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5 November 2015

Spectra Limited
P.O. Box 613
Dunedin 9054

Attention: Bevan Meddings
bevan@spectra.net.nz

Dear Bevan,

**JOHN WICKLIFFE PLAZA – 265 PRINCES ST, DUNEDIN**
**DETAILED SEISMIC ASSESSMENT**

As requested we have conducted a Detailed Seismic Assessment (DSA) of this building. We inspected the building on 28 July 2015 and were able to gain access to the roof and most of the floor areas.

## The Building

We have obtained original structural and architectural drawings for this building. The original structural drawings were prepared by J.R.G Hanlon & Partners, dated September 1996 and include sheets s1-s15 and amendments sheet sB and sheet sC. Original architectural drawings were prepared by Nick Baker Architect Ltd, dated October 1996 and include sheets A101-A115. Sheet s3 showing the basement and ground floor plans is attached in Appendix B along with sheet A106 which shows external elevations of the building.

Located in the central business district of Dunedin, John Wickliffe Plaza is a four storey reinforced concrete and reinforced masonry wall structure. The building is bounded on the east by Bond Street and on the north by Rattray Street. John Wickliffe House sits to the south while the terraced exchange plaza and Princes Street sit to the west. Access to the basement floor is gained through entry from Bond Street while sloping street levels means access to the ground floor is gained from the Princes Street exchange plaza area. Due to the differing external ground levels (see North Elevation sheet A106); we have conservatively assumed an external ground level at Bond Street level (RL 102.940), around the entire structure. Both the lower basement and basement floors are car parking space while the ground floor and first floor levels are used as office space.

The main lateral-load resisting elements in this structure are reinforced masonry walls and precast wall panels. Structural walls dominate the north, south and east elevations while a large number of openings are present to the west (Princes St) elevation above ground floor (RL 105.390). The building is relatively squat, with plan dimensions of approximately 18m by 34m and an overall height of roughly 10m from Bond Street level.

Gravity loads are carried by Stahlton rib flooring which is supported internally by reinforced concrete beams and columns and externally by structural walls. Five major reinforced masonry walls located on the east side of the building separate the lift shaft and stair well from the main building. Lightweight Colorsteel Trimdek profiled metal roofing is supported on Dimond DHS purlins which span between four roof-level portal frames and northern and southern external walls. A large number of precast parapet elements along the north, east and west elevations are supported off these steel portal frames.

## Historical Earthquake Data from the Dunedin Area

Historical Earthquake data from the central Dunedin area suggests that John Wickliffe Plaza has been subjected to very few large scale shaking events. The strong-motion recordings from the Dunedin Civil Defence (DCDS) strong-motion recording centre indicate that the largest shaking experienced at the site is equal to only 7% of the ultimate limit state (ULS) earthquake for an importance level 2 (IL2) structure such as this. Data from the Dunedin Civil Defence station dates back to 2002, with the largest recorded shaking occurring during the Darfield Earthquake of September 2010. This lack of real-life testing unfortunately provides us with little insight into the building's probable earthquake strength.

## Detailed Seismic Assessment Using NZSEE Guidelines

Due to the complete nature of original structural and architectural drawings, very few assumptions have been required during the assessment of this building.

Based on bore hole logs included within structural drawings and previous experience in the area we know that subsoils are predominantly soft marine sediments with clay contents. As a result we have assumed a soil type "D" as outlined in NZS1170.5:2004 which is described as deep or soft soil sites. This is backed up by the Opus study report of 2005 titled *Seismic Risk in the Otago Region.*

The building's strength against earthquake has been assessed based on the following:

(a) The ability of roof-level portal frames to resist in-plane earthquake loads, transferring these to the main concrete structure below. These frames have been assessed by parts under Section 8 of NZS1170.5:2004 with a ductility factor of $\mu_p$ = 3.0.

(b) The ability of steel diagonal bracing within the roof plane to transfer earthquake loads from out-of-plane walls to stiffer in-plane walls.

(c) The ability of cantilever columns along the north (Rattray St) elevation to resist in-plane seismic loads, transferring these to the main structural walls below.

(d) The ability of all reinforced masonry and precast panel wall sections to resist in-plane earthquake forces.

(e) The ability of Stahlton rib flooring to transfer seismic loads from out-of-plane walls to stiff in-plane structural walls.

(f) The ability of all precast parapets and their fixings to resist seismic loads induced under earthquake attack. Precast parapets have been assessed by parts under Section 8 of NZS1170.5:2004 with a ductility of $\mu_p$ = 1.25 for precast fixings and $\mu_p$ = 3.0 for the capacity of precast sections themselves.

Due to the size of this building and its intended use as office space, it is classed as an importance level 2 (IL2) structure. The current loading code (NZS 1170.5:2004) requires an IL2 structure such as this to not collapse in the 1 in 500 year earthquake and for the structure and it's non-structural components to not require repair after the 1 in 25 year earthquake.

Parameters from NZS 1170.5:2004 for the 'Ultimate Limit State' (IL2 – 1 in 500 year event):

Type D subsoils

T = 0.4s

Z = 0.13 Dunedin

R = 1.0 (IL 2)

N(T,D) = 1.0 (D = 250km)

C(T) = 3.0 x 0.13 x 1.0 x 1.0 = 0.390

For in-plane reinforced masonry walls $\mu$ = 2.0, $S_p$ = 0.7, $k_\mu$ = 1.57

so for **ULS** $C_d$ (T) = 0.174

The following standards and literature were used in our assessment of this building:

- NZS 1170.0:2002
- NZS 1170.5:2004
- NZS 3101:2006
- NZS 4230:2004
- NZS 3404:Part 1:1997

- NZSEE "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes." – June 2006 and February 2011.
- "Seismic Design of Reinforced Concrete and Masonry Buildings" – T. Paulay and M.J.N. Priestley.

## % NBS Results

The John Wickliffe Plaza building has an assessed capacity of **100%NBS IL2.** The following table summarises the %NBS IL2 for the various lateral load resisting elements based on the seismic assessment conducted. Refer to attached original drawings in Appendix B for clarity. Transverse direction is taken as east-west.

| Element | Direction | %NBS IL2 Capacity | Commentary |
|---|---|---|---|
| Roof-level steel portal frames | Transverse | >100% | Steel portal frames have sufficient capacity to resist transverse earthquake loads. |
| Precast parapets spanning between supports at steel portal frames | Transverse | >100% | Precast parapets have sufficient flexural overstrength capacity to resist forces induced under seismic loading. |
| Precast parapet connections back to main structure | Both | >100% | Precast parapet connections have sufficient strength to transfer seismic forces back to main structure. |
| Vertical steel members supporting parapets | Both | >100% | Vertical 100x100x10 angles and 150UC30 steel members and connections have sufficient capacity to resist earthquake forces transferred from precast parapets. |
| Cantilever columns on northern elevations | Transverse | >100% | Flexural capacity of cantilever columns is sufficient to resist in-plane seismic forces and transfer these to structural walls below. |
| In-plane capacity of all major reinforced masonry wall sections | Both | >100% | Reinforced masonry wall sections have sufficient in-plane capacity to resist forces induced under seismic loading. |
| Diaphragm action of Stahlton rib flooring to transfer forces to in-plane walls | Transverse | >100% | Connections of Stahlton rib flooring to structural walls are sufficient to permit the transferral of seismic forces from out-of-plane walls to stiff in-plane walls. |
| Diagonal steel bracing within roof plane | Longitudinal | >100% | Steel diagonal roof bracing has sufficient capacity to transfer out-of-plane earthquake forces to stiff in-plane walls. |

Reasons for the listed %NBS IL2 scores are discussed below.

The large amount of precast parapets which are fixed to the roof-level steel portal frames means there are large in-plane forces which must be resisted by the portal frames. Despite this, the lateral restraint provided by the DHS purlins and the strength of the 360UB45 portals means that under combined flexural and axial loading, 100% of the design level earthquake can be resisted.

The precast panels shown on sheet s13 of the original structural drawings in Appendix B have been assessed by parts as outlined in Section 8 of NZS1170.5:2004. The following features were assessed:

- A number of the precast panels have been constructed in separate pieces and the connections between these separate pieces have been found to be sufficient to resist 100% of the design level earthquake.
- The connections of the major panels back to steel portal frames (as seen on Section 2-2 sheet s13), through the use of vertical 150UC30 members, were found to have capacities in excess of 100% NBS along the western and eastern elevations.
- In both the north-eastern and north-western corners of the building, the 150UC30 members have been substituted for 100x100x10 equal angles to allow for easy connections to panels on both faces. These angles and the connections to the precast parapets have sufficient strength to resist 100% of the ultimate limit state design level earthquake.
- Panels along the eastern and western faces typically span horizontally between steel portal frames on each gridline. Under earthquake shaking in the transverse direction, these panels are subjected to considerable face loads. These face loads are accentuated where triangular precast panels such as panel 202 on Princes St Elevation (see sheet s13 Appendix B), are fixed to the face of main panels. This increases the seismic mass of the part and as a result the seismic force which must be resisted. A typical panel such as panel 103 on Princes St elevation has been found to have a flexural overstrength capacity in excess of 100% NBS at the critical middle location (Section 4-4). Under guidelines set out in the NZSEE's publication *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes,* Section 7, the ratio of the overstrength in flexure to nominal flexural strength, $M_o/M_n$, has been taken as 1.25. This overstrength is a result of probable material strengths being higher than those specified and strain hardening effects of steel reinforcing bars.

Cantilever columns along elevation line A (Rattray Street) were assessed under transverse earthquake loading. These 400mm square block columns with 4 H16 reinforcing bars were found to have flexural capacities in excess of 100% NBS.

The regular shape of the John Wickliffe Plaza building and the fact that a large proportion of its seismic mass is tied up in the concrete floors means that the centre of mass is located approximately in the centre of the building. The centre of mass is the point where any lateral seismic load is applied. Due to the large number of openings in the western and northern structural walls of the building at ground floor level (RL 105.940), the centre of rigidity of the structural wall system is located very much in the south-eastern corner. The centre of rigidity is the point where application of a lateral load will not produce a rotation. The large offset between the centre of rigidity and centre of mass at this level produces a large torsional moment as the structure attempts to resist seismic forces. This means that the building effectively twists around the stiff south-eastern corner inducing large forces on the southern and eastern walls and large deflections in the north-western corner of the building. For example, under transverse earthquake loading the southern external wall takes 77% of the total earthquake load. All effects of this torsional moment have been included in our assessment including deflection checks along the northern and western faces of the building. The in-plane capacities of all major reinforced masonry structural walls were found to be in excess of 100%NBS IL2.

Under transverse loading, the concrete floor slabs at each level are required to transfer forces to in-plane northern and southern external walls. The ability of the concrete floor diaphragms to do this depends on the connections to the reinforced masonry walls at each floor level. The D12 reinforcing bars at 400mm centres which are shown on original structural drawings have sufficient capacity to allow the transferral of 100% of seismic design loads.

At roof level, steel diagonal bracing in the form of equal angles is designed to transfer earthquake loads in the longitudinal direction out to eastern and western external walls. Our assessment found these angles have a capacity in excess of 100% NBS.

**Meaning of % NBS**

The Building Code requires new buildings classed as normal structures such as this building (IL2) to have "Ultimate Limit State" (ULS) strength to meet a 1 in 500 year earthquake demand. This is the 100% NBS level assumed in our assessment.

At the Ultimate Limit State, substantial damage is allowed, such as irrecoverable displacement or cracking, as long as there is a margin against collapse and appropriately low life-safety risk.

The following table by NZSEE provides the basis of a proposed grading system for existing buildings as one way of interpreting the %NBS building score. It can be seen that *Earthquake Prone* buildings (%NBS less than 33%) have more than 10 times the risk of collapse than a similar new building. For buildings that are a potential *Earthquake Risk* (67%>%NBS>33%), the risk of collapse is 5 to 10 times greater than that of an equivalent new building. Broad descriptions of the life-safety risk can be assigned to these building grades accordingly.

**Relative Earthquake Risk**

| Building Grade | Percentage of New Building Strength (%NBS) | Approx. Risk Relative to a New Building | Risk Description |
|---|---|---|---|
| A+ | >100 | ≤1 | low risk |
| A | 80 to 100 | 1 or 2 times | low risk |
| B | 67 to 79 | 2 or 5 times | low or medium risk |
| C | 34 to 66 | 5 to 10 times | medium risk |
| D | 20 to 33 | 10 to 25 times | high risk |
| E | <20 | more than 25 times | very high risk |

**Serviceability**

The current New Zealand Building Code also requires that the structure and non-structural components do not require repair after the 1 in 25 year earthquake (SLS1). We are confident that this requirement would be met.

**Liquefaction**

Citing the Opus Study Report of 2005 titled *Seismic Risk in the Otago Region*, this site is listed as being possibly susceptible to liquefaction. The ultimate limit state return period of 500 years for an IL2 structure such as this corresponds to an earthquake with a Modified Mercalli (MM) Intensity of MM7 or greater. The Opus Study Report states for a site possibly susceptible to liquefaction that the liquefaction and settlement hazard following a MM7 earthquake is "liquefaction and settlement of limited layers may occur resulting in minor ground damage".

**Summary**

The existing John Wickliffe Plaza Building has a seismic resistance of 100%NBS IL2 and is therefore a grade A+ building which is regarded as exposing the occupants to **low risk.**

Please do not hesitate to contact us if you have any questions or if we can be of any further assistance.

Yours faithfully
Hanlon & Partners Ltd

David Hand

**Appendices**

» Site Photos
» Original Structural and Architectural Drawings circa 1996

**Figure 1 - Exterior View of Northern Wall viewed from Rattray Street**


**Figure 2 - Exterior View of Western Wall viewed from Princes Street**


**Figure 3 – Interior View of Ground Floor office space looking south-west**


**Figure 4 – Exterior View looking through eastern and southern wall to John Wickliffe House**

**Figure 5 – Exterior View of Eastern Wall viewed from Bond Street**