Jellie Park Recreation & Sport Centre

BU 0266-006 EQ2

Indoor and Outdoor Hydroslides

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Jellie Park Recreation & Sport Centre Hydroslides
Detailed Engineering Evaluation

Quantitative Assessment Report

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Jellie Park Recreation & Sport Centre
Indoor and Outdoor Hydroslides
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Detailed Engineering Evaluation
Quantitative Report – EXECUTIVE SUMMARY
Final

295 Ilam Road
Burnside, Christchurch

Background

This is a summary of the Quantitative report for the two hydroslide structures located at the Jellie Park Recreation & Sport Centre, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. Visual inspection was performed on February 28, 2012. Construction documents made available are noted below:

- Jellie Park Redevelopment architectural as-built drawings by Warren and Mahoney, dated 26 January 2009.
- Jellie Park Redevelopment structural drawings for the gym and changing room areas. Drawings by Powell Fenwick Consultants Limited dated January 2007.
- Jellie Park Redevelopment structural drawings for the new indoor pool. Drawings by Powell Fenwick Consultants Limited dated January 2007.

Key Damage Observed

Key damage observed includes:-

- Cracked slab on grade at the indoor hydroslide stair tower.

Critical Structural Weaknesses

The following potential critical structural weaknesses have been identified with the accompanying %NBS for that portion of the building.

- Outdoor hydroslide platform pipe bracing (69%NBS)
- Outdoor hydroslide platform pipe bracing connections (99%NBS)
- Uplift of outdoor hydroslide platform column foundations (35%NBS)
- Sliding of outdoor hydroslide platform column foundations (50%NBS)
- Uplift of indoor hydroslide brace frame tower structure column foundations (35%NBS)

Indicative Strength of Hydroslides (based on quantitative DEE and CSW assessment)

Based on the information available, and using the NZSEE Detail Engineering Evaluation procedure, both hydroslides have a capacity of 35%NBS. The seismic performance of the outdoor hydroslide is governed by the uplift capacity of the platform's foundations and the seismic performance of the indoor hydroslide is governed by the uplift capacity of the foundations at the braced frame structure supporting the mouth of the slide.

Recommendations

- Further investigation into the thickness of the outdoor hydroslide pipe bracing in order to determine a more exact %NBS of these structural elements,

- A strengthening works scheme be developed to address the inadequate foundations at the outdoor hydroslide platform structure.
- A strengthening works scheme be developed to address the inadequate foundations at the braced frame structure of the indoor hydroslide.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the two Jellie Park Recreation & Sport Centre hydroslides following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the structures are classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

2 Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to structures in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to structure safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a structure is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a structure owner, insurer or mortgagee to carry out a full structural survey before the structure is re-occupied.

We understand that CERA require a detailed engineering evaluation to be carried out for all structures (other than those exempt from the Earthquake Prone Building (Structure) definition in the Building Act). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the structure.

2. The placard status and amount of damage.

3. The age and structural type of the structure.

4. Consideration of any critical structural weaknesses.

Any structure with a capacity of less than 34% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building (Structure) Policy.

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 - Alterations

This section requires that an existing structure complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a structure cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) is satisfied that the structure with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new structure. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Structures

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a structure as dangerous if:

1. In the ordinary course of events (excluding the occurrence of an earthquake), the structure is likely to cause injury or death or damage to other property; or
2. In the event of fire, injury or death to any persons in the structure or on other property is likely because of fire hazard or the occupancy of the structure; or
3. There is a risk that the structure could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the structure is dangerous.

Section 122 – Earthquake Prone Structures

This section defines a structure as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property.

A moderate earthquake is defined by the structure regulations as one that would generate loads 33% of those used to design an equivalent new structure.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any structure defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Structure Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary structures.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Structures, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new structure for structures that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Structures to be strengthened; and,
4. Repair works for structures damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the structure to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.

- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the structure consent application.

2.4 Building Code

The Building Code outlines performance standards for structures and the Building Act requires that all new structures comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on structure occupancy and access must be made with these fundamental obligations in mind.

3 Earthquake Resistance Standards

For this assessment, the structure's earthquake resistance is compared with the current New Zealand Building Code requirements for a new structure constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing structures in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Table 1 below.

<table>
<tr><th>Description</th><th>Grade</th><th>Risk</th><th>%NBS</th><th>Existing Structure Structural Performance</th><th colspan="2">Improvement of Structural Performance</th></tr>
<tr><td>Low Risk Structure</td><td>A or B</td><td>Low</td><td>Above 67</td><td>Acceptable (improvement may be desirable)</td><td rowspan="2">The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.</td><td>100%NBS desirable. Improvement should achieve at least 67%NBS</td></tr>
<tr><td>Moderate Risk Structure</td><td>B or C</td><td>Moderate</td><td>34 to 66</td><td>Acceptable legally. Improvement recommended</td><td>Not recommended. Acceptable only in exceptional circumstances</td></tr>
<tr><td>High Risk Structure</td><td>D or E</td><td>High</td><td>33 or lower</td><td>Unacceptable (Improvement required under Act)</td><td>Unacceptable</td><td>Unacceptable</td></tr>
</table>

Table 1: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table 2 below compares the percentage NBS to the relative risk of the structure failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 2: %NBS compared to relative risk of failure

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

– The Canterbury Earthquake Order[i] Council on 16 September 2010 modified the meaning of “dangerous structure” to include structures that were identified as being Earthquake Prone Structures. As a result of this, we would expect such a structure would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the structure (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an Earthquake Prone Structure.

3.1.2 Cordoning

– Where there is an overhead falling hazard, or potential collapse hazard of the structure, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

– Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

– It should be noted that full compliance with the current building code requires structure strength of 100%NBS.

3.1.4 Our Ethical Obligation

– In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous structures; this would include earthquake prone structures.

[i] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

4 Background Information

4.1 Outdoor Hydroslide

Constructed in 1983, the outdoor hydroslide is located northwest of main outdoor pool at Jellie Park. The hydroslide is an 8.5 metre high steel platform structure with three fibreglass hydroslides and an outer winding steel staircase. Supporting the structure's 4.9 metre by 7.1 metre elevated steel platform are four tubular steel columns laterally braced by tubular steel bracing. The steel staircase is supported off of the platform's steel columns. Gravity loads and seismic loads are resisted by the braced steel frames. The foundations for the four tower legs consist of 700mm diameter concrete piles. Hand dug trial pit excavations show that the piles are at least 1m deep. A mechanical excavator would be required to determine the pile depth beyond this point.

Single cantilevered steel pipe columns support the curved hydroslide. The two straight hydroslides are supported vertically and laterally by single cantilevered steel pipe columns.

4.2 Indoor Hydroslide

Constructed in 2007, the 11.2 metre tall indoor hydroslide is connected to the new indoor pool at the main building's north corner. The hydroslide structure consists of a circular steel concrete filled staircase tower, a steel braced frame supporting the mouth of the fibreglass slide, and steel cantilevered pipe columns supporting the remainder of the hydroslide. The stair tower is laterally supported by the cantilevered stair tower column. Single cantilevered steel pipe columns support the curved hydroslide. The mouth of the slide is supported by a steel brace frame structure, which consists of square steel tubular columns and horizontal members and flat bar tension-only diagonal bracing.

The main tower is supported by a 3.8m square by 0.5m thick concrete foundation pad, while the two legs of the braced steel frame are supported by 1m square by 0.5m thick foundation pads.

4.3 Survey

4.3.1 Site Visit Initial Assessment

An Opus Senior Structural Engineer visited the Jellie Park site on 28 February 2012. This visit was general in nature. Existing documentation was utilised in the walk through. Photographs and notes were taken, including notes and photographs of damage.

4.3.2 Further Inspections

Further inspections were carried out by Opus to survey the hydroslides on 23 March 2012. Since there was missing information on the hydroslides, this survey was conducted for the collection of as-built measurements. A geotechnical site walkover was completed by Opus on 29 February 2012 and 21 March 2012.

4.4 Original Documentation

Copies of the following construction drawings were provided by CCC:

- Jellie Park Redevelopment architectural as-built drawings by Warren and Mahoney, dated 26 January 2009.
- Jellie Park Redevelopment structural drawings for the gym and changing room areas. Drawings by Powell Fenwick Consultants Limited dated January 2007.
- Jellie Park Redevelopment structural drawings for the new indoor pool. Drawings by Powell Fenwick Consultants Limited dated January 2007.

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

No copies of the design calculations have been obtained as part of the documentation set.

5 Structural Damage

The following damage has been noted:

5.1 Surrounding Buildings

Refer to the Qualitative Detailed Engineering Evaluation report regarding the main building structure at Jellie Park for noted damage to the buildings surrounding the hydroslides.

5.2 Outdoor Hydroslide

No structural damage was noted to the outdoor hydroslide. However this was based on a limited investigation with limited access to the structure as a whole.

5.4 Indoor Hydroslide

No structural damage was noted to the indoor hydroslide superstructure. There was cracking of the slab-on-grade that connects the new indoor pool to the indoor hydroslide stair tower. This cracking coincided with the edge of the hydroslide's stair tower foundation, leading to an assumption that the stair tower rocked during the earthquake.

6 General Observations

Both structures appear to have generally performed well during the earthquake, with no apparent structural damage noted. The observed damage to the slab-on-grade between the indoor pool and indoor hydroslide stair tower is consistent with the stair tower rocking and the foundation cracking the slab as it rocks.

7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

7.1 Qualitative Assessment Summary

An initial qualitative assessment of the hydroslides was not undertaken. Because it is the desire of the CCC to investigate the structural integrity of the hydroslides and to determine their seismic capacities, the analysis moved directly into the Quantitative Assessment phase. Based upon these quantitative findings and upon any discovered structural deficiencies, an upgrade strategy would be developed to bring either or both hydroslides up to 67%NBS or more, depending on the cost of the retrofit solution.

7.2 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a structure that could contribute to increased levels of damage or cause premature collapse of the structure. As part of this quantitative assessment, the following potential CSW's were identified for the hydroslides.

a) **Outdoor Hydroslide Platform Pipe Bracing** – A buckling failure of a pipe brace would result in a bending failure of the platform's columns, leading to a collapse of the platform.

b) **Outdoor Platform Pipe Bracing Connections** – Failure of these connections is a brittle failure mechanism, resulting in loss of lateral stability. A loss of lateral stability would lead to collapse of the platform.

c) **Uplift of Outdoor Platform Column Foundations** – Insufficient uplift capacity of the foundation would result in the rocking of the platform. In general, rocking of a structure is an energy dissipating mechanism and would increase the seismic performance of a structure. However, if there is too much rocking, the structure will not be able to right itself, resulting in a collapse of the structure.

d) **Sliding of Outdoor Platform Column Foundations** – A foundation sliding failure of the piles, which are not tied together at ground level, would lead large column base displacements. Large column base displacements could lead to axial instability of the platform columns, leading to collapse of the platform.

e) **Uplift of Indoor Hydroslide Braced Frame Foundations** – Insufficient uplift capacity of the foundation would result in the rocking of the platform. In general, rocking of a structure is an energy dissipating mechanism and would increase the seismic performance of a structure. However, if there is too much rocking, the structure will not be able to right itself, resulting in a collapse of the structure.

7.3 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 3 of the report due to the technical nature of the content. A brief summary follows:

Hand calculations were performed to determine seismic forces from the current building codes. These forces were then distributed to the lateral force resisting systems by tributary area and relative rigidity. The capacities of these lateral elements were then calculated and utilised to estimate %NBS for that element.

7.4 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the hydroslides in an undamaged state. Therefore the current capacities of the structures will be lower than that stated.

The results have been reported as a %NBS and the stated value reported is obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on limited drawings, specifications and site inspections
- The normal variation in material properties which change from batch to batch.
- Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

7.5 Quantitative Assessment

A summary of the structural performance of each hydroslide is reported below in Tables 3 and 4. Only the critical structural element/system of each hydroslide was analysed and noted in these tables, as these effectively define each hydroslide's capacity. Elements below 67% NBS are considered further in the following sections when developing the strengthening options. Elements below 33% NBS need immediate attention since they make the structure (or portion of the structure) earthquake prone.

Table 3: Summary of Seismic Performance – Outdoor Hydroslide

Structural Element/System	Failure Mode or Description of Limiting Criteria Based on Elastic Capacity of Critical Element.	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Platform Pipe Columns	Buckling failure in compression, resulting in collapse of platform	No	>100%
Platform Pipe Bracing	A buckling failure of a pipe brace would result in a bending failure of the platform's columns, leading to a collapse of platform. Since the pipe brace wall thickness (t) are unknown, the thinnest and thickest sections were checked. Since the pipe brace unbraced length (L_e) in compression varies, two ranges of %NBS are presented. The %NBS range to the right reflects these thicknesses and various unbraced lengths.	Yes	Bottom brace, t = 4mm 69% NBS Bottom brace, t = 6mm 97% NBS Central brace, t = 4mm 94% NBS Central brace, t = 6mm 100% NBS
Platform Pipe Bracing Connections	A brittle failure mechanism, resulting in loss of lateral stability, leading to collapse of platform	Yes	99%
Platform Column Foundations – Bearing	A bearing failure of the foundation would result in large displacements of platform. Large platform displacements could trigger a collapse of the structure if large enough.	No	>100%
Platform Column Foundations – Uplift	Insufficient uplift capacity of the foundation would result in the rocking of the platform. In general, rocking of a structure is an energy dissipating mechanism and would increase the seismic performance of a structure. However, if there is too much rocking, the structure will not be able to right itself, resulting in a collapse of the structure. The calculated %NBS indicates a high probability of the structure not being able to right itself when the structure is fully loaded and subject to Code level seismic forces.	Yes	35%
Platform Column Foundations – Sliding	A foundation sliding failure would lead large column base displacements as the piles are not tied together. Large column base displacements could lead to axial instability of the platform columns, leading to collapse of the platform	Yes	50%
Slide Pipe Columns	A bending failure of columns would result in large deformations at the top of the columns. Large deformations of the columns could lead to failure of the slides. Failure of slide fixings to top of columns would result in the slides breaking free of the slide-column fixings.	No	>100%
Slide Pipe Column Foundations	A bearing failure of slide pipe column foundations would result in large deformation at the top of the columns, which could lead to failure of the slides.	No	96%

Table 4: Summary of Seismic Performance – Indoor Hydroslide

Structural Element/System	Failure Mode, or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Stair Tower Column	A bending failure, as the result of yielding reinforcement, would result in permanent deflections of the stair tower.	No	>100%
Stair Tower Foundation	A bearing failure of the foundation would result in large displacements of the stair tower. Rocking of the stair tower could lead to stability issues. Even though there is evidence that the stair tower foundation rocked, calculations indicate that there are no instability issues with the stair tower foundation. A rocking mechanism, in this case, is advantageous because it dissipates energy as the structure rocks.	No	>100%
Hydroslide Braced Frame	If this braced frame were to failure, forces could be redistributed back to the stair tower through the concrete stair/slide landing.	No	>100%
Hydroslide Braced Frame Foundations – Bearing	A bearing failure would lead to a redistribution of forces back to the stair tower through the diaphragm action of the concrete stair/slide landing.	No	>100%
Hydroslide Braced Frame Foundations – Uplift	Insufficient uplift capacity of the foundation would result in the rocking of the platform. In general, rocking of a structure is an energy dissipating mechanism and would increase the seismic performance of a structure. However, if there is too much rocking, the structure will not be able to right itself, resulting in a collapse of the structure. The calculated %NBS indicates a high probability of the structure not being able to right itself when the structure is fully loaded and subject to Code level seismic forces.	Yes	35%
Slide Pipe Columns	A bending failure of columns would result in large deformations at the top of the columns. Large deformations of the columns could lead to failure of the slides. Failure of slide fixings to top of columns would result in the slides breaking free of the slide-column fixings.	No	100%
Slide Pipe Column Foundations	A bearing failure of slide pipe column foundations would result in large deformation at the top of the columns, which could lead to failure of the slides.	No	100%

8 Summary of Geotechnical Appraisal

Both hydroslides are founded on medium dense sands and gravels (1986 borehole SPT N=10-30). These materials become very dense at about 15m below ground level. Minor land damage has occurred at Jellie Park due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake. There appears to have been minor settlement (up to 30mm) of the ground noted in three areas around the site. Liquefaction appears to have been relatively minor at the site, with liquefaction occurring in one location to the east of the main entrance. Cracks in the concrete perimeter footing appear to be minor.

Well logs and CPTs indicate the hydroslides are likely to be founded on interbedded layers of clay, silt, peat and sand, underlain by sand and gravel, with the Riccarton Gravels likely to be encountered from approximately 12m below ground level.

The foundation system for the 2007 addition, perimeter strip footing with pads supporting the portal frame, has performed well. The foundations of the older areas of the Jellie Park complex, although unknown appear to have also performed well.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. We would expect that similar liquefaction and ground damage could occur in a future earthquake dependent on the location of the epicentre.

If CCC wishes to further quantify the potential for differential settlement in future seismic events, consideration could be given to undertaking ground investigations to more accurately estimate the potential differential settlement from liquefaction.

If CCC wishes to further evaluate and quantify the liquefaction potential at this site, additional site specific testing with CPT’s and associated analysis would be necessary. Further investigations are currently not considered necessary.

Further information regarding the geotechnical appraisal can found in Appendix 2 of this report.

9 Remedial Options

Each hydroslide requires repair and strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS.

Prior to the design of any strengthening, the wall thickness of the outdoor hydroslide pipe bracing needs to be determined. This will aid in determining a more exact %NBS for these elements.

Opus was able to obtain the redevelopment geotechnical report written by Geotech Consulting, Ltd, dated 4 August 2006. This report will aid in the design of the new foundations at the hydroslides.

9.1 Outdoor Hydroslide Strengthening

Any concept strengthening scheme for the outdoor hydroslide would entail:

- Based on the actual pipe brace wall thickness, the pipe braces may or may not need to be replaced. If the actual pipe brace wall thickness leads to a %NBS that is below 67% NBS, then the braces will need to be replaced.
- If the pipe braces require replacing, new brace connections at the columns will also be required.
- Since the platform foundations are insufficient for uplift and sliding, the existing foundations will need to be replaced with new foundations.

9.2 Indoor Hydroslide Strengthening

Any concept strengthening scheme for the indoor hydroslide would entail:

- Since the foundation of the braced frame at the mouth of the slide is inadequate in uplift, the two existing isolated pad footings will need to be incorporated into a new footing system.

10 Conclusions

a) The seismic performance of the outdoor hydroslide is governed by the uplift capacity of the platform's foundations, which have an expected strength of 35%NBS and the sliding resistance of the foundations, which have an expected strength of 50%NBS. The outdoor hydroslide is therefore considered to be a moderate risk structure.

b) The seismic performance of the indoor hydroslide is governed by the uplift capacity of the foundations at the braced frame structure supporting the mouth of the slide. These foundations have a capacity of 35%NBS. The indoor hydroslide is therefore considered to be a moderate risk structure.

c) Strengthening schemes are recommended to be developed to increase the seismic capacity of both hydroslide tower structures to at least 67% NBS.

d) Further investigation is required to more accurately determine the wall thickness of the tubular steel bracing on the outdoor tower.

11 Recommendations

a) A strengthening works scheme be developed to increase the seismic capacity of both hydroslide tower structures to at least 67% NBS.

12 Limitations

a) This report is based on an inspection of the hydroslides and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

13 References

[1] NZS 1170.5: 2004, *Structural design actions, Part 5 Earthquake actions,* Standards New Zealand.

[2] NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix 1 – Photographs

OUTDOOR HYDROSLIDE

Figure 1 – Outdoor Hydroslide Platform Structure – Looking North

Figure 2 – Outdoor Hydroslide Platform Structure – Looking South

Figure 3 – Outdoor Hydroslide Platform Structure – Looking Northeast

Figure 4 – Typical Mid-Column Height Pipe Brace-to-Column Connection

Figure 5 – Typical Pipe Brace-to-Column Connection at Platform Column Base

Figure 6 – Platform Column Base Connection

Figure 7 – Typical Column, Slide Fixing and Slide at Straight Hydroslides

Figure 8 – Typical Column, Slide Fixing and Slide at Curved Hydroslide

Figure 9 – Typical Column Base at Hydroslide Columns

Figure 10 – Straight Hydroslides at Top of Platform

Figure 11 – Curved Hydroslide at Top of Platform

INDOOR HYDROSLIDE

Figure 12 – Indoor Hydroslide

Figure 13 – Indoor Hydroslide Stair Tower and Braced Frame at Mouth of Slide

Figure 14 – Braced Frame Supporting Mouth of Slide and Stair Landing

Figure 15 – Top of Stair Tower and Mouth of Hydroslide

Figure 16 – Column, Slide Fixing and Slide at Indoor Hydroslide

Figure 17 – Column, Slide Fixing and Slide at Indoor Hydroslide

Figure 18 – Column, Slide Fixing and Slide at Indoor Hydroslide

Figure 19 – Interior View of Stair Tower Base with Cracked Slab-on-grade (Evidence of Foundation Rocking)

Appendix 2 – Geotechnical Appraisal

Appendix 3 – Quantitative Assessment Methodology and Assumptions

Quantitative Assessment

1.0 Methodology and Assumptions

1.1 Material Strength

Concrete Strength	= 45MPa (30MPa x 1.5)
Structural Steel Tubes, Pipe and Plate Yield Strength	= 300MPa
Steel Reinforcing Bar	= 500MPa
Assumed Soil Bearing Capacity	= 400kPa (based on redevelopment geotech report)

1.2 Loading Actions

Dead Loads – Self weight

Live Load – 4kPa

1.3 Seismic Parameters

T = 0.20 sec (outdoor hydroslide platform)

T = 0.49 sec (indoor hydroslide stair tower)

T = 0.22 sec (indoor hydroslide braced frame)

T (assumed) = 0.40 sec (outdoor hydroslide slide columns)

T (assumed) = 0.40 sec (indoor hydroslide slide columns)

Z = 0.30

Importance Level 2 $R_u = 1.0$, $R_s = 0.33$

N(T,D) = 1.0

Ultimate Limit State C(T) = 0.9

Serviceability Limit State C(T) = 0.3

Location	μ	S_p	k_μ	Cd(T)
Outdoor Hydroslide Platform Braced Frames & Indoor Hydroslide Braced Frame	1.25	0.9	1.07	0.75
Outdoor & Indoor Hydroslide Columns	3.0	0.7	2.14	0.30
Indoor Hydroslide Stair Tower Column	2.0	0.7	1.57	0.37

2.0 Analysis Procedure

A μ = 1.25 was chosen due to the minimal ductile detailing of the above mentioned critical lateral resisting elements. A μ = 3.0 was chosen due to the ductile detailing of the hydroslide column bases. A μ = 2.0 was chosen at the indoor hydroslide stair tower column due to the steel formwork confining the concrete core of the stair tower column.

Hand calculations were performed to estimate the force distribution to the lateral force resisting elements. These lateral resisting elements at the outdoor hydroslide consist of the braced frames at the hydroslide platform and at the cantilever columns at the hydroslides. At the indoor hydroslide, the lateral resisting elements consist of the reinforced concrete cantilevered stair tower column, the braced frame at the mouth of the hydroslide and the cantilever columns at the hydroslides.

Appendix 4 – DEE Spreadsheets

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Indoor Hydroslide - Braced Frame	Reviewer:	Alistair Boyce
	Unit / No: / Street	CPEng No:	209860
Building Address:	/ 295 / Ilam Road	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.62
	Degrees / Min / Sec	Company phone number:	03 363 5400
GPS south:	43 / 30 / 33.02	Date of submission:	18-Sep-12
GPS east:	172 / 34 / 57.93	Inspection Date:	28-Feb-12
		Revision:	Final
Building Unique Identifier (CCC):	BU 0266-006 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	Unknown
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	14.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	16.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	isolated pads, no tie beams		if Foundation type is other, describe:	
Building height (m):	11.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	8.9
Floor footprint area (approx):	7			
Age of Building (years):	5		Date of design:	2004-
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):			Brief strengthening description:	
Use (upper floors):	public			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	Plate, PFC, angles
Floors:	steel deck	type	plate
Beams:	steel non-composite	beam and connector type	RHS
Columns:	structural steel	typical dimensions (mm x mm)	100 x 100
Walls:			

Lateral load resisting structure

Lateral system along:	steel concentric braced frame	**Note: Define along and across in detailed report!**	ıote typical frame sizes and bay length (m)	1.7
Ductility assumed, μ:	1.25			
Period along:	0.22	0.31 from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:				
Ductility assumed, μ:				
Period across:		#N/A enter height above at H31 and lateral system	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	
Wall cladding:	
Roof Cladding:	
Glazing:	
Ceilings:	
Services(list):	

Available documentation

Architectural	full	original designer name/date	Warren & Mahoney / Dec 06
Structural	full	original designer name/date	Powell Fenwick / Dec 06
Mechanical	full	original designer name/date	Powell Fenwick / Dec 06
Electrical	full	original designer name/date	Powell Fenwick / Dec 06
Geotech report	full	original designer name/date	Geotech Consulting, Ltd

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:		Describe how damage ratio arrived at:	
	Describe (summary):	No apparent structural damage		
Across	Damage ratio:	#DIV/0!		
	Describe (summary):	No apparent structural damage		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	none		Describe:
	Building Consent required:	no		Describe:
	Interim occupancy recommendations:	full occupancy		Describe:
Along	Assessed %NBS before:		##### %NBS from IEP below	
	Assessed %NBS after:	35%		
Across	Assessed %NBS before:		##### %NBS from IEP below	
	Assessed %NBS after:	35%		

IEP

Period of design of building (from above): 2004-

h_n from above: 8.9m

Seismic Zone, if designed between 1965 and 1992:

Design Soil type from NZS1170.5:2004, cl 3.1.3:
not required for this age of building

	along	across
Period (from above):	0.22	0
(%NBS)nom from Fig 3.3:		
Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25		1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2		1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)		1.0
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ — $\%NBS_b$: **#DIV/0!** | **#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

		along	across
3.7. Overall Performance Achievement ratio (PAR)		**0.00**	**0.00**
4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Indoor Hydroslide - Stair Tower	Reviewer:	Alistair Boyce
	Unit / No: / Street	CPEng No:	209860
Building Address:	295 Ilam Road	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.62
		Company phone number:	03 363 5400
	Degrees / Min / Sec		
GPS south:	43 / 30 / 33.02	Date of submission:	18-Sep-12
GPS east:	172 / 34 / 57.93	Inspection Date:	28-Feb-12
		Revision:	Final
Building Unique Identifier (CCC):	BU 0266-006 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	Unknown
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	14.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	16.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	isolated pads, no tie beams		if Foundation type is other, describe:	
Building height (m):	11.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	8.9
Floor footprint area (approx):	7			
Age of Building (years):	5		Date of design:	2004-
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):			Brief strengthening description:	
Use (upper floors):	public			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	steel framed	rafter type, purlin type and cladding	Plate, PFC, angles
Floors:	steel deck	type	plate
Beams:	steel non-composite	beam and connector type	RHS
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	850
Walls:	non-load bearing	0	

Lateral load resisting structure

Lateral system along:	other (note)	**Note: Define along and across in detailed report!**	describe system	Cantilever columns
Ductility assumed, μ:	2.00			
Period along:	0.49	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)		describe system	Cantilever columns
Ductility assumed, μ:	2.00			
Period across:	0.49	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	steel	describe supports	
Wall cladding:	other light	describe	
Roof Cladding:	Metal	describe	profile metal roofing
Glazing:			
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	full	original designer name/date	Warren & Mahoney / Dec 06
Structural	full	original designer name/date	Powell Fenwick / Dec 06
Mechanical	full	original designer name/date	Powell Fenwick / Dec 06
Electrical	full	original designer name/date	Powell Fenwick / Dec 06
Geotech report	full	original designer name/date	Geotech Consulting, Ltd

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:		Describe how damage ratio arrived at:	
	Describe (summary):	No apparent structural damage		
Across	Damage ratio:	#DIV/0!		
	Describe (summary):	No apparent structural damage		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	crack slab on grade

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:		##### %NBS from IEP below	
	Assessed %NBS after:	100%		
Across	Assessed %NBS before:		##### %NBS from IEP below	
	Assessed %NBS after:	100%		

IEP

Period of design of building (from above): 2004-

h_n from above: 8.9m

Seismic Zone, if designed between 1965 and 1992:

Design Soil type from NZS1170.5:2004, cl 3.1.3:
not required for this age of building

	along	across
Period (from above):	0.49	0.49
(%NBS)nom from Fig 3.3:		

Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final (%NBS)$_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

	along	across
%NBS$_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

	along	across
3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Outdoor Hydroslide	Reviewer:	Alistair Boyce
	Unit / No: / Street	CPEng No:	209860
Building Address:	295 Ilam Road	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.62
		Company phone number:	03 363 5400

	Degrees	Min	Sec
GPS south:	43	30	33.02
GPS east:	172	34	57.93

		Date of submission:	18-Sep-12
		Inspection Date:	28-Feb-12
		Revision:	Final
Building Unique Identifier (CCC):	BU 0266-006 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	Unknown
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	14.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	16.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	bored cast-insitu concrete piles		if Foundation type is other, describe:	
Building height (m):	8.90		height from ground to level of uppermost seismic mass (for IEP only) (m):	8.9
Floor footprint area (approx):	35			
Age of Building (years):	29		Date of design:	1976-1992
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):			Brief strengthening description:	
Use (upper floors):	public			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:			
Floors:	steel deck	type	UB, angles, plate
Beams:	steel non-composite	beam and connector type	UB
Columns:	structural steel	typical dimensions (mm x mm)	Pipe columns
Walls:			

Lateral load resisting structure

Lateral system along:	steel concentric braced frame	**Note: Define along and across in detailed report!**	note typical frame sizes and bay length (m)	4.9
Ductility assumed, μ:	1.25			
Period along:	0.22	0.31 from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	steel concentric braced frame		note typical frame sizes and bay length (m)	4.9
Ductility assumed, μ:	1.25			
Period across:	0.22	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):
east (mm):
south (mm):
west (mm):

leave blank if not relevant

Non-structural elements

Stairs:
Wall cladding:
Roof Cladding:
Glazing:
Ceilings:
Services(list):

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:		Describe how damage ratio arrived at:	
	Describe (summary):	No apparent structural damage		
Across	Damage ratio:	#DIV/0!		
	Describe (summary):	No apparent structural damage		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS(before) - \%NBS(after))}{\%NBS(before)}$$

Recommendations

Level of repair/strengthening required:	none	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:		##### %NBS from IEP below
	Assessed %NBS after:	35%	
Across	Assessed %NBS before:		##### %NBS from IEP below
	Assessed %NBS after:	35%	

IEP

Period of design of building (from above): 1976-1992

h_n from above: 8.9m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.22	0.22
(%NBS)nom from Fig 3.3:		
Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25		1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2		1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)		1.0
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E

$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

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