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RESIDENTIAL CONCRETE SLAB PERFORMANCE AS A RESULT OF THE CHRISTCHURCH EARTHQUAKE SEQUENCE 2010-2012

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ABSTRACT

The sequence of over 3,700 earthquakes and aftershocks greater than Magnitude 3.0 and the related liquefaction which occurred in Christchurch from 4^{th} September 2010 have proved a testing ground for structural components.

The humble domestic concrete slab is a very important component in getting things right first time. Because there are so many of them, the cost of poor detailing and construction techniques have had a significant effect on the economy and wellbeing of the community running into billions of dollars worth of damage in Christchurch and causing displacement and trauma to people unnecessarily.

However many habitable houses with unreinforced cracked concrete floors are being written off unnecessarily, due to the wording of insurance policies and ignorance with respect to the reparability of such.

Although the nominally reinforced non-ductile 665 mesh floors had their failings and seismic mesh is now required, such floors also had their successes holding buildings together while their neighbours fell apart.

Unreinforced slabs are now no longer permitted for new construction of residential houses, and seismic reinforcing is now the mandate.

Experience as a result of the earthquakes has proved the superiority of waffle slabs. However, subsequent DBH guidelines for such are in the writer’s opinion “over design” and there is some scope for relaxing the current 2m cantilever requirement.

INTRODUCTION

A series of some 58 earthquakes and aftershocks greater than magnitude 5 have occurred in the 22 months since the 4^{th} September 2010 event. On the positive side, this gives a very good opportunity to study the effects of earthquakes first hand, learn from them and develop improved construction techniques.

When I first volunteered to assist in Christchurch following the 4 September 2010 earthquake and inspected 2 groups of council flats that were write offs due to unreinforced concrete slabs, I thought at first that someone had made a mistake and left the reinforcing out.

I was absolutely astounded to learn that this was standard practice, and have concluded that this practice has cost the country and insurers billions of dollars.

Having been actively involved in the design of waffle slabs or Rib-Raft Slabs, I was also keen to see how these performed.

My prime area of interest has been the performance of residential concrete slabs under seismic conditions, particularly with respect to liquefaction and my observations are outlined in this paper.

GENERAL OBSERVATIONS

Foundations and floor slabs are crucial and extra effort and cost is warranted to get these right first time. If the roof or a wall is damaged it can be replaced or fixed relatively easily. However, if the foundations and floor slab are ruined chances are the house is a "write-off."

A house is not just a building; it is someone's home and sanctuary, their shelter, their prime asset, filled with memories and heirlooms. For their home to be written off, it can be devastating for most people. Even if the home is badly damaged but repairable, for many the sanctity of their home has been violated and they want to move out, put the bad memories behind them and make a fresh start somewhere else.

Houses where the foundations and concrete floor are badly damaged can be very difficult to rectify because of internal partitions, services, floor coverings, etc. Houses with concrete floors in Christchurch tend to be brick veneer clad with heavy tile roofs. They are generally not economic to relocate.

While older double brick buildings pre 1931 earthquake collapsed, generally houses built in accordance with the NZ Standard for Timber-framed Buildings NZS3604 performed well, even with brick veneer and heavy tile roofs.

It is noted however that with the high vertical accelerations generated particularly in areas founded on rock, heavy roof tiles were often thrown upwards and smashed on impact when they hit the roof again. There is certainly merit in having a lightweight roof in a seismic area. It was also noted that there was generally less damage to gib linings where a light weight roof was used.

Overall, I was surprised how well 70 series brick veneer and gib board generally stood up to the earthquakes and generally the resilience of New Zealand homes built in accordance with NZS 3604 and the Building Code.

However, the Achilles heel of residential buildings in Canterbury has been foundations and slabs built in accordance with NZS 3604 on three counts;

- Under NZS3604 definitions, Liquefiable Soils constituent "Good Ground". (Even though all PIM reports warned that the area could be subject to liquefaction, building consents were issued on the basis of the NZS3604 definition). Note that while Canterbury has now been "ring fenced" as a special case, this definition still applies to the rest of NZ and change is needed.
- NZS3604 permitted the use of unreinforced concrete slabs in residential single story buildings on "Good Ground". (It still does, but the Department of Building and Housing has over-ridden with an amendment to the building code).
- NZS3604 permitted the use of uncompacted round river gravel under slabs. This material often rattled like marbles and settled under seismic conditions or dropped like the proverbial stone though the liquefying material underneath, leading to floors settling and internal partition walls pulling away from the floor and/or the ceiling. Use of uncompacted river gravel has since been banned by an amendment to the Building Code.

A further factor to consider is that without satisfactory regulation or enforcement, in a competitive market, matters can tend to sink to the lowest common denominator, as responsible builders who would prefer to incorporate mesh were competing against those who did not and for the sake of $800 worth of mesh have lost work. The writer is concerned that a similar situation is developing in the North Auckland Silverdale area where the effects of shrinkage of expansive clay fills is being ignored as a cost cutting measure on a similar basis.

INFORMATION ON THE CURRENT CODES AND GUIDELINES

Things with the current codes and Guidelines are a little disjointed. NZS 3604:2011 came out after the 4 September 2010 earthquake. It was abundantly clear at the time that ground which could be subject to liquefaction did not constitute “Good Ground”, and it was further oblivious that uncompacted river gravel under concrete slabs, and unreinforced concrete slabs for houses were particularly stupid ideas in light of the experience at Christchurch, yet this folly followed through in the new version of NZS 3604;2011 released several months after the event.

I was pretty hot on this and rattled a few cages. To misquote Shakespeare “Me thinks something still not smell right in the state of New Zealand”.

On 19 May 2011 **the definition of ‘good ground’** was changed **(for the Canterbury earthquake region only)** to exclude land where liquefaction and/or lateral spread could occur. Designers can refer to Guidance on using NZS 3604 construction on ground with potential for liquefaction. [5]

There is some pretty woolly logic somewhere. Earthquakes and liquefaction are no respecters of lines on a map. My advice to anyone building elsewhere in New Zealand on potentially liquefiable soils would be to not treat it as “Good Ground” and use the Guidelines intended for Canterbury.

To their credit, DBH did make amendment requiring seismic reinforcing in slabs. This was a fairly drastic move as the steel industry was bobbing a long producing 665 mesh without any seismic mesh on the horizon despite concern having been expressed in the engineering community over a number of years.

NZS 3604:2011 is now referenced with modifications. All concrete slabs-on-ground must be reinforced with ductile Grade 500E steel mesh which is tied to perimeter foundation reinforcement; you can no longer use unreinforced slabs anywhere in New Zealand. [5]

With respect to the Guidelines produced by DBH, while the need for urgency is appreciated, these did not go through the full consultative process as used to be the case with NZ Standards and are supposed to be a “Living Document” subject to constructive criticism and continual upgrade. Constructive criticism so that the final document becomes the best possible is to be encouraged.

Some of the current ideas contained, such as excavating out an existing small residential site in liquefiable country to stabilise and form a densified stabilised crust 2 m thick with neighbour’s houses both sides are highly questionable and need a re-think, Or the next house likely to be on your client’s section, will be his neighbours!

This proposed methodology also takes no cognisance of the soil profile. Often there is an upper crust, or “Soil raft” anyway. So attempting to provide a soil raft by destroying the one that already exists may not be the wisest solution.
The critical depth that requires strengthening is often from 3 to 6m, and forming a raft to 2m does not solve this problem.

Further, the water table is normally at 1.2m to 1.6m deep so excavating to 2m deep will be below the water-table and really asking for problems. The jib answer is dewatering, it will inevitably add to cost. It may work in some cases but not in others and does add to the possibility of induced settlement of neighbouring properties.

While the DBH Guidelines are a commendable start there is room for improvement. However, do not use these guidelines without thinking very carefully, or the next time you see your client's neighbour may be in court.

RELEVELLING

Before attempting to repair a slab, if it is out of level or differential settlement across a crack has occurred, the slab should be relevelled if possible using an underground grouting technique if appropriate such as Highly Expansive Polymer Injection Grout (HEPIG) or Low Mobility Grout (LMG).

Highly Expansive Polymer Injection Grout (HEPIG) is a product that although expensive (about $7,000/m3) can in the right conditions be very cost effective in restoring a building that might otherwise require demolition and rebuild.

I had previously used Highly Expansive Polymer Injection Grout (HEPIG) on a job in Auckland where differential settlement in expansive clays of the order of 150mm had occurred at the end of a house opening up cracks in the walls and floor and in the ceiling up to 5mm wide. HEPIG was used and the 5mm cracks closed up to a pencil line that was only visible when looked for prior to replastering.

In Christchurch I have also had experience using this product. For re-lifting the slab, the perimeter first needs to be lifted and relevelled. This can be a challenge as the outer walls carry the weight of the brick cladding as well as the roof load transferred by the roof trusses to the outer walls.

The HEPIG product works best when it has something solid to press against, but unfortunately in the Christchurch situation the loose saturated sands below the water table do not offer a good reaction surface to press against to provide the desired lift to the building. Therefore it is necessary to try and densify this underlying soil raft first before focusing on lifting the building.

An appropriate current method of densifying the soil raft with a building already in place may be the use if Low Mobility Grout LMG columns injected into the ground these swell out in diameter compressing the adjacent soils and causing densification. An alternative is to use HEPIG "Power piles" which consist of inserting a split pipe into the ground containing a geotextile "sausage skin" which is then pumped full of HEPIG which expands in diameter compressing the adjacent ground. Another method currently being imported from Japan is the use of cement jet grouting to cement the soil matrix together increasing its strength, and thus providing a suitable solid mass for the HEPIG to react against.

I am satisfied that HEPIG is a good product that is unlikely to break down and will have a life well in excess of 50 years. However it is not so much the product that is being wrestled with, as the ground conditions that can liquefy under a significant seismic event. The threshold for liquefaction appears to be a Magnitude 5.6 to 6.3 earthquake depending on local soil conditions and distance to epicentre etc.

However densification of the underlying soil raft can improve the performance of the land and the slab above.

Observations in Christchurch where buildings were treated with HEPIG after the 4 September 2010 event but before the 22 February 2011 event indicate that the use of such grouting techniques stiffens the underlying ground locally and buildings so treated fare better than their previously less affected neighbours in the subsequent events.

A paper by three Turkish authors [1] following an earthquake in Turkey in 1999 tested the effectiveness of Highly Expansive Polymer Injection Grout (HEPIG) in improving local soil density and reducing proneness to liquefaction or consolidation and found it to be worthwhile.

Once the underlying soil raft has been densified, the actual lifting of the building and slab can be concentrated on.

With the river “marbles” under most Christchurch slabs, it has been found better to target the underlying material about 600mm below the floor slab causing this to swell and lift the marbles as well as the slab. Otherwise considerable product can be lost within the void space between the clean washed river gravel or “marbles” increasing cost unnecessarily.

A further point to consider when deciding whether to use LMG or HEPIG, is relative density. The soils will generally have a saturated density of about 18KN/m3 compared with LMG at 24KN/m3 and HEPIG at less than 18KN/m3. Should liquefaction occur again, unless LMG is formed as a well founded column bearing onto a non-liquefiable layer, LMG at 24KN/m3 is going to drop like a stone through the liquefying soil having a density of 18KN/m3.

If a building has already suffered settlement as a result of liquefaction, it makes perfect sense when considering buoyancy to utilize a grout for re-lifting with a specific gravity less than the liquefying soil to provide some positive buoyancy to offset the superimposed load of the building On this basis alone HEPIG would be preferred to LMG.

UNREINFORCED CONCRETE SLABS

While many unreinforced slabs survived the earthquakes without notable damage where they were on good ground and not subject to liquefaction, many did not and accumulatively would have cost the country and the insurers billions of dollars and resulted in displacing people from their homes.

Unreinforced concrete floor slabs generally tended to perform badly, tearing apart principally along saw cut control joints, and with little or no resistance to differential settlement, or rupturing under liquefaction conditions.

Never the less, to some degree unreinforced concrete slabs have attracted a worse reputation than they have deserved and many good houses have been condemned to demolition because of insurance policies that undertake to ‘replace as new”, and perhaps a lack of understanding of repair methods available.

If an unreinforced concrete slab has cracked. The logic goes; “How do you <u>replace</u> a cracked unreinforced concrete floor?” To do so one must remove and then reinstate the roof, walls and services. By the time one does so it is easier and cheaper to demolish the house and rebuild.

However this logic has tended to overlook that cracked slabs can often be economically repaired.

One aspect is that unreinforced slabs were often cracked due to concrete shrinkage before the house was finished, with the crack disguised under carpet and floor coverings. While the earthquake would have exacerbated matters, the truth is that the crack was often there before the earthquake and not noticed until people got nervous and started lifting carpets and tapping tiles. In one instance observed by the writer, the tiles in the house entry foyer were cracked about 1.5mm wide. However upon lifting the tiles it was found that the crack through the underlying slab was 4.5mm wide containing 3mm of flexible latex gap filler. Thus the slab had already cracked while the house was still being built. The crack had not been repaired and the tile layer had simply laid the tiles across the crack which virtually guaranteed that the tiles would crack if there was the slightest movement.

Another aspect is that concrete shrinks. In the laboratory, concrete shrinks about 1mm per metre, but in the field resisted by surface friction with the ground below, it may only shrink 1/6th of this amount and the concrete sits there in equilibrium with tensile forces built up within it. When an earthquake occurs, everything is shaken and the friction between the slab and the ground is disturbed allowing the slab to shrink like a stretched spring and the control joints and existing hairline cracks to instantly open up leading to the misunderstanding that either; the building has “spread”, or the earthquake has opened a fissure under the house.

It has generally been found that the DPC Membrane under the slab is unlikely to be compromised providing the crack is less than 100mm wide. Most cracks are less than 10mm wide and therefore waterproofing should not normally be an issue.

Observations have indicated that with successive shakes the slab reaches a point of equilibrium where the cracks grow no larger. In other words the concrete has shrunk as much as it is going to. At that point the control joint has done its job and it is probably best sealed solid with a 2 part epoxy or hybrid Urethane grout.

One suitable proprietary product is a modified Urethane developed for NATO to repair bombed airfields. Fine silica sand is poured down the crack as a filler and this 2 part hybrid urethane, with a surface tension 1/3rd that of water, is poured in which penetrated the sand and deep into micro fissures of the concrete to permanently micro-weld the broken concrete back together. This product sets hard in 10minutes and within 24 hrs has to be removed with a grinder. It is a very good product which I have used myself and highly recommend. It is not cheap and the product would cost about $2,000 to do a 200m² house, but if it can make the difference and save a $300,000 house from potential demolition it is very cost effective.

Another alternative is to maintain the control joint as a flexible joint using a backing tube of foam and a flexible filler. However this does not provide good overall diaphragm action of the slab or provide resistance against vertical shear.

Retro reinforcing of an unreinforced slab is possible. As a precaution an unreinforced concrete floor which is covered with carpet, can be retro-reinforced using stainless steel 1mm thick brace strap laid across the house and fixed to the concrete floor using concrete nails in predrilled holes. Usually the local form of construction has reinforced perimeter strip footings with D10 starters bent into the reinforced slab at 600mm centres. The idea is to well nail one end of the strap in this zone, then stretch it across the house through doorways or under internal partition walls at about 5m centres. The brace strap is kept temporarily taught by use of a temporary spring at the other end hooked onto a temporary nail protruding from the concrete.

A slow settling non-brittle adhesive can be pasted between the concrete and the brace strap and temporally weighted down as holes are drilled and nails hammered into place holding the strap to the slab, commencing at the nailed end and working towards the spring so as to keep the brace strap taught.

Alternatively or as a supplement to retro reinforcing right across the building, the joint or crack can be stitched using 600mm lengths of brace strap at 1m centres at right angles across the joint of crack. If nailed at the ends and left “free” at right angles across the crack, the brace strap will stretch over say a 400mm length approximately 4mm and remain elastic drawing the joint together again when the shaking stops. The steel strap forms a frangible connection which will fail rather than causing the slab to crack elsewhere under ultimate load.

As a guide the tensile strength of the perimeter of the slab reinforced with D10’s at 600mm centres will be about 39 KN/m width. The 20 MPa concrete slab 100mm thick should have a tensile capacity of about 20KN/m width and the brace strap should have an ultimate strength of 8.8 KN, hence if placed at 1m centres the brace strap will be the weak frangible link saving the concrete in the case of overloading.

This photo is of a previously "written off" house with cracked unreinforced concrete floors, saved from the demolition by crack repair methods and brought back to code compliance condition. In this particular case there was no liquefaction on site and floor slab levels were within new house tolerances and did not require relevelling. This is now the writer's new residence.

The neighbouring house with the gray roof was also a "Write off", but this one although "Safe and Sanitary" has differentially settled 54mm. It is intended that this be relevelled in due course using ground injection densification and lifting techniques.

REINFORCED CONCRETE SLABS

Reinforced slabs have historically been reinforced using 665 mesh which was made of drawn wire welded in a 6" or 150mm square grid. Because the wire was drawn in manufacture it had already been stretched past its yield point and although strong was brittle and was prone to failure across a narrow crack under seismic conditions. Further, it was often local practice to cut control joints so deep that the mesh was often cut or nicked and compromised, thereby providing no shear or tensile capacity across the control joint, rendering such slabs little better than unreinforced slabs.

However 665 mesh was better than nothing particularly if left continuous across control joints.

The writer has viewed one slab which cracked only part way across an otherwise unreinforced concrete driveway near the Darfield epicentre. A check with the metal detector revealed that the crack stopped in an otherwise unreinforced slab where a sheet of 665 mesh left over from the house had been laid at random in the drive.

Within the parklands neighbourhood, while most houses had unreinforced slabs, I am aware of 4 that were reinforced with 665 mesh and their performance compared with their neighbours is of interest.

The first building, a 2 storey house with a 665mesh floor and ply bracing survived with virtually no damage while single storey neighbours were write offs.

The second building, another 2 Storey house with a 665 slab and a light weight roof in a particularly bad area (M 5.7 Liquefaction) tilted diagonally100mm with a horizontal crack along a gib board joint in the bathroom and a few cracked bricks at the back of the garage the only apparent damage. The neighbouring house was a write off.

The third building a single story house with a 665 slab had a substantial drop out under one corner running back about 2 to 3 m along each wall. This has been temporally wedged up with timber with little apparent damage where the neighbouring house with an unreinforced concrete floor is a wracked and broken write off.

The fourth building was a single story heavy reinforced block masonry single storey house. This one however straddled a 105mm wide ground fisher which opened up running under the house and with conventional footings at both sides tying the house into the ground like grader blades, the house was literally torn in two despite the 665 mesh floor.

As a result of the Christchurch earthquake, 665 mesh has how been phased out in favour of seismic mesh, which is commendable and will enhance floor slab performance in the future.

However, it should be noted that larger seismic steel has not performed as well in Christchurch as anticipated possibly due to the concrete, which tends to be made with river gravel, developing isolated concentrated cracks, forcing the steel to yield at one point rather than micro cracking over a longer development length of steel which is more likely to occur where crushed aggregate has been used.

With on-grade slabs there would therefore be merit in de-bonding the steel for 50mm each side of the control joint, (Denso tape wrap the steel locally, or by using crack inducers (CANZAC or similar) in a grid across the slab at about 750mm to 1m centres to induce micro cracking as used in supermarket floors.

WAFFLE OR “RIB-RAFT”TM SLABS

Waffle or Rib-raft Slabs (see photo) historically have been constructed as a deep slab typically 305mm to 385mm total thickness on top of the ground, without conventional foundations dug into the ground. The waffle slab consisting of a grid of reinforced ribs in both directions typically 100mm wide with a 300 or 400mm wide reinforced concrete perimeter beam, with a reinforced topping slab typically 85mm thick, all poured in one concrete pour, with void spaces formed typically 1.2m x 1.2m using polystyrene pods set out in a chess board fashion separated by the ribs.

Waffle slabs are typically about 17 times as strong as a conventional reinforced concrete slab and foundations while using a similar amount of concrete plus nominal additional steel. This extra strength is due to the depth of the ribs.

The other advantage of waffle slabs is that they are relatively light compared with a solid concrete slab of similar thickness. This is seen as a plus during liquefaction conditions where a heavy slab will tend to sink under its own weight. Further on soils prone to liquefaction, the heavier a building the more disturbance the buildings inertia is likely to cause to the underlying soils thus encouraging liquefaction.

Prior to the Canterbury earthquakes, Waffle slabs were not common in Christchurch. However the irony is that in particularly bad areas because they were particularly bad areas waffle slabs were occasionally used and there were sufficient to monitor performance.

Between one supplier of Rib-raft TM pods, [2] and the writer approximately 28 rafts were reviewed. From investigation they all performed well structurally with no rupture or significant structural damage. Some draping and tilting did occur in some cases but this was redeemable.

It needs to be borne in mind however, that all these slabs were designed using what were the relatively light designs of the day and all performed well before DBH climbed into the act with the current criteria which add unnecessarily to the cost of such slabs. The key problem with the current DBH criteria is the requirement for the slab to cantilever for 2m over some imaginary gaping chasm and the ripple on effects.

For a start, such a design is basically red zone material where one is not allowed to build anyway. Even so existing relatively lightly reinforced as they were, waffle slab buildings in Red Zones survived remarkably well. The present design is complete overkill. We are looking at a design that will survive an ULS Design Event in a Red Zone when a percentage of slab damage could and should be expected if over all nationwide economics are considered and optimised on a cost-benefit basis.

Because of the 2m cantilever requirement, the slab needs to be about 400mm deep or deeper, bringing into play code requirements for shear steel event though the concrete is strong enough to take the shear in an ULS event with the slab sitting on the ground where it is not going to fall down and kill somebody.

In the guideline the DBH also published a typical cross section of a waffle slab and claimed that if it were used, specific design was not required. A quick check of the typical cross section reveals that it is far too light to measure up to the DBH's own Guideline criteria.

Looking at the house as a whole, the secondary effects such as the deep beam effects of walls are not taken into account. With the HEPIG grouting of the house in Auckland, (our job no 6682) previously mentioned, as an aside a 305mm deep nominally reinforced waffle slab had been used and effectively in combination with the deep beam effects of the walls had draped but was effectively cantilevering about **3m**. This was using a slab designed to cantilever about 900mm! The house was restorable and was relatively easily made good to the satisfaction of the owner and the tenants.

I have not done a costing but I would anticipate that if the current DBH specific design guidelines are religiously followed, the waffle slabs for TC2 areas will cost about twice what they should cost. I would recommend that the cantilever requirement be reduced to 1.3m

Because historically waffle slabs were marginally more expensive than conventional slabs and recognised as being superior, they were nearly always used on potentially problematic sites. In some bad areas of liquefaction they sometimes had a tendency to tilt (the worst the writer observed was 150mm) as did other slabs on grade. They did however keep their integrity without rupture or damage and the house remained liveable, functional and restorable.

In badly hit areas failure of public infrastructure including sanitary sewer and water supply applied to all house construction types, and temporary supply of portaloos and water tankers was provided. However relatively lightly designed waffle slabs served their function and remained intact.

The following excerpt from the DBH guidance document for earthquake reconstruction- is of interest;

"An observation from the Canterbury earthquake is that there are significant advantages in people being able to remain in their homes for as long as possible after the event. This means employing building practices to limit the damage so that buildings remain habitable and ultimately gain a Green (Inspected) placard from council. Encouraging wide, stiff foundation systems such as stiff rafts (eg, waffle slab) or stiff inter-connected footings is considered to be the best way of improving performance with respect to both amenity and collapse, and thereby improving homeowners' confidence in repairing or rebuilding these locations."

Waffle slabs are cast on top of the ground rather than having foundations. However they did not tend to slide as one would imagine. In one case movement was measured relative to services after the 7.2 magnitude earthquake and found to be 5mm in one direction and 10mm in the other.

This lack of sliding is thought to be possibly due to a "Limpet" effect where atmospheric pressure bearing down on the top of the slab without the chance of air rapidly getting underneath is sufficient to hold the slab down. For whatever reason waffle slabs were not observed to slide and ground shear keys were not used.

There are advantages in casting on top of the ground as should a rupture occur directly under a building the waffle slab is free to slide rather than be pulled in half having foundations locked into the ground each side of such a rupture with conventional slabs on grade and strip footings.

Waffle slabs have enough integrity to be relevelled should such be required. It is further anticipated that it should be possible to cut if necessary and uplift, transport and relocate a waffle slab home complete with brick cladding. However where a heavy tile roof (10 tonnes) has been utilised, it would be best to replace it with a light weight roof (1.7 tonnes).

Waffle slabs if adequately reinforced (recommend DH16s in the bottoms of the ribs) and less than 25m long act as one monolithic block and do not tend to crack. This may be partially due to being built on a polythene sheet on top of the ground without foundations locking it into the ground like grader blades and not permitting the concrete to naturally shrink and take up dry shrinkage.

Historically for slabs over 25m long, a dowelled construction joint has been recommended. However, under seismic conditions this compromises the integrity of the building. With larger slabs the writer considers that it would be preferable to set up a grid of crack inducers across the slab under the mesh on top of the polystyrene pods at the points of contra-flexure of the topping slab spanning between ribs but in such a manner as to not compromise the edge cantilever action of the slab.

The writer is firmly of the view that the current requirement for waffle slabs to cantilever 2m in a TC2 Zone is unreasonable and should be reduced to 1.3m for design purposes to optimise cost / benefits.

TILTING

Under severe liquefaction conditions buildings founded on slabs on grade can tend to tilt. However this tendency does not tend to happen on light timber framed buildings raised off the ground and founded on shallow pile foundations. Possibly part of the reason is that under liquefaction conditions ground water tends to migrate upwards. Under a house on piles it can come up and flood the subfloor area releasing local pore-water pressure. Whereas under a slab on grade water is entrapped without adequate drainage, leading to the slab acting like a “waterbed”, very sensitive to weight distribution, like a top heavy boat.

Liquefaction also does not tend to occur evenly, as evidenced during the earthquakes by liquefaction on the roads causing cars in several instances to partially sink with the front disappearing down into liquefaction and the rear wheels still on firm pavement.

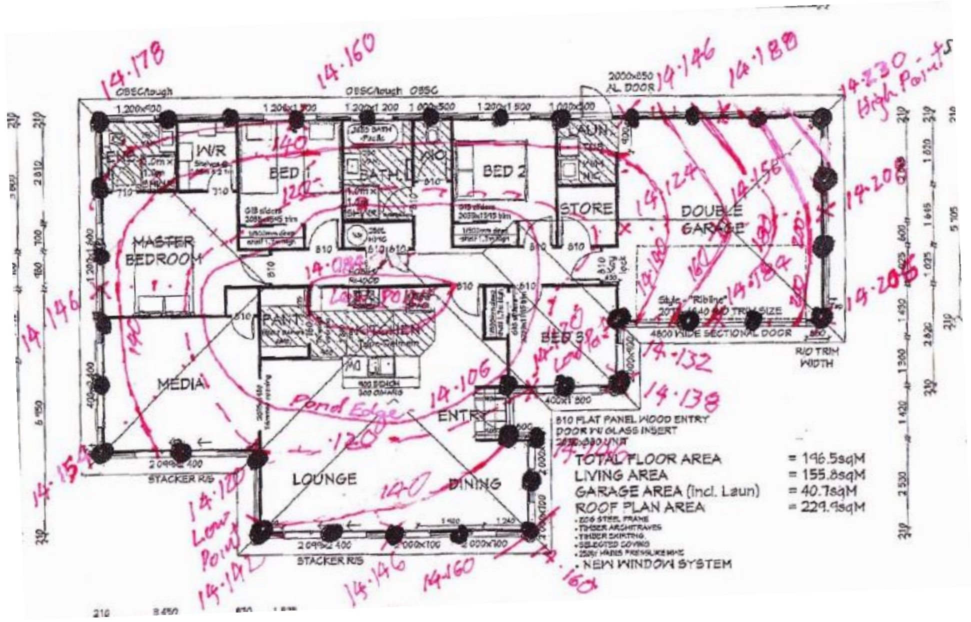
The secret to overcoming the problem of tilting would be to build a reinforced earth sub-grade raft extending out beyond the foot print of the building perhaps 3m all around and of the order of 300mm to 1m thick. Like a boat this will give the structure wider beam and keep it more stable under liquefaction conditions. In addition some consideration should be given with respect to incorporating a lightweight fill into such a reinforced subgrade rafts such as large 4.2m x 1.2m x 0.6m polystyrene blocks as used in the construction of the Albany Motorway.

Such a subgrade raft should be constructed of compacted graded metal (not marbles) containing interlocking fines but free of plastic fines, founded on a layer of a combination

Geogrid/ filter cloth which has very high strength at low strain, immediate interlocking characteristics and works as a filter cloth / drainage blanket keeping liquefiable fines from polluting the subgrade raft. Possibly subsoil drainage should be run through the compacted graded metal to relieve pore water pressure build up.

Buildings should also be considered like ships, to ensure that the centre of gravity is not eccentric from the centre of foundation support or tilting could result. Buoyancy should also be considered and light weight materials or compensating light weight fill or a shallow or partial basement should also be considered.

PILING



Piling does work well in combination with a waffle slab flooring system usually supporting every second nodal point, where liquefiable materials are shallow and competent load bearing gravels are near surface such as at Halswell where good bearing is located at depths of typically 1 to 2m. Piles however can be worse than useless where deep liquefiable soils are encountered such as in the eastern suburbs of Christchurch where potentially liquefiable soils are as much as 32m deep before peat is encountered, and depth to suitable gravel is uncertain. In such an area the writer has seen timber piles probably about 6m long used as friction piles, around the perimeter of an ordinary reinforced slab only (this was <u>NOT</u> a waffle slab) sink differentially of the order of 120mm. The piles then lock the structure in place preventing relevelling of the building using grouting techniques.

On the other hand on the St Andrews school site a new building was piled to an intermediate depth of 7m to competent gravel. The 22 February 2011 earthquake then occurred and the upper poorly consolidated soils liquefied and consolidated 200mm, taking adjacent and interconnected buildings, services and access down with it, leaving the new building sticking out of the ground by 200mm. Thus making the piling “cure” worse than the “disease”.

Christchurch central is underlain by the Riccarton gravels at a depth of 20 to 25m, but even if it were economic to pile to such depth for a house, the layer is an aquifer with its own set of problems, and differential settlement within these gravels could still occur.

Lateral flow of the ground of over 1m has been observed by the writer and under such circumstances the stresses and eccentricities induced into piles would probably render then worse than useless.

The DBH TC3 Guideline in Table C5.3 indicates that where lateral displacement is likely to exceed 300mm piles are inappropriate. Such Lateral displacement in a ULS event can relatively easily occur through much of the TC3 areas if one does the calculations.

Piling therefore is not a silver bullet in liquefiable ground and needs to be thought about carefully. Skin friction under seismic conditions cannot be relied upon and end bearing at times can be dubious. There is also risk that settling liquefiable soils can leave a building out of the ground dislocated from services etc. There would be merit in providing a socket end to piles so that should differential settlement occur they would not hold the building down but would permit relevelling using a grouting method.

There would also be merit in retro adjustable piles, the heads of which would be able to be accessed from within a building so as to adjust its level if deemed necessary.

BASE ISOLATION

Although Base Isolation was practiced by the Ancient Greeks and more recently was largely developed in New Zealand by Robinson Seismic of Wellington, it unfortunately has not been as widely used in NZ as overseas.

When one considers the damage sustained, base isolation even for residential buildings needs to be seriously considered and a cost effective solution developed.

The writer has given this matter serious thought, but has not as yet unlocked the riddle.

To put Base Isolation into perspective, it can reduce the impact experienced by the structure and its occupants and contents, significantly reducing the impact by as much as 70% to 90%. Many of the older double brick buildings if they had had such a foundation would still be standing. Pallet racks would not topple, fridge doors would not fly open spilling their contents, many buildings would not have collapsed or been write offs, lives would be saved, injuries reduced, property damage minimalised, insurance premiums would be kept within reason, etc.

One however has to be aware that the natural frequency of some base isolation systems can be dangerously close to the longer critical periods induced in the Christchurch case by the deep sedimentary bowl effect under the city. There is therefore the risk that some base isolation systems could without appropriate damping could resonate making the situation worse than without them.

The other concern is that someone will sooner or later fowl it up by filling in a seismic gap or doing a non seismic extension or planting a tree in the wrong place. Christchurch excluded, the frequency of need may be so small that it is not worth the effort of incorporating base isolation for NZ domestic houses as they are reasonably resilient.

While base isolation would protect contents, NZ houses on the whole, if built in general accordance with NZS 3604, but taking into account that liquefiable land does not constitute “Good Ground”, the latest guidelines and code amendments, and particularly if incorporating a light roof and a light waffle slab foundation are likely to perform exceptionally well.

CONCLUSION

The humble domestic concrete slab is a very important component in getting things right first time. Because there are so many of them, the cost of poor detailing and construction techniques have had a significant effect on the economy and wellbeing of the community running into billions of dollars worth of damage in Christchurch and causing displacement and trauma to people unnecessarily.

Over 10,000 homes have been written off and a high percentage of these, particularly among newer homes are due to poor slab design and construction. This has been the “Achilles heel” of houses built in accordance with the New Zealand Standard NZS3604 which otherwise have generally fared well.

However many habitable houses with unreinforced cracked concrete floors are being written off unnecessarily, due to the wording of insurance policies and ignorance with respect to the reparability of such.

Unreinforced slabs are now no longer permitted for new construction of residential houses.

Experience has proved the worth of even nominally 665 reinforced slabs and the new seismic mesh on the market as a result of findings from the Christchurch earthquakes should certainly improve things further.

Existing light waffle slabs constructed prior to the earthquakes all appeared to have functioned extremely well, maintaining the integrity and habitability of the building, although in a few cases due to insufficient detailing, tilting occurred as was the case with other on grade slabs.

The current DBH criteria of requiring waffle slabs to cantilever 2m for specific design in TC2 areas is unreasonable over kill in view of performance and risk, and is therefore contrary to the intent of the Resource Management Act which requires that costs and benefits be weighed. A lesser Cantilever requirement of 1.3m in view of secondary effects would seem reasonable.

Relevelling of slabs using grouting methods is practical but does set up some challenges with weak sub grades to react against in the Christchurch area. However in densifying sub-grades to react against for relevelling, the seismic performance of the building platform and the foundations and slab, are likely to be enhanced.

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