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Horowhenua District Council

126 Oxford Street
Levin

ISPS CONSULTING ENGINEERS

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Attention: Ann Clark annc@horowhenua.govt.nz

Dear Ann,

RE: Report on the Structural Behavior of Existing Building at 126-148 Oxford Street, Levin

We have now completed the assessment on the building located at 126-148 Oxford Street, Levin. The assessment was carried out after completing an inspection of the building and a review of the available archive information; these included the original structural calculations, drawings and specifications for the original construction and also later reports on the seismic performance of the building.

Executive Summary

The building at 126-148 Oxford Street, Levin is two storey building with a basement that is used as the offices for the Horowhenua District Council. It was designed by Kenin O'Connor & Associates in 2006.

ISPS have undertaken the inspection of the building and completed a review of the structural documentation and the report on the structural performance.

The building is considered a grade A, low risk building according to the grading scheme of the New Zealand Society for Earthquake Engineering in terms of seismic performance, as assessed by Opus International Consultants. This classification corresponds to a normal commercial building of Importance Level 2 as defined in NZS1170.0.

The capacity of the building to transfer gravity loads to supporting subsoil has been shown to be sufficient in terms of both design and actual performance. Differential settlement appears to be well within the Building Act requirements and no related damage has been observed.

The building has developed cracks in various locations which are mainly the result of concrete shrinkage due to the longitudinal dimension of 73.6m. These cracks do not affect the capacity of the structure to resist earthquake or gravity loads. Also they are not related to any differential settlement at the foundation level which remains well within the limits of the building act.

The building is considered safe for normal occupancy

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Introduction

The building at 126-148 Oxford Street, Levin is two storey building with a basement that is used as the offices for the Horowhenua District Council.

ISPS Consulting Engineers have been engaged by the Horowhenua District Council to report on the cause of concrete cracking in the concrete floors and walls in the building and the potential impact on the structural integrity of the structure.

Fig.1: Horowhenua District Council Building

References

We have been provided with copies of the original documentation of the building, including structural calculations, specifications and drawings. We have also received a Detailed Seismic Assessment report on the seismic performance of the building.

Specifically, our references include:

i. The Building Act 2004
ii. Structural Design Actions Part 5: Earthquake actions - New Zealand NZS 1170.5:2004
iii. New concrete building standard NZS 3101.1:2006
iv. Steel Structures Standard NZS3404:1997
v. Assessment and Improvement of the Structural Performance of Buildings in Earthquake 2006 by New Zealand Society for Earthquake Engineering
vi. Original Design Structural Calculations, Drawings, Specifications by Kevin O'Connor & Associates
vii. Detailed Seismic Assessment report by Opus International Consultants Ltd
viii. Repairs to structural cracking to existing concrete floor by Horowhenua District Council.
ix. Preliminary Survey on the Levels of the building
x. Non-intrusive building inspection by ISPS

Building Description

The building located at 126-148 Oxford Street, Levin was designed and built in 2006. The building comprises two storeys and a basement. It is rectangular in plan with dimensions 73.6m X 30.0m for the basement and ground floor. The first floor is also rectangular with dimensions 73.6m X 10.0m.

The structure is a combination of reinforced concrete frames and reinforced concrete pre-cast infill panels for the basement. The section of the ground floor which has the first storey above it consists of reinforced concrete frames. The section of the ground floor without a first storey above it consists of steel portal beams supported on reinforced concrete columns. The first storey also consists of steel portal beams supported on reinforced concrete columns. The ground floor and first floor slabs are double "Tees" with a topping of varying thickness. The roof is lightweight steel with diagonal braces that provide the diaphragm action.

The 1st floor slab is supported on pre-stressed concrete beams which span along the transverse direction of the building (North-West to South-East).

The foundation of the building consists of shallow pads, ground beams and strip footings. The basement slab is a 125mm concrete slab over compacted hard fill.

Original Design and Construction of the Building

The building was designed by Kevin O'Connor & Associates in 2006. The seismic code used for the design was NZS 4203:1992. Please note that the current earthquake actions code is NZS 1170.5:2004.

While a detailed review of the design was not the subject of this report, the calculations appear to have taken into account all seismic, wind and gravity actions properly.

The analysis of the building was based on 2D computer analysis and hand calculations. Today a 3D computer analysis would be expected for a building of this importance and size, although for a two storey building with a lightweight roof the approach adopted by the original designer would still be considered adequate.

The length of the building is 73.6m. This length is a significant parameter that has to be considered for a reinforced concrete structure as shrinkage and temperature effects have an impact to the long term performance and durability of the structure.

Furthermore, the inclusion of pre-stressed beams reinforced with highly stressed tendons imposes significant forces that result in long term differential shrinkage due to the creep effect between the pre-stressed beams and the normal floors.

The construction of the building was up to normal commercial standards.

Structural performance of the building

The seismic capacity of the building has been assessed by Opus International Consultants. The result of their assessment was that the building compares to

85%NBS as an Importance Level 2 building and 50%NBS as an Importance Level 4 building. An explanation of the Importance Level is provided in table 1 below from NZS1170.0.

Consequences of failure	Description	Importance level	Comment
Low	**Low** consequence for loss of human life, *or* **small or moderate** economic, social or environmental consequences	1	Minor structures (failure not likely to endanger human life)
Ordinary	**Medium** consequence for loss of human life, *or* **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels
High	**High** consequence for loss of human life, *or* **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)
		4	Post-disaster structures (post disaster functions or dangerous activities)
Exceptional	Circumstances where reliability must be set on a case by case basis	5	Exceptional structures

Table 1: Consequence of Failure for Importance Levels (NZS1170.0, Table 3.1)

The rating of the building in terms of seismic capacity is compared to the earthquake action that corresponds to a specific probability or statistical return period of occurrence. An Importance Level 2 building must be designed to resist an earthquake with a return period of 500 years. An Importance Level 4 building must be designed to resist an earthquake with a return period of 2,500 years.

The New Zealand Society for Earthquake Engineering have proposed a grading system for buildings as one way of interpreting the %NBS building score (Table 2 below).

	Percentage of New Building Standard (%NBS)	Approx. Risk Relative to a New Building	Building Grade
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 79	2 to 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

Table 2: NZSEE proposed Grading System for Buildings

This building has been classified as a grade A, low risk building as a commercial office building and as a grade C, medium risk building as a post disaster building.

The design of the building for gravity loads does not raise any concerns.

The longitudinal dimension of 73.6m is considered a sufficient length to raise concerns about the capacity of the slabs to tolerate shrinkage and temperature effects efficiently.

Reinforced concrete structures tend to evaporate water during their life. This phenomenon is more intense during the first days and months after concrete pouring. It would be expected that the contractor would pour concrete in stages and zones in order to minimize this effect. However, this effect continues with lower intensity over years.

The use of pre-stressed beams may contribute to a combination of creep and shrinkage. Creep is the effect of permanent deformation of concrete under constant actions, for example gravity, but in this case also the axial pre-stress force. This may create an incompatibility of deformation between the pre-stressed beams and the concrete topping. The pre-stressed floor panels would not compensate for this effect as their pre-stressed force is perpendicular to that of the beams and also they are usually allowed to cure before their transportation to the construction site.

Concrete Cracking

As a result of the shrinkage of concrete the building has developed cracks in various locations.

The most severe crack was observed on the ground floor slab, between grid lines E and F, as shown on Figure 2 below.

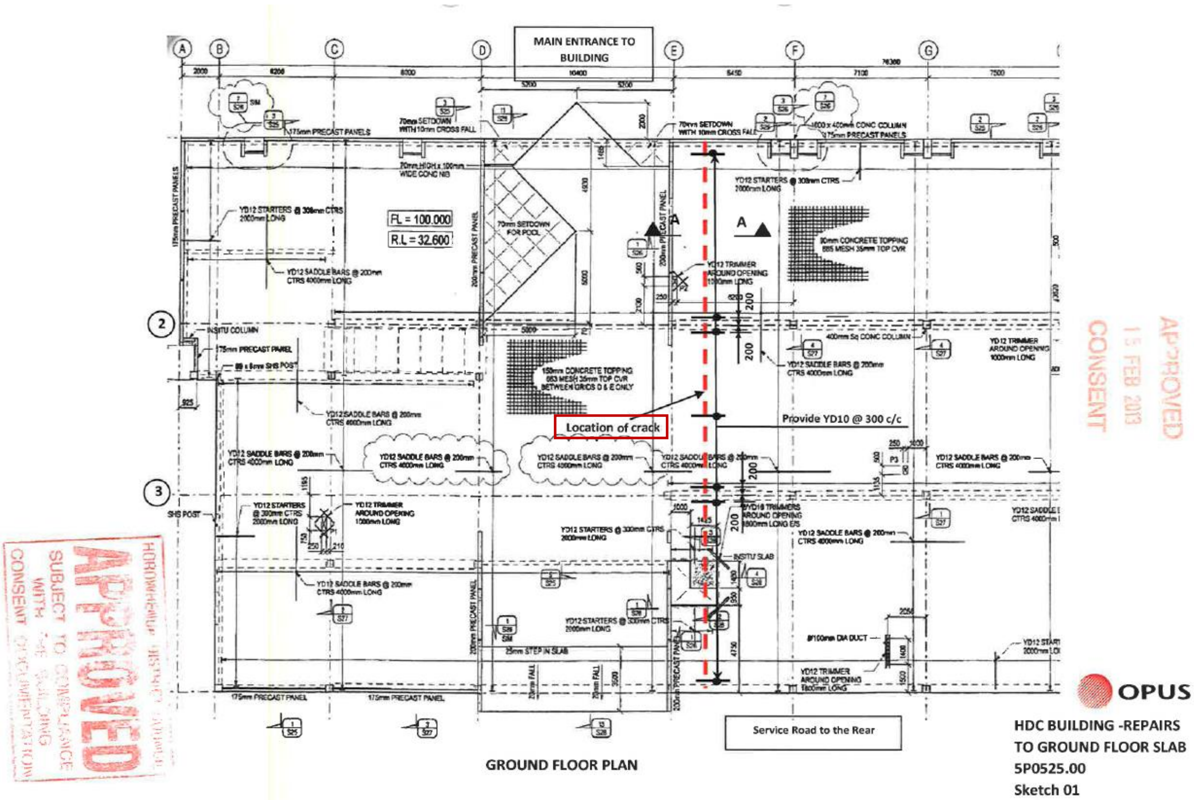

Fig.2: Location of crack on ground floor slab

The crack occurred at the joint between two adjacent floor panels and runs in the transverse direction of the building. It developed through the full depth of the concrete topping. The retrofitting is expected to have restored the full capacity of the diaphragm at this location.

The crack seems to be a result of concrete shrinkage as explained above. It was reported that light could be seen through the crack. This is an indication of the development of uniform axial tension and strains that exceeded the tolerance of the concrete topping. If the cause of the crack was a structural action of a different nature (e.g. differential settlement of the foundation) the crack would be categorized as a bending crack with an opening at the top face and close at the bottom (or reversed) but in any case light should not pass through the crack.

It may be expected that similar cracks may have developed or will develop at other locations. Ground shaking during an earthquake could expose these cracks. However, the configuration of the structure does not create a collapse mechanism with inelastic rotations at the locations of the observed cracks.

The basement slab on grade has also developed cracks which are attributed to concrete shrinkage. While the design has allowed for joints to compensate for shrinkage these joints appear to have inadequate depth. This has resulted to various cracks, distributed around the area of the car parking area.

The pre-cast walls have developed cracks of various widths. Most of the cracks are close to the corners indicating that they may have resulted due to a combination of factors, from initial concrete curing to minor ground shaking.

A horizontal crack was observed at the North-Eastern corner column. The shape and especially the horizontal plane of the crack indicates that it is not due to an external action but rather because of staging in concrete pouring during construction. Cracks due to structural actions are expected to form diagonal cracking in columns.

The structure has developed other cracks that are not considered to have an important impact to the structural performance or durability of the building.

All the cracks that have been observed do not appear to pose any danger for the occupants of the building. The structural performance is not degraded as a result of these cracks.

The long term durability of the building may be affected as the coastal environment has the potential to cause corrosion to the steel reinforcement with a reduction of the steel cross section and strength. Rusted steel rebars tend to expand in volume and the surrounding concrete cover will spall. The visual inspection of the cracks does not indicate that this has happened thus far.

However, it is recommended to fill the cracks with appropriate products to avoid future corrosion.

Foundation Settlement

The basement structure comprises reinforced concrete with substantial stiffness. A potential differential settlement at the foundation level would cause severe cracks. This behavior would be especially apparent at the beam column joints and the pre-cast panels where significant cracking would occur.

The structure does not appear to have sustained any damage due to differential settlement.

The Horowhenua District Council have performed a level survey at the rear side of the building. The results of this survey show that the difference of two adjacent gridlines does not exceed 17mm (gridlines D-E)

According to the Building Act (Appendix-B, B1/VM4) the foundation could allow for a differential settlement of up to 25mm for every 6m horizontal distance.

STRUCTURE
FOUNDATIONS

Appendix B B1/VM4

Appendix B (Informative)

B1.0 Serviceability Limit State Deformations (Settlement)

B1.0.1 No specific method is given for determining foundation settlement. The following information is provided for guidance only.

Amend 8
Dec 2008

B1.0.2 Foundation design should limit the probable maximum differential settlement over a horizontal distance of 6 m to no more than 25 mm under serviceability limit state load combinations of AS/NZS 1170 Part 0, unless the structure is specifically designed to prevent damage under a greater settlement.

Fig. 2. Building Code Extract

The dimension between grid lines D and E is 10.4m and the allowable differential settlement is: $25mm \times \frac{10.4m}{6.0m} = 43.3mm > 17mm$

Consequently the differential settlement does not appear to be an issue.

If more detailed investigation of this issue would be required, a registered surveyor could perform a full survey of the basement slab. Based on our observations this action is not necessary.

Conclusions

ISPS have undertaken the inspection of the building and completed a review of the structural documentation and the report on the structural performance.

The building has a probable performance of a grade A, low risk building for seismic actions, as defined in the New Zealand Society for Earthquake Engineering grading scheme for a normal commercial building when compared to the latest building standard.

The capacity of the building to transfer gravity loads to supporting subsoil has been shown to be sufficient in terms of both design and actual performance. Differential settlement appears to be well within the Building Act requirements and no related damage has been observed.

The building has developed cracks in various locations which are mainly the result of concrete shrinkage due to the longitudinal dimension of 73.6m. There may be more cracks in the ground floor and first floor slabs that may have not been observed or appeared and future shaking could bring them to the attention of the occupants and the engineers.

The building is considered safe for normal occupancy of an Importance Level 2 structure.

Yours sincerely,

ISPS CONSULTING ENGINEERS NZ LTD.

Manos Bairaktaris

Director