BEFORE THE ENVIRONMENT COURT
AT WELLINGTON

ENV-2015-WLG-024

IN THE MATTER	of the Resource Management Act 1991
AND	
IN THE MATTER	of applications for resource consent by Site 10 Redevelopment Limited Partnership and Wellington City Council in respect of the area known as Site 10

STATEMENT OF EVIDENCE OF ADAM WILLIAM THORNTON
ON BEHALF OF SITE 10 REDEVELOPMENT LIMITED PARTNERSHIP AND WELLINGTON CITY COUNCIL

3 July 2015

Simpson Grierson
Barristers & Solicitors
Simpson Grierson
Solicitors
PO Box 2402
Wellington 6410
Tel: 04 499 4599
Fax: 04 472 6986
Solicitor Acting:
Duncan Laing / Lizzy Wiessing
Email: duncan.laing@simpsongrierson.com / lizzy.wiessing@simpsongrierson.com

INTRODUCTION

1. My full name is Adam William Thornton. I am a civil/structural chartered professional engineer based in Wellington.

Qualifications and experience

2. I graduated with a Bachelor of Engineering (Honours) degree in civil/structural engineering from Canterbury University in 1974. I am a Distinguished Fellow of the Institute of Professional Engineers and I am a past President and Life Member of the Association of Consulting Engineers of New Zealand. I am also a past member of the executive board of the International Federation of Consulting Engineers (**FIDIC**) and a past member of the Institution of Professional Engineers New Zealand (**IPENZ**) Practice Board.

3. I am or have been a member of the following:

(a) New Zealand Society for Earthquake Engineering (**NZSEE**) Study Group on Earthquake Prone Buildings;

(b) New Zealand Government Department of Building and Housing – Advisory Board Member;

(c) New Zealand Government Department of Building and Housing – Sector Reference Group Member for 2011 Building Bill Amendment;

(d) Canterbury/Auckland Universities Seismic Retro Fit Research Programme – Review Board;

(e) DBH Expert Panel for Christchurch Earthquake Investigations;

(f) MBIE Engineering Reference Group for the implementation of The Royal Commission Recommendations (and other recommendations);

(g) SESOC – Structural Engineering Society for New Zealand;

(h) NZSEE;

(i) New Zealand Timber Design Society; and

(j) EENZ – Earthquake Engineering New Zealand Cluster.

4. After graduating from Canterbury University, I was employed for several years as a design engineer before becoming a director and shareholder of Dunning Thornton Consultants Ltd (**DTC**) in 1986.

5. DTC is a Wellington-based consultancy engineering company primarily offering structural design services to clients, architects, contractors and suppliers in the construction industry.

6. I have over 40 years' experience as a consulting structural engineer, working primarily in New Zealand but also in the Asia Pacific region in the construction industry. My work has included:

   (a) Structural design and monitoring of commercial and high rise buildings, many and varied new buildings and structures for private, institutional, local and central government clients;

   (b) Seismic assessment and retrofit of over 50 heritage and earthquake prone structures, some of which are unreinforced masonry buildings; and

   (c) Relocation of a number of heavy buildings and structures.

7. Significant new buildings I have been involved in include:
   (a) IBM Centre – Petone;
   (b) Hewlett Packard building – Willis Street;
   (c) ASB Tower – Jervois Quay;
   (d) Wellington Hospital A&E building (base isolated);
   (e) Wellington Stadium walkways;
   (f) Bhuj Hospital (base – isolated in conjunction with HCG) – India;
   (g) Chews Precinct development – Willis Street;
   (h) New Wellington Regional Hospital;
   (i) The Wellington Hotel and Apartments – Cuba Street;
   (j) The Hub – Victoria University of Wellington;
   (k) Clyde Quay Wharf (ex-overseas passenger terminal);
   (l) One Market Lane – Taranaki Street; and
   (m) National Containment Laboratory for MPI (base – isolated) – Upper Hutt.

8. The seismic strengthening/assessment projects I have been involved in include:
   (a) The St James Theatre;
   (b) The Embassy Theatre;
   (c) The Futuna Chapel;
   (d) The Dominion Museum (Massey University);
   (e) State Insurance (Te Puni Kokiri);

(f) The Hope Gibbons Building;

(g) Turnbull House;

(h) Numerous buildings in the Courtenay Place/Allen/Blair Streets Precinct;

(i) Dockside;

(j) Shed 5;

(k) Shed 21 – Waterloo Apartments;

(l) Odlins Building (NZX Centre);

(m) Shed 22 – now Mac's Brewery;

(n) The John Chambers Building;

(o) The Huddart Parker Building; and

(p) The Whitcoulls Building.

9. Relocation projects I have designed include:

(a) The Museum Hotel;

(b) The Waihi Cornish Pumphouse;

(c) The Rob Roy Hotel (Birdcage) in Auckland;

(d) Buckle Street Creche; and

(e) Arthur Street Boys Institute.

10. I was a member of the expert panel appointed by the government (DBH) to investigate the performance of four significant buildings during the Christchurch earthquakes. The buildings were the CTV, the PCG, the Hotel Grand Chancellor and the Forsyth Barr buildings. I led the investigation into the performance of the Hotel Grand Chancellor and reported our findings to the Royal Commission.

11. I have made significant contributions to leadership in professional associations for Association of Consulting Engineers New Zealand (**ACENZ**), IPENZ and FIDIC.

12. I have written a number of publications and presentations from 2004 through to 2014. I have won a number of awards with ACENZ, New Zealand Institute of Building (**NZIOB**), IPENZ and NZSEE for my designs and contributions to the engineering profession, particularly in the field of structural engineering.

Code of conduct for experts

13. I can confirm that I have read the Expert Witness Code of Conduct set out in the Environment Court Practice Note 2014. I have complied with the Code of Conduct in preparing this evidence and I agree to comply with it while giving oral evidence. Except where I state that I am relying on the evidence of another person, this written evidence is within my area of expertise. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed in this evidence.

SCOPE OF EVIDENCE

14. I have been engaged by the applicants to provide evidence in relation to the construction methodology for the basement of the proposed building on Site 10, Kumutoto, Wellington Waterfront and other matters relating to the structural design of the proposed building.

15. Key issues addressed in my evidence include design to mitigate the likely effects of natural hazards and methodologies to deal with the existing site contamination during construction of the basement and foundations.

16. I have been leading the structural design team for the proposed building since the applicant engaged DTC as structural engineers early 2013. I have also had some involvement in previous proposals for the site.

17. In forming my view, I have also made reference to the following documents:

(a) Appendix 21 to the AEE – Natural Hazards Assessment (Dr Richard Sharpe, Beca);

(b) Brief of evidence of Dr Michael Revell;

(c) Brief of evidence of Mr Reinen-Hamill;

(d) Brief of evidence of Mr Peter McGuinness;

(e) Groundwater and Contamination Assessment – Tonkin & Taylor (as part of the applicants' further information response – 1); and

(f) Geotechnical Concept Design – Tonkin & Taylor (as part of the applicants' further information response – 2).

18. My evidence will cover the following matters:

(a) Structure and foundation mitigation of natural hazards;

(b) Contamination mitigation methodologies;

(c) Comments on submissions; and

(d) Comments on the Councils' section 87F reports.

EVIDENCE

19. I prepared the technical report titled "Site 10 – Basement Construction Method Statement" included as Appendix 17 of the AEE for the applications. A copy of the technical report is attached as **Appendix A** of this evidence.

20. In summary, the report outlined the proposed methodology for constructing the basement of the proposed building at Site 10, in particular:

(a) How construction and excavation materials would be prevented from entering the storm-water system.

(b) That a bund would be created around the site perimeter by excavating to 400mm deep to provide against spills or flooding.

(c) How a perimeter wall around the building footprint would be constructed to provide a cut-off wall to minimise the likelihood of groundwater flow towards the harbour and as a temporary retaining wall for further excavation works.

(d) That the requirement for localised de-watering down to the underside of the proposed basement would be minimised by the construction methodology. Any required de-watering would continue until the building mass exceeds the groundwater uplift pressure.

(e) The order and method by which the various structural elements of the basement will be constructed.

21. I confirm that the proposed methodology is essentially unchanged since that outlined in the technical report. The methodology described the construction of a grid of subterranean walls, extending from the basement down through the reclamation fill and into the original seabed alluviums utilising a process known as Deep-Soil-Mixing (**DSM**). This process involves mixing cement with the existing fill materials to form concrete. The grid of walls eliminates the effects of potential liquefaction and also forms a platform to support the building super structure.

22. In addition to DSM, an alternative, known as Continuous-Flight-Auger (**CFA**) is being evaluated. This involves forming the walls with a more conventional piling rig, i.e. drilling a hole and filling it with concrete. The difference is that the concrete is delivered to the base of the pile via the shaft of the auger. To form the wall, the piles are drilled contiguously, i.e. overlapping.

23. The resulting wall is much the same from a structural perspective whether it is constructed using DSM or CFA. CFA might be preferred if there is a risk that the combination of cement with the existing fill materials does not form a suitable concrete. From a site management viewpoint, however, CFA will generate additional excavated material, which will in turn need to be disposed off-site.

24. A decision as to the use of DSM or CFA is likely to be made following on-site testing.

25. I also note that the proposed grid of walls beneath the basement will have added long-term benefits of encapsulating any remaining contamination within the site and slowing the flow of contaminants from the west of the site towards the harbour.

26. I also prepared the technical report titled "Site 10 – Structural Effects" included as Appendix 20 of the AEE for the applications. A copy of the technical report is attached as **Appendix B** of this evidence.

27. In summary, the report outlines the various measures that are proposed to mitigate the likely effects of seismic shaking, liquefaction, lateral spreading, tsunami and sea-level rise.

28. I confirm that my views on these matters are unchanged.

29. In my report I refer to seismic damage avoidance. Typically, in zones of high seismicity, like Wellington, modern structures are designed to dissipate some of the energy of major seismic shaking by utilising ductility within the structure, i.e. they are not designed to resist the largest likely earthquake without some yielding of the structure. This usually means that the reinforced concrete or structural steel is detailed to allow post-elastic yielding and deformation in a controlled and pre-determined manner.

30. A hierarchy of strength methodology, referred to as Capacity Design, is followed to give a high level of protection against any potential collapse mechanism. This conventional design methodology, while meeting the life-safety provisions of the Building Act 2004, can result in large inter-storey drifts (lateral displacements between adjacent floors) with consequential damage to non-structural elements such as façades and partitions. The structural yielding and non-structural damage can be expensive and time-consuming to repair.

31. Damage avoidance design involves detailing to make the structure more easily repairable and to separate the non-structural elements from the effects of the inter-storey displacements.

32. Base-isolation, as is proposed for the Site 10 building, is regarded as giving the most effective form of damage avoidance. With base-isolation, the post-elastic yielding and the displacements are concentrated at the base-isolator level, below the ground-floor slab. Above the isolators, the displacement and accelerations are significantly reduced by comparison to a conventional structure, giving enhanced protection to non-structural elements, building contents and occupants. The risk of business disruption is greatly reduced.

33. I also note that from reviewing the draft evidence of Dr Revell and Mr Reinen-Hamill that a ground floor slab built at RL2.5m (as proposed) will ensure the building is not likely to be subject to inundation in a 1-in-100 year event during the building's 100 year lifespan.

SUBMISSIONS

34. Reviewing the submissions on the applications, I note a number of identical submissions were made by Shed 21 Body Corporate or owners of units at the Shed 21 Apartment Building (submissions 19 (Ponatahi Trust), 20 (Body Corporate), 21 (Pledger), 22 (Fergusson), 23 (Hayes), 24 (Constable and Compain) and 44 (Farley)).

35. These submissions express a requirement for a robust tolerance control process around potential structural or other damage caused to Shed 21 as a result of piling and other works. This is being addressed in a statement of evidence by Mr McGuinness of LT McGuinness, but I would make the following additional comments:

(a) The proposed piling and other construction work is extremely unlikely to have any significant effect on the Shed 21 structure.

(b) The proposed piling is to be drilled and augured rather than driven, i.e. there will be no pile-driving so vibrations should be minimal.

(c) The proposed basement is shallow compared to the separation between the buildings and in any event Shed 21 is founded on deep piles so the risk of under-mining is insignificant.

(d) The depth of ground water drawdown is likely to be extremely small as the ground water is readily replenished by the harbour and the large surrounding volume of ground water. In any event, Shed 21 is founded on deep piles so the risk of any loss of support is insignificant. I note this is confirmed in Tonkin & Taylor's Groundwater and Contamination Assessment.

36. I also note that submission 26 (Horne/Mitcalfe) refers to the proposed building site as being inappropriate due to it being on reclaimed land and subject to liquefaction. In this respect, I would comment as follows:

(a) Liquefaction can lead to loss of support in buildings with shallow foundations. It can also lead to lateral spreading adjacent to lowered ground such as a river bank or harbour edge. Both these issues were identified in the Geotechnical Concept Design prepared by Tonkin & Taylor and Beca's Natural Hazard Assessment. The foundation and structural design incorporates deep foundations and ground improvement that will mitigate the potential for liquefaction and lateral spreading. I have included a description of the mitigation within paragraphs 39-40 of my evidence below.

SECTION 87F REPORT

37. I have read the section 87F reports prepared for this matter.

38. I have the following comments on the reports:

(a) Paragraphs 129-130 of the Wellington City Council's section 87F report refer to anticipated sea level rise and the possibility of inundation at the site. I note that this matter is being covered in the evidence of Dr Revell, which confirms a lower likely inundation level than that predicted by Dr Iain Dawe and in the section 92 response by Mr Reinen-Hamill. I also note that Mr Reinen-Hamill has since updated his predictions in his evidence on the basis of Dr Revell's evidence. This indicates that a ground floor slab at the proposed level of RL2.5m is not likely to be at risk of inundation during the next 100 years.

(b) I have reviewed Dr Dawe’s memorandum dated 20 February 2015. I note Dr Dawe’s comments that details of the liquefaction mitigation are not included.

39. The Tonkin & Taylor Geotechnical Concept Design includes a description of the proposed grid of sub-ground walls, formed using a DSM methodology. The concrete walls, formed by in-situ mixing of cement with the reclamation materials, will extend from the underside of the basement down and into to the original seabed, below the zone of liquefiable material.

40. As I noted in paragraph 22, the wall methodology may be changed from DSM to an alternative but similar process known as CFA. The resulting mitigation is however effectively the same. The benefits of the grid of walls include the following:

(a) The walls confine the liquefiable material and prevent liquefaction.

(b) The walls provide the foundation structure to transmit the building loads down to competent bearing strata.

(c) The walls act as bracing elements (underground shear walls) to resist potential lateral spreading.

(d) The walls contain existing contamination within the site footprint.

41. The Greater Wellington Regional Council's section 87F report suggests at paragraphs 9.1.1 and 9.1.2 that dewatering will potentially mobilise contaminants. In this respect, I would comment:

(a) Dewatering will not commence until the perimeter cut-off grout/concrete wall has been completed. This means that any potential contaminant mobilisation will be contained within the site and is highly unlikely to result in mobilised contaminants flowing toward the sea/harbour. In any event, the existing sea wall also restricts this flow.

(b) Draw down of water outside of the site is likely to be non-existent or minimal. This is because 1) the sub-ground walls should provide an effective, continuous cut-off curtain, and 2) the depth of draw-down of the water table is not great, around 1m, and so there will not be much ‘head’ or pressure to push water through the contiguous sub-ground wall. I note this is confirmed in the Groundwater and Contamination Assessment.

(c) Some water may come up through the base of the contained excavation. This is not expected to be significant because of the long flow path under the base of the sub-ground walls and through the denser and less porous materials at the base of the reclamation. Contaminants mobilised through this process are likely to be removed from the general flow of ground water towards the sea, i.e. they will become contained beneath the building.

(d) I note that there will be long term benefits resulting from either the DSM or CFA walls, in relation to contamination. The walls will have the effect of encapsulating all the existing contaminants that lie beneath the site footprint. This eliminates their potential to further contaminate the harbour. In addition the walls will significantly slow/reduce the ongoing flow of contaminated ground water towards the harbour.

CONCLUSIONS

42. It is my opinion that natural hazards and in-ground contamination have been adequately addressed with the mitigation measures proposed for the Site 10 development.

Adam William Thornton
3 July 2015

APPENDIX A: Site 10 – Basement Construction Method Statement

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Site 10 – Basement Construction Method Statement

The Site 10 basement will be constructed in reclamation fill, adjacent to the harbour edge but inside the existing Seawall. Construction activities will include excavation, removal of existing foundations (from previous structures on the site), piling, Deep-Soil-Mixing [DSM], de-watering and construction of the reinforced concrete basement slab and walls. The following steps outline, in concept, the construction methodology that will be used.

1. Site establishment, fencing, site sheds etc.
2. Storm-water protection/diversion etc. Temporary filters, kerbs etc. to prevent construction and excavation materials entering the storm-water system.
3. Site-wide excavation to approximately 400mm deep. This is to provide a bund against any spills or flooding. The excavated material shall be assessed for contamination, treated if required and disposed to landfill/cleanfill as appropriate.
4. Construction of perimeter DSM wall around the entire building footprint. This will provide a cut-off wall to minimise the likelihood of groundwater flow towards the harbour. It will also act as temporary retaining for excavation inside the perimeter wall.
5. General excavation to expose the existing, remaining foundations and lower general ground surface levels. The excavated material shall be assessed for contamination, treated as required and disposed to landfill/cleanfill as appropriate.
6. Demolition of existing foundations. Debris will be removed to landfill.
7. Additional proof-drilling to determine depths for piles and the internal DSM walls.
8. Drilling and concreting of piles.
9. Construction of the internal DSM grid of walls over the whole building footprint.
10. Localised de-watering down to underside of the proposed new basement. Note that the grid of DSM should minimise groundwater flow from adjacent cells with the grid. Water will be pumped to settling tanks, treated as appropriate and disposed to the storm-water system.
11. Excavation will then continue to the underside of the basement slab. The excavated material shall be assessed for contamination, treated if required and disposed to landfill/cleanfill as appropriate.
12. Construction of a concrete tidy slab across the basement footprint.

13. Construction of the structural, reinforced-concrete basement slab in a checkerboard, hit-and-miss process.
14. Construction of the basement walls, generally poured directly against the perimeter DSM walls.
15. Progressive construction of the internal basement columns, base-isolators, ground floor slab and building superstructure.
16. De-watering would continue until the building mass exceeds the groundwater uplift pressure.

Note that the sequence described above would be a progressive one starting from one end of the site. I.e. Slabs and walls may be in progress at one end of the site while excavation is still underway at the other end.

Dunning Thornton Consultants Ltd
140919

Consulting Structural Engineers
94 Dixon Street, PO Box 27-153, Wellington 6141
Telephone (644) 385-0019, E-Mail: dtcwgtn@dunningthornton.co.nz

APPENDIX B: Site 10 – Structural Effects

Site 10 – Structural Effects

Site 10 presents a reasonably significant challenge from a natural hazard perspective with its high shaking hazard, liquefaction/lateral spreading potential and susceptibility to Tsunami/Seiching waves. The site is also relatively low and could be subject to potential, occasional, future inundation, particularly as a consequence of prolonged sea-level rise. The structural and architectural design mitigates these issues with a high-performance structure expected to perform well in excess of code minimum requirements. In addition the development will result in the remediation of existing in-ground contamination.

The liquefaction and lateral spreading arises through the relatively un-compacted reclamation fill dating from 1900. As part of the development significant sub-ground mitigation will be provided to address the liquefaction/lateral spreading potential through the use of Deep-Soil-Mixing [DSM] a technology that uses a deep drilling auger to mix cement with the underlying soils to form a grid of weak-concrete walls, spaced at around 4m centres, over the whole site. This grid of walls provides five functions; confinement of the liquefiable soils, lateral bracing down to the non-liquefiable sub-strata, a platform on which to found the new structure, encapsulation of contaminated material and effective cut-off of contaminated ground-water flow towards the harbour.

Localised de-watering will occur during construction to enable formation of the basement slab. The de-watering will be restricted to zones within the grid of DSM wall and typically will only lower the water table by 1m to 1.5m. This will be a localised effect, due to the presence of the DSM walls and the ready re-supply of water from the groundwater and the sea. Accordingly, water-tables beneath neighbouring buildings will not undergo significant change. In any event, the buildings in the immediate vicinity are deep-founded and so would not experience loss of support as a result of ground water changes.

Above the foundation the building superstructure will be base-isolated to provide an extremely high level of seismic life-safety protection coupled with damage avoidance, business continuity and protection of contents. Base isolation will provide seismic, life-safety performance in excess of Importance Level 3 [IL3]. Above the base isolators the structure will be predominantly steel-framed to provide the strength and resilience at the least weight. The upper floor slabs will be reinforced concrete.

Below the isolation plane will be a single-level, tanked basement to provide space for services and off-waterfront parking. Excavated material from the basement will be tested for contamination and treated/disposed as appropriate.

The new building ground floor will be set as high as practicable while still providing access from existing waterfront levels. This means that the ground floor and basement levels may become susceptible to occasional inundation as a result of sea-level rise after approximately 100 years. Future mitigation to prevent flooding of the ground floor will be possible by simply raising the building at the isolator level. Lifting technologies capable of raising the building structure are already in existence. Mitigation options to prevent flood waters from entering the basement will include raising of the crest to the vehicle ramp, together with the surrounding ground surface levels and/or the installation of flood-gates that could be used for the duration of an exceptionally high tide event.

As with other low-lying properties around the Wellington region, ground floor spaces may be inundated during Tsunami or Seiching waves. The first floor level has been set sufficiently high to avoid damage, based on maximum wave height predictions. While significant damage could be expected to the ground floor non-structural elements, the primary structure will have sufficient resilience to resist the wave actions.

Dunning Thornton Consultants Ltd
141022

Consulting Structural Engineers
94 Dixon Street, PO Box 27-153, Wellington 6141
Telephone (644) 385-0019, E-Mail: dtcwgtn@dunningthornton.co.nz