nil Steel	Nil Steel	Nil Steel
Visual description		Homogeneous Concrete						
Average core diameter	(mm)	94.0	94.0	94.1	94.0	93.9	93.9	94.3
Average core length	(mm)	186.3	190.1	193.4	193.6	190.0	191.7	144.3
Mass of core prior to cappin	(g)	3073	3083	3114	3271	3095	3109	2327
Density	(kg/m^3)	**2380**	**2340**	**2320**	**2430**	**2350**	**2340**	**2330**
Height diameter ratio		1.98	2.02	2.06	2.06	2.02	2.04	1.53
Conditioning		Tested dry as received						
Load at failure	(kN)	330.9	174.4	228.6	300.7	288.2	316.7	289.1
Compressive strength	(MPa)	**47.5**	**25.0**	**33.0**	**43.5**	**41.5**	**45.5**	**41.5**
Type of fracture		Cone	Columnar	Cone	Cone	Cone	Cone	Cone

Test Methods	Notes
Testing of Cores, NZS 3112 : Part 2 : 1986, Clause 9	Cylinder 166/2 (7 & 8 Grid B) was cracked upon receipt
Compression, NZS 3112 : Part 2 : 1986, Clause 6	
Density, NZS 3112 : Part 3 : 1986, Clause 5	
Capping, NZS 3112 : Part 2 : 1986, Clause 4 (amendment No.2 2000)	

Date tested : 14 June 2011
Date reported : 14 June 2011

Sampling is not covered by IANZ Accreditation. Results apply only to sample tested.
This report may only be reproduced in full

IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 14 June 2011

new zealand accredited laboratory

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

PF-LAB-095 (18/12/2010)

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Christchurch Laboratory
Quality Management Systems Certified to ISO 9001
52C Hayton Road
Wigram, Christchurch 8042,
New Zealand
Telephone +64 3 343 0739
Facsimile +64 3 343 0737
Website www.opus.co.nz

Appendix F

Evaluation & Analysis

F) Evaluation / Analytical Detail

This appendix contains a summary of the evaluation and analysis of the Hotel Grand Chancellor structure.

F.1 Analysis of Wall WRT NZS3101:1982 & NZS4203:1984

The as-built structure has been modelled on Etabs to determine periods and displacements under code loading. Foundation flexibility has been included. Screen shots from the model are included in Figures 1 & 2. A load case with support removal from shear wall D5-6 was included to check the deformed shape. This matched the post February shape on site.

F.1.1 Axial Actions

Derived Axial Loads (at each end of wall)	**@-5D**	**@-6D**
Gravity Loads ($D+L_R$)	6300kN	8500kN
Mass contributing to Vertical Earthquake forces	450T	680T
Range of Vertical Earthquake Loads (V_E) *(range of 0.5g to 1.5g, Note NZS4203:1984 required 0.9g on parts)*	2300kN to 6800kN	3400kN to 10200kN
Seismic Overstrength Beam Shears (Voe) *from upper tower*	10100kN	-2800kN
Displacement Induced Seismic *from transfer beams*	3000kN	3000kN
D+1.3L + E	21700kN to 26200kN	12100kN to 18900kN
1.4D+1.7L	8800kN	12100kN
Total Load on Wall5-6 D+1.3L + E : (with $V_{E)}$: (without $V_{E)}$ 1.4D + 1.7 L	33,800 – 45,000kN 28000kN 20,900kN	

The components of axial load are illustrated on Fig 3.

F.1.2 Moment

In-plane code moment	
maximum from Etabs Analysis	6000kNm
Building over-strength of approximately 2	
Assessed moment Range	10-15MNm
Out-of-plane moment	600kNm

F.1.3 Shear

In-plane shear	800kN

F.1.4 Combined Actions – *possible maximums*

Axial load	33-45MN
Seismic in-plane moment	10-15MNm
Shear	1.5-2MN

Original Design Actions

Axial Load	=	17MN
Moment	=	8MN
Shear	=	800kN

F.1.5 Capacity of Wall - as derived from NZS3101:1982

Using the specified wall dimensions, reinforcing and concrete strength interaction diagrams have been derived (refer Fig.4). For an axial load of 30MN, a flexural capacity of around 35MNm is available. This suggests that the flexural strength of the wall is adequate and in fact had considerable overstrength. At lower axial loads there would have been ability for the wall to move beyond first yield (of the building) without ductility demand.

Curvature ductility capacity has also been computed for varying axial loads (refer Figs 5 & 6). From these it can be seen that under high axial loads the wall had very little available ductility, without confinement.

At ultimate limit state for an axial load of 30MN the neutral axis depth is around 2.8m long. This is over half the length of the wall and means, in effect, that a large portion of the wall, within the hinge region, requires confinement in order to provide adequate ductility.

F.1.6 Slenderness and Confinement

For a rectangular wall, Clause 10.5.2.1 of NZS3101:1982 imposes a slenderness limit of Ln/bw <10 unless the neutral axis depth is less than 4bw or 0.3Lw

For the D5-6 shearwall:

Lw/bw	= 12.75
4bw	= 1.6m
0.3Lw	= 1.46m

Therefore for load cases which result in a neutral axis depth in excess of 1460mm, the wall did not meet slenderness requirements.

Within the end zone of the wall, the reinforcing ratio ρ_l = As/bs$_v$ exceeds 2/Fy and so requires transverse reinforcing in accordance with clause 10.5.4.3. This requirement was met in the original design.

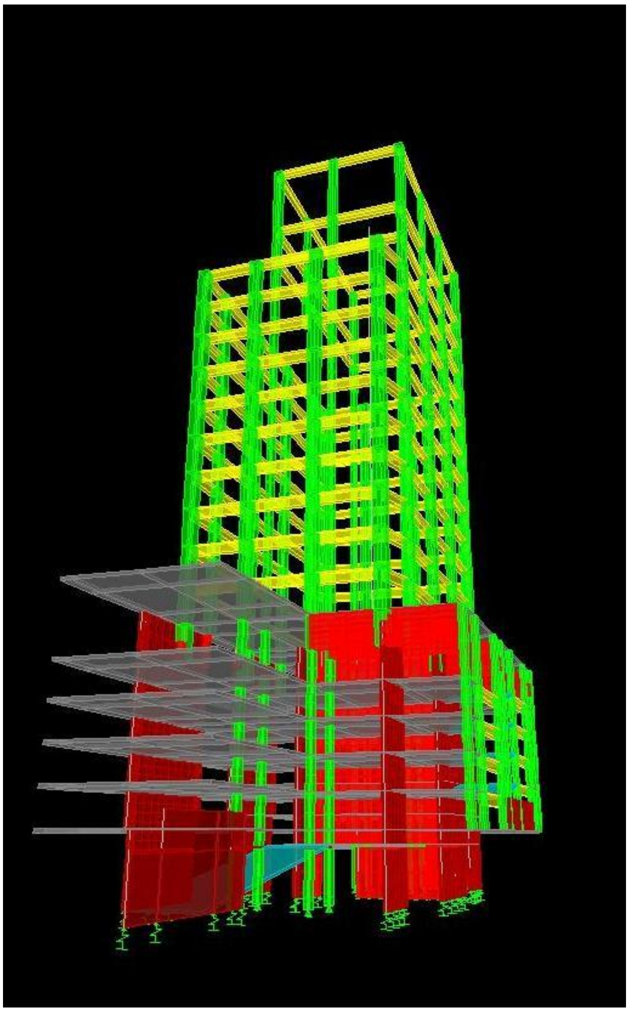
Clause 10.5.4.5 required confinement reinforcing when the neutral axis depth exceeds the critical value Cc,

Cc = $0.1Ø_oSL_w$
= 1000mm for $Ø_o$ = 2.5
= 2000mm for $Ø_o$ = 5.0

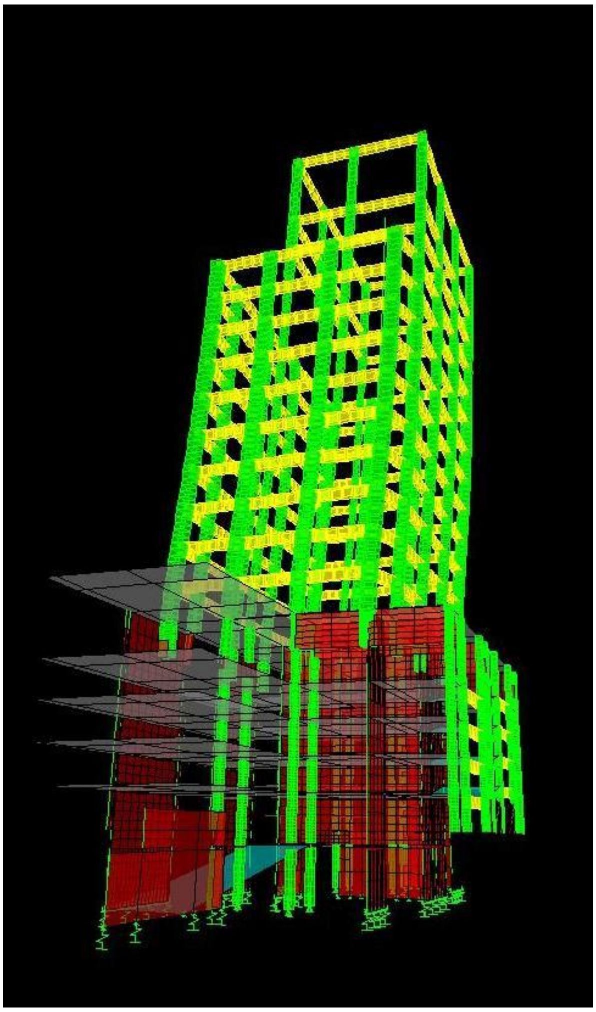
As noted in section F.1.5, this suggests confining reinforcing is required as the neutral axis depth of 2.8 exceeds Cc. In the original design Cc had been assessed at 2363mm, based on an overstrength of 6.04 and a calculated neutral axis depth of 1800mm.

F.2.0 Comparison of Base Shear Coefficients

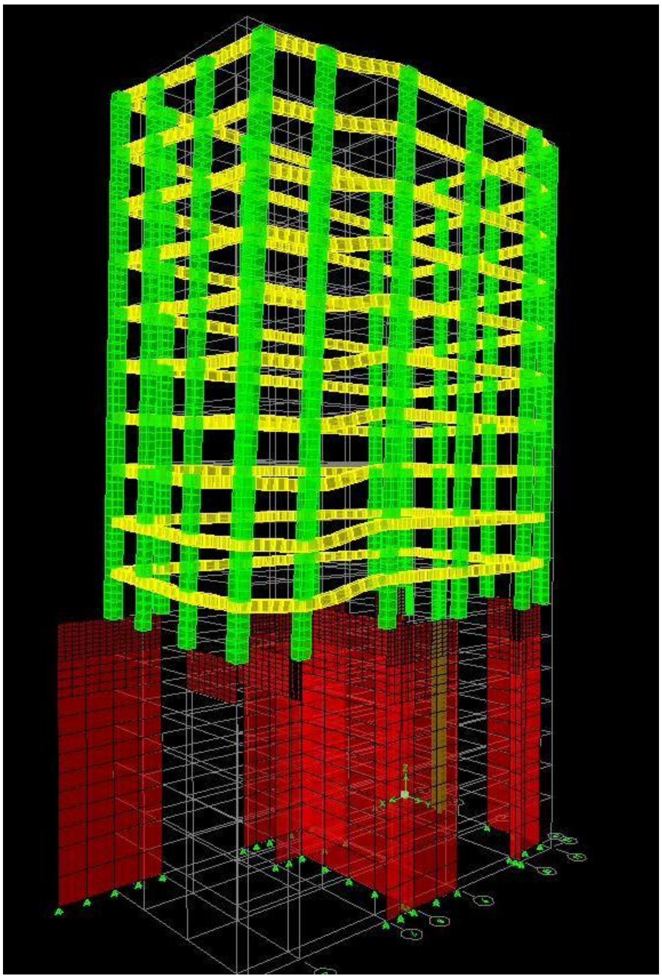
Original design assessment	Cd=0.048
Assessed value to NZS4203:1984	Cd=0.06
Assessed value to NZS1170.5:2004 (μ=3)	Cd(2.5)=0.045
Assessed value to NZS1170.5:2004 (μ=5)	Cd(2.5)=0.026



South (Front) View

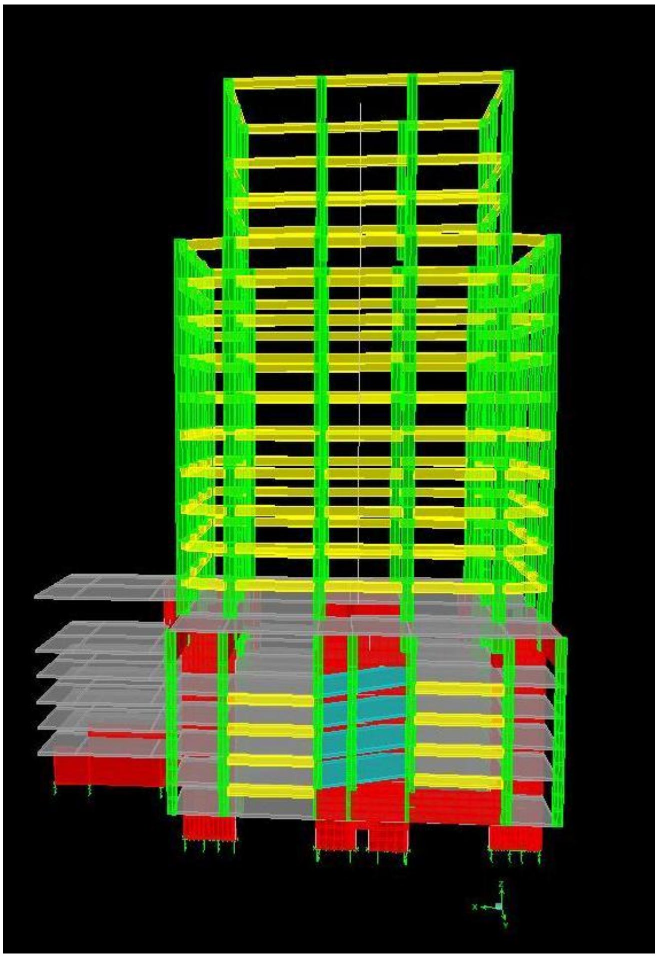


South View -Deformed

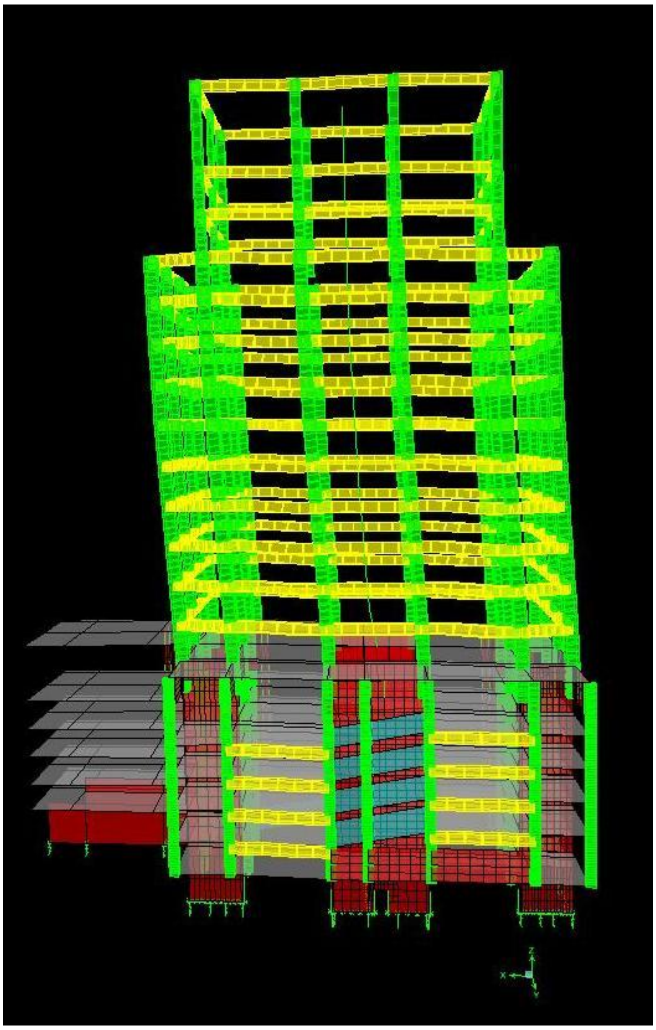


Corner Displacement

Etabs Screen Shots - 1

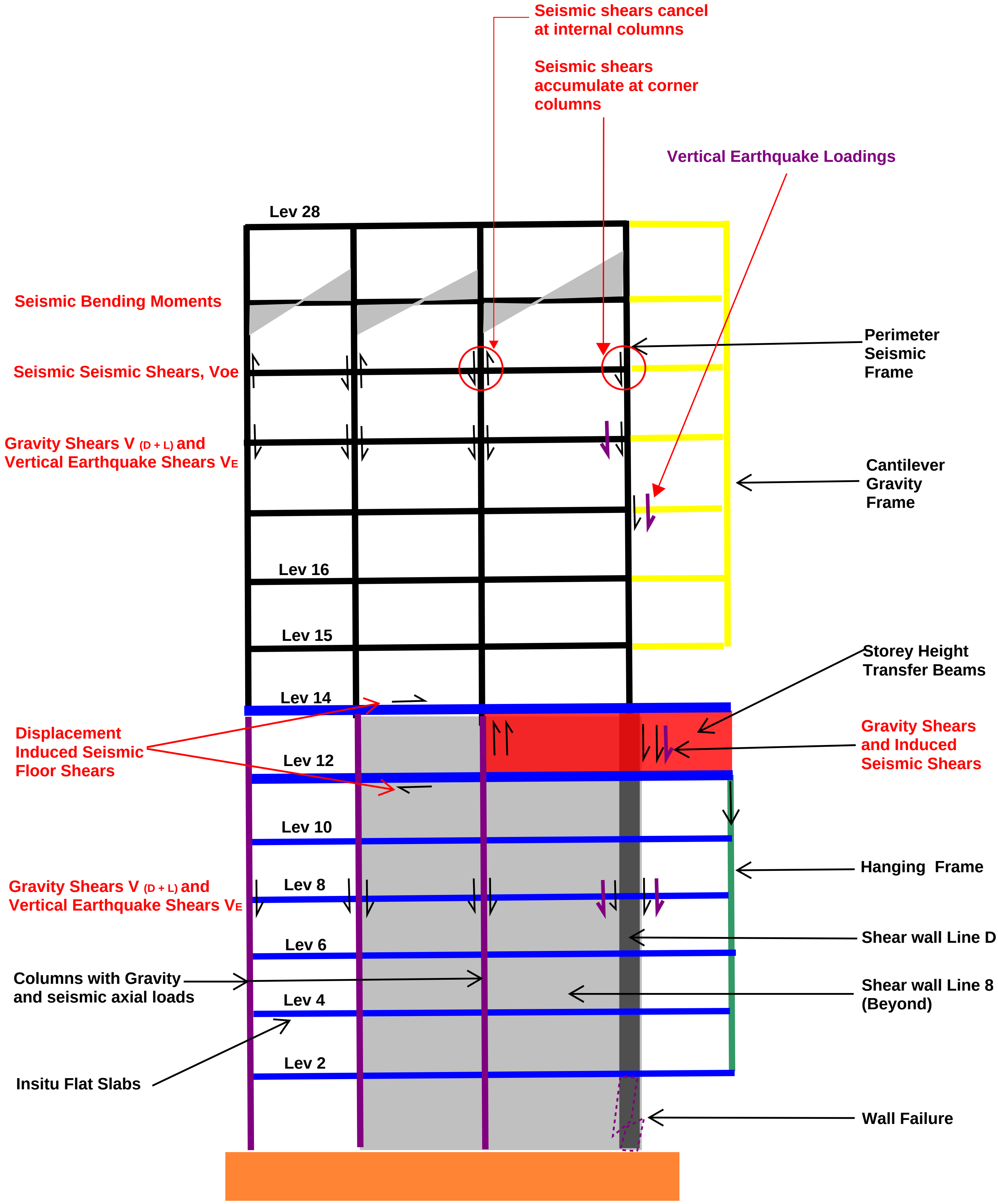


East Side View

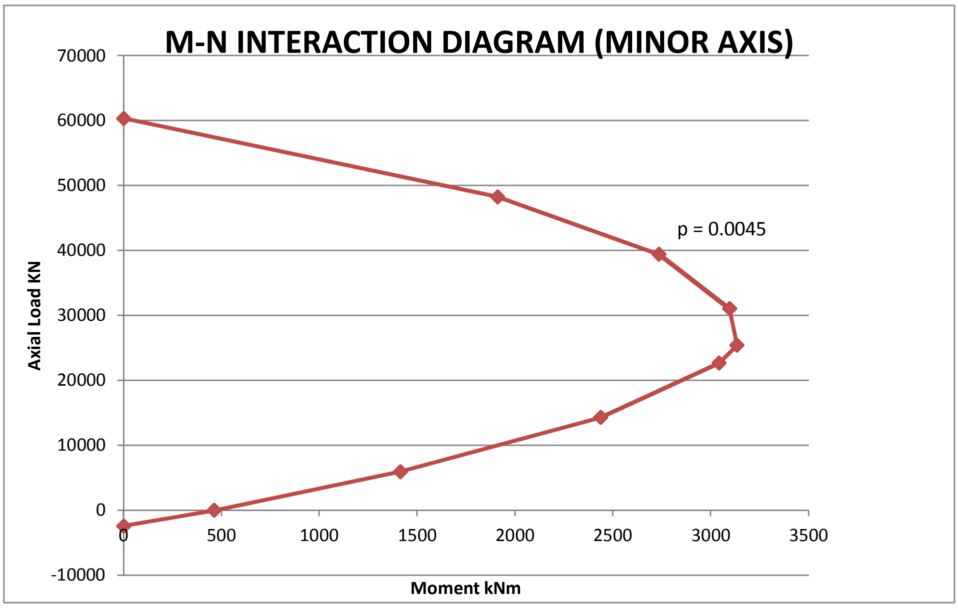


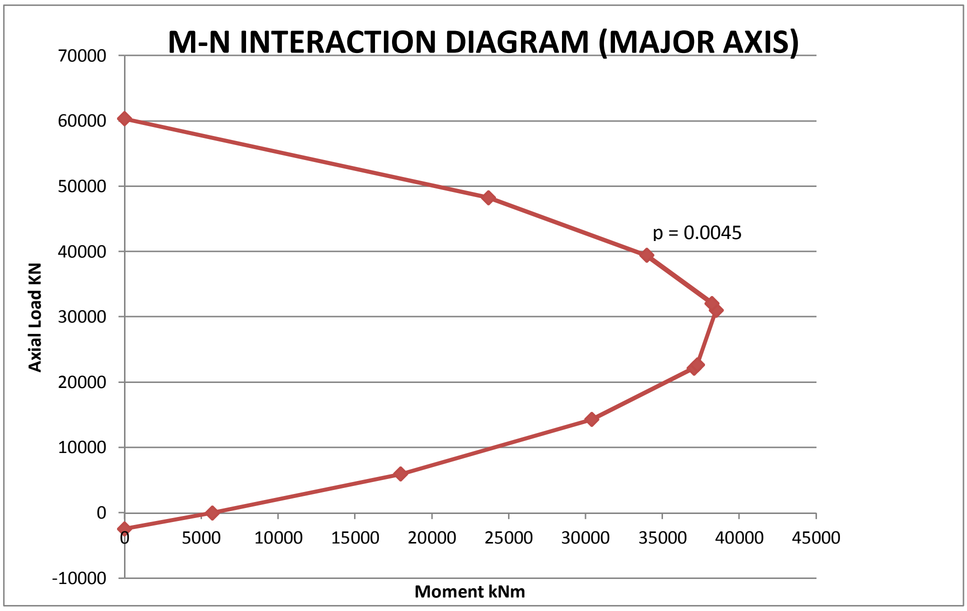
Etabs Screen Shots - 2

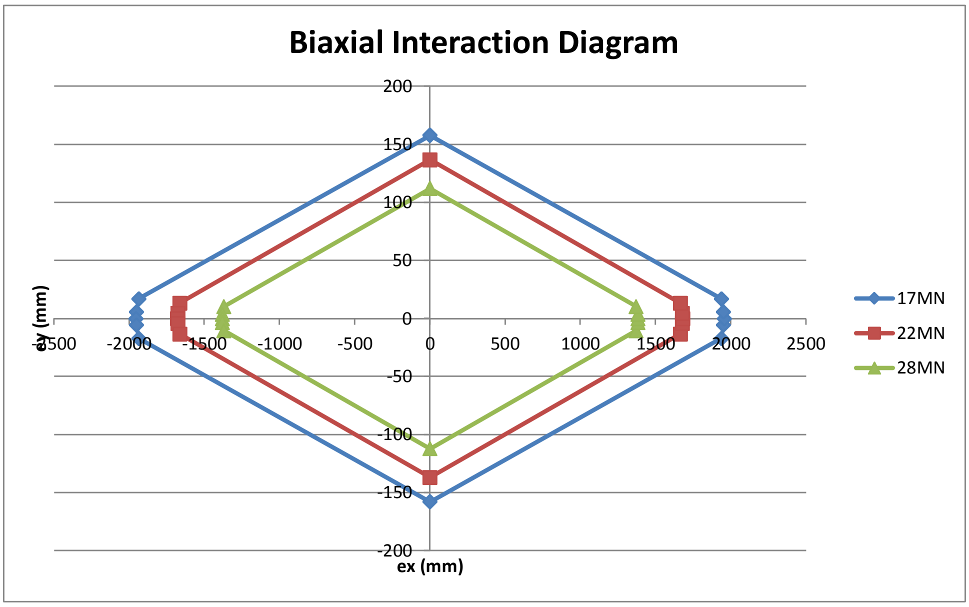
East Side View - Deformed



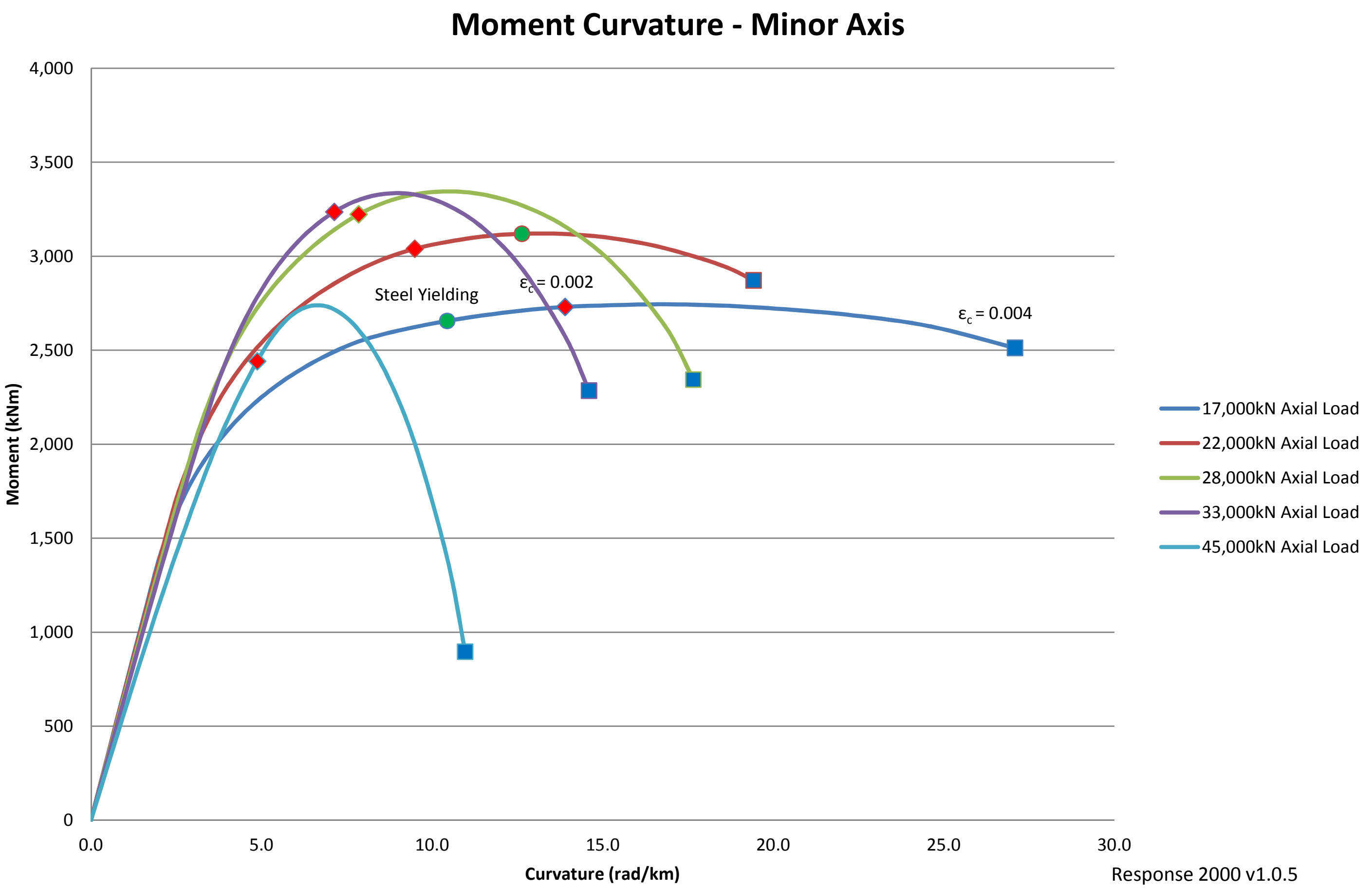
HGC Frame Line 6
Axial Load Actions



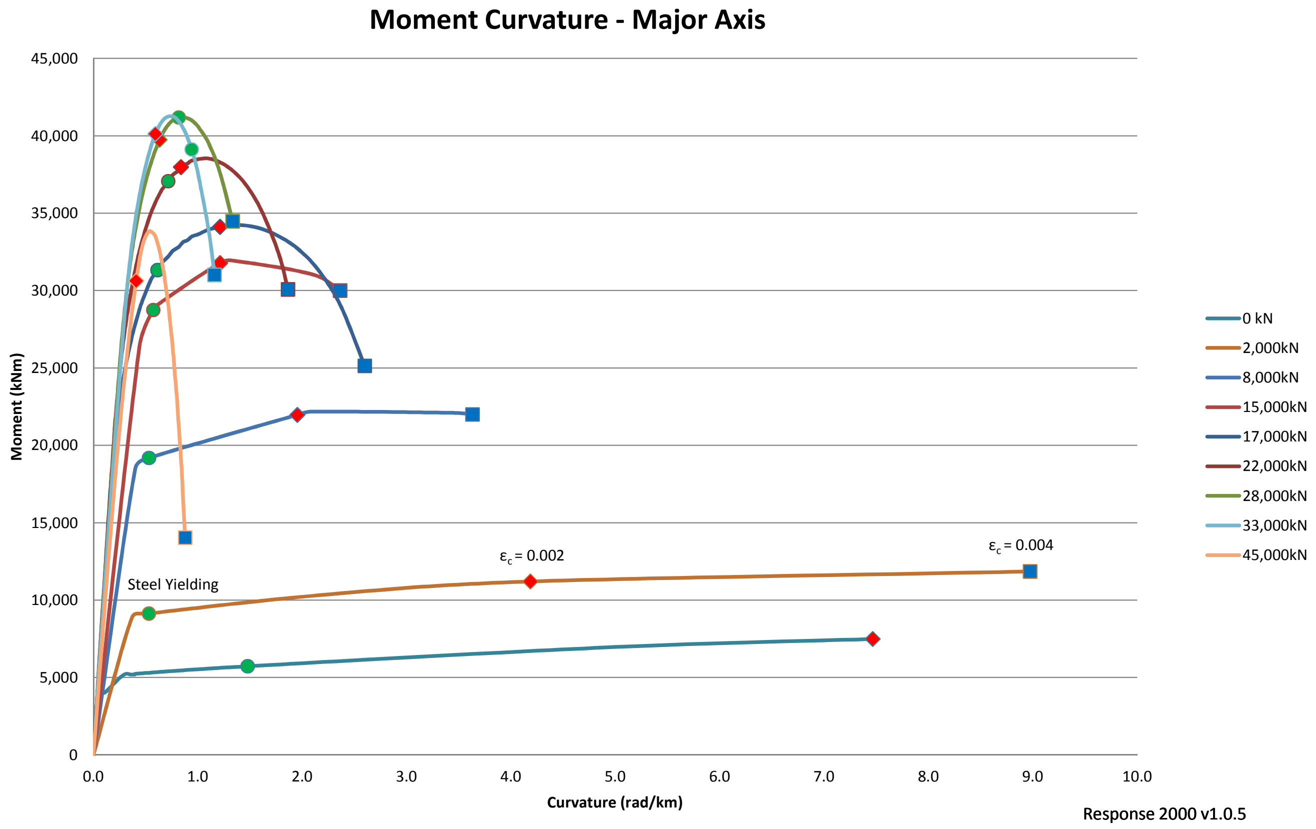


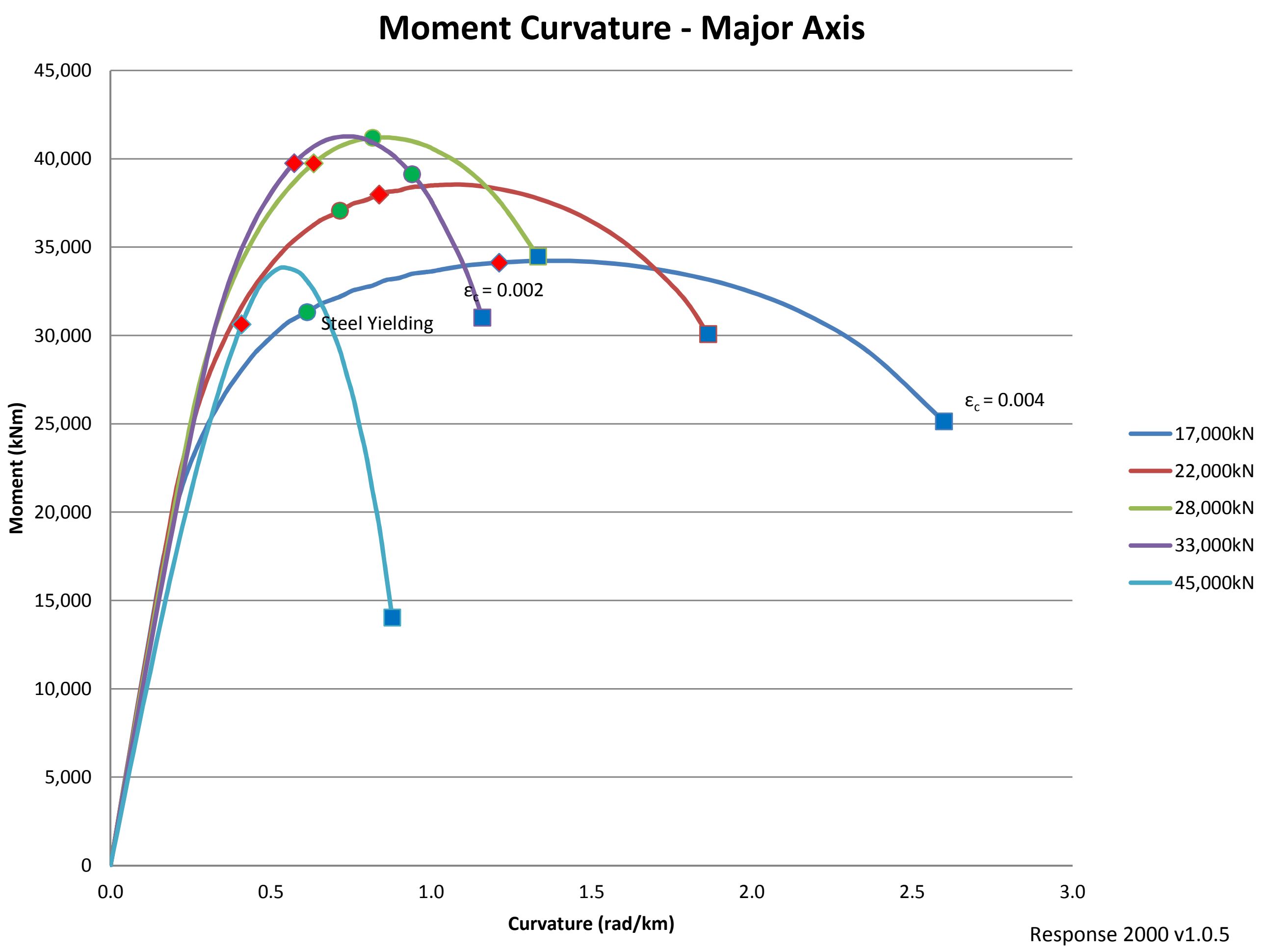


Wall D5-6 - Interaction Diagrams



Wall D5-6 - Moment Curvature Diagram - Minor Axis





Wall D5-6 - Moment Curvature Diagram - Major Axis

Appendix G

Geotechnical Information

Extracts from Original Specifications

5.3 NATURE OF SITE

The site is generally clear, except for an existing MED substation and some fencing.

Bore-holes drilled on the adjacent site to the west have indicated sandy silts, silty clays and some fine sand overlying gravel at approximately 6m below ground level. Test bore logs are included with this Specification, and the following should be noted:

Borehole 1 was in the footpath in Cashel Street, at the west boundary of the present site to be piled. Borehole 2 was close to the service entry of Smith and Browns, at the north-west corner of the present site.

Depths are in feet. The water table, as measured in July 1974, is shown on the borelogs.

The adjacent parking building to the west is founded on Franki-type piles. The National Bank, Reserve Bank and the building currently under construction at the north end (Hereford Street frontage) of the right of way are piled. The United Building Society building on the corner of Cashel and Manchester Streets is thought to be not piled. It is likely that the older buildings in the area are not piled.

5.9 PILES

The piles as detailed are 500mm diameter. Each is to be driven to found firmly in gravels. Refer to the drawing for lengths and reinforcing and load requirements.

Final structural design work has yet to be completed. This may vary the number of piles, and will determine the precise position of each. Ground conditions may vary, and this may in turn cause the length of piles to vary.

Drive casings and form bulbs to achieve the loads shown on the drawing. The Contractor shall provide evidence to substantiate his determination of driving set of casing, and of bulb volume and formation, to achieve the required loads. This information shall be supplied to the Engineer in advance of construction.

CLIENT

SUBSURFACE DATA FROM
Gough Gough & Hamer site.
on footpath Cashel St. - East body line.

Method CDB rotary rig.

Date

Data log by

Plan N°

Job file

BORE DIAM. 9"

BORE N° 1

Cashel St end

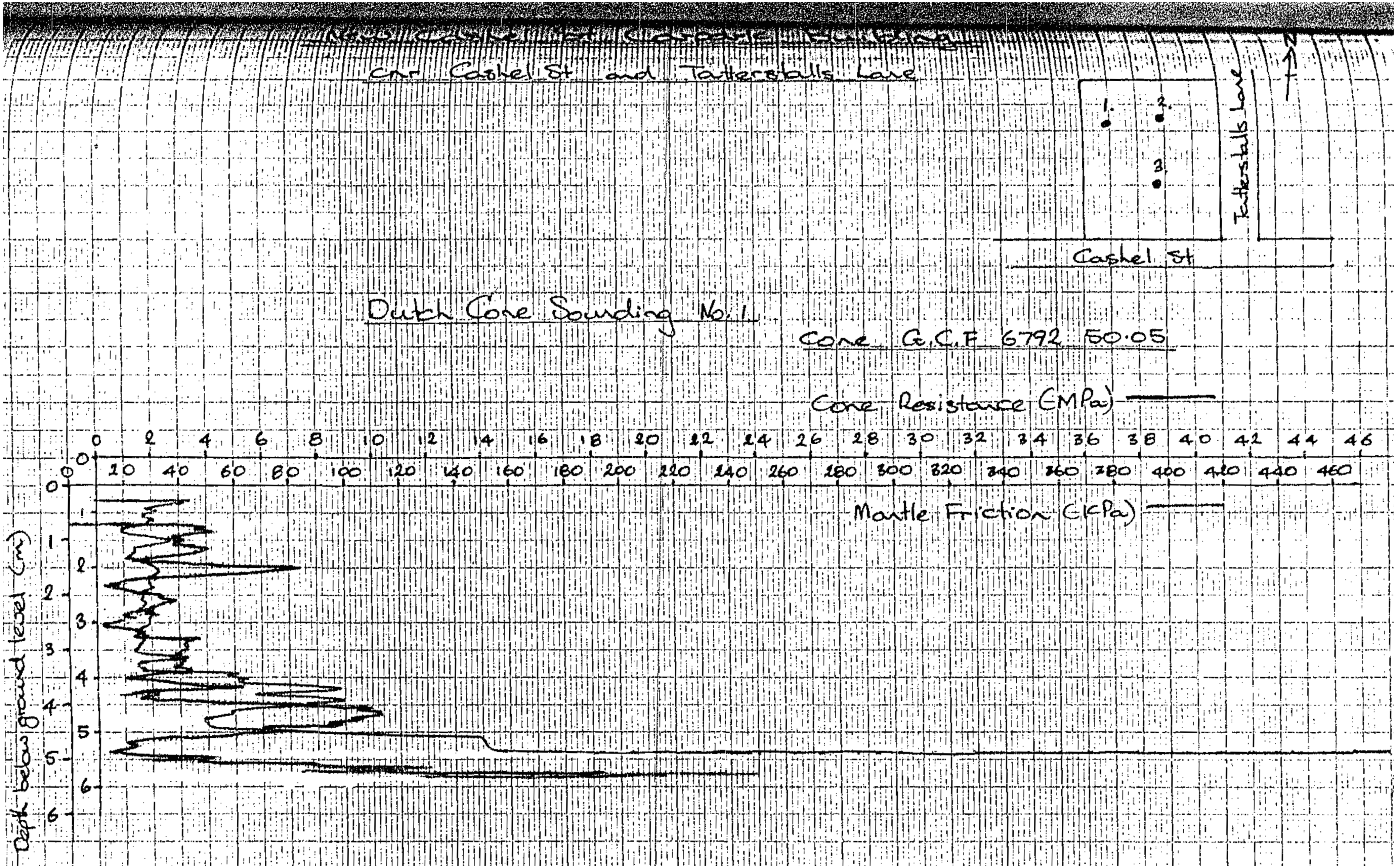
redcd level	depth	strata	Description of soils		Samples & Tests : field/laboratory
	1		12" seal and road metal.		
	2		light brown silty clay		
	3		damp - slow dilatancy.		
	4				
	5				vane test - initial remould
	6		Sample (1) top - firm brown silt mottled / bottom - lt brown fine sandy silt.		5'-8" 1280 670
	7				1510 900 / 1050 600.
	8				
	9				
	10		water level		
	11		sample attempted - did not stay in tube / lt brn fine sandy silt on tube		
	12				
	13		disturbed sample		
	14		lt. brown fine sandy silt saturated.		
	15		blue grey fine silty sand on point of drill.		
	16				
	17				
	18				
5.8	19				
	20		gravel		SPT. 0-9" - 3 blows
	21		S.PT.		9-12" 3 blows / 12-15" 3½ "
	22		grey sandy gravel		15-18 3 "
	23		max size 1" rounded.		approx 12 blows/ft.
	24				
	25				SPT.
	26		SPT.		30 blows / 12"
	27				
	28		gravel		
	29				
9.1	30		SPT.		SPT. 30 blow / 4".

clay | silt | sand | gravel | clayey silt | silty sand | sandy grvl | organic

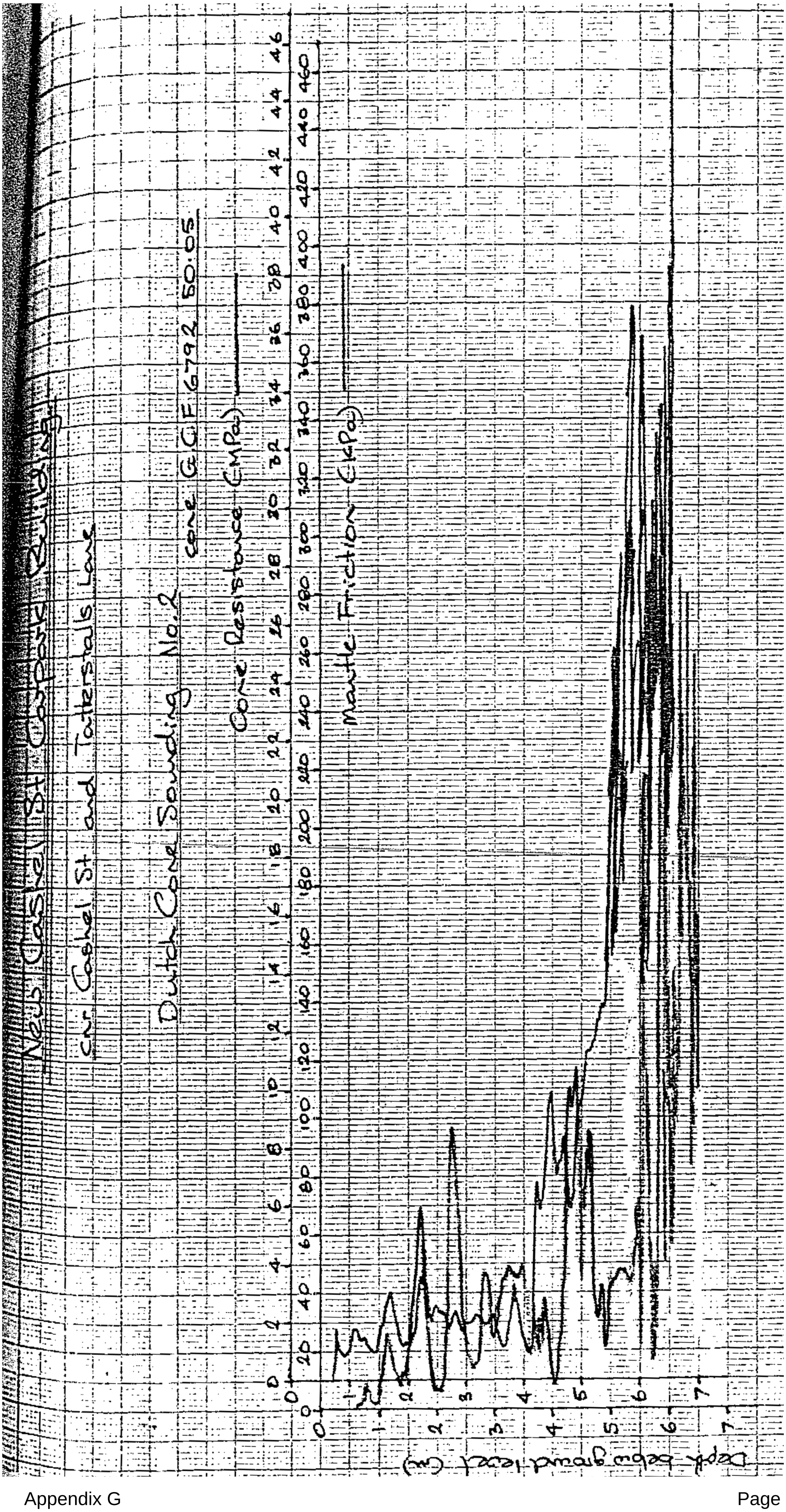
CLIENT	
SUB SURFACE DATA FROM Gough Gough & Hamer site. Inside bldg East side.	
Method C.D.B. Rotary auger	Plan N°
Date / Data log by [illegible]	Job file
	BORE DIAM. 9" / BORE N° 2.

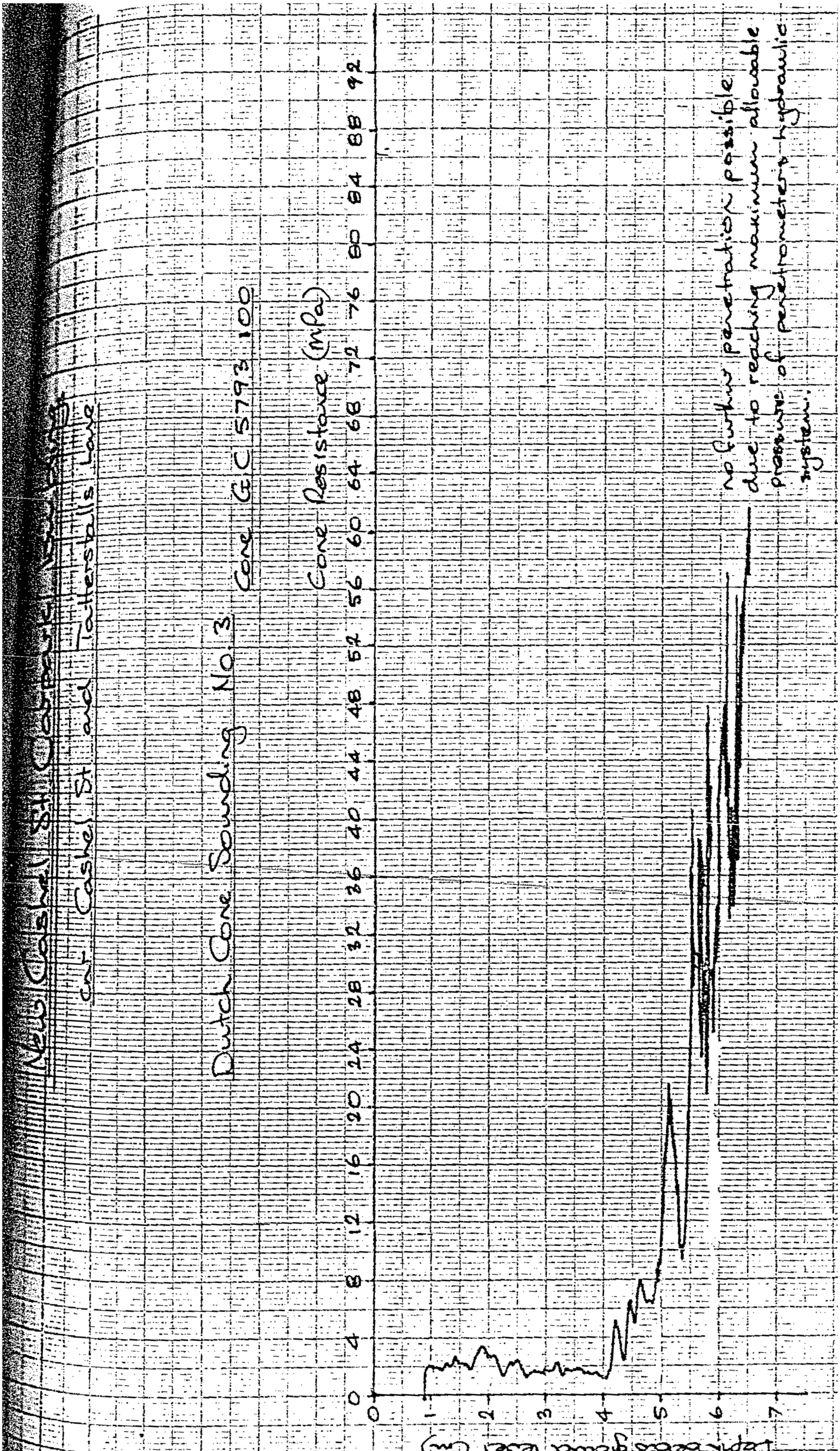
redcd level	depth (0)	strata	Description of soils	Samples & Tests : field/laboratory
	4"		Concrete floor.	
	1		stoney silt filling.	North end
	2			≈ 2 k/ft above WT
	3		lt brown silt - damp	1 k/ft below WT
	4			
	4'6"			
	5		SPT sample (1)	
	6		light brown fine sand	SPT. 6 blows / 12".
	7		∇ Water level	
	8			
	9		fine light brown sand	
	10			
	11		SPT sample (2) fine lt brn sand top. " " bottom lt brn silty clay	SPT 6½ blows / 12"
	12			
	13			
	14			
	15			
	16		SPT sample 5" - grey silty clay (3) 4" finer silty clay lot of organic ~~transition to grey clay with~~ large organic fibres.	SPT 6 blows / 12"
	17			
	18			
	19			
	20		Sand run in when inner rods withdrawn	12" - 4 blows
6.4	21		SPT sample (4) firm grey sand	SPT 6" - 4 blows; 3" 10 blows = 40 blows/ft
	22			
	23		shingle.	
	24			
	25		feel of rig - finer material.	
	26		Coarse sand.	
	27			
	28		Inspection of flights on withdrawal.	
	29		Coarse sand to 36'-0	
9.1	30		36-40 sands scattered gravel	

clay · silt · sand · gravel · clayey silt · silty sand · sandy grvl · organic



cnr Cashel St and Tattersalls Lane
1.
2.
3.
Tattersalls Lane
N
Cashel St
Dutch Cone Sounding No. 1
Cone G.C.F 6792 50.05
Cone Resistance (MPa)
Mantle Friction (kPa)
Depth below ground level (m)


News Cashel St Carpark Building
Cnr Cashel St and Tattersalls Lane
Dutch Cone Sounding No. 2
cone GCF 6792 50.05
Cone Resistance (MPa)
Mantle Friction (kPa)
Depth below ground level (m)


New Cashel St Carpark
cnr Cashel St and Tattersalls Lane
Dutch Cone Sounding No. 3
Cone GC 5793 100
Cone Resistance (MPa)
0 4 8 12 16 20 24 28 32 36 40 44 48 52 56 60 64 68 72 76 80 84 88 92
Depth below ground level (m)
0 1 2 3 4 5 6 7
no further penetration possible due to reaching maximum allowable pressure of penetrometer's hydraulic system.