EXECUTIVE SUMMARY

1.0 Background

Spencer Holmes Limited has been commissioned by the Greater Wellington Regional Council (GWRC) to review the Greater Wellington Regional Council buildings at 142 Wakefield Street, Te Aro, Wellington.

Our brief is to prepare a report on the probable strength of the building with respect to current code requirements for the purpose of informing the Greater Wellington Regional Council and associated parties on the of likely seismic performance of the building in light of the recent Christchurch Earthquakes.

The GWRC buildings consist of a 10-storey tower block constructed along the Wakefield Street frontage to the property at 142-146 Wakefield Street and an independent 5-storey car parking and office annex building constructed at the rear of the tower block. For operational purposes the two buildings are one structure but are in fact separate buildings.

For the purpose of reporting on the performance of each part of the building these are referred to in the report as;

- The GWRC Main Tower Building, (10-Storey Tower)
- The GWRC Car park Building, (Adjacent 5-Storey Car Park and Office Building)

The building was designed in 1985 and built soon after, which was close to the peak period in the development boom prior to the Stock Market Crash November 1987.

The building was designed for Williams Property Holdings by Alun Wilkie Associates, Architects and D. J. Falloon, Consulting Engineer both of Christchurch.

In May 1990, Wellington Regional Council staff discovered cracks in the columns on Level 3 and an investigation of the columns established a poor standard of workmanship in general for the building construction with low concrete strength in the columns. Strengthening of selected columns was undertaken as was repair of the seismic joint.

Significant inadequacies in the fixing of the pre-cast concrete panels were also identified soon after the column strengthening was undertaken, and work was undertaken in upgrading the fixings of the panels.

In 2004, a review of the performance of the hollow core flooring systems used throughout the building was undertaken and reported on in June 2005.

There have been other minor requests for structural advice in respect of secondary damage to the building resulting from the construction of the Museum of New Zealand and other staff concerns.

The building was designed in accordance with the 1976 loadings code and/or the 1984 update however, there has been significant advancement in material detailing requirements and understanding in the last 26 years since then.

As the loading codes from 1976 to 2008 have evolved, the equivalent lateral load requirements to less ductile wall elements such as those found in the GWRC building have increased significantly.

This is mainly due to the early design codes such as the 1976 code assuming, but not requiring, an acceptable level of structural ductility or detailing in the building and the adoption of capacity design procedures to prevent brittle structural failures.

There have also been substantial increases in code loadings for buildings on "flexible soil types", now recognised as susceptible to site amplification of ground shaking.

The site is located adjacent to the old pre 1880s shoreline and to the west of a historic stream with its head waters in Aro Valley and has a subsoil classification in accordance with NZS 1170.5: 2004, of Class D or "Deep subsoil".

The area is also classified on the Greater Wellington Regional Council maps as "High" potential for liquefaction. This means the site and the general surrounding area around the building are likely to be subject to high ground shaking amplification and significant liquefaction under the 2 likely large earthquake scenarios defined by the GWRC assessment. i.e. a large, distant, shallow earthquake or a large earthquake centred on the Wellington Fault.

EXC 2.0 Explanation of Building Code Requirements - Definitions

In order to undertake this assessment we have undertaken a three-dimensional computer analysis of each of the buildings to check the strength of the shear walls, and to include a review of the gravity systems, including the Dycore flooring, and non-structural elements such as the pre-cast panels and in accordance with expected code performance.

The three levels of review in the report in accordance with the building code are summarised below;

EXC 2.1 Ultimate Limit State - Definition

Performance Under a Large - 1/500 Year Level Earthquake - Definition

We have undertaken this assessment based on this building being an importance level 2 (IL2 or ordinary) type structure* from NZS 1170.

**Note: the structure not being designated for post-disaster functions or use.*

If post disaster functions or use are intended for the GWRC building then assessment as an IL 4 building would be appropriate. However, the expected performance requirements of an IL 4 building are beyond the capability of this building to withstand seismic actions.

The ultimate limit state design earthquake load for an IL2 building from the seismic provisions of NZS 1170.5 is based on a return period of 1/500 years.

EXC 2.2 Serviceability Limit State – Definition

Performance Under a Serviceability - 1/50 Year Earthquake - Definition

The serviceability limit state earthquake for an IL2 building is defined as an event of ¼ of the intensity of shaking of an ultimate limit state event and is based on a return period of 1/50 years.

EXC 2.3 "Moderate Earthquake" and Earthquake Prone Building Requirements - Definition

Performance Under a Moderate – 33% of the 1/500Year Earthquake - Definition

The building act also requires existing buildings to be assessed with respect to performance with a "moderate" level earthquake.

From section **122 of the Building Act.**

A building is earthquake prone for the purposes of this Act if, having regard to its condition and to the ground on which it is built, and because of its construction, the building -

(a) will have its ultimate capacity exceeded in a moderate earthquake (as defined in the regulations); and

(b) would be likely to collapse causing -

(i) injury or death to persons in the building or to persons on any other property; or
(ii) damage to any other property.

With a moderate earthquake defined as

> ***moderate earthquake means,*** *in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is* ***one-third as strong*** *as, the earthquake shaking (determined by normal measures of acceleration, velocity and displacement) that would be used to design a new building at that site.*

To summarise this; To be considered earthquake prone – the building code definition requires a building as a whole or part to have its ultimate strength exceeded in a moderate earthquake, (33% of the 1/500 year earthquake), and be likely to cause injury or death to persons within the building.

EXC 2.4 Status of GWRC Building - Earthquake Prone Building Requirements

It is our opinion the Greater Wellington Regional Council Building as a whole is not expected to be classified an earthquake prone building in this respect.

This report has identified several elements of structure and also non-structural elements that may not perform to the required 33% levels of the code and that could meet the definitions of the act, i.e. like the fixings to the exterior pre-cast cladding panels.

However further detailed study of these and the post elastic behaviour of the main structural elements, foundations, and interaction with other elements is required before a definitive conclusion on these can be reached.

EXC 3.0 GWRC Main Tower Building Performance

We have undertaken an analysis of the tower buildings to check the strength of structural and non-structural elements in accordance with expected code performance levels. The results are summarised below;

EXC 3.1 Drift Displacements

The allowable maximum drift displacements under the current code are up to 2.5% for structures of the type of the GWRC building.

The code also requires critical elements, such as the floor and gravity structure, not to collapse at displacement levels greater than the design level earthquake called the maximum considered earthquake or MCE, this is 1.8 x the displacement of the design level and based on a 1/2500 year return period.

The main walls in the longitudinal direction are within the current code requirement however, the transverse walls exceed these limits. Due to diaphragm flexibility and torsion the drift at the exterior of the building is further amplified and is over 75% more than current code requirements for the exterior corner columns.

High drift displacements have the following effects on the building;

- Increased ductility demand on shear walls
- Increased risk of failure or damage of the pre-cast "Dycore" flooring system.
- Increased forces and ductility demand on the floor diaphragm.
- Increased P-Delta effects on columns (additional forces due to displacement).
- Higher ductility demand on the non-seismic gravity structure.
- Increased pounding issues with the adjoining car park building and buildings on adjoining sites.
- Increased demand on stair detailing and seating problems.
- Increased demand on non-structural fixings and elements such as windows, ceilings, pre-cast panels, and services.

EXC 3.2 Main Shear Walls

Flexural Strength

The review found the tower building shear walls are designed to approx 75% of the flexural capacity required by the current code for "ductile" detailing.

The original design intent for the building was for the walls to behave in a ductile manner and reach a sustained flexural yield without loss of significant strength.
The "as built" details of the shear wall however do not meet the current code requirements for this and do not have sufficient shear reinforcing to meet the expected demand placed on the walls due to capacity design and over strength actions. Consequently, the walls are not expected to behave in the ductile manner assumed by the designer but behave in a more brittle manner where strength is lost more quickly once flexural yield is attained.

The original building design and code assumed that the building would be detailed for a ductility of at least $\mu = 3$ however our assessment of the building is that without major upgrading work being undertaken, the building is unlikely to achieve a ductility much in excess of $\mu = 1.25$ based on current code levels of detailing.

The effective level of strength of the main shear walls for the building is therefore decreased from the design level for ductile detailing and based on nominally elastic $\mu = 1.25$ design actions from the current code are in the order of ;

Transverse Sear walls	25% to 35% NBS
Longitudinal Shear walls	35% to 45% NBS

% NBS is defined as "Percent New Building Standard"

We would note that there is a level of conservatism built into the current design codes (and the assessment is based on comparison to these) to give a high level of certainty that any detailing of new walls will guarantee the desired ductile failure mechanism.

There is less certainty around the performance of walls not detailed to the current code levels and they may behave better than the above gross percentage figures suggest.

Further detailed study of the failure behaviour of these walls, and interaction with other elements, in particular with regard to the effects of foundation conditions is recommended.

Shear design

Modern design requires significant over strength to be built into the walls such that they would yield in flexure in a sustained manner prior to a shear failure.

This design procedure called "capacity design" and allows a ductile response of the wall, i.e. where the wall can accommodate large deflections due to seismic demand without significant loss of strength.

We have found that there is insufficient shear reinforcing in the building walls to meet this ductility requirement and less certainty about the overall behaviour of the structure.

The review also identifies other critical structural weaknesses and deficiencies with respect to other current code detailing requirements for these walls.

Rectifying these weaknesses and providing confinement of the shear walls to increase the ductility would considerably increase the seismic performance of the building

EXC 3.3 Vertical And Horizontal Irregularities In The Structure

What has been observed in the recent Christchurch (2010 and 2011) earthquakes and other significant earthquakes such as the Kobe (1995) and North Ridge (1994) earthquakes is that significant changes in stiffness or strength of building elements can lead to unexpected local failures in building that can initiate a more catastrophic failure.

Undesirable structural features found in our review of the GWRC building were;

- Change in Frame Stiffness Level 5
- Reduction in Column Shear Reinforcement above Level 5
- Reduction in Shear Reinforcement in walls above Level 5
- Horizontal Steps in Columns B/6, B/15, C15
- Stiff block work walls over 2 stories in the loading dock
- Uneven distribution of mass throughout the building
- Rigid stair connections leading to un-anticipated loads and stiffness effect
- Floor at the north (Wakefield St) end is supported by a cantilevered 600x300 concrete beam loading the main support beam in torsion.
- The longitudinal shear wall on the west side is on the edge of the foundation beam.
- Rigid connection of the pre-cast "spandrel" panels to the gravity beams along the western and southern sides of the building.

EXC 3.4 Diaphragms and Dycore Pre-Cast Flooring Performance

Based on the findings of the report we are of the opinion that there is a potential for damage to the floor diaphragm and the pre-cast Dycore flooring to the Tower.

The damage is expected to be more pronounced in the Tower building than the Car park due to the higher drifts that will initiate an earlier onset of damage to the floor system in this building.

In the case of a Serviceability 1/50 yr earthquake or the slightly larger "Moderate Earthquake" (33% code) level earthquake defined by the Building Act, the recommendations and observations from the University of Canterbury and the DBH study group on Dycore floors indicate that the support to the pre-cast floor is not likely to be damaged to the extent to initiate collapse at the predicted drift levels for this building for either a "serviceability" or "moderate" level earthquake.

We are of the opinion the floor structure will suffer some considerable visible damage with the formation of flexural and shear cracks in the flooring and possible rupture of mesh in the topping slab but not collapse.

In a 1/500 year or larger event, the DBH study group findings indicates that there is increased potential for greater damage to the floor at the building drifts expected in a large earthquake that could possibly lead to a localised collapse of the Dycore floor units, particularly in areas subjected to greatest deformation and/or drift.

The areas of high damage are generally near the shear walls at the centre of the building, whereas the areas of greatest drift are at the Wakefield St frontage of the building.

We understand that there was no total collapse of any Dycore flooring observed in the Christchurch earthquakes, but that the Dycore flooring to some buildings had suffered severe damage, loss of support, and structural cracking.

EXC 3.5 Gravity Structure

Seismic induced loading to the gravity structure occurs when frames have to displace and move with the rest of the building during an earthquake. The configuration and geometry of the building is important and buildings with torsional or irregular geometry, or poorly located lateral load resisting elements allow greater displacements than regular and well proportioned systems.

This is the case with the GWRC tower building, the outer frames at the front northern end of the building displace up to 50% more than the shear walls due to torsion and diaphragm flexibility.

Based on the estimated code level displacement the flexural capacity of the columns for combined earthquake and gravity loading have been assessed to be in the order of 70% of current code 1/500 strength levels.

The code also requires critical elements, such as the gravity frames, not to collapse at a displacement level greater than the design level earthquake.

The ground floor columns in particular undergo high inelastic actions in an earthquake to accommodate the building drift and are subject to higher axial load than the columns further up the building. This makes confining and shear reinforcing in the ground floor columns critical to the overall performance of the gravity frames to prevent brittle failure or collapse mechanisms occurring.

The columns at the base of the building were reviewed for shear and confining steel requirements in accordance with the current concrete standard NZS 3101:2006, and do not meet the current code requirements.

A positive aspect of the previous strengthening works undertaken in the 1990s on the building is that at least some of the columns were retrofitted with steel or FRP jacketing as part of the earlier remedial works, which will significantly improve the performance of these columns.

EXC 3.6 Pre-Cast Stair Performance

There were a number of pre-cast concrete stair failures observed in multi-storey buildings after the Christchurch 22 February 2011 earthquake.

Typically, pre-cast stairs are designed and detailed such that there is a fixed connection and a free sliding connection. The failure mode surmised in the 22 February 2011 earthquake was, as the earthquake was larger than expected, the displacements were also greater than the seating provided, and the stairs lost support and collapsed, with some stairs also crushing under axial loads induced (due to detailing) shortening the stair lessening the bearing support.

The GWRC building design does not incorporate this detail and the stairs are fixed support at both ends. This precludes the stair failure mechanisms above observed in the Christchurch earthquakes, however the detail used on the GWRC building is problematic for different reasons.

The fixed connections cause very high axial compression and tension forces to be induced in the stairs, as the building moves in an earthquake which has an undesirable effect on both the building performance and the stair.

Further investigation into the behaviour of the stair is recommended.

EXC 3.7 Pre-Cast Panel Performance

In 1991, remedial works were undertaken on some of the panel fixings that were identified as a potential problem for the then code.

The detailed archive records on the actual remedial work completed to the panels on site were unable to be located. However, it is understood that a reasonable number of the panels were checked on site and those found to have poor tolerances, workmanship issues or potentially under strength were improved to suit the code requirements of the time – or as near as practical to these requirements.

There have been significant increases in requirements in the (non structural parts and portions) sections of the loadings of the code for the fixings of since this remedial work was undertaken and the east wall panels in particular are a concern.

These panels extend over two levels and are affected by building movements and inter-storey displacements during an earthquake far more than panels fixed to a single level only.

As the building (and each separate floor) displaces under earthquake loading the fixings of the panels to the building transform with the building into a diamond shape, however the stiff concrete panel, which is a rigid rectangle, does not.

The result is that the connections are required to be flexible or allow for sliding horizontal and vertical movement without locking up other wise considerable forces will be generated in the connections which are not designed for this.

The strength of the panels and the level of displacement at which lock up of the connections may occur have been assessed as considerably below current code levels.

The level of strength of the panels on the eastern side of the building are potentially less than would be considered a "moderate earthquake" or less than 33% current code. The failure of the heavy pre-cast cladding panels in "moderate level earthquake" would cause a life safety and property hazard to building users and the public, and the building may be deemed an "Earthquake Prone" hazard due to this issue in accordance with the current legislation.

We do not have sufficient details of the as built construction to support this conclusion and accordingly, we would recommend that a more detailed investigation into the construction and remedial work undertaken in 1991, and how this complies with the current code, including possible retrofit options be undertaken.

EXC 3.8 Non-Structural - Ceilings

We understand the GWRC has had a programme of removal of the older "heavy tile" ceilings and replacement and retrofit with lightweight ceiling tiles has occurred to a number of levels in the building as redecoration, fit outs and alterations are progressively undertaken.

This will improve the performance of the ceiling for life safety and functionality, however based on the Christchurch earthquakes, damage to the ceiling grid and loss of tiles was observed to be a problem even for new ceilings with a level of engineering design to the current code.

If the building is required to be operational in the period immediately following a moderate to major earthquake, it is likely similar damage to that observed to Christchurch would occur to the GWRC building.

EXC 3.9 Non-Structural - Shelving, Office Furniture, etc

Unless all shelving, desks, computer terminals, storage units, etc are effectively restrained they are a potential danger to staff in a moderate to major earthquake and can be damaged so that the essential facilities are not operational in the period immediately following the earthquake.

We understand that a programme for providing seismic restraints to all tall and heavy office furniture has been implemented.

EXC 3.10 Non-Structural - Window Systems

There are no records available of the design or construction details of the windows for the building, however it is unlikely they were installed to accommodate the anticipated level of movement of the building under current code requirements.

EXC 3.11 Non Structural - Partitions

High demand is expected on the partitions to the building with inter-storey drifts in a moderate to major earthquake event. Where the internal partitions are fixed from floor to floor, significant damage can be expected such damage will result in less than desirable operational conditions following a moderate to major earthquake.

EXC 3.12 Other Services / Sprinkler Systems

The adequate performance of the sprinkler systems and other services in a moderate to severe earthquake can only be assured when a rigorous and integrated approach to securing to the structure of the building and providing sufficient flexibility where required to accommodate building displacements is undertaken.

For example the failure to restrain ceiling tiles and sprinkler pipes can result in activation of sprinklers that will result in water damage to the premises in a small to moderate earthquake. This resulting damage will result in less than desirable operational conditions following an earthquake.

It is our observation the services installation have been installed in an ad hoc manner over the last 20 years rather than being considered in a total integrated manner, and therefore the performance is expected to be varied.

EXC 3.13 Non Structural - Underground Services

Based on the experience with the Christchurch earthquakes it is likely that some of the services would be disrupted or damaged in a moderate earthquake that caused liquefaction of the upper level soils. There may also be some damage to the main water reticulation and drainage systems along Wakefield St and Jervois Quay that could limit the use or occupation of the building after a moderate earthquake.

In a very large earthquake due to the significant liquefaction and additional influence of lateral spreading along the Wellington waterfront it is expected that considerable damage would occur to the services to the building and to Wellington City Councils main water reticulation and drainage systems along Wakefield St and Jervois Quay.

EXC 3.14 Seismic Joint Performance

Concern with the insufficient seismic separation between the GWRC tower building and the parking building was highlighted soon after its construction and extensive remedial work undertaken was on the seismic joint circa 1992 to improve the situation.

It is understood this joint was opened up as much as possible within the physical constraints of the existing structure and the separation adjoining structure was improved to about 80mm.

The drift displacements required in accordance with current code requirements of both the tower and car park building exceed these limits and there is potential for severe pounding damage to these buildings.

We would recommend that a detailed investigation into possible damage scenarios to the car park and /or retrofit options for the seismic separation and joint to the building be undertaken.

EXC 3.15 Foundation Design

In a large (full code level) earthquake it is very likely that the ground will liquefy and the building foundations will be subjected to unfavourable foundation conditions not accounted for in the original design.

The implications on the building structure are serious and could lead to a potential failure of the , piles, gravity structure, severe damage of the shear wall foundations, and unpredictable behaviour of the seismic resisting system.

We have undertaken a preliminary assessment of the foundations and this indicates the following issues;

- The piles appear to not to comply with current code ductile plastic hinge detailing requirements.
- The piles appear to have insufficient geotechnical and lateral load capacity to comply with current code levels or over strength requirements of the seismic system.
- The piles are very lightly reinforced near the base of the piles with only R6 wire spirals at 400mm crs – this does not comply with current minimum confinement or shear reinforcement for gravity or earthquake.

Further detailed geotechnical assessment of these is strongly recommended and should include;

- comprehensive site testing and drilling with bore log and CPT to verify the site liquefaction analysis
- site specific hazard assessment for the site
- settlement analysis (under liquefaction)
- finite element modelling for the soil structure interaction and loads and displacements
- analysis of the soil effects on the building structure

EXC 4.0 GWRC Parking Building Performance

The GWRC Parking Building is a five story building similar in construction to the main tower constructed with reinforced concrete gravity frames and concrete shear walls.

A basic three-dimensional computer analysis of the car parking building was also undertaken with similar issues to the tower building discovered.

The results are summarised below;

EXC 4.1 Drift Displacements

The walls in the longitudinal direction are within the current code requirement, however the transverse walls exceed these limits.

As with the Tower, due to diaphragm flexibility and torsion the drift at the exterior of the building is amplified in the transverse direction and is over current code limits.

EXC 4.2 Main Shear Walls – Flexural Strength

The review found the Car parking building shear walls are designed in a similar manner to the main Tower and would be badly damaged under even a moderate level earthquake.

Based on the corresponding level of ductility of $\mu = 1.25$ from NZS1170.5, the estimated level of the strength of the main shear walls of the building to be approximately;

Transverse Shear walls	25 -35% NBS*
Longitudinal Shear walls	25-35% NBS*

Notes: *%NBS is defined as "Percent New Building Standard".*
** Ductile behaviour limited by shear failure of the wall prior to sustained flexural yield.*

EXC 4.3 Main Shear Walls – Shear Strength

We have reviewed the details for the car park and found as with the main tower building there is insufficient shear reinforcing in the main walls to meet current code requirements.

The assessed level of shear reinforcing for the walls were found to be;

Transverse shear walls	48% of current code requirements
Longitudinal shear walls	56% of current code requirements

The result is that the walls will not behave in a ductile manner but will fail in a more brittle manner once flexural yield is obtained and the strength capacity will drop off significantly once the walls shear strength is exceeded.

This review also identifies other critical structural weaknesses and deficiencies with respect to other current code detailing requirements for these walls.

Further detailed study of the failure behaviour of these walls, and interaction with other elements, in particular with regard to the effects of liquefaction on the foundation conditions is recommended.

EXC 4.4 Vertical And Horizontal Irregularities In The Structure

Undesirable structural features found in our review of the parking building were;

- Significant change of wall stiffness at lower levels
- Irregular general wall and building mass layout
- Sloping ramps at lower levels rigidly connected to adjacent floor levels
- Double height walls at south west corner (around ramp).

EXC 4.5 Diaphragms and Dycore Pre-Cast Flooring Performance

Based on the findings of the report we are of the opinion that there is a potential for damage to the floor diaphragm and the pre-cast Dycore flooring to the Car park .

The damage is expected to be less pronounced than in the Tower building. due to the higher drifts which will initiate an earlier onset of failure of the floor system in this building.

In the case of a Serviceability 1/50 yr earthquake or the slightly larger "Moderate Earthquake" (33% code) level earthquake defined by the Building Act, the recommendations and observations from the University of Canterbury and the DBH study group on Dycore floors indicate that the support to the pre-cast floor is not likely to be damaged to the extent to initiate collapse at the predicted drift levels for this building for either a "serviceability" or "moderate" level earthquake.

We are of the opinion the floor structure will suffer some damage with the formation of flexural and shear cracks in the flooring and possible rupture of mesh in the topping slab but not collapse.

In a 1/500 year or larger event, the DBH study group findings indicates that there is increased potential for greater damage to the floor at the building drifts expected in a large earthquake that could lead to a localised collapse of the Dycore floor units, particularly in areas subjected to greatest deformation and/or drift.

We understand that there was no total collapse of any Dycore flooring observed in the Christchurch earthquakes, but that the Dycore flooring to some buildings had suffered severe damage, loss of support, and structural cracking.

EXC 4.6 Gravity Structure

The car park building is a shear wall building similar to the Tower with the main walls used for resistance of seismic lateral loads, and gravity concrete frames.

The concrete gravity frames were not designed to be part of the seismic resisting system, (as with the Tower) however these frames are still subject to seismic induced loading.

Specific columns were reviewed for shear and confining steel requirements and found to not meet the current concrete standard NZS 3101:2006 requirements.

EXC 4.7 Pre-Cast Panel Performance

The east wall panels to the parking building are similar in construction to the main tower building and extend vertically over two levels

The strength of the panels and the level of displacement at which lock up of the connections may occur are considerably below current code levels.

The level of displacement the panel connections on the eastern side of the building are able to accept make them Potentially vulnerable in a "moderate earthquake" and the panels could be deemed "Earthquake Prone" in accordance with the legislation. We do not have enough information on the as built construction of these panels to support this conclusion however, and recommend that a further more detailed investigation work be undertaken.

EXC 4.8 Secondary Damage –Windows, Partitions, Ceilings and Services

The car park building has similar detailing of non-structural items to the tower and similar comments and recommendations apply.

EXC 4.9 Seismic Joint Performance

The drift displacements required in accordance with current code requirements of both the tower and car park building exceed these limits and there is potential for severe pounding damage to these buildings.

We would recommend that a detailed investigation into possible damage scenarios to the car park and /or retrofit options for the seismic separation and joint to the building be undertaken.

EXC 4.10 Foundation Design

The possible effect on the car parking building structure due to liquefaction is similar to the main Tower building. Further detailed geotechnical assessment of this is recommended.

EXC 5.0 Summary of Findings

There has been significant advancement in material detailing requirements and understanding in the last 26 years since the building was designed circa 1985.

Many of these lessons are from recent earthquakes.

There have also been substantial increases in code loadings for buildings on "flexible soil types", now recognised as susceptible to site amplification of ground shaking.

In particular, with the introduction of the most recent loadings codes NZS1170.5 in December 2008 these increases are in many instances are greater than 50% of the previous NZS4203: 1992 loadings code.

The ability to analyse buildings in a more detailed manner has also improved dramatically over the last 5 to 10 years with the increase in availability of more powerful desktop computer systems and user-friendly commercial building analysis software.

The building was assessed in 1991 and found to have some deficiencies with respect to the gravity structure requiring remedial work. It addition the compliance with various seismic detailing aspects were found not to fully meet the then current 1984 code but were judged broadly tolerable.

The development of codified knowledge and understanding of earthquakes over the last 25years has culminated with the most recent loadings code NZS1170.5 being gazetted in December 2008.

Although the Greater Wellington Regional Council Building is a relatively modern building the building (along with similar buildings of the era) has some inadequacies with respect to seismic performance that are below current code requirements when reviewed with respect to the current code in 2011, and the additional lessons learnt from the Christchurch Earthquakes.

Further study of these is recommended however it is our opinion the Greater Wellington Regional Council Building as a whole is not expected to be classified an earthquake prone building with respect to the Building Act due to these inadequacies.

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Report Revisions

Date	**Revision**
Thursday 30th June 2011	Revision 1– Client Review