Hydraulic uplift forces on basements subject to liquefaction

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ABSTRACT: As a result of the 22 February 2011 earthquake in Christchurch a building with a single storied basement underwent differential movement and shear reinforcement within part of the basement slab failed. The building was under engineering observation prior to the first earthquake in September and has been closely monitored. The building has provided a unique opportunity to assess the forces applied to the basement slab under the effects of liquefaction.

The evidence suggests that pressures considerably in excess of the hydrostatic pressures occurred beneath the slab and it is assessed that those pressures are likely due to the soils behaving as a dense fluid when they liquefy. It is recommended that basements founded in soils which are subject to liquefaction should be designed for uplift pressures that are greater than hydrostatic pressures.

1 BACKGROUND

Between September 2010 and December 2011 Christchurch experienced four significant earthquakes which caused liquefaction of the soils beneath significant parts of the city area. Several buildings with basements in the area suffered damage and settlement. One building in Bealey Ave provided a unique opportunity to assess the probable forces on the basement slab due to liquefaction because the building was already under engineering study immediately prior to the first earthquake event and was closely inspected after each event.

From observation of the area surrounding the building it is apparent that no specific liquefaction around this particular building occurred as a result of the first earthquake event, but significant liquefaction occurred during the second earthquake event in February 2011. This liquefaction resulted with the south end of the building changing level in relation to the north end by about 120mm. This change may have been due to differential settlement or differential heave or both.

For uplift to have taken place the pressures on the basement slab needed to have exceeded full hydrostatic pressures equivalent to the site with groundwater at the ground surface. Analysis of a shear connection detail within the basement slab confirms that the pressures beneath the basement slab exceeded such hydrostatic pressures.

This paper considers that the pressures acting on the base of the slab would have been equivalent to those of a dense fluid with a density equivalent to the bulk density of the surrounding soils and refers to this pressure as the hydraulic uplift pressure.

2 BUILDING DETAILS

The building consisted of a three and a half storied complex consisting of 42 apartments built in two parallel rows approximately 8m apart and aligned north south along the site. Each wing of 21 apartments had a full single stored basement beneath. Each wing was 17.4m wide by 37.7m long with the two basement wings interconnected with a 16m wide full depth basement at the south end of the structure.

The basement area was used for carparking and access to the basement was by way of a ramp off the street frontage at the north east corner of the complex. The concrete floor level of the basement was originally founded at 2.2m below ground level with the lower level of the apartments at 2.8m above this level or 600mm above the ground level.

The basement walls were precast concrete and the basement slab was cast insitu concrete with internal columns resting on the basement slab. The general arrangement of the basement is shown in Figure 1. Along each side of the basement the insitu concrete basement slab was 375mm thick to 5.7m from the perimeter wall with thickening to 725mm beneath column points. The central 6m wide strip of slab was 425mm thick and a construction joint with water stops was formed between the central and outer portion of the slabs.

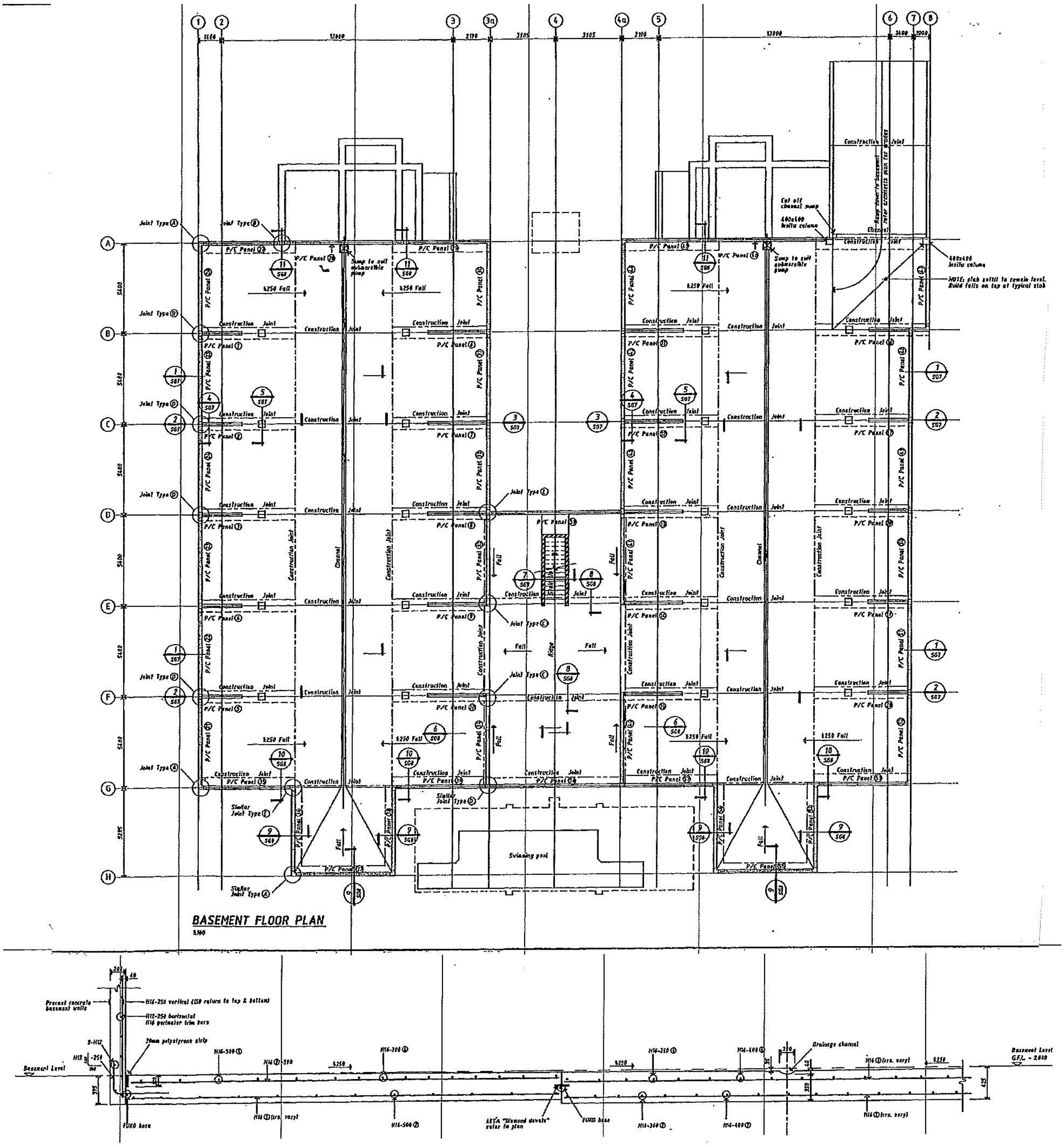

Figure 1. Layout of Basement and Slab Reinforcement Details

The connection between the central and outer slabs was designed to transfer uplift in shear using a system known as a Lesa diamond dowel which is designed to allow shrinkage movement across the construction joint but no moment transfer. The Lesa dowel is a rectangular steel plate which is set into the edge of each side of the concrete slab at regular intervals with its diagonal axis aligned along the construction joint.

Around the perimeter of the basement walls was a drain at 0.5m below ground level which was designed to control the maximum level to which groundwater could rise and therefore control the maximum hydrostatic uplift pressure on the underside of the slab. At this site the Lesa dowels were designed to resist 1.2 times the anticipated maximum hydrostatic uplift force on the base of the slab. Subsequent finite analysis of the slab and dowel system showed that the dowels would resist 1.4 times the hydrostatic uplift pressure (Robinson, 2013).

Several of the Lesa dowels were found to have failed following liquefaction at the site.

The total mass of the basement and superstructure (excluding live load) exceeded 1.3 times the hydrostatic uplift pressure if the site was fully saturated, that is the groundwater level coincident with the ground surface.

3 GROUND CONDITIONS

The ground conditions at the site were investigated prior to the development of the site by Riddols & Grocott in June 1999 and subsequent to the earthquakes by Tonkin & Taylor Ltd in March 2012.

The subsurface conditions can be summarised as follows:

1. 0- 0.5m Topsoil (silty sandy organic stained soil)
2. 0.5 -1.5m Sandy silt, firm to stiff
3. 1.5- 2.5m Peat organic silt, soft
4. 2.5 – 3.5m Silty Sand/Sandy Silt, loose to compact
5. 3.5- 8.0m Sandy Gravel/gravelly Sand, Compact to dense with some loose layers
6. 8.0 – 22.0m Silty Sand, compact to dense
7. 22.0 – 22.5 Organic silt, stiff
8. 22.5^{+} Sandy Gravel, dense

The underside of the basement slab is located at about 2.6m below ground level within the loose to compact layer of silty sand and sandy silt strata.

In March 2012 the ground water level was measured at 1.2m below the ground surface. In June 1999 the ground water level at the site was measured at 0.8m below the ground surface. Observation of the basement prior to the earthquakes observed water running out of the perimeter relief drain during times of winter high groundwater levels indicating that the groundwater rose to within 0.5m of the ground surface.

Cone penetrometer testing (CPT) was carried out down to 3.5m depth. Standard penetrometer testing was carried out in the layer below 3.5m and identified some loose lenses within this deeper strata. From the test boring, standard penetrometer testing and cone penetrometer testing carried out in 2012, it is apparent that the ground at the south end of the site was more dense than at the north end of the site however a borehole put down post the earthquakes some 50m south of the site showed loose to compact layer of silty sands extended to more than 8m depth.

4 SEISMIC ACTIVITY

In Table 1 is listed the strong ground motions recorded by the GeoNet station network and the approximate distance between the epicentre of the earthquake and the site.

Table 1. Peak Ground Accelerations

Magnitude (Mw)	Date	Peak Horizontal Ground Acceleration (PGA) at Chch Botanic Gardens	Distance and direction to epicentre from site (km)
7.1	4 September 2010	0.18g	39km W
6.2	22 February 2011	0.43g	7 km SSE
6.0	13 June 2011	0.17g	11Km SE
5.9	23 December 2011	0.24g	8km E

5 LEVEL OF INTENSITY OF SHAKING

The New Zealand design codes require consideration of the performance of buildings under the Ultimate Limit State (ULS) condition and that under such conditions while deformation can occur they should not collapse, but under the Serviceability Limit State (SLS) the building should remain functional. In Table 2 comparison is made with these design requirements and the ground accelerations which occurred in the September and February earthquake events.

Table 2. Comparison of Peak Ground Acceleration with Design Requirements

	SLS	ULS	4 Sept Event	22 Feb Event
Return Period	25yr	500yr		
Magnitude	7.5	7.5	7.1	6.2
PGA	0.13g	0.35g	0.18g	0.43g

From the above it is apparent that the peak ground acceleration for the 4 September event exceeded the SLS level of acceleration but not the ULS but the February earthquake exceeded the ULS event.

6 EVIDENCE OF LIQUEFACTION

On the basis of the stratigraphic profile and CPT testing carried out on the site and using the methods described by Idriss & Boulanger (2008) it is apparent that under an ultimate limit state earthquake event the soils from the ground surface down to a depth of about 2.5m would be unlikely to undergo any liquefaction (but some strain softening of these soils might take place with increased cycles of vibration).

However, analysis indicates that the layer between 2.5m depth and 3.5m depth would be expected to liquefy under repeated earthquake shaking under an ULS earthquake event and under the 22 February event but was unlikely to liquefy under a SLS event. This layer extends beneath the entire basement floor slab from the north end to the south end.

Below 3.5m depth, based on the insitu test results most of the strata would not be expected to liquefy, however, lenses of loose materials within this layer could be expected to undergo some liquefaction.

Following the 22 February 2011 earthquake known ejecta from liquefaction was mapped around the greater Christchurch area. No such mapping was undertaken following the 4 September 2010 event. From aerial photographic evidence after the 22 February event evidence of ejecta surrounding the general area where this building was located was apparent.

7 ASSESSMENT OF SETTLEMENT DUE TO LIQUEFACTION

Assessment of the amount of settlement which might be expected to occur in the soils post liquefaction for level free field conditions was made based Idriss & Boulanger method and this predicts that between 50 and 150 mm of settlement of the ground might be anticipated following liquefaction.

Based on the borehole information at either end of the building it is predicted that more settlement could be expected at the north end of the site than the south end.

8 LEVELS ON BASEMENT FLOOR SLAB

Because the building was under observation prior to the first earthquake levels had been obtained across the basement floor level. Contours showing the relative difference in level across the basement floor are shown on Figure 2. From this it is clear that the basement floor was constructed with a fall from south to north. These initial levels were tied into a benchmark established in the kerb on Bealey Ave. The benchmark was used to compare the results of the level survey conducted before the 4 September earthquake with those after the 4 September earthquake. No appreciable change in the basement floor levels was found as a result of the 4 September earthquake event confirming that no apparent liquefaction occurred at this site as a result of this event.

As a result of the 22 February 2011 earthquake movement occurred in the kerb and thus the bench mark in the kerb could not be relied on to obtain new absolute levels across the floor slab. However differences in levels depicted as contours in respect to the same origin within the basement are shown on Figure 3. This shows that post the 22 February earthquake the north end of the basement is 110 to 130mm lower than it was prior to the 22 February earthquake.

However, from these levels it cannot be determined whether the south end of the building has risen in relation to the north end or the north end has settled or a combination of both actions has taken place.

9 OBSERVATION OF THE GROUND SURFACE SURROUNDING THE BASEMENT

Lidar survey data has shown that the general ground surface in the Christchurch area has settled as a result of the earthquake events. At this site the ground immediately alongside the basement walls has settled but this local settlement was observed to be greatest alongside the portions of the basement at the south end of the structure. The cause of this local settlement of the backfill could be due to loose backfill alongside the basement walls consolidating due to the vibrations from the earthquake events or it could be due to settlement as a result of some of the backfill material flowing into the void created as a result of the basement slab lifting under the excess hydraulic pressure.

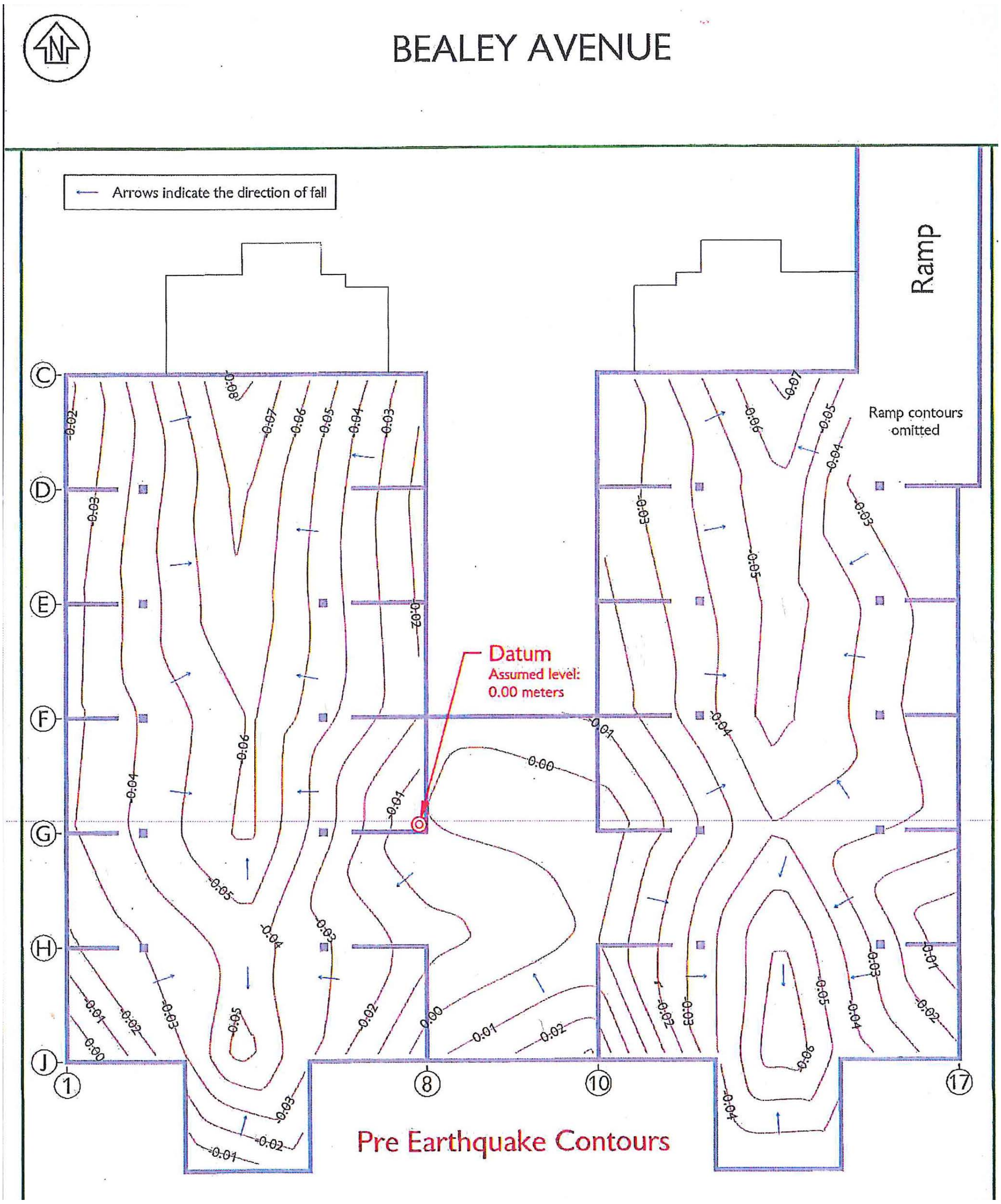

Figure 2. Levels on Basement Slab prior to Earthquakes

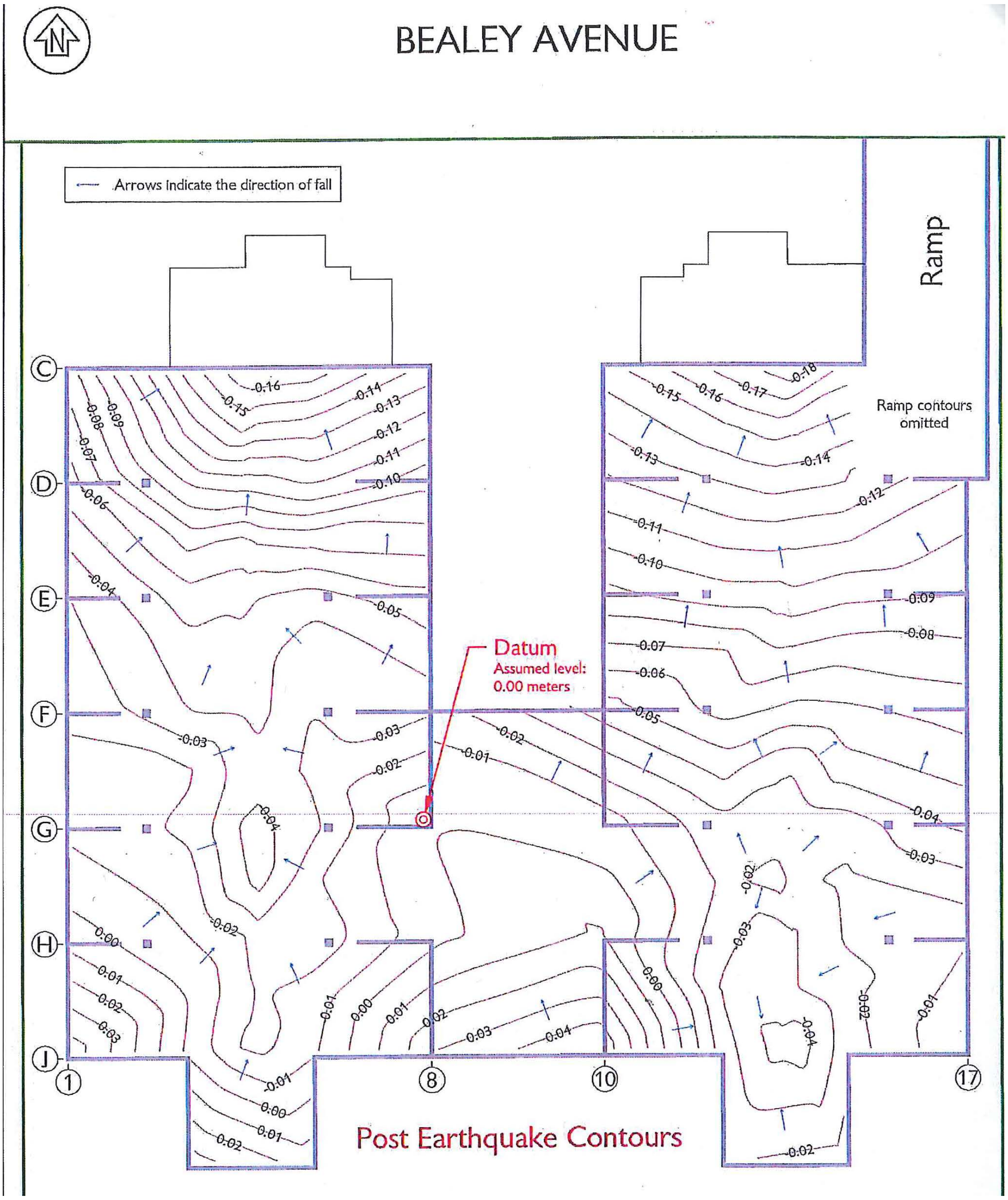

Figure 3. Changes in Basement Levels after February Earthquake

Had the settlement been relatively uniform around the basement then it most likely would have been due to settlement of loose backfill. Because the amount of settlement of the backfill is greatest at the south end and least at the north end it is probable that the south end of the basement has risen.

10 UPLIFT FORCES

Under hydrostatic pressure with the groundwater controlled by the perimeter drain at 0.5m below the ground level the uplift pressure beneath the basement is about $20kN/m^2$ and the dead weight of the building and basement has been estimated at about $36kN/m^2$. Thus the average factor of safety against uplift is about 1.6. However at the southern end of the building, because of the crosslink in the basement where uplift forces are only resisted by the adjacent building loads, there would be a greater hydrostatic uplift on the building and this would reduce the factor of safety against hydrostatic uplift from the average value of 1.6 to a local value of 1.4 at the south end of the building.

However, if the groundwater table were to have risen to be coincident with the ground surface, the hydrostatic uplift force would increase to about 25kN/m^2 and the factor of safety against uplift would have reduced to 1.3 on average and to 1.15 at the south end.

11 LIQUEFACTION PROCESS

In a cohesionless soil individual soil particles (typically silt, sand and gravel) carry the load from the overlying strata and buildings by interparticle stress at points of contact between individual particles and the void space between particles where such strata is below the groundwater table is filled with water. With the onset of seismic shaking, where layers are saturated and the individual sand particles are loosely packed together the individual particles try to assume a more dense packing, displacing the water in the voids. If this water cannot drain away quickly enough, the stress applied to the individual soil particles transfers to the pore water and the pressure in the water increases. If the pore water pressure equals or exceeds the interparticle stress, the particles in effect become buoyant, the soil loses its ability to resist load and liquefies.

The excess pore water pressure is then equal to or exceeds the overburden pressure, be that due to the soil or a building load. The pressurised water then flows towards the ground surface via fissures and fractures in the ground taking some fine particles with it until the pressure subsides. These fine particles form the ejecta which is evident after liquefaction has occurred.

It is important to note that although the soil liquefies during the process of shaking, it is often several minutes before ejecta typically reaches the ground surface, sometimes being as long as 5-10 minutes or longer after shaking stops. However, until water and ejecta can escape and relieve the increased pore water pressure, the liquefied layer behaves as a dense fluid. The pressure in the liquefied layer increases with increased intensity and duration of shaking until the layer liquefies and the process leading to such liquefaction ceases as soon as shaking stops.

However this pressure is not relieved until fluid escapes either by natural seepage into the surrounding formations or by flowing up onto the ground surface. To differentiate from hydrostatic pressure this paper refers to the pressure which exists in the liquefied layer as a dense fluid as the hydraulic pressure. It generally will not exceed the equivalent bulk density of the soil and typically will range from 1.6 to 1.8 times the hydrostatic pressure. Studies of forces on pipelines have identified such uplift pressures when the soils liquefy (Teh et al (2006), Sumer et al (2006)).

The basement at Bealey Ave property had a stratum of liquefiable soil which extended from very near to the underside of the basement floor to about 1m below the underside. (Some lenses and layers in the strata at a greater depth will also have liquefied). As this 1m thick layer liquefied it would apply a hydraulic pressure to the underside of the basement slab equivalent to the total overburden pressure. This is approximately 1.6 to 1.7 times the pressure which would be caused by the maximum possible hydrostatic pressure which the groundwater could exert if the groundwater table rose to be coincident with the ground surface.

Such increases in pressures beneath the basement would be sufficient to cause the uplift pressures beneath the basement to exceed the weight of the structure and for the basement to 'float' and the difference in uplift pressures at the south end would cause the south end to rise more than the north end.

12 CAUSE OF DIFFERENTIAL DISPLACEMENT AND FAILURE OF LESA DOWELS

Surveys taken before and after the February earthquake clearly show that this basement has differentially displaced with the north end lower than the south. Ejecta associated with liquefaction has been observed surrounding the site and the soils immediately beneath the structure have been identified as being liquefaction prone.

Unfortunately, because bench marks used for the level survey could not be reliably re-established it was not possible to quantify whether the differential movement measured across the basement was due to differential settlement or differential uplift. Settlement calculations indicated that the north end of the structure is likely to have settled more than the south end.

Thus two potential mechanisms could explain the change in levels, differential settlement or differential hydraulic uplift or a combination of both.

However, the structural connections in the basement floor slab failed and detailed finite analysis indicates that the pressure beneath the structure had to exceed more than 1.4 times the design hydrostatic pressure. This suggests the base of the slab was subject to significant uplift pressures greater than could be explained by a mere rise in the groundwater table to the ground surface.

13 PRESSURES BENEATH THE BASEMENT

The September 2010 earthquake occurred during the end of winter when groundwater elevations would have been at their highest. Observations indicate that high winter conditions were sufficient for the perimeter drain to flow. Thus it can be expected that at the time of this quake the groundwater level will have been at its highest and possibly only 0.5m below the ground surface.

Although there is no evidence of liquefaction having taken place at this site as a result of this earthquake event (and as confirmed by the pre and post survey of levels on the basement) this does not mean that the water pressures within the liquefiable layers did not increase, but rather the increase was not sufficient to cause liquefaction. In all probability the pressures under the slab will have been close to or greater than the pressures representing the groundwater table coincident with the ground surface. Post earthquake inspections found no evidence of failure of the structural connections.

Thus the shear connections within the basement floor probably experienced uplift pressures at least as great as 1.25 times the design pressures.

On the other hand the February earthquake occurred in the latter part of summer when the groundwater level would have been low. Post earthquake measurements show this level at 1.2m depth in March and pre development measurements were at 0.8m in June. Hence the hydrostatic pressure on the base of the slab immediately prior to the February quake would likely have been considerably less than the design value.

Hence the most probable explanation for the increase in pressure beneath the floor slab sufficient to cause failure of the shear connections is that the hydraulic pressures created by the liquefying of the soils beneath the slab applied a force equivalent to 1.6 to 1.8 times the hydrostatic pressures.

14 IMPLICATIONS ARISING

Basements below the groundwater table are routinely designed to resist hydrostatic uplift forces. However, this site indicates that where such foundations are located in soils which are subject to liquefaction the uplift pressures on the basement can exceed the hydrostatic uplift pressures and increase to a hydraulic pressure equivalent to the soil behaving as a dense fluid with the density equivalent to the bulk density of the soils.

Thus basement slabs should be designed for such hydraulic pressures.

15 CONCLUSION

The basement at this site underwent differential movement and shear connections within the basement floor slab suffered failure. Observation around the perimeter of the basement suggests that the south end of the basement probably rose in relation to the north end.

Analysis confirms that a greater uplift force existed at the south end when compared with the north end. Analysis indicated that the north end of the building probably settled more than the south end.

The shear connections in the floor slab had to sustain more than 1.4 times the design hydrostatic uplift pressures before failure would occur. The performance of the building suggests that hydraulic pressures beneath the basement slab equivalent to the liquefied soils behaving like a dense fluid have caused the pressures beneath the slab to exceed the design strength of the shear connections.

Basement floor slabs should consider the uplift force associated with soils becoming a dense fluid where basements are founded in soils that can liquefy.

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