MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

File No. DOIA 1819-0239

12 SEP 2018

Mr Jake Preston
fyi-request-8510-94430293@requests.fyi.org.nz

Dear Mr Preston

Thank you for your email dated 16 August 2018 requesting under the Official Information Act 1982 (the Act) the following information:

> *In item number 28 of the Minister Briefing document dated 2013, and referenced 432 12-13 the author states that the guidance documents had been reviewed by international experts.*
>
> *I would like to request qualifications of the international experts that provided reviews. I request all correspondence engaging these international experts, along with any terms of engagement, and all correspondence from these experts including all drafts reviews as well as final reviews provided.*

Searches of the Business, Innovation and Employment's information system found three documents within the scope of your request and are being released to you. Some information has been withheld under section 9(2)(a) of the Act, to protect the privacy of natural persons.

Please note that the Final Report from the international reviewers, dated 25 May 2012, states:

> *The reviewers are satisfied that the DBH response addresses reviewer comments at an appropriate level of care and detail, and that the TC3 guidance report of 27 April 2012 reflects suitably the reviewer's comments.*

The Final Report from the reviewers then proceeds to record comments they had made when commenting on earlier drafts of the guidance. It is important to note that all these comments were acted on prior to the final 27 April 2012 document. The Department of Building and Housing response document, attached, clarifies the response to each of the reviewer comments, demonstrating how they have been addressed in the final document. This is acknowledged in the reviewer Final Report and in the accompanying email, attached, from Professor O'Rourke dated 27 May 2012.

Where information has been withheld for section 9 reasons, in terms of the Act, I am satisfied that, in the circumstances, the decision to withhold this information is not outweighed by other considerations that render it desirable to make the information available in the public interest.

You have the right under section 28(3) of the Act to request a review by the Ombudsman. The relevant details can be found here: www.ombudsman.parliament.nz

Yours sincerely

Dave Robson
Manager, Building Performance and Engineering
Building System Performance

s 9(2)(a) [redacted]

From: Mike Stannard
Sent: Thursday, 7 August 2014 10:14 a.m.
To: s 9(2)(a) [redacted]
Subject: FW: Final Report with Review Comments for DBH TC3 Guidance Document
Attachments: 25 May 12 Final Review Report DBH TC3 Guidance.docx; 25 May 12 Final Review Report DBH TC3 Guidance.pdf

From: Thomas Denis O'Rourke [mailto:tdo1@cornell.edu]
Sent: Sunday, 27 May 2012 1:46 a.m.
To: Mike Stannard
Cc: Jonathan D. Bray (bray@ce.berkeley.edu); Misko Cubrinovski (misko.cubrinovski@canterbury.ac.nz); Kate Percy; Peter Highton; David Kelly; s 9(2)(a) [redacted]
Subject: Final Report with Review Comments for DBH TC3 Guidance Document

Hello Mike,

Attached please find a final report with our review comments on the DBH TC3 guidance document. We are well satisfied that the DBH response addresses our comments at an appropriate level of care and detail, and that the new TC 3 guidance report reflects suitably the reviewers' comments.

The report adds to the version of our report, dated 27 April 2012, with our acknowledgement of the DHB response to reviewer comments of 9 May 2012. Our report contains a new Section G that provides clarification of select issues as requested in the DHB response as well as a few additional clarifications we identified after our examination of both the DHB response and currently available TC3 guidance document. I am attaching both pdf and Word versions.

We are pleased to help with this important project, and again extend our sincere good wishes for the Christchurch recovery.

Please confirm receipt by return e-mail.

Best regards,

Tom

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DBH Response

Review of Interim Guidance for Repairing and Rebuilding Foundations in TC3
by
T D O'Rourke, J Bray and M Cubrinovski, 16 April 2012 & 27 April 2012

A. Review Comments on Lateral and Vertical Ground Movement Effects on Structures:

1. Ground movement

- Ref Fig 1 – diagram showing effects of vertical and horizontal differential ground movements (Boscardin & Cording)
- Combined effect of differential settlement and horizontal ground strain is more damaging – see Table 3.1
- Lateral stretch is most important in terms of building damage
- Stiff foundation slab best at reducing lateral stretch and foundations that allow for horizontal slip
- Refer Japanese examples – successful use of polyethylene sliding layers
- Even a modest slab can provide restraint against lateral strain
- Foundation designs that tend to anchor are less effective

DBH Response:
We have taken these comments into consideration in looking at options for stiffening of foundation slabs and response to lateral stretch. New designs have been proposed that include polythene sliding layers and stiffening of foundations slabs. Specifically, the guidance proposes new options which include:

- Several ground improvement options now include two layers of geogrid to allow the solution to be used in areas of major lateral stretch (eg stabilised crust methods Types 2A and 2B)
- Type 2 surface structure foundations – flexible re-levellable platforms (Type 2A and 2B) both include two layers of polythene as well as two layers of geogrid reinforcement
- Type 3B surface structure foundation – steel beams over pre-stressed concrete beams (stiff platform) includes a polythene slip layer beneath the beams
- Table C5.5 in the Guidance Document provides a summary of which of the surface structure types proposed in the document is able to respond to differing levels of vertical land settlement and lateral stretch

2. Ejecta

- Most damaging aspects of liquefaction is soil loss through ejecta – TC3 Guidance Document does not specifically refer to this source of ground movement
- Reviewers unanimous that primary cause of ground deformation and resultant structural damage is ejecta

DBH Response:
The Guidance Document has been amended, by adding a new section on design principles, to identify that liquefaction ejecta results in soil loss and that this is the

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a primary cause of ground deformation, while acknowledging that quantification of its effects are not currently possible. It has also been highlighted in the document that foundation options that disturb the crust will cause liquefaction ejecta, such as where piles are driven or some surface structure options. A statement has also been added in the design principles section that minimising penetration of the site will lessen liquefaction effects.

3. **Settlement calculations**
   - Focus on performing liquefaction-induced settlement calculations at just SLS is problematic
   - Liquefaction is a brittle phenomenon
   - I&B magnitude scaling factors should have been used – calculation method is suggested in text

DBH Response:
Settlements are routinely assessed at both SLS and ULS. The requirement for foundations at ULS however is life safety, and the recent events have demonstrated that this particular requirement is met by even the most rudimentary foundation system. The DBH foundations systems are however far more robust than this, to provide for both life safety as well as repairability at ULS. To this extent, calculating the actual magnitude of ULS settlements in the TC3 zones (remembering that the areas of the worst damage have been red zoned and therefore are not addressed in this document) is not quite as important as trying to determine likely movements at or around SLS, where the requirement is to 'preserve amenity'. This is in fact the more difficult aspect to achieve, and therefore the document does place a bit more emphasis on it. Our aim is to provide 'ready repairability' at SLS. We are aware of the sensitivity of settlement calculations at 'a little over SLS' – however in TC3 typically it is not a 'brittle' on/off phenomenon – almost all sites will show liquefaction settlements calculated from a little under 0.1g, to about 0.3 – 0.35g where increases in settlements typically start to decrease rapidly.

We are treating the calculated settlements more as 'indices' of where a property might sit in the spectrum form 'good' to 'bad', and using an arbitrary cut-off point at 100 mm to subdivide into 'better or worse', to aid in choosing a level of robustness for the foundation system. The GNS study indicates an SLS PGA of as low as 0.1g depending on assumptions; to build in a degree of conservatism 0.13g was chosen as the design value – this both recognises the 'steepness' of the settlement vs PGA curve, and allows for other aspects such as the magnitude scaling factor issue mentioned. It has also been noted that measured settlements tend to be over estimated by most methodologies in the range we are particularly interested in (ie, the SLS range).

4. **ULS event displacements**
   - Statement that structures which can tolerate lateral displacements in a ULS event are unlikely to suffer significant damage due to lateral ground movement at SLS is overly optimistic

DBH Response:
Surface structures Type 1 and Type 2 have been designed so that either the soil adjacent to the buried foundations is pushed aside in a spreading event (Type 1) or slide over the stretching ground (Type 2), thus accommodating expected ULS spreading, and minimising damage which might occur should ground movement

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occur in smaller events. The guidance document has been updated to clarify that such foundations are expected to "tolerate" ground movements which might occur in a SLS event (within the target performance expectation for SLS), rather than necessarily be entirely undamaged.

To provide an indication of the low level of lateral spreading damage that could be expected at SLS in most parts of TC3, it is useful to consider the September 2010 earthquake, which subjected most of TC3 to levels of shaking modestly greater than SLS-level. In this event liquefaction occurred extensively alongside the Avon River and smaller waterways and free-faces, however significant lateral spreading was only observed in a limited number of such locations. This suggests (at least for the local situation) that a certain severity of liquefaction and strength-loss is required before significant post-seismic lateral spreading occurs under static driving stresses.

The vast majority of these most-susceptible locations have since been placed in the Residential Red Zone by CERA, and so are excluded from TC3. Therefore the land in TC3, where lateral spreading might occur in large earthquakes but was not observed in September 2010 (or February 2011 in many cases) despite liquefaction occurring, could generally be considered to have performed adequately from a lateral spreading perspective when "tested" at SLS-level. This "testing" at SLS has been in terms of both the intensity of shaking (driving co-seismic movement), and the severity and extent of liquefaction strength-loss and void redistribution (driving post-seismic movement).

5. Ground stretching

- Item 3 suggests that in some areas of significant suburb-wide global lateral movement, only minor ground stretching may be observed – evidence of this should be provided.

DBH Response:
Item 3 is intended to apply to the two specific suburb-areas listed in Table C2.2, where co-registration analysis of LiDAR data has identified suburb-scale ground movements which appear to be driven by the gently-sloping sand dune topography. The statement in Item 3 regarding minor ground stretching is intended as a background explanation to engineers why they might not observe ground cracking effects when they inspect sites in these specified areas, rather than as a general rule for all areas of major global movement.
In the next revision of the guidance document, we will rephrase this sentence to make it clearer that this is a specific rather than general case.

6. Foundation performance

- Need more evidence in document about the performance of foundations, particularly piles

DBH Response:
Our assessments regarding the ability of the several "approved" pile types to withstand global lateral movement have been based on our computations involving standard pseudo-static analysis procedures for kinematic interactions for in-ground pile:

1. We have made simplified assumption regarding a typical soil profile (described in footnote to Table C5.3)
2. We have assigned simple bi-linear p-y curves for each soil layer using standard correlations
3. We have assumed typical pile material properties

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4. We have assigned a simple soil deformation pattern for the case of 300 mm ground surface lateral movement (parabolic).
5. We have used a "Winkler spring" model to calculate the pile bending moments.
6. We have applied judgement as to whether or not the resulting pile strains are acceptable

Note that we are still working on the concrete pile analyses.

While it would be desirable for these analyses to be fully reported, referenced, and reviewed, we have not had the time or resources to do so to date.

It should also be noted that there are not many observational reports to draw on as yet for Christchurch as not all of the foundation options outlined for the TC3 sites have been constructed and tested.

B. Review Comments on "Deep Pile" Design Concepts:

1. Implication that deep piles preferred

- Deep pile foundations may not perform as well as concrete raft slabs
- Loss of ground due to the formation of liquefaction ejecta can lead to ground damage

DBH Response:
It was not the intention of the Guidance Document to imply that deep pile foundations are a preferred option. This has been made clear in the document.

2. Lateral stretch

- Should consider horizontal stretch – Table C5.2 should include lateral stretch under deep piles

DBH Response:
We agree that lateral stretch will be a serious issue for deep piles. We are in the process of assessing typical details to potentially make sliding connections at the pile heads because we do not think that piled foundations will be able to withstand the 200 mm lateral stretch across the building footprint.

Areas have been identified where these options (ie, fixed pile heads) are more suitable (ie, less severe areas of lateral stretch).

The amendment to include lateral stretch for deep piles in Table C5.2 was agreed and the table will be changed accordingly.

3. Evidence to show piles are resilient / ductile

- Need evidence about resilience / ductility of deep pile types with respect to lateral movement, vertical movement after hinge formation and p-delta effects.

DBH Response:
The lateral movement issue has been addressed under item A.6 above.

Vertical movement equals 8 mm for our typical case, which we have decided to ignore.

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We have studied Bhattacharya et al. 2004 and make the following comments: These TC3 piles are intended for residential purposes only and carry very light axial loads compared to their material capacity. For instance, a 150 diameter screw pile stem will only be carrying factored axial loads of about 10% of the Euler buckling load even assuming that the pile is pinned at the top and bottom of the assumed 6 m thick liquefiable layer. The authors of the paper stress the prime importance of the Euler buckling case and consider P-delta effects as a secondary concern.

4. **Deep pile-slab connection**
    - Need more detail on deep pile-slab connection
    - Design assumption for deep pile head fixity should be stated

    DBH Response:
    Prompted by the comments in your review, further consideration is to be given to the pile-slab connections for locations where significant lateral stretch has occur or could occur. This will include further modelling of pile head details. The initial version notes (p 48) that conventional pile to slab connections can be used where there is no evidence that lateral stretch has occurred at the site (in areas that can be judged to have been well tested at SLS).

5. **Capacity design equation**
    - Simplify equation (section C5.2.5) by deleting down drag term
    - Should emphasise the critical design check for deep piles is that the neutral plane under the anticipated loads remains well within the bearing stratum below any liquefiable soils
    - Critical that deep piles are driven into bearing stratum
    - Document should warn that jetting/pre-drilling must not be used to advance fully the deep pile to its target depth

    DBH Response:
    We have simplified the design procedure further by removing the need to consider down-drag for driven piles. However, we believe that it should be considered for bored piles and CFA piles (we have discouraged the use of such piles).

    We agree with the discussion on neutral plane and have already included a requirement that the pile resistance should only be calculated from end bearing and side resistance within the bearing layer. Requirements for depth of embedment into the bearing layer are included in the Meyerhof procedure.

    We have provided additional emphasis on the need to drive the pile into the bearing layer with a suitable hammer.

    In addition, in the Guidance Document (Section C5.2 Deep piles) we have included the point that it is critical that deep piles are driven into the bearing stratum.

6. **Settlement issues for bored piles**
    - Load transfer mechanism of bored piles differs from that of driven deep piles

    DBH Response:

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We have repeated the discussion on problems with bored piles (loss of side resistance and settlement etc) in both CFA and bored pile sections of the Guidance Document.

C. <u>Review Comments on "Ground Improvement" Design Concepts:</u>

1. Compaction methods

- Use of a steel drum vibratory roller compactor should also be specified to form a more resistant soil fabric

DBH Response:

The QE2 trials were carried out at 95% standard compaction and the results were satisfactory. While we agree that higher levels of compaction would be better, the use of a vibratory roller in the base of the excavation, where the water table will be directly below or within the layer being densified, typically leads to ground heave problems in the slightly silty Christchurch soils. Most compaction equipment is vibratory, so will likely be used where appropriate - we have specified the use of vibratory compaction methods for shallow foundation treatments (eg densified raft of re-compacted soil or replacement fill) when sufficiently clear of the water level to avoid fluidising or heaving the ground.

2. Geogrid

- Need two layers of geogrid to improve performance

DBH Response:

We have adopted this advice and are proposing the use of geogrid layers in the Guidance Document. We are recommending two layers of geogrid as suggested to improve the performance of the densified soil.

3. Deep soil mixing

- Deep soil mixing must extend through entire thickness of potentially liquefiable soils

DBH Response:

The requirements have been adjusted to extend the depth of treatment where the depth of liquefiable soils exceeds 10 m. Geotechnical members of the advisory team consider deep soil mixing to 8 m provides an acceptable result for typical residential dwellings (where the depth of liquefaction is limited to this 10 m depth) and that mixing to a deeper level does not justify the extra costs incurred.

4. Densified or stabilised crust methods

- Densified or stabilised crust methods deliver improved performance if excavation extended 1.5m beyond footprint of structure

DBH Response:

Much discussion was held regarding the extension of excavations beyond the footprint of the structure. Constraints on extending the excavations from 1 m to 1.5 m included – cost (estimated as an extra $4000 per site for the 0.5 m addition); boundary issues (side yards) and the sloping of the excavation (1.5 m at base would equate to an extension of around 2 m on the surface). It was agreed that the extension beyond the footprint of a structure would be changed to a minimum of 1 m (at the base of the excavation) within the Guidance Document which would equate to around 1.5 m at the surface.

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5. **Type 3 improvements**
   - Need evidence re: Stabilised crust ground improvements with geogrids

   DBH Response:
   The guidance document has been updated to include the following options for concrete raft slabs in areas of major lateral stretch (but only where SLS settlement < 100 mm).

   Ground Improvement of either:
   Type 2a (stabilised crust excavate / mix / replace, plus 2 layers geogrid), or
   Type 3 (deep soil mixing)

   In conjunction with enhanced concrete raft slab option of either:
   Option 2 (thick slab), or
   Option 4 (waffle slab)

   Satisfactory performance is expected for these options because:

   - Ejection of material from beneath the foundation will be reduced, limiting the severity of local differential settlements. This is achieved by either providing confinement of the underlying liquefied material via the stabilised crust with tensile reinforcement (to improve toughness and limit the development of large open cracks), or by reducing the likelihood and severity of liquefaction of the underlying material via deep soil mixing.
   - If large-scale (multiple-block type) lateral spreading occurs, cracks will be encouraged to form in the unimproved ground in front and behind the building, rather than though the improved block beneath the building, reducing the stretch imposed on the foundation.
   - If more localised severe lateral ground deformation were to occur (such as edge-failure), or if the block of improved ground beneath the building was otherwise compromised, the robust raft slab foundation is expected to hold the building together and reduce the severity of stretching and differential settlement transferred from the ground to the structure.

D. **Review Comments on "Shallow Foundation" Design Comments:**

1. **Robust foundations**
   - A robust foundation that is not locked into the ground will reduce the level of stretch and movements of the ground that is transferred to the superstructure

   DBH Response:
   The foundation slabs have been designed to resist the forces expected to be generated by the friction of half of the structure on the underlying soil.

2. **Type 1 surface structure**
   - Concerns with SS Type 1 detail/concept – need to confirm integrity

   DBH Response:
   The Type 1 surface structure has been designed to bulldoze soil ahead of the isolated shallow foundations around the perimeter and discrete interior lines in the event of lateral spreading. In the event of lateral stretch up to 200mm

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beneath the floor, the isolated, unbraced piles will pivot about the connection of wire dogs and skewed nails between the pile and the bearer. Some re-piling is expected in the event of maximum expected spreading. The plywood bracing system on the perimeter and at discrete locations between (to ensure effective floor diaphragm load transfer) serves to resist ULS lateral shaking actions. Under vertical differential settlements, the dwelling is expected to distort but the distortions are expected to be manageable with the option of packing piles to recover a level platform. Layout constraints have been specified to minimise distortions in the superstructure.

3. Type 2 surface structure Concept 2.1 inertial load (Now called Type 3A)

- SS Type 2 – not clear how structure's inertial load considered in this design philosophy

DBH Response:
For the ULS case the superstructure will move over pads but within their boundary. Inertial loads have been considered. Net movement is not thought to be great at ULS. This concept is an option for specific design with the expectation that the engineer will design a foundation along the lines of the example but calculating inertial forces and providing appropriate fixings/stops.

This relates to what are now Type 3A and 3B concepts. In the former case (ie, Type 3A) the "brass anchor" bolted connection is essentially frangible at anticipated ULS "lateral spread" loads/deformation. Lateral ULS loads are transferred to the supporting blocks via friction between the bearers and the concrete blocks. There will be a DPC in place which will reduce the available friction and therefore a single M12 bolt has been added at one end of each bearer line to supplement the resistance.

4. Type 2 surface structure Concept 2.1 unknown spreading direction (Now called Type 3A)

- Concept 2.1 – same issues with unknown spreading direction. This solution not providing confidence for acceptable performance for ground-induced and inertia-induced loads/ deformations

DBH Response:
Should lateral spreading occur in the direction transverse to the bearers, the brass frangible connections are expected to shear, allowing the bearers to translate on the concrete blocks and the brackets can be re-fixed post event. As noted above, this option is a concept requiring specific design where the expectation is that the engineer will design a foundation along the lines of the example but calculating inertial forces and providing appropriate fixings/stops.

5. Type 2 surface structure Concept 2.2 inertial loads / spreading direction (Now called Type 3B)

- Type 2 Concept 2.2 – better version because stiffer but still issues with inertial loads and spreading direction

DBH Response:

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This option does have a preferred spreading direction, which is in the line of the pencil beams. The connection between the steel beams and the concrete pencil beams has been detailed more thoroughly (see Figure C5.20 in the final Appendix C) to ensure that a grid of beams is maintained. However, this is now identified as a concept for specific design. The now double layer of polythene beneath the pencil beams will aid the ability of the ground to spread beneath the beams without dragging the beams with it. Under inertial loads, because the beams are not buried deeply in the surrounding soil, some sliding may be expected. Removal of one layer of polythene would increase the frictional resistance and there will be a need to balance the resistance to inertial loads against allowance of the beams to move on the compacted hardfill under spreading actions.

6. Type 2 surface structure Concept 2.3 (Now called Type 2A and 2B)

- Type 2 Concept 2.3 – most robust

DBH Response:
This concept has been developed further taking on board peer review comments into two solutions where either a 150 mm thick or 300 mm thick slab is supported on a reinforced gravel raft foundation (now re-labelled as Type 2 surface structure foundations). In each case the timber piles are encapsulated in the reinforced concrete slab and the stiffness of the slab in combination with the reinforced gravel raft will serve to flatten any curvatures that may arise from differential settlement of the ground beneath the raft. The 150 mm thick "underslab" and raft is capable of resisting vertical liquefaction induced settlement of the land of up to 100 mm at SLS. The 300 mm thick "underslab" and raft is considered capable of resisting vertical liquefaction induced settlement of the land of up to 200 mm at SLS.

Lateral spreading in any direction is resisted by the reinforced concrete slab and both options can accommodate up to 250 mm SLS and 500 mm ULS spreading. Because the slabs are buried in the ground this will resist any lateral inertial forces. The piles may cantilever above the slab by up to 1 m maximum including the depth of the bearer and will resist lateral inertial loads without further bracing. A plywood skirt has been included around the perimeter of the floor plate to resist any service level displacements that may result from looseness caused by shrinkage of the timber piles away from the concrete.

7. Type 2 site investigation requirements

- All types of shallow foundations would benefit from proposed site investigation requirements
- It might be difficult to select appropriate foundation solutions or ground improvement methods if no specific information can be inferred from the area-wide investigations

DBH Response:
The overall philosophy is that deep investigation is required for piles but that surface structures are readily reparable therefore deep site investigations are not as crucial. For these options we will draw on the area-wide investigation data – this should provide CPT or SPT data that is very close to the site and allow the required assessment of settlements. We are expecting movement of surface structures but are requiring a site specific shallow investigation to check for 'normal' static issues such as organic layers etc. The footnote to Table 3.1 on

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p19 does allow for more investigation than is required at the discretion of the engineer.

The area wide investigation requirements specifically state that the number of investigation points can only be reduced (ie, from a nominal two per site) if the area wide results show consistency. Furthermore, the surface foundations have now been altered in such a way that requires deep investigations to be carried out - refer Figure C3.1.

8. **Polythene slip layer**
   - Figures C5.17- C5.19 – polythene slip layer is not shown, better to have 2 layers

DBH Response:
Two layers of polythene have been provided between the concrete slab and the reinforced gravel raft in the (new) Type 2 options. Concept design Type 3B has a single layer of polythene. One layer is sufficient and with the reinforcing present this could be dispensed with.

E. **<u>Review Comments on Methodology for Compiling DBH Foundation Technical Category Map:</u>**

General comments:
Detailed review has not been possible due to timeframes, and because the complexity of the analysis is difficult to completely explain and understand in a short methodology statement.

DBH Response:
The Technical Category analysis methodology will continue to evolve as additional information becomes available and a greater understanding is developed of the performance of the various foundation options. We would be happy to continue to liaise with the reviewers as this work progresses, to enable the complexity to be discussed in more detail.

1. **Seismic demand issues / weighting of factors**
   - Weakness in methodology – does not discriminate between areas subject to high seismic demand and those that were not; does not give proper weighting to data layers and types of information

DBH Response:
We agree that variation in seismic demand across the region is critically important to consider when interpreting post-earthquake observations. This was a point raised by one of the reviewers early in the development of the methodology, and was incorporated as a fundamental component of the analysis. However, the methodology document is perhaps unclear regarding the mechanism used to incorporate seismic demand into the analysis and this may have lead to some confusion.

There are two aspects of the TC analysis where the level of shaking which was experienced is incorporated:

- To normalise the damage observations – this is important for ensuring that different parts of the region are assessed fairly. For example, it would be

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unfair to automatically assume that a site with severe liquefaction is poorer ground than a site with minor liquefaction if the only reason for this difference is because the first site was closer to the epicentre and shaken twice as hard as the second.

- To determine the appropriate weighting of observations of actual performance in the earthquakes to date against theoretical predictions of performance based on analysis of ground conditions. For example, when assessing a site which was subjected to strong shaking (well in excess of SLS-level) and yet suffered little liquefaction damage, there is strong evidence to support the conclusion that significant liquefaction is unlikely in a future SLS-level event, regardless of what theoretical analysis of the ground conditions might suggest. However, the observation of a similar low-level of damage on a site where the shaking was well below SLS-level would provide little evidence towards an assessment of likely future performance, and a greater reliance would need to be placed on predictive analysis.

Normalisation of damage observations is achieved as follows:

- The performance expectation of land categorised TC1 / TC2 / TC3 is defined in terms of severity of land damage expected at SLS and ULS levels of shaking (Table 1, on pg 2 of the methodology statement, or Table 3.1 on pg 24 of the November 2011 guidance document).
- For the purposes of this analysis, the land performance expectation at intermediate levels of shaking between SLS and ULS is interpolated using a backbone curve to represent the typical predicted trend of liquefaction consolidation settlement (or expected severity of surface-effects of liquefaction in general) vs PGA for Christchurch soils – ie, a rapid increase in predicted severity from about 0.1g to 0.2-0.3g with any further increase in PGA only giving a slight increase in severity of effects. This is the basis for the values in Tables 1b and 2b on Pg 11 of the methodology statement.
- A similar approach is taken when considering performance expectations regarding building damage – ie, an expectation of only minor (readily repairable) building damage at SLS, and greater damage at ULS. Again this performance expectation is interpolated for PGA values between SLS and ULS (Table 3b on Pg 11), however with less non-linearity than for liquefaction severity above.
- The normalised index value (FDI) which forms the basis of the TC categorisation compares the observed land/building damage in each area in each earthquake against the performance expectation for TC1 / TC2 / TC3 <u>for the level of shaking that was experienced at that location</u> in the relevant earthquake. So in effect, the observed damaged data is normalised relative to the intensity of earthquake shaking which caused the damage.

Weighting of actual observations against theoretical predictions is achieved as follows:

- It was considered that even though the September 2010 earthquake caused less severe shaking and liquefaction in the Christchurch urban area than the February 2011 event, observations of areas which did or did not liquefy in September provide important information to differentiate highly susceptible areas from those which are slightly less susceptible.

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Therefore a reasonable weighting should be given to observations from areas of moderate shaking in September, rather than strongly favouring observations from strong shaking in February.

- It was identified that the weighting between observation and prediction for a given site should depend not only on the maximum strength of shaking experienced in any earthquake, but also the number of earthquakes which caused moderate to strong shaking. For example, a site which experienced strong shaking in both September 2010 and February 2011 could have very high weighting towards observation. If it had strong shaking in February but only moderate in September then a moderately-high weighting could be appropriate. Moderate in one and low in the other could give a moderate weighting, and low in both would give a low weighting.
- To achieve this objective, a weighting mechanism was set up which self-adjusted to the "quality" of the total dataset available at each residential property – ie, how reliably the ground was tested over the course of the various earthquakes, and the amount of data available (important because many properties do not have data for all the layers). So rather than setting the weighting factor for a layer based only on the level of shaking in the relevant earthquake, it also considered how the quality of data in a layer compares to the other layers. Absolute weighting values were assigned to each layer based on the level of shaking experienced (with a zero value for layers with missing data). The % weighting for each layer is then calculated as the ratio of its absolute weighting value to the sum of values from all layers. This may be better explained by the example table below, which outlines the calculation for the four earthquake combination scenarios outlined above.

Location	Experienced PGA (g)		Experienced CSR ratio to SLS		Layer weighting - absolute weighting values					Layer weighting - resulting % weight				Resulting overall weight	
	Sep-10	Feb-11	Sep-10	Feb-11	Sep land damage	Feb land damage	Building post-Feb	Crust & soil type	Sum of all values	Sep land damage	Feb land damage	Building post-Feb	Crust & soil type	Actual Observation	Theoretical Prediction
Heon Hay	0.26	0.42	1.51	[illegible]	[illegible]	0.26	0.26	[illegible]	[illegible]	26%	27%	27%	20%	80%	20%
Richmond	0.21	0.42	[illegible]	1.74	[illegible]	0.26	[illegible]	0.20	0.92	21%	29%	29%	22%	78%	22%
Mairehau	0.19	0.30	[illegible]	[illegible]	0.15	0.20	0.20	0.41	[illegible]	16%	21%	21%	42%	58%	42%
Belfast	0.19	0.20	[illegible]	[illegible]	0.15	0.07	[illegible]	0.55	[illegible]	17%	7%	17%	59%	41%	59%

2. Definition of categories

- Crust thickness practically only determinant for defining technical category in areas where the level of shaking experienced was low; other factors need to be considered

DBH Response:

While a weighting of near-100% would be given to theoretical predictions (ie, based on ground conditions) for areas where near-zero shaking was experienced, in practice the level of shaking experienced in the main residential areas was such that at least some weighting can be given to observations (see example of Belfast in table above).

The theoretical prediction layer is based not only on crust thickness, but also on whether the saturated shallow soils (2-5 m depth) are predominantly gravelly. The intention of this criteria is to ensure that all areas with near-surface saturated sandy/silty soils are classified as at least TC2. This is based on the general assumption that soils of this type in the Christchurch region are usually loose and susceptible to liquefaction. This assumption is potentially somewhat conservative, but was necessary given the time constraints for production of the initial TC categorisation. This assumption is considered appropriate at this stage given that the intention of the TC categorisation to provide a general screening tool to

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ensure that appropriately-resilient foundations are constructed, rather than to create a detailed liquefaction hazard map.
Nonetheless, as more geotechnical investigation data is collected and analysed, this theoretical prediction data layer will be refined to incorporate additional liquefaction evaluation detail.

3. **Building damage**
   - Building damage not necessarily related to land damage or liquefaction

   DBH Response:
   This limitation in the use of overall building damage information was recognised during development of the TC analysis, however at the time this was the only building-related data available (specific foundation-damage data has only recently been compiled). As an interim measure to filter out some of this shaking-only damage, for any sites where no or only minor land damage was observed, the building damage data was given a very low weighting.

   Also, the effect of this issue will have been reduced by the area-wide averaging process which was undertaken. This provides a moving-average over a 60 m radius window, so the influence of individual properties with shaking-only damage is diminished (although there is still susceptibility to area-wide trends in construction-type).

   When the TC methodology is refined in future, foundation-specific building damage data will be incorporated where available.

F: Additional Review Comments:

1. **Idriss & Boulanger (2008) monograph**
   - TC3 doc references I&B monograph but this does not reference the preferred method of Zhang et al.; use of this CPT-based method should be recommended

   DBH Response:
   Have now included use of Zhang et al. in the Guidance Document instead of the Yoshimine (2006) method from Idriss and Boulanger

2. **Use of I&B without soil samples**
   - Means for using I&B procedure without retrieving soil samples should be provided

   DBH Response:
   Idriss and Boulanger do mention a number of methods, including Suzuki et and as well as Robertson and Wride. While we agree that soil sampling is the better way to determine fines content, this adds disproportionate costs, complexity and time to the process, requiring separate boreholes for sample retrieval as well as laboratory testing (then, with the typically highly variable subsoil profiles, there is no certainty that the limited number of samples that can be practically retrieved and lab tested are necessarily representative of the soil profile). Given that carrying out CPT testing and having a geotechnical engineer involved in the process is a quantum leap in quality over the previous practice, the inaccuracies

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inherent in using a CPT-fines relationship are considered acceptable. We have (as suggested) specified the Robertson & Wride methodology, but request some clarification as to why this is preferred over the Suzuki et al method that is also mentioned (and illustrated) in Idriss & Boulanger 2008).

3. **Need for engineering design**
   - Inconsistencies in document between pages 5 and 35 of the draft on the need for specific engineering design

DBH Response:
Page 5 (now page 10) notes that all of the new foundation types require (at least some) specific engineering design input, including in relation to land performance and option selection. The later reference was to the adoption and documentation of the solution.

4. **International structural engineering review**
   - It would be advantageous for the document to be reviewed by international structural engineering experts familiar with residential construction practices

DBH Response:
At this stage we are satisfied with the expert review undertaken by yourselves. We have had to complete the guidance and the review in a very limited timeframe in order to develop foundation solutions for those who need to use them. It may be possible, in future, for international structural engineers to provide comment to the Department about the solutions proposed. The Guidance Document is to be updated over time and this will allow for refinement of the methods proposed.

5. **Independent peer review for rebuilt structures**
   - It would be advantageous to have a peer review of suitability of proposed designs of rebuilt structures in TC3 areas

DBH Response:
We are actively working with Christchurch City Council in developing a panel to review designs as they are initially submitted, and subsequently on a representative sample basis. Furthermore, for Surface Structures Types 2 and 3, we have asked for the Department to be involved in reviewing the initial designs.

6. **Meyerhof method**
   - Meyerhof method – error with basis noted

DBH Response:
We have made the correction to the Meyerhof method. We have added the CPT-based procedure of Eslami and Fellenious (1997) into the Guidance Document.

7. **Low load capacities for deep driven piles**
   - Table C5.3 – capacities presented are low for deep driven piles and should be described as conservative

DBH Response:
The capacities may seem low, but are adequate for residential situations where pile loads are limited by practical spacing requirements (span) between piles. We have recalculated these upwards by removing down-drag. We have added comments as suggested, but are mindful that pile driving equipment will be light

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and in many cases the embedment depth will be limited by the available thickness of the bearing layer.

8. SPT method

- SPT sampler and standardised method of performing SPT should be described

DBH Response:
We are not sure that this level of detail is necessary in such a document, for example we do not describe the (more complex) CPT process in detail either, yet the vast majority of liquefaction assessments will be done with CPT data.

9. Liquefaction susceptibility criteria

- Use of liquefaction susceptibility criterion of I_c >2.6 is inappropriate
- The recommended liquefaction susceptibility criterion of I_c > 2.6 should not be used without performing some representative soil index testing to confirm that this criterion is appropriate for Christchurch soils

DBH Response:
More research is required to refine this criterion for the Christchurch situation. We believe in the meantime it is overly conservative to discontinue its use as a screening tool.

Developing a Christchurch specific I_c-PI relationship has been previously identified as research work that should be carried out as soon as practicable. Once coordinated area-wide information begins to be collected and collated, this work should be relatively easy to carry out.

The latest version of the TC3 Guidance Document can be found at the following web link: http://www.dbh.govt.nz/guidance-on-repairs-after-earthquake. Scroll down from the main document to Appendix C – the Interim Guidance Document for TC3.

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FINAL REPORT

25 May 2012

To: Mike Stannard, Chief Engineer, Department of Building and Housing
From: T. D. O'Rourke, J. Bray, and M. Cubrinovski
Re: **Review of "Guidance for Repairing and Rebuilding Foundations in Technical Category 3 (TC3)"**

This report summarizes review comments pertaining to the referenced report, "Guidance for Repairing and Rebuilding Foundations in Technical Category 3 (TC3)." It also contains comments pertaining to the Department of Building and Housing (DBH) response to reviewer comments.

The reviewers were contacted by DBH on 30 March 2012, and agreed to perform their joint review within a short time frame to comply with deadlines for editing and publishing. The reviewers examined other documents directly related to the technical issues addressed in the referenced report, including "Christchurch Ground Improvement Trials" (Tonkin and Taylor, 2012), "Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence" (Department of Building and Housing, 2011), and "Draft Methodology for Compiling DBH Foundation Technical Category Map" (DBH Residential Advisory Group, 2012). The reviewers engaged in several international conference calls to discuss and develop their joint review, and provided an advanced draft copy of their review comments on 16 April 2012 to DBH. Representatives of DBH were briefed on the review comments on 16 April 2012 during a conference call with the reviewers. A final draft copy of reviewer comments, dated 27 April 2012, was provided to DBH, and the DBH response to reviewer comments was received on 9 May 2012.

The reviewers find that "Guidance for Repairing and Rebuilding Foundations in Technical Category 3 (TC3)" represents a detailed and careful evaluation of the characteristics of potential liquefaction-induced ground deformation in TC3 areas of Christchurch, and it provides valuable guidance for designing and constructing the foundations of residential housing in these areas. The reviewers acknowledge the high standard of work that underpins the referenced document and commend DBH for undertaking such a comprehensive study.

The reviewers also acknowledge that the DBH response to reviewer comments is comprehensive and detailed. The DBH provided an explanation for its response to each review comment. Moreover, the reviewers have examined the TC3 guidance document, "Interim Guidance for Repairing and Rebuilding Foundations in Technical Category 3, dated 27 April 2012, available through DBH. The reviewers are satisfied that the DBH response addresses reviewer comments at an appropriate level of care and detail, and that the TC 3 guidance report of 27 April 2012 reflects suitably the reviewers' comments.

The reviewers' comments with respect to the DBH guidance document are addressed in the first six sections (Sections A through F) that follow. In addition, Section G provides clarification of select issues

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as requested in the DHB response and as identified by the reviewers after their examination of the DHB response and TC3 guidance document.

A. Review Comments on Lateral and Vertical Ground Movement Effects on Structures:

1. The document indicates that the design of repaired or rebuilt foundations in TC3 areas should accommodate minor to moderate levels of: 1) global lateral movement (0 - 300 mm) [pp. 10-13], 2) lateral stretch across a building (0 – 200 mm) [pp. 13-14], and 3) settlement (< 100 mm) [pp. 14-15]. Global lateral movement as large as 300 mm can have a significant effect on deep foundation performance, as discussed under deep foundations below. Lateral stretch of 200 mm and settlement as large as 100 mm can also have a potentially significant effect on residential structures.

   There has been substantial research, with guidelines for practice, which has been accomplished with respect to the combined effects of differential settlement and horizontal ground movements on buildings similar to Christchurch residential structures (e.g., Boscardin & Cording 1989, and Son & Cording 2005). It is instructive to review briefly some of the salient features of these works to provide a framework for the ways buildings respond to underlying soil movement and the characteristics of resulting structural damage.

   Figure 1 summarizes the effects of vertical and horizontal differential ground movements on buildings developed by Boscardin & Cording (1989). The figure was developed from field measurements and observations as well analytical models of building response to ground movements imposed by adjacent, deep excavations and tunneling, and by mining subsidence. The buildings include unreinforced masonry and timber frame structures. The figure is based on data and observations, which were collected and validated over many years, and has been applied successfully on many projects. The figure correlates various degrees of building damage with combined levels of angular distortion (differential settlement divided by the horizontal distance separating the locations of settlement) and horizontal ground strain. Table 1 provides a description of damage categories used in Figure 1, based on the classification of building damage proposed by Burland et al. (1977).

   Assuming typical residential building plan dimensions like those in Figure C5.3 in the TC3 guidance document, a lateral stretch of 200 mm over 13 m results in horizontal strain of 15×10^{-3}, which is well above the level of horizontal strain that causes severe to very severe damage. If we assume that "differential settlement would be expected to be approximately half the total settlement" as assumed in relation to Table 3.1 in the document, the angular distortion is calculated as the ratio of 50 mm differential settlement to 13 m or 3.8×10^{-3}, assuming there is no rigid body tilt across the structure. This value exceeds the threshold for moderate to severe damage in Figure 1. When differential settlement is accompanied with horizontal ground strain, the resulting combined effect is more damaging.

   From the previous work, we see that minor to moderate ground movement, as classified by DBH, will likely produce moderate to very severe damage to residential structures unless mitigated by the foundation and superstructure design. To reduce the potential damage associated with the upper bounds of moderate to severe ground movement, the horizontal strain and angular distortion need to be on the order of 2 x 10^{-3} and 4 x 10^{-3}, respectively. Most of the potential for damage is driven by lateral stretch of the building. To provide protection against horizontal ground strain damage, the foundation and superstructure must be able to reduce the effective lateral stretch in the ground to about 25 mm across the structure. This reduction can be accomplished best by a stiff foundation slab that resists lateral ground strain and by foundation features that allow for horizontal slip between the ground and structure (e.g., plastic slip layer).

Earthquake experience in Japan shows that deliberate stiffening of the foundation has been successful in accommodating liquefaction-induced differential lateral and vertical movement (Tokimatsu & Katsumata 2012, Yasuda & Ishikawa 2012). For example, based on data from Urayasu (Japan), where nearly 8,000 residential houses were affected by soil liquefaction during the 2011 Great East Japan Earthquake, Tokimatsu and Katsumata (2012) found that: *"Even where foundations settled or tilted, few superstructures suffered damage. This was because many buildings had adopted mat foundations or highly rigid foundations to prevent damage to superstructures from liquefaction or uneven settlement."* Observations after the 1992 Landers earthquake (Bray 2001) show that polyethylene sliding layers were effective in reducing lateral ground movements effects on buildings subjected to surface fault rupture. Observations of mining subsidence mitigation techniques provide useful insights as well (e.g., Kratzsch 1983).

Boscardin & Cording (1989) provide analytical results showing that even a modest slab can provide restraint against lateral strain and thereby reduce substantially the influence of horizontal ground strains on building damage. Conversely, foundation designs that tend to anchor the superstructure into to the underlying soil through elements that are not interconnected or supplemented with horizontal structural reinforcement are likely to convey ground strain to the structure and be markedly less effective in reducing the detrimental influence of horizontal ground strains.

2. The Canterbury Earthquake Sequence (as well as other earthquakes) shows that one of the most damaging aspects of liquefaction is the loss of soil through ejecta. If a sand boil forms at the edge of a foundation and a considerable volume of soil is carried away, significant damage can occur. The TC3 guidance document does not refer explicitly to this source of ground movement, but instead focuses on post-liquefaction reconsolidation settlement, using a calculated vertical land settlement of 100 mm at SLS as the threshold of potentially significant damage (see Table C2.5, p. 15). Loss of ground due to the formation of soil ejecta is more important than post-liquefaction reconsolidation settlement because it induces excessive and localized ground deformation/failure. The reviewers unanimously concur with the relative importance of ejecta as a primary cause of ground deformation and resulting structural damage. We encourage recognition of this important source of movement within the TC3 guidance document, and the inclusion of recommended practices for addressing this phenomenon.

3. The focus on performing liquefaction-induced settlement calculations at just the SLS is problematic. Liquefaction is a "brittle" phenomenon. A site that does not liquefy at the SLS level

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may liquefy at a slightly higher acceleration level. In addition, a site in which a relatively thin layer of soil liquefies at the SLS level may liquefy over a significantly greater thickness of the soil deposit at a slightly higher acceleration level. Thus, a site that does not liquefy or only marginally liquefies at the SLS and is judged to undergo minimal settlement may undergo significant liquefaction and settlement when shaken just slightly harder.

Additionally, the magnitude weighting factors employed in the development of the magnitude-weighted design PGA values differ from those recommended by Idriss & Boulanger (2008), which is the liquefaction triggering procedure recommended to be used. To achieve the document's stated objective of consistency, the Idriss & Boulanger (2008) magnitude scaling factors should have been used. As an example of the potential impact, for a $M_w = 6$ event, GNS Science used an inverse magnitude weighting factor of 0.57, whereas Idriss & Boulanger (2008) use 0.67 (i.e., a ratio of these values of 1.18).

To correct for this inconsistency and to identify sites that do not liquefy or only marginally liquefy at the SLS PGA = 0.13 g level but could liquefy over a significant thickness of its soil deposit at a slightly higher PGA level, consider requiring that liquefaction-induced settlement calculations be performed at the SLS PGA = 0.13 g (as already required in the document) and at a performance sensitivity assessment PGA = 0.16 g. This additional level has a PGA that is about 25% greater than the SLS PGA. The use of the higher PGA partially compensates for the different magnitude scaling factors and "softens" the transition between sites that do not liquefy or only marginally liquefy at the SLS PGA and sites that liquefy significantly at just a slightly higher PGA. It differentiates sites that are not significantly affected by liquefaction at both PGA = 0.13 g and PGA = 0.16 g from sites that are significantly affected by liquefaction at PGA = 0.16 g but not at PGA = 0.13 g.

Repeating the liquefaction triggering and settlement calculations at PGA = 0.16 g does not require much additional effort, but useful insights are gained. The case for which liquefaction occurs within a significant thickness of the soil profile at an acceleration level slightly higher than the SLS should inform the engineer about site's sensitivity to liquefaction triggering and its resulting effects (i.e., liquefaction-induced settlement). When presented with a site whose seismic performance is sensitive to slight variations in the PGA level used in the calculations, the engineer may opt for additional foundation strengthening to offset the potential for additional movements from small incremental increases in the PGA.

4. The statement on p. 10 that structures that can tolerate lateral displacements in a ULS event would "... *suffer no significant damage due to lateral ground movements in a SLS event*" is overly optimistic. If liquefaction is triggered in the SLS event, significant lateral spreading can occur in sloping ground or near a free-face, as once liquefaction occurs, the lateral movement is largely gravity controlled (not shaking controlled).

5. Item 3 in the list on p. 12 states that a type of "... *large-scale global lateral movement has the potential to cause significant global lateral ground movements. However, ... it causes only minor ground stretching, and thus little damage to surface structures ...*" It is very difficult to exclude

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the possibility of significant ground stretching in cases when global lateral ground movements are significant. Evidence supporting this statement should be provided. It is also important to acknowledge that liquefaction-induced horizontal movements can also impose lateral stretching on deep pile-supported foundations. Both global lateral movement and lateral stretch can affect deep pile foundations.

6. There are a number of places in the document where claims are made about the performance of foundations, but no supporting evidence in the form of references to technical publications or work performed for DBH are given. For example, on p. 40 the document states that screw piles should easily withstand lateral movements of the ground surface up to 300 mm. Again on p. 47 it states that timber and screw piles (concrete filled steel tubes) are considered to have sufficient resilience or ductility for sites with moderate potential for lateral movement to occur. These claims may be true, but the substantiating evidence for them is absent. It would be helpful to provide references to support such claims where they are made throughout the report.

<u>B. Review Comments on "Deep Pile" Design Concepts:</u>

1. This document implies that deep pile foundations are preferred because they are often listed first (e.g., Section C5.1 and Table C5.1, p. 34) and described favorably (e.g., "*…[piles] provide the greatest flexibility for the superstructure configuration and weight.*" Section C5.2.1, p. 39). The referenced document entitled "Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence" dated November 2011 also states that deep piles will likely be required at TC3 sites (Section 1.4, p. 8) and states: "*Sites categorised as, TC3 are considered incapable of supporting structures with shallow foundations.*" (Section 3.3, p. 27). Deep pile foundations may not perform as well as a ductile reinforced concrete slab foundation, especially in areas undergoing horizontal ground stretch. Additionally, the interface between a pile and the ground surrounding it will likely form a gap that will enable liquefied sand to be transported more easily through the crust to form ejecta. Loss of ground due to the formation of liquefaction ejecta can lead to ground damage, and ejected sand and water can damage the supported house. These issues need to be considered carefully when evaluating the relative benefits of deep pile foundations.

   Robust (stiff) pile foundations that resist ground movement performed very well during 1995 Kobe and 2011 Tohoku earthquakes (high-rise buildings). It is recognized by the reviewers, however, that these types of deep foundations will rarely be economically feasible for the buildings that the TC3 guidance document applies to.

2. Deep pile design should consider horizontal "stretch" of the ground across the building footprint. In addition to the requirement noted in Item 5 of Section C5.2.2 (p. 39) of designing pile foundations to withstand lateral movement at the ground surface relative to the bearing stratum, deep pile foundations should be able to withstand differential horizontal ground displacements across the building (i.e., lateral stretch of the ground). Table C5.2 (p. 36) should include lateral stretch under deep piles at the top of the table. Its absence implies that such deformation may not need to be considered. Thus, Table C5.2 should include lateral stretch categories in part (a) "Deep

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piles" as it is just as important for deep piles as it is for part (b) "Site ground improvement and surface structures."

3. The document states that several deep pile types (e.g., timber and screw piles) "… *have sufficient resilience or ductility for sites with moderate potential for lateral movement [i.e., up to 300 mm]* …" (p. 47). As stated previously, the analytical and observational basis for this claim should be provided. It might be useful to include an illustrative diagram such as that shown in the provided Figure 2. Lateral movement of the crust can induce plastic hinges in deep piles at the approximate locations shown in Fig. 2. Even though screw piles have the ductility to yield without rupture and timber piles can accommodate deformation beyond hinge development, the formation of two or three (including the weak connection of deep pile to slab) hinges results in a mechanism that must move downward under vertical load until soil resistance is mobilized against further displacement. The amount of horizontal displacement before hinge formation will depend on the pile material, pile cross-sectional dimensions, crust and liquefied layer thicknesses, tip embedment, and details of the pile connection to the overlying building. $P\text{–}\Delta$ effects should be considered in cases where the slenderness ratio, defined as Euler's effective length of the pile divided by the minimum radius of gyration, is greater than 50 (Bhattacharya et al. 2004). This reduction can be performed using the first-order correction proposed in MCEER/ATC-49-1 (ATC/MCEER Joint Venture 2003).

   MCEER/ATC-49-1 also provides a simplified tool for estimating the bending moment (M) developed in the deep pile. It gives: $M = 6\, E_{pile}\, I_{pile}\, D\, /\, L_{pile}^{2}$, where E_{pile} = Young's modulus; I_{pile}=moment of inertia of the pile; D=relative lateral displacement between the ends of the pile; and L_{pile}=equivalent length of the pile, i.e., $L_{pile} = H + (h_{top}+h_{bottom})\,(2R)$; H=total thickness of the critical layer; $2R$=diameter of the pile; and h_{top} and h_{bottom} are factors that indicate how many pile diameters above and below the liquefiable layer that the piles are fixed against rotation. In this simplified approach it is normally assumed that h_{top}=h_{bottom}, and their value is between 2 and 5, with the restriction that (h_{top}*2R) cannot be greater than the vertical distance between the top of the liquefiable layer and the bottom of the pile cap.

4. Sufficient details are not provided regarding the deep pile-slab connection. The connection of the top of the deep pile to the foundation slab is a critical design consideration, yet few details are provided in the document. Illustrative diagrams such as Figures C5.3 and C5.4 (pp. 45-46) suggest that the deep pile fits within a socket that is only 75 mm to 100 mm deep. The top of the deep pile should be prevented from pulling out of its socket within the slab. The design assumption employed regarding deep pile head fixity should be stated in the document (e.g., is it pinned?), and the deep pile-slab connection should achieve the assumed deep pile head fixity.

5. The deep pile capacity design equation that is provided in Section C5.2.5 (p. 43) can be simplified by deleting the down drag term (this term is also included in Step 9 of Annexure C3 on p. 82). The use of a capacity reduction factor of 0.4, load factors of 1.2 and 1.5 for dead load and live load, respectively, and inclusion of only the capacity of the deep pile in the bearing stratum ensures that the conventional Factor of Safety (FS = Capacity / Demand) is well above 3. Given this conservatism, there is no need to include the down drag force due to liquefaction.

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The document should emphasize that the critical design check for deep piles is that the neutral plane under the anticipated loads remains well within the bearing stratum below any liquefiable soils. The neutral plane is located at the point in which the soil and pile have displaced downward an identical amount so that the relative soil-to-pile displacement is zero. Above the neutral plane, the soil has displaced downward relative to the pile (and hence already loaded it), and below the neutral plane, the pile has displaced downward relative to the soil (to develop bearing). The neutral plane under working loads should be in the bearing stratum for driven deep piles. With additional loading the neutral plane will rise, but as long as it remains in the bearing stratum, there will be no additional effect due to down drag load of the soils above the liquefied layer. The soil above the neutral plane is already imposing a drag load downward along the top of the deep pile due to residual driving stresses. There is no need to add it in again.

Removing the calculation of the down drag load of the crust will simplify the design calculations and make them consistent for each driven deep pile type. By checking that the deep pile resistance in the bearing stratum is well above the load applied at the pile head, the neutral plane will be in the bearing stratum so there will be negligible settlement of the driven deep pile because of liquefaction. Numerous seismic case histories confirm this concept.

It is critical, however, that the deep pile be driven into the bearing stratum so that it will develop full tip resistance and residual driving stresses so that its neutral plane is within the bearing stratum and below liquefiable soils. The document states that jetting or pre-drilling may be required to install driven wooden piles (p. 41). The document should warn that the jetting/pre-drilling must not be used to advance fully the deep pile to its target depth. The deep pile must be driven into the bearing stratum so that it develops full tip resistance and residual driving stresses. In many cases, a deep pile should be driven at least 5 pile diameters after jetting/pre-drilling to develop its full capacity in the bearing stratum.

6. The load transfer mechanism of bored deep piles differs from that of driven deep piles because there are no residual driving stresses and the tip resistance of the deep pile has not been developed fully under working loads. The load transfer of the bored deep pile is from top-down, such that if skin friction along the shaft of the deep pile is lost due to liquefaction, the load will transfer to the bottom of the pile, which will lead to settlement. The document states that it is unlikely that bored deep piles will be used, but this warning should be emphasized as opposed to the general statement that is now made that "*bored piles face similar issues to CFA piles.*" (p. 42)

<u>C. Review Comments on "Ground Improvement" Design Concepts:</u>

1. It is uncertain what is meant by the term "standard compaction" in the document. As opposed to what is specified in Item 1 of Section C5.3.4 (p. 50), compaction practice in the U.S. indicates that the minimum density of compacted sandy soils (with < 15% nonplastic fines) should be greater than 100% standard Proctor relative compaction (as per ASTM D698-07) to ensure that they are placed with a relative density of at least 75% (Lee & Singh 1971). The use of a steel drum vibratory roller compactor is generally used to develop a more resistant soil fabric.

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2. In addition to what is specified in Item 4 of Section C5.3.4 (pp. 50-51), a second layer of geogrid should be placed above the base layer of geogrid to improve the performance of the densified block of soil. Two horizontal layers of geogrid are significantly more effective in spreading out underlying ground movements and in maintaining the integrity of the densified block of soil (Bray, 2001).

3. Deep soil mixing (Section C5.3.4, p. 50 & p. 57; and in other parts of the document, e.g., Annexure C4, p. 89) should extend beyond a minimum depth of 8 m if dense sands or gravels are not encountered in the upper 8 m of the soil profile. Deep soil mixing or any other deep ground improvement technique (e.g., stone columns; Annexure C4, p. 89, and low mobility grout columns) should not just extend "*... down to a depth of 8.0 m below ground level or to found on dense or gravel.*" They must extend through the entire thickness of potentially liquefiable soils at the site, so that satisfactory seismic performance is achieved. The field trials at QE2 Park showed clearly that the effectiveness of deep soil mix was reduced by liquefaction that occurred beneath the soil mix at 8 m depth.

4. The densified or stabilized crust methods (Type 1 or Type 3, Section C5.3.4, pp. 50-56) would deliver improved performance if the excavation base extended 1.5 m beyond the footprint of the proposed structure.

5. The analytical or observational basis for stating that Type 3 ground improvement with geogrids will perform satisfactorily in "Major" lateral stretch zones, wherein lateral ground movements exceeding 300 mm are possible, should be provided.

D. Review Comments on "Shallow Foundation" Design Concepts:

A number of surface structures with shallow foundations options have been proposed in the document specifically addressing the significant demand on the foundation and superstructure due to large and non-uniform lateral ground displacements and strains. It would be useful to explain briefly or indicate the types of analyses and calculations that have been conducted in the development and verification of the proposed solutions. The comments below are aimed to enhance further these solutions or point out some potential improvements in their design and performance.

1. The proposed Surface Structure Type 1 is a modified NZS 3604 solution for shallow foundations capable of withstanding minor to moderate differential settlement from liquefaction (less than 100 mm) at SLS level, and minor to moderate lateral strain across the building footprint (up to 200 mm) at ULS level. While this solution is proposed for areas of minor to moderate ground strains and relies on an easy repair rather than a significant reduction in damage, it is useful to have in mind that a robust foundation that is not locked into the ground will effectively reduce the level of stretch and movements of the ground that is transferred to the superstructure (as described in Section A).

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2. The proposed Surface Structure Type 1 shallow foundation presumes a well-defined (known) predominant spreading direction and has much lower stiffness and resistance in the direction which is orthogonal to the anticipated spreading direction. There are several concerns with this detail/concept: 1) the spreading direction cannot be always well defined (anticipated), or even if it is well defined, it may not be aligned with the foundation axes; 2) the lack of stiff connections in the orthogonal direction may result in differential movement and deformation of the foundation causing racking or twisting of the superstructure; and 3) by embedding the "pile" supports in the ground, the strains due to ground stretching will be transferred to the foundation, and then the superstructure. In this context, it is useful to confirm the integrity of this foundation and its performance under earthquake loads, including the effects of liquefaction and lateral spreading, using an appropriate analysis (if this has not been done already).

3. The proposed Type 2 Surface Structure Foundation - Concept 2.1 assumes that the floor plate can slide on the concrete pads in a ground spreading event (but will remain in place when subjected to wind loads). It is not clear how the structure's inertial load was considered in this design philosophy. The inertial load will be three-directional and could be of a magnitude similar to the lateral spreading load. With this in mind, it is not clear what will prevent the superstructure from sliding on top of the foundation during the dynamic action of earthquake accelerations and whether this could be achieved in a controlled manner.

4. Type 2 Surface Structure Foundation - Concept 2.1 has the same issues with regard to the unknown spreading direction or spreading direction not aligned with the foundation axes. Also, it does not offer significant resistance to differential vertical displacements. Overall, there are uncertainties with this type of foundation related to good/acceptable performance both for ground-induced and inertia-induced loads/deformation.

5. Type 2 Surface Structure Foundation - Concept 2.2 is a better version of Concept 2.1 because of its greater stiffness. It still has similar issues with regard to the inertial loads and spreading direction. Also, as in the previous solutions, the foundation consists of many detached or weakly connected members/elements and does not have the integrity of a robust foundation that would be most effective in reducing/controlling the damaging effects of liquefaction-induced differential settlements or spreading-induced lateral stretching of the ground.

6. Type 2 Surface Structure Foundation - Concept 2.3 appears to be the most robust of the proposed solutions that addresses many of the uncertainties related to Concepts 2.1 and 22.

7. Type 2 shallow foundations are the only option for which deep site investigations are not required. Ground conditions are a key factor in the foundation performance, and this is particularly true for liquefiable soils. All types of shallow foundations will benefit from the site investigation. Type 2 shallow foundations should not be treated differently from the other considered solutions. Also, it might be difficult to select the most appropriate foundation solution or ground improvement method at a given site if no specific information can be inferred from the area-wide deep investigations.

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8. In Figures C5.17 to C5.19, the polythene slip layer is not shown. When it is beneficial to use a slip layer, it would be better to have two layers of 1.0 mm-thick HDPE. At a minimum, two layers of a relatively thick polythene plastic sheet should be used because a single layer of thin plastic, which will likely be damaged during installation, will not be as effective in decoupling the underlying ground movements from the foundation elements. Lastly, dry rot may occur for the Detail A of Figure C5.18.

E. Review Comments on Methodology for Compiling DBH Foundation Technical Category Map

This report summarizes the methodology used for the zoning of Christchurch into TC1, TC2 and TC3 foundation technical categories. It presents a comprehensive area-wide methodology including characterization of the seismic demand, land damage, building damage and shallow subsurface conditions, and their combination into various factors and index measures for eventual compilation into a foundation technical category map. The map is of paramount important, because it defines the soil investigations and appropriate types of foundations for any given area/location in Christchurch. There was not sufficient time for us to conduct a thorough review of this methodology, and the document itself does not provide all the details of the methodology. Based on our rapid screening of the document, we do have some positive impressions about the robustness of the adopted methodology, but also have concerns about some of the assumptions and their implications. Some of our concerns are expressed as follows:

1. Parts of Christchurch were subjected to high seismic demands during the 2010-2011 earthquakes (e.g., the south, south-east, and eastern suburbs), while the seismic demand was significantly lower in other areas (e.g., west and north-western suburbs). One may argue that in the areas of very strong ground shaking, the observed performance of land and foundations would provide the best indicator of their performance in future earthquakes. Conversely, the observations will have much less weight in areas of low seismic demand, and therefore, the results of liquefaction analyses would provide more reliable input for their classification. The proposed methodology does not discriminate between areas that were subjected to a significantly high seismic demand and those that were not. Moreover, it does not give proper weighting to various data layers or types of information. For example, the most damaging 22 February 2011 earthquake that produced high seismic demands in nearly half of Christchurch contributes at most 30% in the weighted evaluation. The reasoning in support of the weighting factors listed in Table 5 should be provided.

2. It appears from Tables 4 and 5 that in areas of low seismic demand ($PGA < PGA_{SLS}$) that the crust thickness is practically the only factor defining the technical category. TC3 is assigned for a water table depth of $0m \le z \le 1m$, TC2 for a water table depth $1m < z \le 3m$, and TC1 for $z > 3m$. While the crust thickness is recognized as an important factor affecting the liquefaction resistance and particularly the consequences of liquefaction, there are also other factors that need to be considered in the classification of sites using more detailed liquefaction evaluation.

3. Building damage has been used together with land damage in the classification. In the areas of high accelerations, however, building damage was also induced by the severe shaking, and hence the building damage was not necessarily related to the land damage or liquefaction. In this

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context, and for the purpose of the DBH TC Map, the use of foundations damage (instead of building damage) seems more appropriate.

F. Additional Review Comments:

1. The document references the Idriss and Boulanger (2008) monograph, but this monograph does not reference the preferred method for estimating seismic post-liquefaction settlements of Zhang et al. (2002). The use of this CPT-based method should be recommended.

2. A means for using the Idriss & Boulanger (2008) CPT-based liquefaction triggering evaluation procedure without retrieving soil samples should be provided so that the CPT can be used (when appropriate) without soil borings as suggested by the document. Robertson and Wride (1998) provide a relationship for estimating fines content (FC) as a function of the CPT-estimated soil behavior type index (I_c). It is: (a) if I_c <1.26, then apparent FC = 0%; (b) if 1.26 ≤ I_c ≤ 3.5, then apparent FC (%) =1.75 $I_c^{3.25}$ - 3.7; and (c) if I_c > 3.5, then apparent FC =100%. Alternatively, conservatively, the FC can be assumed to be 0% for all cases when using the Idriss & Boulanger (2008) CPT-based procedure. Another possibility is to develop a conservative interpretation of the Robertson and Wride (1998) relationship for use with the CPT.

3. On p. 5, the document states that "*... all foundation repairs and reconstruction in TC3 require specific engineering design ...*", but later on p. 35, it states "*... some elements can be adopted and specified directly from these Guidelines without further engineering design.*" This apparent inconsistency should be clarified.

4. It would also be advantageous to have the document reviewed by international structural engineering experts who are familiar with residential construction practices to incorporate their differing experiences in addressing similar problems.

5. It would be advantageous to involve an independent peer reviewer in assessing the suitability of proposed designs of rebuilt structures in the TC3 areas. The California Seismic Hazards Mapping Act's requirement to use independent peer reviewers led to significant improvements in the state-of-the-practice in the relatively novel field of "liquefaction engineering." It is an evolving field with nuances and significant sources of uncertainty. Independent peer review is an important component of advancing the state-of-the-practice and in improving resilience.

6. The document provides an example deep pile design using Meyerhof's SPT-based method. The Meyerhof (1976) method is likely based on N_{60} values (not $(N_1)_{60}$ values). Deep pile capacity should increase with overburden pressure as well as relative density so the use of N_{60} values is most reasonable. Additionally, as the document (and we) prefers the use of the CPT when it can be advanced economically, the document should also provide a CPT-based deep pile design method so that it is readily available. The Eslami and Fellenious (1997) CPT-based method is a sound method.

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7. Table C5.3 (p. 44) presents low load capacities for driven deep piles. The low capacities of the driven deep piles appear to result from the assumption that they have only been embedded 1 m into medium density sand. It would be more appropriate to describe these values as "conservative capacities," and inform the reader that in most cases, the use of higher load capacities can be justified through a proper site investigation and appropriate design calculations.

8. The standard SPT sampler and the standardized method of performing the SPT should be described or referenced (e.g., to ASTM (2011 & 2012) D1586 and D6066).

9. The recommended liquefaction susceptibility criterion of $I_c > 2.6$ should not be used without performing some representative soil index testing to confirm that this criterion is appropriate for Christchurch soils. Soils with $I_c > 2.6$ have been found to liquefy, so the use of a PI-based criterion such as that of Bray and Sancio (2006) is preferred. This may not be a concern in parts of Christchurch, as we have been informed that there are few deposits of slightly plastic soils with $I_c > 2.6$ that could potentially liquefy or undergo cyclic failure. However, we understand that in some areas of Christchurch, soils with $I_c = 2$-3 are often located at depths from 1.5 m to 3.5 m. The presence of these shallow soils can potentially affect the liquefaction evaluation of sites with residential buildings. Hence, the proper classification of the liquefaction susceptibility of these soils is important. It would be advantageous to investigate representative sites in Christchurch where such soils exist to retrieve soil samples and test them in the laboratory to better understand their fines content, plasticity, and water content so that an I_c liquefaction susceptibility criterion could be employed with confidence.

G. Clarification of Select Issues Related to DBH Response to Reviewer Comments:

The DBH personnel and their consultants should be commended for the care they exercised in carefully considering our review comments. They have addressed satisfactorily our key concerns. The revised TC3 document is well done. A few responses are provided below for clarification. These issues could be addressed at some time in the future.

1. B.3. Evidence to Show Piles Are Resilient/Ductile: Although Bhattacharya et al. (2004) provides an excellent centrifuge study of foundation buckling effects, one should be cautious in extending the buckling analogy too broadly at the expense of P-delta effects. The Bhattacharya et al. (2004) centrifuge tests were performed in a uniform loose sand layer with minimal non-liquefiable crust and no increasing lateral spread displacement. This condition may not match conditions in Christchurch where 1) sand density is variable and not uniformly susceptible to liquefaction, 2) the non-liquefiable crust may have significant thickness, and 3) global lateral spread displacement is imposed on the deep foundations. Other studies, such as Goh and O'Rourke (2006), provide additional insight regarding the effects of a non-liquefiable crust and steadily increasing lateral movement. Analytical simulations in that paper, which show very good comparison with observed deep foundation displacements and failure for the Niigata Family Court House (NFCH) Building, were strongly influenced by the vertical loads carried by the piles and attendant P-delta effect as lateral movement increased. Please note that the NFCH Building settled differentially

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and tilted 1 degree, thus demonstrating the downward movement of piles subjected to excessive moments during lateral spreading.

2. <u>C.1. Compaction Methods:</u> A definition or reference for the term "standard compaction" should be provided in the revised document so that it is clear what is meant by "95% standard compaction." If this term is intended to indicate standard Proctor relative compaction (ASTM D698-07), then 95% standard Proctor relative compaction equates roughly to a relative density of just 50% for sandy soils with less than 15% nonplastic fines (Lee & Singh, 1971). Soil reinforcement works better in compacted fill if the soil is dense so that it dilates when sheared. To avoid liquefaction of compacted sandy soils that could become saturated, the soils should be compacted to a relative density of at least 75%.

3. <u>C.3. Deep Soil Mixing:</u> For stone columns (Type 4) on p. 62 of the revised document, the phrasing of "*Depth of columns should be determined by The Engineer but is expected to be a minimum of 8 m below ground level ...*" is used. This same phrasing is not used for the other deep foundation treatments of deep soil mixing (Type 3) and low mobility grout columns (Type 5). This phrasing would be appropriate for all three types of deep foundation treatments. There are cases where very loose soils from a depth of 8 m to 10 m can undergo significant shear-induced deformation under the overlying improved ground, so it is useful for the Engineer to make a site-specific assessment.

4. <u>E. Methodology for Compiling DBH Foundation Technical Category Map:</u> The reviewers encourage DBH to proceed with their intention to gradually improve the methodology for technical land zoning as new geotechnical data and foundation-specific building data become available, and to further scrutinize the weighting values and combinations of different contributing factors that form the basis for land classification into different technical categories.

5. <u>F.2. Use of I&B Without Soil Samples:</u> The CPT-based liquefaction triggering procedure of Robertson & Wride (1998) is not referenced in the reviewer comments. Instead, the reference is to the I_c-FC correlation that is presented in their paper. It is judged to be the best CPT-based FC correlation available.

   Idriss & Boulanger (2008) discuss both the Robertson & Wride (1998) and Suzuki et al. (1997) CPT-based liquefaction triggering evaluation procedures and how they modify their procedures for soils with increasing fines content. As shown in Idriss & Boulanger (2008), the Suzuki et al. (1997) I_c-based adjustment to CPT tip resistance is more aggressive than that of Robertson & Wride (1998), and the Robertson & Wride (1998) I_c-based fines content adjustment is greater than that of Idriss & Boulanger (2008) and Moss et al. (2006). The reviewers understand that DBH is using the Idriss & Boulanger (2008) liquefaction triggering evaluation procedure, and we are only suggesting that the Robertson & Wride (1998) I_c-FC correlation be used when necessary to estimate FC in the Idriss & Boulanger (2008) procedure.

   The reviewers draw attention to the following sentence: "*For fines corrections where soil samples have not been retrieved and tested, the method of Robertson and Wride (1998) should be*

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used.", which appears on p. 24 of the revised document. This sentence could be misinterpreted to imply that when soil samples are not taken and a fines content correction is required to be made, the Robertson & Wride (1998) procedure should be used. This is not the intent of the revised document after reading ahead to the next page. The reviewers suggest a sentence in the future to be clear on this issue, such as the following: "*When soil samples have not been retrieved, the I_c-FC correlation of Robertson & Wride (1998) should be used to estimate FC for use in the Idriss & Boulanger (2008) procedure.*"

6. F.8. SPT Method: The CPT method is more standardized, and the reviewers have not often encountered issues with its use. However, the SPT method varies widely in how it is employed, which is why ASTM developed the additional standard D6066 to emphasize the importance of performing the SPT in as standardized a manner as possible. By referencing the standards D1586 and D6066, the reviewers hope to avoid the use of non-standard SPT procedures. This is a major problem in the U.S. and in many other countries.

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Table 1. Categories of Building Damage

TABLE 2. Classification of Visible Damage

Class of damage (1)	Description of damage[a] (2)	Approximate width[b] of cracks, mm (3)
Negligible	Hairline cracks	<0.1
[redacted]	[redacted] ire [redacted]	[redacted] <5 [redacted]
[redacted]	[redacted] de [redacted] ne r [redacted]	[redacted]
	and ndow n	
	out and p	5 to 15 or several
[redacted]	fle d po [redacted] small a nor [redacted] ng equ pu	[redacted]
	ows ng. Util may be rrupted. fren mpa ed. o pair 1 an replacement ns walls, espe y o or do ows required. s	15 to 25 also depends on number of cracks
	redecoration. Perhaps isolated slight fractu of building. Cracks in exterior brickwork visible upon close inspection. ice	
Slight Very ov ro	Cracks easily filled. Re-decoration probably required. Several slight fractures insi building. Exterior cracks visible, so c- pointing may be required for ng. Weathertightness. Doors wi s may stick slightly.	usually >25 depends on number of cracks
Moderate	Cracks may require cutting atching. Recurrent cracks c by suitable linings. Tuck-point an ssibly replacement of a a mount of exte	

their d considered when classifying
Crack width is only be used on alone as a direct
asure of
Note: M dif

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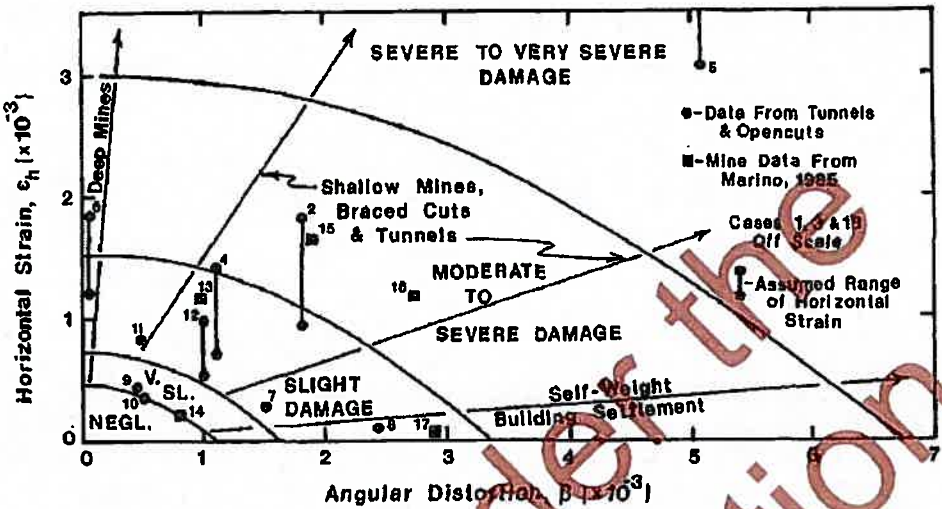

Figure 1. Relationship among Building Damage, Horizontal Strain, and Angular Distortion (after Boscardin and Cording, 1989)

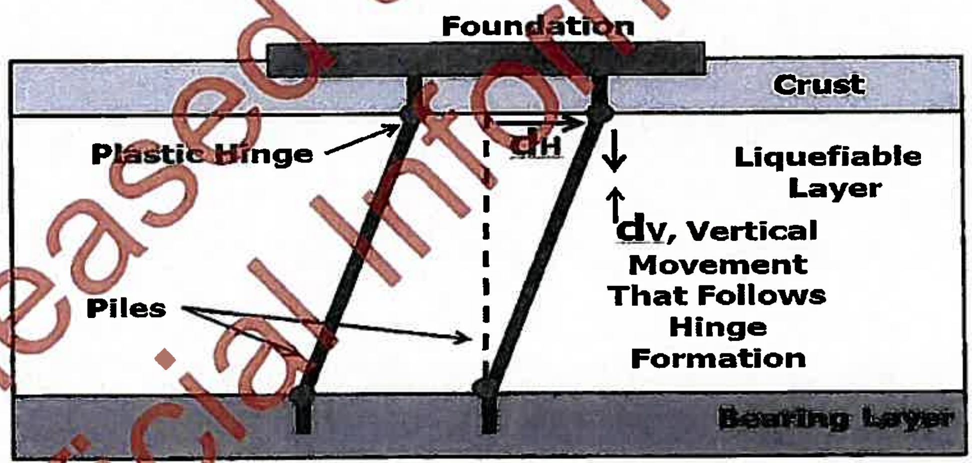

Figure 2. Global Lateral Movement of Pile Foundation

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