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Analysis of the CTV Floor Diaphragm Adequacy

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Report to Hyland Consultants Ltd for the Department of Building and Housing investigation into the collapse of the CTV Building during the 22 February 2011 earthquake after-shock.

EXECUTIVE SUMMARY

This document, prepared for Hyland Consultants Ltd for the Department of Building and Housing investigation into the collapse of the CTV Building during the 22 February 2011 earthquake after-shock, presents calculations determining the adequacy of the diaphragm connection between the floors and the north side lift/services core of the CTV building. It also presents calculations estimating the demands on those diaphragms in the February 22nd 2011 earthquake and consideration of the failure modes of these diaphragms and the building on the basis of the calculations.

Assessment of diaphragm in-plane capacities adjacent to the north side lift/services core are based on the tested material properties with the steel decking contribution used where it has participated across the failure plane. Loading demand on the diaphragm is based on the average of the peak ground accelerations recorded from the three closest strong ground motion recording stations to the site factored by 1.6 at all levels. An approximate assignment of diaphragm actions to the north side lift/services core and the south side coupled shear wall has been undertaken, excluding effect of the reinforced masonry infill wall on Line A, although the potential contribution of this wall is discussed separately.

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1. SCOPE OF REPORT

This short report presents findings from an assessment of the actual diaphragm strength of the floors attached to the main lift/services core of the CTV building.

The report commences with key details of the building layout.

This is followed by summary results from the assessment of diaphragm strength and demand, details of which are presented in the Appendices.

A background to these calculations is then given.

This is followed by a short discussion on the potential contribution of the west side block wall to lateral deformation in the north-south direction

The next section presents a postulated collapse mechanism based on the results of this assessment and details from other references, especially (Hyland, 2011) and (Burns, 2011).

The final section contains the references. The calculations are presented in the appendices.

2. BUILDING DETAILS

Figure 1 shows a general plan view of levels 2 to 6, with the north side lift/services core highlighted. Figure 2 shows a close-up of this core and the interface with the floors. This report focuses on the capacity of the floor to lift/services core interface, which is along the left hand side of the plan view in Figure 2, as indicated in that figure.

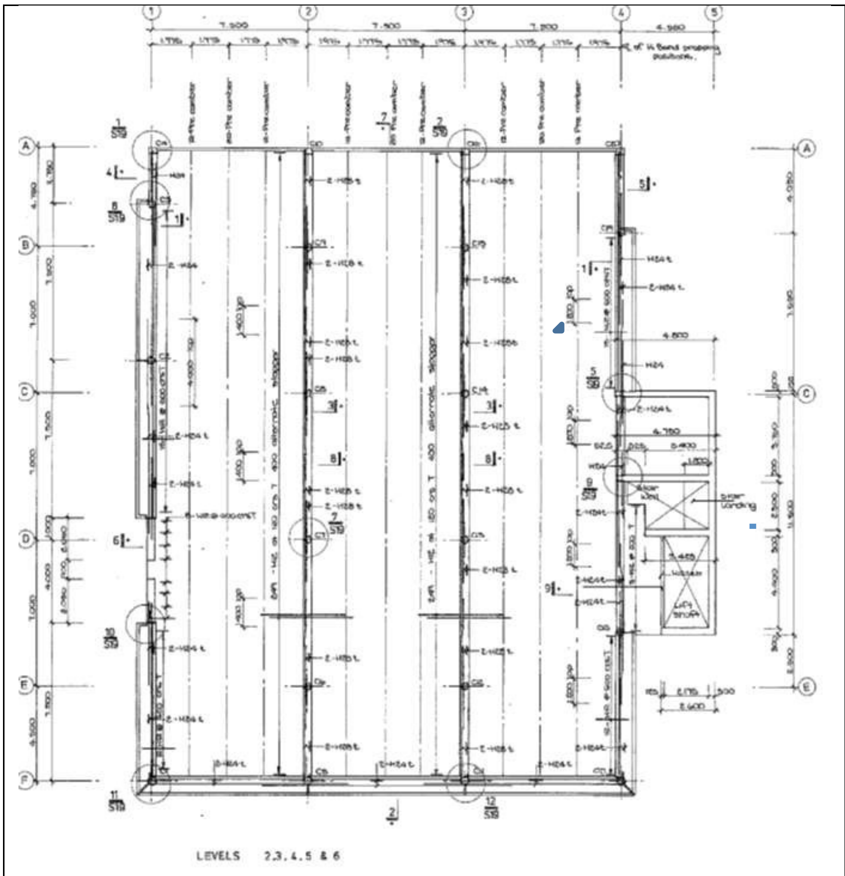

Figure 1 Plan view of typical floors with North side lift/services core highlighted (from (Hyland, 2011))

Interface region on which this report is based

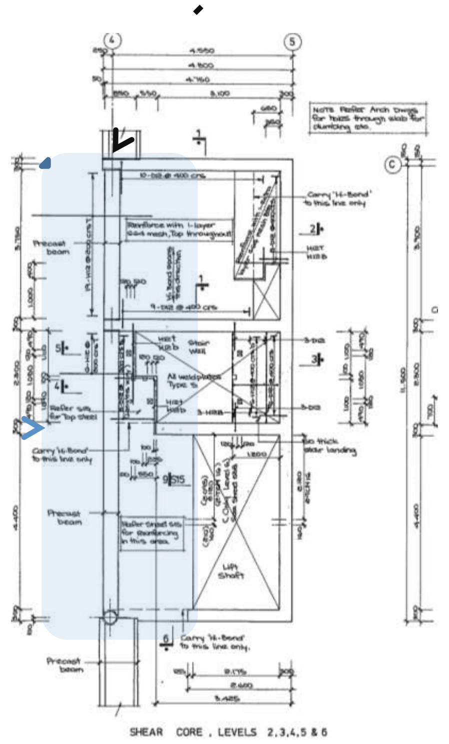

Figure 2 Close-up view of lift/services core (from (Hyland, 2011))

Details of the floor to core connection used in these calculations have been taken from (Hyland, 2011). This includes additional ties between the floor and core installed on levels 4 to 6 which significantly enhance the capacity of the diaphragm connection on these levels compared with those on levels 2 and 3.

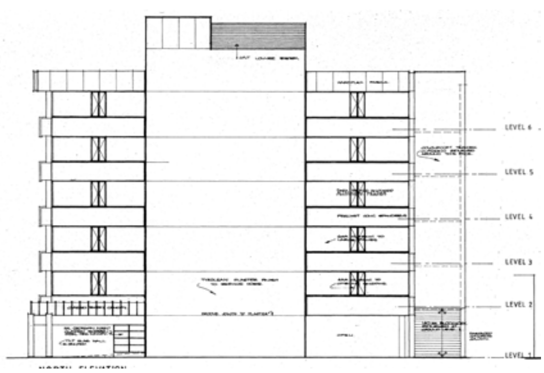

Figure 3 North side elevation

Figure 3shows the north side elevation with the levels indicated. These levels are shown again in Figure 6 with reference to the north side lift/services core.

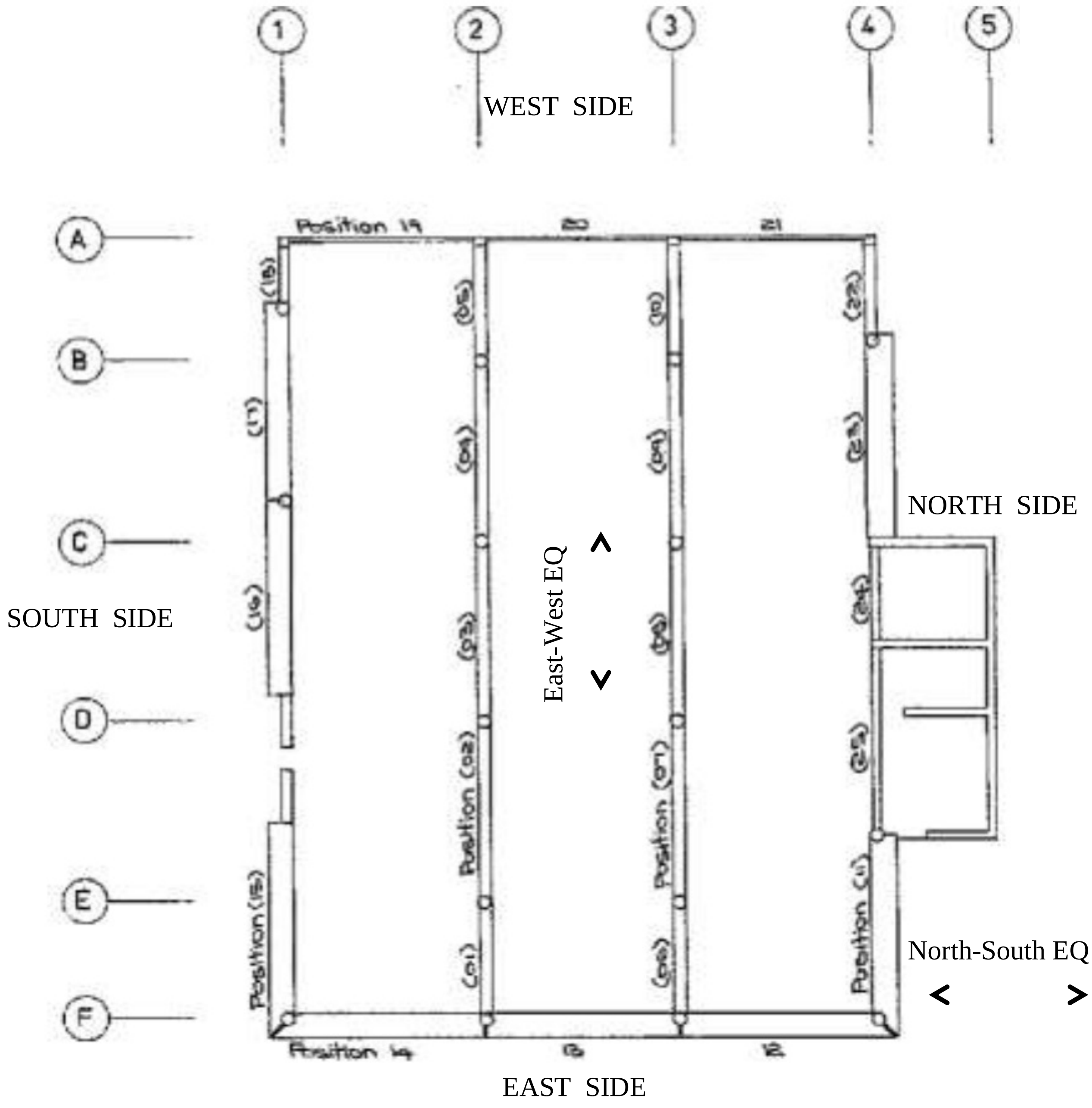

Figure 4 Plan view showing orientation of building and earthquake directions

Figure 4 shows the floor plan with the compass directions and the directions of earthquake action included. The calculations for diaphragm demand and capacity have been undertaken for ground shaking in the north-south direction and for ground shaking in the east –west direction, separately. Concurrent action would make the demand/capacity ratio worse than what is calculated in this report.

3. RESULTS FROM ASSESSMENT

These are given in Tables 1 and 2, relating to the earthquake directions shown in Figure 4. The background to these calculations is given in section 4

Table 1 Diaphragm Demand and Capacity for Earthquake in East-West Direction

Level	Diaphragm Capacity		Diaphragm Demand (Note 1)		Demand/Capacity Ratio (Note 2)	
	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment	Shear
2 and 3	4,967	980	15,630	3,941	3.15	4.02
4 to 6	12,523	5,286	15,630	3,941	1.25	0.75

Notes to Tables 1 and 2:

1 it is likely that the diaphragm demand was slightly lower on level 2 than on level 3

2 a value > 1.0 indicates failure is likely, given the capacity is based on actual strengths of components along the failure plane and demand based on expected peak demand

Table 2 Diaphragm Demand and Capacity for Earthquake in North-South Direction

Level	Diaphragm Capacity	Diaphragm Demand (Note 2)	Demand/Capacity Ratio (Note 2)
	Tension (kN)	Tension (kN)	Tension
2 and 3	2,912	2,859	0.98
4 to 6	4,042	2,859	0.71

Notes to Tables 1 and 2:

1 it is likely that the diaphragm demand was slightly lower on level 2 than on level 3

2 a value > 1.0 indicates failure is likely, given the capacity is based on actual strengths of components along the failure plane and demand based on expected peak demand

Floor accelerations obtained from numerical integration time-history analyses have shown that the building superstructure magnifies the peak ground accelerations by a factor of approx 1.5 to 1.75 and that this magnification is near constant up the height of the structure, with a slight reduction on the lowest floor above ground (Uma et al., 2009). This is the background to note 1. A factor of 1.6 has been used in this assessment, based on (Clifton, 2011) and as detailed in section 4.1.

As described in section 4, especially section 4.2, the diaphragm capacity has been assessed using the tested material properties of the relevant components in the failure planes and the appropriate material property relating to the failure plane. This includes contribution from the steel decking as appropriate where it crosses the failure plane, taking into account the anchorage conditions relating to that decking.

The diaphragm demand is based on an estimate of the actual loading that would have been on a floor at the time of the earthquake, comprising the self weight of the floor and frame, with an additional 1.0 kPa for partitions, floor coverings, ceiling and services and a live load of 0.5 kPa, which is that used in offices for floor vibration checks and represents the minimum likely live load on the floor at any instant and on the average of the peak ground accelerations recorded from the nearest three strong motion recording stations to the CTV site, factored by 1.6 at all levels.

Concurrent action has not been considered acting on the diaphragm interface with the north side lift/services core. This is unconservative, however the clear result from the uniaxial considerations make it unnecessary to consider concurrent actions and there is a high degree of uncertainty in how they would have combined in practice.

Adding to this uncertainty and also not included in this assessment has been the influence of a blockwork wall along the west side (along gridline A; see Figure 4), which was built up to level 4 and only partially isolated from the concrete gravity frame along that gridline. The extent of potential interaction is discussed qualitatively in section 5 Interaction of that wall against the movement of the building in the north-south direction would have shifted the centre of rigidity closer to the west side, increasing the tension demand on the diaphragm interface with the north side lift/services core, either due to tension action from north-south direction ground movement or due to moment-induced tension action from east-west ground movement. Either would have been detrimental, effectively increasing the demand/capacity ratio beyond that shown in Table 1 and Table 2.

The results show that failure is most likely to have occurred due to diaphragm failure at level 3, where the demand/capacity ratio is at its highest. It is sufficiently high that failure could have commenced at level 3 and possibly level 2 in the September 4 2010 earthquake which was approx 1/3 the capacity with a PGA in the CBD region of around 0.18g. This is explored further in section 6

4. BACKGROUND TO CALCULATIONS

The calculations have been based on the following:

1. The failure planes in the floor diaphragms as advised by Clark Hyland, identified in his report (Hyland, 2011) and in the police photographic records published in (Burns, 2011). As seen in Figure 6, the failure plane on levels 6 and 5 is within the floor slab in front of the North side lift/services core while that on levels 3 and 2 is at the front of the core along the west portion only. (See Appendices A/B and C for more details)
2. The actual strengths of the materials crossing the assessed failure planes, taken principally from (Hyland, 2011) and including the anchorage offered to each component
3. The diaphragm demand is based on the method given in section 9.4 of the seismic resisting system steel design notes from the CIVIL 714 course (Clifton, 2011), modified to use the actual peak ground acceleration from the nearest of the three strong motion recording stations to the site.
4. Background to the assessment of diaphragm demand is given in section 4.1 and the assessment of diaphragm capacity in section 4.2.

4.1 Assessment of Diaphragm Demand

The expression for the design diaphragm shear force at level i is given by equation 9.4 from (Clifton, 2011):

$$V_{dia,i} = C_{dia} W_{t,dia,i}$$

where:

$V_{dia,i}$ = the design diaphragm shear force for the floor at level i

$C_{dia} = C_{h0\,modal} Z R_u S_p C_{Hi,diaphragm}$ (9.5 of (Clifton, 2011))

$C_{h0\,modal}$ = the spectral shape factor for T=0 seconds for the modal response spectrum from Table 3.1 of NZS 1170.5

Z = the zone factor

R_u = the return period factor from Clause 3.1.5

S_p = the structural performance factor for the category of structural system from NS 3404 Clause 12.2.2.1(b)

$C_{Hi,diaphragm}$= 1.6

In equation 9.5, the combination of $C_{h0\,modal} Z R_u S_p$ is used to give the design peak ground acceleration, which is then magnified by the building response through the variable $C_{Hi,diaphragm}$. In this evaluation, that combination is replaced by the actual peak ground acceleration recorded. This has been taken from the strong motion recordings of the closest three strong motion recording stations to the CTV building (Carr, 2011), all of which are within approx 1 km. These are the recording stations REHS, CHHC and CCCC respectively. The average value of PGA recorded for the north-south (NS) direction and for the east-west (EW) direction from these three stations is 0.37g and 0.52g respectively. The calculation of $V_{dia,i}$ is given in Appendix D; for which an estimated PGA of 0.5g in both directions was originally used and was replaced by the above figures in order to give the more accurate determination of diaphragm demand presented in this report.

4.2 Assessment of Diaphragm Capacity

For the north-south direction, the north side lift/services core is the principal lateral load-resisting system. It is located reasonably centrally along the north side and so the building has minimal torsional eccentricity for north-south earthquake excitation (if the contribution from the west side block wall is ignored). In that direction, therefore, the calculations have focussed on the diaphragm action in tension and the resistance of the interface between the floor system and the north side lift/services core to resist that tension. This resistance is made up of the tensile resistance of components transverse to the assessed failure plane and the shear resistance of components parallel to the assessed failure plane.

For levels 4 to 6, two failure planes are assessed. The first is shown on page A1 of Appendix A and is based on the floor slab visible at level 6 in Figure 6 following the collapse and also similar in extent at levels 5 and 4, as advised by Hyland. For this failure plane, the full contribution of the decking in tension is taken, as the decking was continuous across the failure plane and observed to be fractured in the remnant of floor remaining with the north side lift/services core standing after the collapse. The second failure plane is shown on page B1 of Appendix B and involves failure of the interface between the slab and the north side lift/services core walls. Along this failure plane, the decking is considered to develop its full tensile contribution for the length of interface between the two north core walls that intersect the building slab along gridline 4. This may be overly optimistic in terms of the decking contribution. However, this interface does not provide contribution further east than that region, due to lack of interconnection of this region of the slab with the north side lift/services core wall on gridline D and the eastern most wall between gridlines D and E. For that region, the interconnection on levels 4 to 6 is via the 150x150x10 EA’s bolted to the walls and the floor slab. No other connection to those walls is considered dependable across that interface.

The calculations show that the latter interface is critical; see page E1 of Appendix E.

For levels 2 to 3 in the north-south direction, the effective width of slab contributing to tension capacity is very much smaller, as shown on page C1 of Appendix C. The tension capacity is assessed from failure across this interface. The construction drawings and failure photos show the mesh fractured across this failure plane and the decking appeared to have also fractured; thus the tension contribution of both were included in the failure plane capacity. There is no effective interconnection between the floor slab and the north side lift/services core wall on gridline D or between the floor slab and the wall between gridlines D and E, as the interconnecting angles are not present at those levels.

For the east-west direction, there are two seismic-resisting systems. These are the north side lift/services core with the 11.5m long wall running east-west along the north side and two coupled shear walls on the south side, around gridline D. See Figure 4. After the collapse it was observed that the south side coupled walls and coupling beams had undergone significant inelastic demand, as evidenced by fan cracking and testing on reinforcing bar from the east end of the wall showing approx 3% strain hardening compared, while the north side wall was effectively undamaged. There was also some cracking in the south side wall coupling beams. This combination of system layout and damage to the weaker walls only created a significant torsional imbalance in this direction, with this determined in Appendix D, section D2 (2). For east-west acting ground shaking, the building will tend to pivot about the centre of rigidity which is very close to the 11.5m long north side wall. This puts significant moment and shear across the interface between the floors and the north side lift/services core. The design moment and shear is assessed in Appendix D3.

The moment and shear capacity is calculated in Appendix A and B respectively for levels 4 to 6 and in Appendix C for levels 3 and 2. For levels 4 to 6, two failure planes are assessed. The first is shown on page A3 of Appendix A and is based on the floor slab visible at level 6 in Figure 6 following the collapse and also similar in extent at levels 5 and 4, as advised by Hyland. For this failure plane, the full contribution of the decking in tension is taken, as the decking was continuous across the failure plane and observed to be fractured in the remnant of floor remaining with the north side lift/services core standing after the collapse. The second failure plane is shown on page B1 of Appendix B and involves failure of the interface between the slab and the north side lift/services core walls. Along this failure plane, the decking is considered to develop its full tensile contribution for the length of interface between the two north core walls that intersect the building slab along gridline 4. This may be overly optimistic in terms of the decking contribution. However, this interface does not provide contribution further east than that region, due to lack of interconnection of this region of the slab with the north side lift/services core wall on gridline D and the eastern most wall between gridlines D and E. For that region, the interconnection on levels 4 to 6 is via the 150x150x10 EA’s bolted to the walls and the floor slab. No other connection to those walls is considered dependable across that interface..

The calculations show that the latter interface is critical; see page E1 of Appendix E.

For levels 2 to 3 in the east-west direction, the effective width of slab contributing to tension capacity due to bending moment or able to carry diaphragm shear is very much smaller, as shown on page C1 of Appendix C. The construction drawings and failure photos show the mesh fractured across this failure plane and the decking appeared to have also fractured; thus the tension contribution of both were included in the failure plane moment capacity. There is no effective interconnection in tension between the floor slab and the north side lift/services core wall on gridline D or between the floor slab and the wall between gridlines D and E as the interconnecting angles are not present at those levels. This makes direction of moment that puts the east side of the interface into tension critical, which is why the assessed capacity calculated in Appendix C is based on moment acting in this direction; see the figure on page C4.

5. POTENTIAL CONTRIBUTION OF WEST SIDE BLOCK WALL TO NORTH SOUTH LATERAL DEFLECTION

Drawing S17 of the construction drawings (ACRL, 1986), shows the infill blockwork wall built along the West side between the columns and floor beams of levels 1, 2 and 3 over the three bays from gridline 1 to 4 inclusive.

This wall is separated from the columns by a 25mm specified gap which is filled with "Thioflex 600 sealant and PTF backing strip and asbestos rope" in order to provide a seismic separation between wall and frame and yet allow a 3 hour Fire Resistance Rating (FRR) to be developed. The wall is cantilevered off the base at each level and held laterally at the top with greased bars set into inset fasteners in the base of the concrete beam above with the top layer of blockwork then presumably filled to allow fire resistance to be developed. Each bay of wall is also split into 3 with vertical joints intended to facilitate separation from the structural frame in the north-south direction.

The wall comprises three bays each 7.5m long. There are no corresponding walls on the east side of the building or internally, meaning that if effective separation is not achieved, the torsional impact of the wall on the building's response is potentially significant and the direction of torsional rotation is such to develop a moment from north-south earthquake action at the interface of the diaphragm to the north side lift/services core that will be additive to the moment from east-west earthquake action and raise the design moment demand considerably above that assessed in Table 1.

Effective separation may not have been achieved in practice for one or more of the following reasons:

1. The interstorey deflection demand during the February 22nd earthquake would have been at least 1% of storey height or 37mm and probably higher. That figure is based on the known response of a structure which have performed well (the HSBC Tower with 0.75% drift) and assessed drifts of structures that have suffered greater damage, such as the Westpac bank building which exhibited a total drift of 2.6% (1.5m lateral deflection - assessed by the writer during the earthquake – over 14 storeys).
2. The effective separation of the wall joints joints in compression would have been less than 10mm due to the combination of the sealant and asbestos rope fitted into the 25mm specified gap, provided they were properly installed which is expected to be the case given the high FRR. Any tendency to mortar the joints for fire resistance (as has been reported in some buildings) would have further reduced that gap
3. Figure 5a shows a 5 storey EBF building in Riccarton. This building underwent minor tensile movement across a seismic separation, causing tension across the sealed join between two precast panel units. Figure 5b shows that the sealant was sufficiently strong in tension not to fail but to cause failure in one concrete unit adjacent to the sealed joint. Sealants have improved in quality since the CTV building was built in 1988 and the outer layer of sealant would have been exposed to the weather since that time, however it is possible that the resistance of the sealant would have prevented effective opening of the tension joint
4. The ability of the slip plane between the top of the wall and the underside of the beam above to allow relative slip must also be questioned, especially if this plane was mortar filled to provide fire resistance

These factors mean that the effective separation of that west block wall from the frame was probably no more than 10mm which would have been inadequate during the 22 February earthquake and also potentially during the 26 December 2010 and 4 September 2010 earthquakes.

The torsional influence of that inadequate separation may have contributed to premature failure of the floor diaphragm at level 3 as discussed further in section 6.

Figure 5 Shopping mall on Dilworth St and Clarence St, Christchurch [Photos by G. MacRae]; (a) View from the East, (b) Fracture of Precast Spandrel Beams on South side

6. POSTULATED COLLAPSE MECHANISM FROM DIAPHRAGM FAILURE

The postulated collapse mechanism from diaphragm failure is that this occurred first on level 3. This failure occurred at the interface between the floor slab and the north side lift/services core, forcing the gravity system columns at that level into double curvature due to the floor diaphragm remaining connected to the lift/services core at levels 6 and 5 and maybe some of level 4. Because of the layout of that interface, the critical direction of moment is that causing tension in the east end of the interface. The gravity columns could not withstand this double curvature and failed, initiating a collapse at one or both of level 2/level 3 within the region of floor approximately bounded by grids F, 2, D? and 3. This failure pulled the support out from under the upper floors, causing them to fall vertically downwards into the centre of the building, finally ripping the floor at level 6 along the failure plane shown in Figure 6.

Note that this may have been one of several collapse mechanisms operating simultaneously or in sequence during the February 22nd 2011 earthquake.

Supporting evidence for this postulated collapse mechanism comes from the following:

1. The strongest indicator is the failure plane at level 6 and to a less clear extent at levels 5 and 4. The failure plane in the slab at level 6 is clear in Figure 6 and is shown dimensionally in Appendix A pages A1 and A3. This involves a front face of the break some 1.2m into the floor slab from the precast beam along the front of the north side lift/services core. Calculation of the design capacity of this floor slab in Appendix A and B shows that the capacity of this interface is much greater than that between the edge of the slab and the lift/services core walls, shown in Appendix B pages B1 and B2 under floor diaphragm actions, especially moment from east-west ground shaking. Therefore, had the failure been by slab pull-out of the lift/services core from the top under diaphragm action, the failure plane would have been in a different location to that observed.
2. The failure plane observed at level 6 can be explained by a vertical downwards failure of the main body of the floor initially, which broke off the floor slab at level 6 at the position shown in Figure 6. Similarly with level 5 and probably with level 4, however it is getting harder to see in that photo.
3. The edge of the decking at the failure plane on level 6 is turned downwards (see Figure 20 of (Hyland, 2011)); similarly with level 5 and level 4. The rebar crossing the failure plane is similarly turned down. This is most pronounced at level 6, indicating the general downwards movement of the floor slab on the interior side of the failure plane at that level and not showing indication of a lateral tearing failure.
4. The severity of the collapse. If the middle levels fell down before the top, pulling the top down into the collapsed middle, the collapse would be more compact and complete internally, minimising the chance of survivors. This is what was observed, compared with the collapse of the PGC building which is more laterally displaced.
5. The final shape of the collapsed remains of the building is more consistent with a bottom up collapse rather than a top down vertical or partially sidesway collapse
6. The receptionist, who escaped from the ground floor near the north-east corner, reported that the building collapsed very rapidly behind her, which indicates an internal collapse and one starting close to the ceiling to level 1

As stated in section 3, if these calculated assessments are accurate, it may be possible to test that failure of the floor diaphragm to the north side lift/services core began at levels 2 or 3, probably at level 3. This is because the calculations show that the ratio of demand/capacity should have been greater than 1.0 in the September 4 2010 earthquake and also the 26 December 2010 earthquake, leading to partial failure of the diaphragm at level 3 and possibly at level 2. Any contribution to torsional imbalance from the west side block wall would have exacerbated the tendency for diaphragm failure at level 3 from those two earlier events.

If this is the case, then the building may have felt softer and exhibited minor movement not previously recorded, especially within the eastern half of the floor and especially the region bounded by grids F, 4, D or C and 1, but only or most noticeably on levels 2 and 3 and maybe 4. The effect would have got less going towards levels 6 and 7. This would have been caused by the failure of the interface details as shown in Figure B herein.

It has been reported from survivors who worked in the building that it had “softened considerably” following the September and December 2010 earthquakes. It may be possible to ascertain where in the building this influence was most pronounced and if that is towards the east side of levels 2 and 3 and not at the upper levels it would support the above postulated collapse mechanism.

7. REFERENCES

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8. APPENDIX: CALCULATIONS

The following calculations comprise:

Section A: Approximate capacity of diaphragm connection to North side lift/services core levels 4, 5 and 6, based on failure plane at edge of observed slab overhang following failure

Section B: Approximate capacity of diaphragm connection to North side lift/services core levels 4, 5 and 6, based on failure plane at the line of attachment of the floor system to the core walls

Section C: Approximate capacity of diaphragm connection to North side lift/services core levels 2 and 3, based on failure plane at edge of observed slab following failure

Section D: Estimate of floor diaphragm actions at the face of the North side lift/services core (with updates to the original hand calculations as noted)

Section E: Estimate of adequacy of diaphragm

Figure 6 View of north side lift/services core following collapse identifying the levels (photo from (Burns, 2011))

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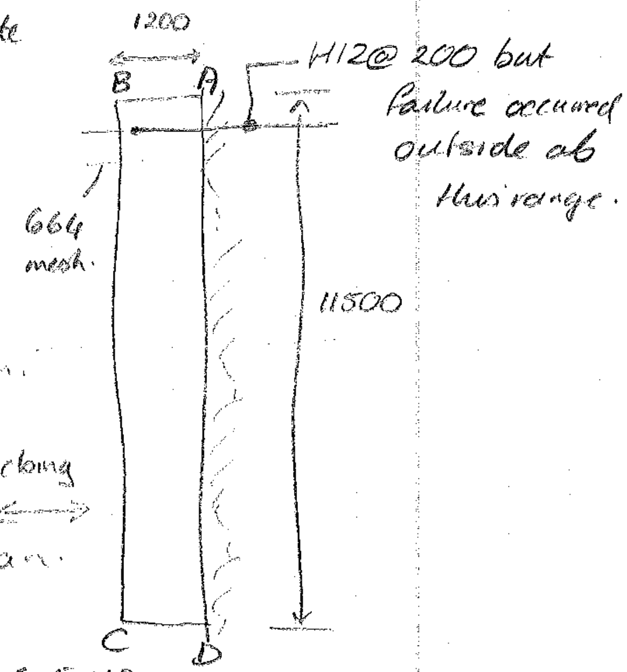
A Levels 6, 5, 4. Approximate Capacity.



decking span.

664 mesh
— 6mm ϕ at 150 centres
— 188 mm^2/m width

Reinforcing steel props
664 Mesh → R_{eL} = 615 MPa.
R_m = 665 MPa.

Concrete slab strength f_c', av = 24.7 MPa.

Concrete slab thickness = 200 mm.
$t = t_o - h_e/2 = 200 - 55/2 = 173$ mm.

Decking – 0.75mm BMT f_y & f_u = 617 MPa (page 111) average

Decking cross section A_{deck} = 1058 mm^2/m.

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A.1 Calculation of tension capacity.

Lines CD, BA in shear

– assume no capacity as concrete cracked in shear.

– $V_{CD,conc}$ $(200-85) \times 1200 \times 0.17 \times 10^{-3}$ = 29 kN.

– $V_{BA,conc}$ = 29 kN.

Lines CD, BA in shear.

Mesh crossing this crack.

186 mm²/m R_m = 665 MPa.

– $V_{CD,mesh}$ = $0.6 \times 665 \times 186 \times 1.2 \times 10^{-3}$ = 89 kN.

Line BC in tension.

– tension resisted by 664 mesh and decking

– ignore effect of concrete

$N_{BC,mesh}$ = $11.5 \times 665 \times 186 \times 10^{-3}$ = 1422 kN.

$N_{BC,deck}$ = $11.5 \times 1058 \times 617 \times 10^{-3}$ = 7507 kN.

Total tension capacity across rupture

N = $7507 + 1422 + 2 \times 29 + 2 \times 89$ = 9165 kN.

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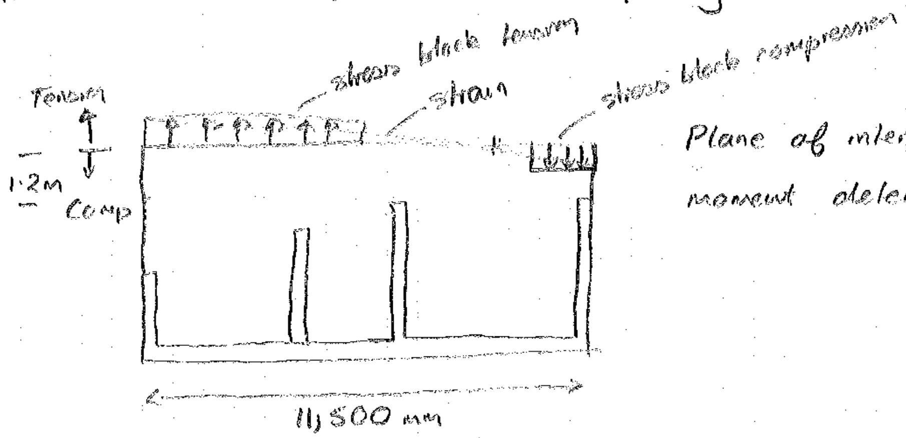
A·2 Calculation of Moment Capacity

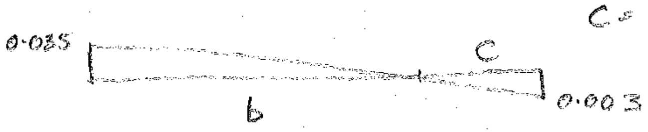


Plane of interest for moment determination

Decking elongation at fracture ≈ 3·5 %

Concrete compression at failure = 0·003

Assume PNA from compression face proportional to strains for first estimate



c = depth compression face to PNA

$\frac{b}{c} = \frac{0{\cdot}035}{0{\cdot}003} = 11{\cdot}67 \Rightarrow b = 11{\cdot}67\,c$

$\Rightarrow c + 11{\cdot}67\,c = 11{\cdot}50 \Rightarrow c = 0{\cdot}91$ m.

$R_c = h_e\, 0{\cdot}85 f_c'\, c$

$= 178 \times 0{\cdot}85 \times 24{\cdot}7 \times 0{\cdot}91 = 3305$ kN

Length of tension yielding

ε_y (deck + reo – similar) $= \frac{620}{205{,}000} = 3{\cdot}02 \times 10^{-3}$

$L_{y,\text{tens}} \approx 11{\cdot}67 \times 0{\cdot}91 - \frac{0{\cdot}91}{2} \times \frac{3{\cdot}02 \times 10^{-3}}{3 \times 10^{-3}} = 9{\cdot}70$ m.

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$R_t = (665 \times 186 + 1058 \times 617) \times 10^{-3} \,/\mathrm{m}$ = 776 kN/m

$R_{t,\,total} \approx 776 \times 9.70$ = 7531 kN

⇒ tension is stronger, need to recalculate based on limiting compression strain.

Take $c = 1.75$ m

⇒ $R_c = 3305 \times 1.75 / 0.91$ = 6355 kN

$L_{y,\,tens} \approx 11.50 - c - \overbrace{\frac{1.01c}{2}}^{\text{elastic region}}$ = 8.87 m

$R_t = 776 \times 8.87$ = 6882 kN

Take $c = 1.85$ m

$R_c = 3305 \times 1.85 / 0.91$ = 6719 kN

$L_{y,\,tens} = 11.5 - 1.85 - 1.01 \times 1.85/2$ = 8.72 m

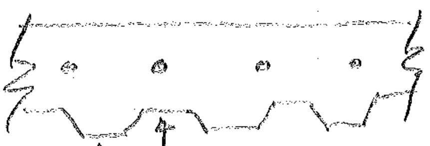
$R_t = 776 \times 8.72$ = 6763 kN

– close enough ✓ adopt.

Moments about centroid of compression force

$M = R_t \left(11.50 - \frac{8.72}{2} - 0.85 \times \frac{1.85}{2} \right)$ = 42,973 kNm

A.3 Shear capacity horizontal direction BC



Shear contribution deck horizontal lengths

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$$V_c = 0.17\sqrt{24.7}\, h_c\, L$$

$$= 0.17\sqrt{24.7} \times 173 \times 11{,}500 \times 10^{-3} \quad = 1680 \text{ kN}$$

$$h_c = 200 - 54/2 \quad = 173 \text{ mm}$$

– ignores contribution of decking in shear

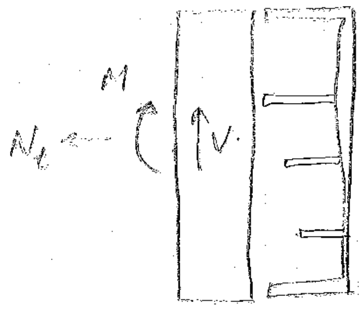
Compression on face AB.

$$N_c = 0.85 f'_c\, 1200 \times 173 \times 10^{-3} \quad = 4359 \text{ kN}$$

$$\Rightarrow V_c = 1680 + 4359 \quad = 6039 \text{ kN}$$

– excludes contribution from decking.

A.4 Summary of Resistance of Slab Remnant, Levels 4 – 6.



$$M = 42{,}973 \text{ kN/m}$$
$$N = 9{,}166 \text{ kN}$$
$$V = \underbrace{5{,}256}_{\text{conservative}} \text{ kN}$$

} Level 6 slab remnant.

Project: B: Resistance of Levels 6, 5, 4 Date: 2/10/2011
Subject: Slab at Attachment to Lift / Services Core
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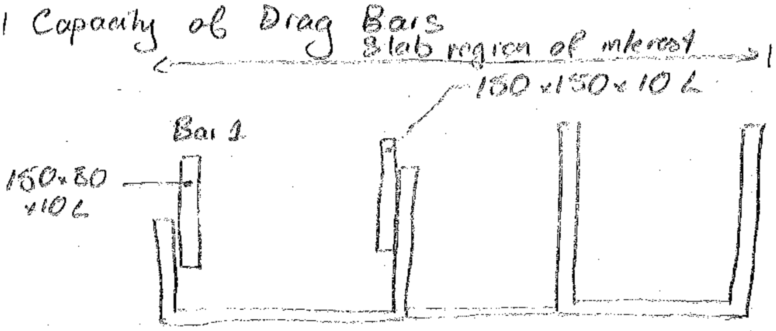
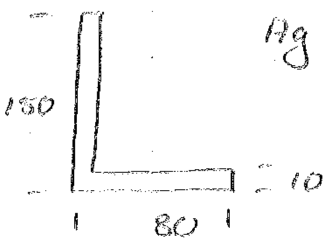
B: Levels 6, 5, 4: Approximate Capacity – Connection to Lift / Services Core.

B.1 Capacity of Drag Bars



See Hyland report Fig. 25.

Bar 1 $f_{y,nom} = 250$
Angle 150×80×10 Grade 250? $f_{y,act} = 1.18\, f_{y,nom}$



$A_g = 150\times10 + 70\times10 = 2200\,mm^2$

M20 bolts into slab

$A_n = 2200 - 22\times10 = 1980\,mm^2$

$N_{t,avail} = \begin{cases} 0.85\, k_t\, A_n f_u = 0.85\times0.85\times1980\times410/10^3 = 587\,kN. \\ A_g f_y = 2200\times250/10^3 = 550\,kN. \end{cases}$

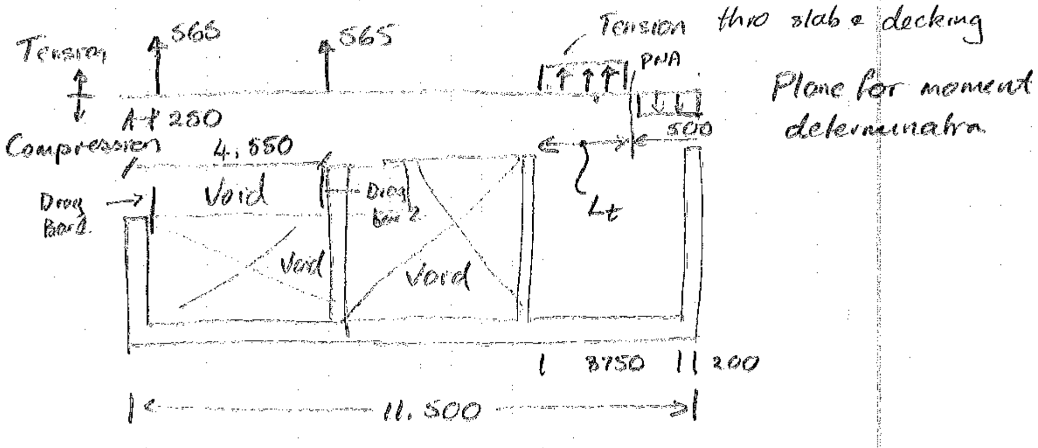
However attachment into slab is 6 M20 bolts.

Resistance each bolt (on basis is shear stud) $= 0.62 \times 410 \times \frac{\pi 20^2}{4} \times 1.18 = 94\,kN.$

V_t for 6 bolts $= 6\times94 = 565\,kN.$

⇒ Adopt both shear as limit.
N_t, diag bar = 565 kN. Use for both.

B.2 Determination of moment capacity slab attachment to shear wall.



Try PN at 500 from compression end c = 500

R_c = 173 x 0.85 x 0.50 x 24.7 = 1816 kN.

R_T = 2x565 + (3.75+0.2-0.5-0.2) x 776 = 3652 kN.

rebar / deck near PNA

Try PN at 800 from compression end

R_c = 1816 x 0.8/0.5 = 2906 kN

R_T = 2x565 + 2.95x776 = 3419 kN.

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Try PN at 900 from compression end

$R_C = 1816 \times 0.90/0.50$ = 3268

$R_T = 2 \times 565 + 2.85 \times 776$ = 3841

- close enough; adopt.

Moment capacity - about centroid of concrete in compression $(0.85 \times 900/2)$ = 383 from RHE

$M = 565 \times (11.50 - 0.25 - 0.383)$
$+ 565 \times (11.50 - 4.55 - 0.383)$
$+ 2.85 \times \underbrace{776 \times 0.5}_{\text{half in yield assumed}} \times (0.9 - 0.383 + \frac{2}{3} \times 2.85)$

$= 12,523$ kNm.

B.3 Determination of Tension Capacity.

$N = 2 \times 565 + 3.78 (665 \times 186 \times 10^{-3}$
$+ 1058 \times 617 \times 10^{-3})$ = 4042 kN

B.4 Determination of Shear Capacity Across Front of Services Core

Take as from A.3 V = 5286 kN.

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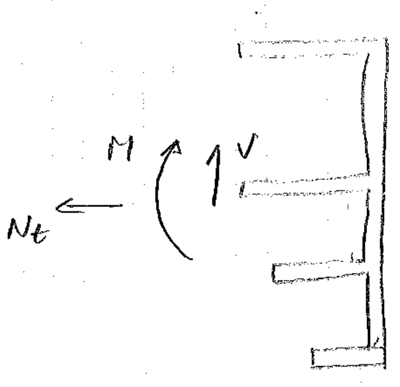
B.5 Summary of Capacities Across Slab at Level 6, 5, 4 Attachment to Lift / Services Core.



$M = 12{,}523$ kNm

$N_t = 4{,}042$ kN

$V = 5{,}286$ kN

Project: C: Resistance of Levels 3 and 2 Date: 3/10/2011
Subject: Slab at Attachment to Lift / Services Core.
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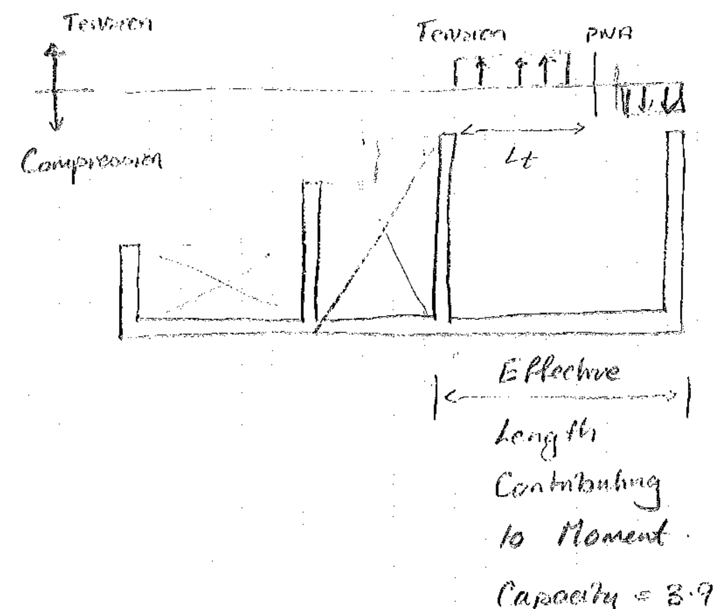
C: Levels 3, 2 : Approximate Capacity - Connection to Lift / Services Core.



Plane for moment determination

C.1 Moment Capacity.

Determine length beyond which steel yields:

f_y, rebar = 615 MPa.

ε_y, rebar = 3.08×10^{-3}

=> distance to yielding on tension side of PNA = $\frac{3.08}{3} \times c = 1.03c$

c = depth of comp. zone.

Try depth conc. in compression = 0.50 m.

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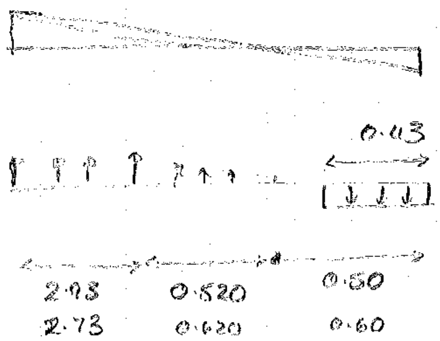

$R_c = 173 \times 0.85 \times 0.50 \times 24.7 = 1816$ kN.

$R_T = 2.93 \times 776 + 0.520 \times 776/2 = 2475$ kN.

Try increasing concrete comp stress to 600 mm.

$R_c = 1816 \times 0.6/0.5 = 2180$ kN.

$R_T = 2.73 \times 776 + 0.50 \times 776/2 = 2312$ kN.

Try increasing conc. comp. stress block to 700 mm

$R_c = 2180 \times 0.7/0.6 = 2543$ kN

R_T now $< R_c$.

Adopt 650 mm for depth concrete around lift shafts.

$M = 2.63 \times 776 \times (3.980 - 0.85 \times 0.7/2 - 2.63/2)$

$+ 0.62 \times 776/2 \,(0.7 - 0.85 \times \frac{0.7}{2} + \frac{2}{3} \times 0.620)$

$= 4967$ kNm.

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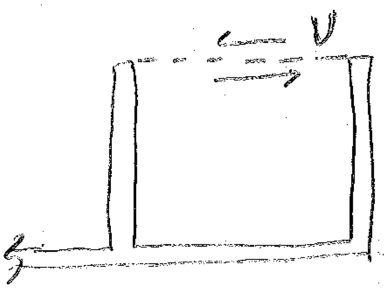
C.2 Tension Capacity

$N = 3.75\ (665 \times 186 + 1058 \times 617) / 10^3 = 2912\ kN.$

C.3 Shear Capacity

This is along shear plane shown below



$V_c = 0.17 \times \sqrt{24.7} \times 173 \times 3{,}750 \times 10^{-3} = 548\ kN.$

Decking contribution:

$V_d \approx 3750 \times 0.5 \times 0.83 \times 0.6 \times 617 \times \frac{0.75}{10^3} = 432\ kN$

$V_{total} = 548 + 432 = 980\ kN.$

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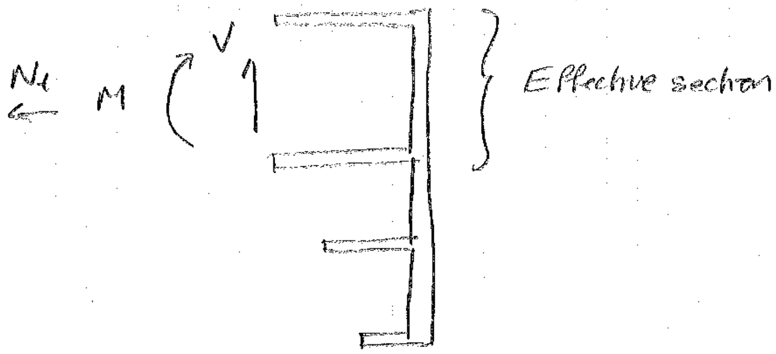
C.4 Summary of Capacities Across Slab at Levels 3, 2 Attachment to Lift / Services Core.



$M = 4967$ kNm.

$N_t = 2912$ kN

$V = 980$ kN.

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Project: D: Estimate of Floor Diaphragm Date: 04/10/2011

Subject: Actions at Face of Lift/Services Core

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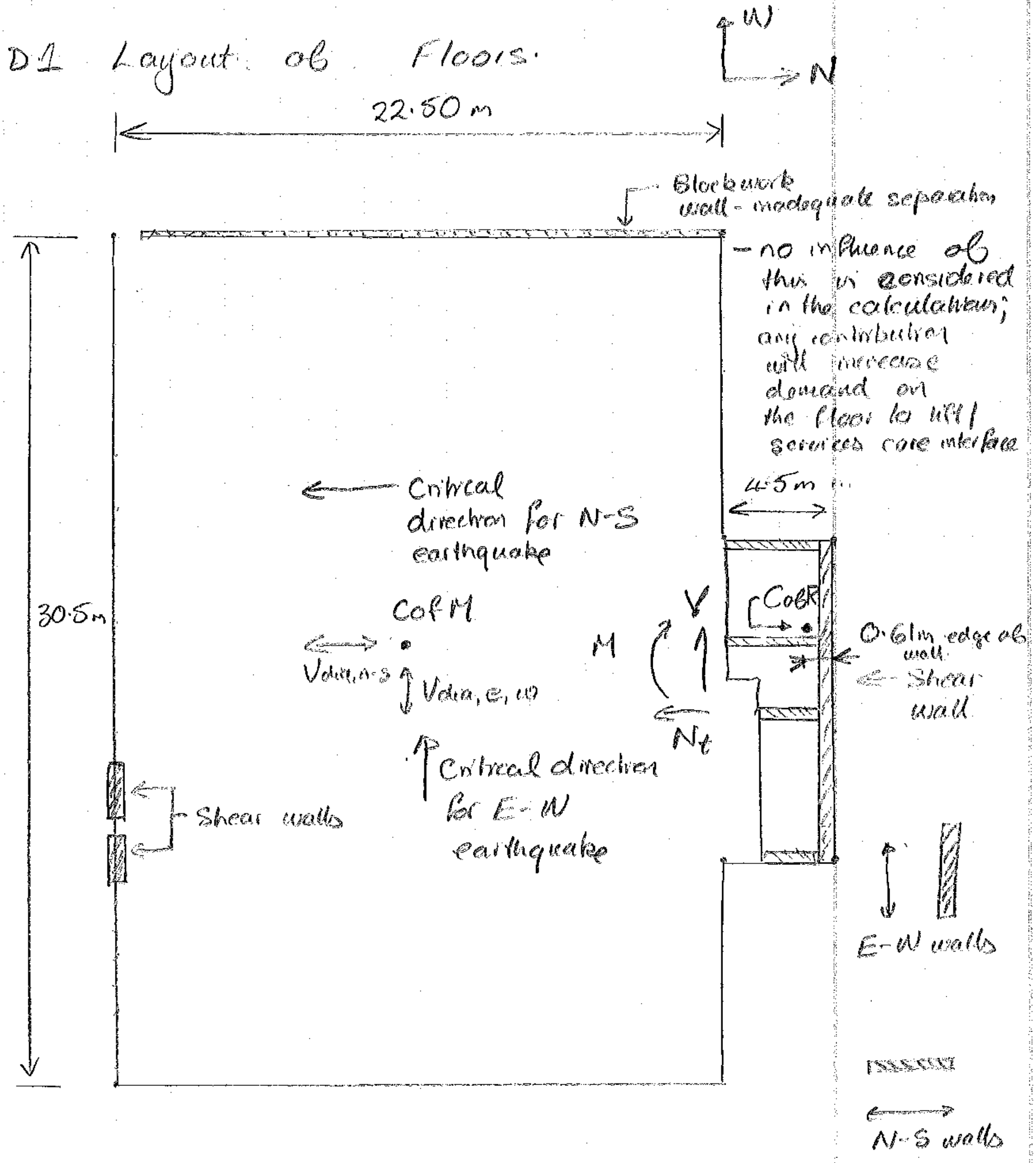
D1 Layout of Floors.



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D2 Determination of Centre of Rigidity – Approx.

(1) In N-S direction will be close to middle of building.

(2) In E-W direction will be near lift/services core.

How close?

Shear walls on south side cracked, by eq, use $0.4 I_g$
– each 2m long, 0.2m wide, coupling beams damaged

$$I_{xx,south} = \underbrace{0.4}_{cracking} \times \left(\underbrace{\frac{1}{12} \times 0.2 \times 2^3}_{I_{xx}\ each\ wall} \times 2 + \underbrace{2 \times 0.2 \times 1.1^2 \times 2}_{Ar^2} \right) = 0.49\,m^4$$

North side – single wall 11.5m long not cracked

$$I_{xx,north} = \frac{1}{12} \times 0.2 \times 11.5^3 = 25.4\,m^4$$

$\Rightarrow$ Centre of rigidity $= \frac{0.49}{25.4 + 0.49} = 0.02$ from North side wall.

North side wall $= 0.02x$ from North side wall

$\Rightarrow$ CoR $= 0.02 \times (22.50 + 4.50) = 0.51\,m$ from North wall.

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D.3 Estimate Seismic Diaphragm Actions Generated By Floor on Lift / Services Core.

(1) Estimate seismic weight / floor.

Floor is 200 mm slab on Hibond, he = 173 mm.

$G \approx 0.173 \times 24 + \underbrace{1.0}_{\text{frame}} + \underbrace{0.5}_{\text{partitions}} + \underbrace{0.5}_{\text{services, ceilings}}$ = 6.15 kPa.

$G_{clad} \approx \underbrace{2.5}_{\text{2.5 kN/m around bldg / floor}} (22.50 + 30.50) \times 2$ = 265 kN

Q – use value for commercial building from floor vibration check – likely low side = 0.5 kPa.

$W_i = (6.15 + 0.5) \times 22.5 \times 30.50 + 265$ = 4829 kN

(2) Estimate diaphragm shear / floor.

Base on PGA × C_{Hi} = 1.6 for effect of lightly damped building ← Used in diaphragm design Civil 714 lecture notes

PGA – from 22 Feb record, CNS = 0.5g near CTV
(in practice probably bit higher)

NOTE: The estimate of PGA used here has been superseded by the average of the NS and EW components recorded from the three closest strong motion recording sites to the CTV building. Details are in section 4 and the results in Tables 1 and 2 incorporate those updated values not the diaphragm demands given below. In the NS direction the average PGA is 0.37g and in the EW direction the average PGA is 0.52g. The figures are updated below but not in Appendix E.

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$V_{dia} \approx W_i \; PGA \; C_{Hi} = 4829 \times 0.5 \times 1.6 = 3863 \text{ kN}.$

Using strong motion data, $V_{dia.NS}$ = 2859 kN and $V_{dia.EW}$ = 4018 kN

(3) Diaphragm actions on lift / services core for north-south earthquake.

C of M & C of R align so no moment. All N-S action taken by lift / services core walls.

$N^*_{tension\ or\ compression}$ 2859 kN

(4) Diaphragm actions on lift / services core for east-west earthquake.

V_{dia} applied through C of M.

$V^* \approx \left(\frac{25.4}{25.4 + 0.49} \right) \times$ 4018 kN = 3941 kN

- this is proportion diaphragm force through the North side wall, on basis this is uncracked one south side wall is cracked.

M^* at connection of slab into core

= 4018 (4.50 − 0.61) = 15,630 kNm

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Project: E: Estimate of Adequacy of Date: 04/10/2011
Subject: Diaphragm
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E.1 Adequacy of Levels 4-6.

(1) For North-South direction earthquake motion

$N^*_{tension}$ (critical direction) from D.3(8) = 3863 kN

$N_{diaphragm}$ = minimum $(\underbrace{9,165}_{\text{Section A}};\ \underbrace{4042}_{\text{section B}})$ = 4042 kN

✓ OK.

(2) For East-West direction earthquake motion

V^* from D.3(4) = 3789 kN

M^* from D.3(4) = 14,743 kNm

V from section A = 5 286 kN

M = minimum $(\underset{\text{section A}}{42,973};\ \underset{\text{section B}}{12,523})$ = 12,523

– moment capacity is exceeded, but not by much

E.2 Adequacy of Levels 2-3

(1) For North-South direction earthquake motion

$N^*_{tension}$ from D.3(8) = 3863 kN

N_{dia} from section C.2 = 2912 kN

– overloaded; likely to fail

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(2) For East - West direction earthquake motion:

N^*	from	D.3(4)	= 3789 kN
M^*	from	D.3(4)	= 14,743 kNm
V	from	section C.3	= 980 kN
M	from	section C.1	= 4967 kNm

- both moment and shear are well below what is required

E.3 Conclusion:

Slab separation from lift / services core at levels 2, 3 will have occurred and initiated complete building collapse due to pancaking failure at those levels.

Both directions overload diaphragm; the E-W earthquake especially so.