



OPAL TOWER INVESTIGATION INTERIM REPORT

**Independent Advice to NSW Minister for Planning and
Housing**

14 January 2019

OPAL TOWER INVESTIGATION – INTERIM REPORT

Executive Summary

At the request of the Hon. Anthony Roberts, Minister for Planning and Housing, an investigation was carried out into the cause or causes of structural damage to the Opal Tower at Sydney Olympic Park, which was first observed at Christmas 2018. The investigation also considered possible remedial action to repair the damage to the building. This interim report presents the preliminary findings of that investigation.

Further information is required to enable definitive conclusions to be made about the cause or causes of the damage to this structure and the proposed remediation. However, based on the information available to date the following opinions and findings of the investigation are presented:

1. The building is overall structurally sound and not in danger of collapse, but significant rectification works are required to repair and strengthen damaged hob beams and in some cases panels that rest on them.
2. A number of design and construction issues have been identified, a combination of which probably caused the observed damage to some structural members in the Opal Tower building. Further information is required to ascertain the relative materiality of these issues to the cause of damage.
3. The viability of residents re-entering the building extends beyond the structural issues considered here and hence beyond the scope of this investigation. Nevertheless, notwithstanding point 1 above, before residents re-occupy the building the designers must ensure that no structural member is overloaded as a result of any load redistribution likely to have occurred as a consequence of the observed damage to the structure of the building.
4. A proposal for staged rectification of the damaged structural members of the building has been considered and agreed in principle. However, it is recommended that independent and qualified structural engineers should be engaged to check the final proposal in detail before major rectification works commence.
5. Further analysis should be undertaken on the structural design of the hob beams and associated structural members with similar details and consideration should be given to strengthening them wherever they occur throughout the building.

Introduction

The Opal Tower is a high-rise residential building located in Sydney Olympic Park, NSW. It consists of 36 storeys above ground and 3 basement levels below ground. Construction of the building was completed in 2018 and occupation of the 392 residential apartments commenced in the second half of 2018.

A photograph of the Opal Tower is shown in Figure 1. The building is characterised by its overall triangular prismatic shape, with a number of insets in the three external faces of the building (see Figures 1 and 2). These architectural features are referred to as “slots” on some design drawings.

On Christmas Eve 2018, residents of the Opal Tower reported loud noises, including a loud “bang”, reportedly of internal origin, and presumably associated with the structure of the building. Early investigations of the source of these loud noises identified cracks in a load-bearing panel on Level 10 of the building, forming one of the exterior walls at the base of one of the inset slots. Later investigations revealed further cracking of the hob beam supporting the cracked load-bearing panel. Subsequent investigations also identified other cracked concrete structural members at Level 4 in the building, again at the base of an inset feature.

Because of safety concerns, residents of the building were evacuated, first on Christmas Eve 2018. They were subsequently allowed to re-enter and then asked again to depart the building on 27 December 2018 following more detailed checking of structural elements and specifically the identification of the additional structural damage on Level 4 of the building.

On 27 December 2018, the NSW Department of Planning and Environment (DPE) engaged Professors John Carter and Mark Hoffman to investigate a number of matters related to the cracking of the concrete structural members in the building, including the likely causes of the observed cracking. Following initial investigations, Professor Stephen Foster was also engaged to assist in the investigations, on the recommendation of Professors Carter and Hoffman.

This interim report contains a brief description of the investigations carried out to date and presents preliminary findings and opinions. These findings and opinions may be modified as more information comes to light during the ongoing investigations.

Terms of Reference

The terms of reference of the investigation have been provided verbally by the Secretary of the NSW Department of Planning as follows:

1. *“Determine the basis of the failure, what happened and how?”*
2. *The immediate steps that need to occur to ensure the safety of the building for its occupants.*
3. *Any other recommendations on what needs to happen to avoid incidents like this in the future.”*

This interim report provides preliminary findings for the first two of these terms of reference, viz., the basis of the failure and the steps to ensure the safety of the building, which have been the focus of our investigations to date.

Building Structure

The Opal Tower is a reinforced concrete building with post-tensioned concrete floor slabs. It has a reinforced concrete central core structure, which houses the lifts and fire stairs. The floors of the building are supported by the core walls and reinforced concrete columns and precast concrete elements. A particular feature of the building is the slots located on each external face of the building (see Figures 1 and 2). The walls of these inset slot sections of the building are constructed largely from precast reinforced concrete panels (with some cast in situ panels). The walls composed of these panels have been designed to carry gravity loading, effectively acting as columns,

transmitting vertical loads (from floors above the inset slots and from floors intersecting them) to the individual supporting columns below each inset slot feature. The columns of the building are founded on individual pad footings and the central core is supported on shallow spread footings. All footings are founded on shale bedrock.

The major structural design of the building was carried out by WSP, an international engineering services company. The design of the post-tensioned concrete floors was carried out by Australasian Prestressing Services (APS). The precast wall panels were fabricated by Evolution Precast Systems (Evolution). The building was constructed by Icon Co, an Australian building contractor and part of the Kajima Corporation of Japan (Icon).

Investigation Activities

To date, our investigations have included the following activities:

1. Multiple visits to the Opal Tower site and inspection of the damaged structural members in the building, and members in similar locations;
2. Review of the design of relevant sections of the building and related documentation;
3. Review of construction records and quality control records;
4. Viewing of security camera recordings of the garden area in the slot on Level 10 where damage to the panel was first observed; and
5. Discussions with representatives of the building's structural designer (WSP) and builder (Icon).

(Note that at the time of compiling this interim report we had yet to receive all requested materials relating to the design and construction of the building.)

These activities have focused on various structural elements in sections of the building located on Levels 3, 4, 9, 10, 16 and 26, as well as the basement level B3. All areas of known structural damage, both major and minor, were inspected. In particular, we inspected all inset slot regions of the building focusing on the structural panel walls and their supporting hob beams, and the floor plates adjacent to damaged hob beams.

Observed Damage

During the numerous visits to the site of the Opal Tower, we inspected and re-inspected a number of locations where significant damage had occurred to load bearing concrete members.

The areas of significant structural damage are located on Levels 4 and 10 of the Opal Tower. The approximate locations of these damaged regions are shown in Figures 3 and 4.

Photographs of some of the damaged concrete structural members are shown in Figures 5 to 11. Specifically:

On Level 10:

1. A hob beam spanning between columns C21 and C38 (along grid line C shown in Figure 4) and the Panel A resting on it – see Figures 5 to 8. Cracking was also

observed in the floor plate adjacent to column C21 – see Figure 9. This was the damage observed on Christmas Eve and is considered to be major damage.

On Level 4:

2. A hob beam spanning between columns C16 and C34 (along grid line A shown in Figure 3) – see Figure 10. This also appears to be major damage.
3. A hob beam spanning between columns C2 and C22 (along grid line B show in Figure 3). These cracks could be considered minor *at this stage*.
4. Cracking was also observed in the floor plate between Levels 3 and 4 – see Figure 11.

It is noted that the vertical load lines along which these three pieces of observed damage occur all appear to be different and hence the three areas of damage are likely to be unrelated to each other. But this deduction requires further detailed examination before it could be confirmed.

From the security camera footage referred to previously, we observed cracking in the bottom corners of the bottom panel (Panel A) on Level 10. The time stamp on this video recording indicated that the cracking of the panel commenced at approximately 2.16 pm on Monday 24 December 2018 and continued for approximately 8 seconds.

Possible Cause(s)

After inspecting the damaged areas of the building, we initially hypothesized a number of factors that may have been a contributing cause of the observed cracking of the concrete hob beams on Levels 4 and 10, the damaged precast panel on Level 10 and the damaged floor plate between Levels 9 and 10 and Levels 3 and 4. These factors are categorized as follows:

1. Environmental factors such as major storms, heavy rainfall, high winds and extreme changes in temperature causing unexpected and potentially damaging loading of the building;
2. Poor quality construction materials;
3. Issues with the foundations, namely differential settlement of the pad footings supporting the building's columns;
4. Flaws or errors in the design of the structural systems; and
5. Poor quality workmanship or errors during construction.

We considered and assessed each of these factors in some detail and ultimately concluded that not all of the factors were relevant to the damage observed to the Opal Tower. Further details of this assessment are provided as follows.

Environment

In particular, the environmental factors were considered to be highly unlikely to have contributed to the damage because the meteorological records for the few months preceding the failure, and particularly in the period immediately prior to and on 24 December 2018, show no extreme or adverse conditions. The rainfall records show some significant downpours in the months leading up to Christmas, but they were considered not to be unusual.

Materials

There is also no evidence in the documentation we have reviewed to date to indicate that the materials used in construction were inferior in quality or did not meet the specifications required. We therefore infer that poor quality construction materials are unlikely to have contributed to the damage observed.

Foundations

In general, differential settlement of the footings of a building can occur for a variety of reasons. For example, neighbouring columns may experience large differences in their compressive loading or the ground beneath neighbouring footings may vary markedly in terms of stiffness and strength. Differential settlement is also likely if the ground beneath some footings, but not others, may soften over time, perhaps due to local wetting of the ground beneath the footing causing softening of the foundation material.

The records we inspected reveal that all column footings for the Opal Tower structure were founded on shale of low to medium strength, with the majority being medium strength. The records we reviewed indicate that the spread footings supporting the tower core and all but two of the 40 individual pad footings supporting the tower columns were inspected by a geotechnical engineer prior to the pouring of concrete to form the footings. All inspected footings were certified by the geotechnical engineer as suitable to carry a maximum allowable bearing pressure of 3.5 MPa. We could not find inspection records for columns designated as C8 and C40 (see Structural Drawing 4419 S02.051 A for column designations and locations).

However, if differential footing settlements had been a contributing cause of damage to the building, and specifically the damage observed on Levels 4 and 10, we would have expected to observe cracking in the floor slabs and at floor column connections in the lower levels of the building. Our inspections of these areas of the building indicated no such damage. So on the basis of this observation and the documentary evidence of the condition of the shale foundation at the time the footings were poured, we concluded that differential settlement of the column footings is unlikely to be a contributing factor to the structural damage observed on Levels 4 and 10 of the Opal Tower.

Design

Preliminary consideration of the bearing capacities of the hob beam at the locations of the connection of the beams with columns C21 and C38 on Level 10 and with columns C16 and C34 on Level 4 indicate factors of safety lower than required by Standards. To date, we are still awaiting further details of design analyses to further consider this factor.

Construction

There are a number of points noted on Level 10 of the building where construction differed from design and / or Standards:

- (a) Grouting: design drawings indicate that full grout coverage was expected between the panel and the hob beam. However, during construction only the inner surfaces of approximately 110mm width appear to have been grouted, leading to an eccentric bearing load on the hob beam on Level 10. It should also be noted that approved shop drawings show the grout extending over only the inner portion of the hob beam to panel connection;

- (b) Location of reinforcing steel in the vicinity of the connection between the hob beam and columns C21 and C38 on Level 10, in particular, inadequate cover concrete due to encroachment of discontinued (anchored) column bars into the cover zone, and the placement of an electrical conduit within the cover zone in this area;
- (c) A dowel bar between the hob beam and the panel was observed to be incomplete, possibly cut during construction;
- (d) The original design drawings of the building indicate precast concrete panels that were the same width as the hob beam upon which they rest (180 mm). These were subsequently manufactured to be 200 mm in width. The panels were erected so that they overhang the inside face of the hob beam by approximately 20 mm; and
- (e) Inadequate tensile capacity in the horizontal direction in the bottom region of Panel A that rests on the hob beam spanning columns C21 and C38. There is compelling evidence indicating that the wrong size reinforcing bars were placed in this area during manufacture of this particular panel – 20 mm diameter bars were used instead of 28 mm diameter bars (see Figure 8).

At this stage it is not possible to state a definitive cause for the failure of the hob beam on Level 10. However, it is likely that a combination of some of the above design and construction issues led to the observed structural damage on Level 10. There were other construction issues observed, such as incorrectly anchored shear reinforcement in the hob beam, which were considered immaterial to the failure.

In regards to the timing of the observation of damage, it is likely that the damage occurred after progressive build-up of load on the structure as apartments became occupied, culminating with the observed failure at Level 10 on 24 December.

The architectural design where the major damage to the hob beam has been observed on Level 4 is quite different to that of the damaged hob beam on Level 10. Namely, the panels commencing at Level 4 are manufactured in two sections, rather than one, and the width of the hob beam is 320 mm under the inner panel and 180 mm under the outer panel.

The cause of the damage observed on Level 4 is still being assessed and the bearing of the damaged hob beam on its supporting columns is being studied in detail.

Proposed Rectification

We understand that soon after the structural damage to the building was observed, WSP instigated a program of installing props under the damaged areas, as a temporary measure, to ensure the safety of these areas of the building.

WSP has briefed us on the structural principles behind their proposal for permanent repair of the damage observed on Levels 4 and 10 and strengthening of the associated structural members, viz., the hob beam and lowest panel at these locations. We understand that WSP is proposing that these rectification works should be carried out as a three-stage process.

It is our opinion that the structural *principles* behind the WSP proposal for rectification are sound. In particular, the defective structural members will be strengthened by the proposed measures. However, we have not received details of a structural analysis of

the proposed rectifications. We recommend that WSP's detailed plans for the proposed rectification works should be checked by an independent qualified structural engineering organisation.

We understand that strengthening of the hob beams at all levels of the building with similar details, i.e., Levels 4, 10, 16 and 26, is also being considered.

The observed damage on levels 4 and 10 will have caused load, designed to be taken by the damaged elements, to be redistributed to other parts of the structure. Consideration should be made of these load redistributions to confirm that the other parts of the structure have satisfactory capacity to carry the new loads.

Conclusions

At this interim stage, we are still awaiting further information to enable definitive conclusions to be made. However, we can state the following:

1. In our opinion, the building is overall structurally sound and not in danger of collapse, but significant rectification works are required to repair and strengthen the damaged hob beams and in some cases the panels that rest on them.
2. We have identified a number of design and construction issues, a combination of which probably caused the observed damage to some structural members in the Opal Tower building.
3. The viability of residents re-entering the building extends beyond the structural issues considered here and hence beyond the scope of our investigation. Nevertheless, notwithstanding our first conclusion, before residents re-occupy the building the designers must ensure that no structural member is overloaded as a result of any load redistribution likely to have occurred as a consequence of the observed damage to the structure of the building.
4. We have considered the proposal by WSP for staged rectification of the damaged structural members of the building and agree in principle with the rectification solution proposed by WSP. However, we recommend that independent and qualified structural engineers should be engaged to check the WSP proposal in detail before major works commence.
5. We also recommend that further analysis be undertaken on the structural design of the hob beams and associated structural members with similar details, i.e., levels 4, 10, 16 and 26, and consideration be given to strengthening them at these other locations throughout the building.

John Carter, Mark Hoffman and Stephen Foster

Date: 14 January 2019



Figure 1. General view of the Opal Tower. Note the inset slots on the external faces of the building, each extending for multiple storeys (some 6 and others 10 storeys).



Figure 2. Close-up view of a 6 storey slot. Note the 6 panels forming the load bearing wall of the inset slot, each one storey high.

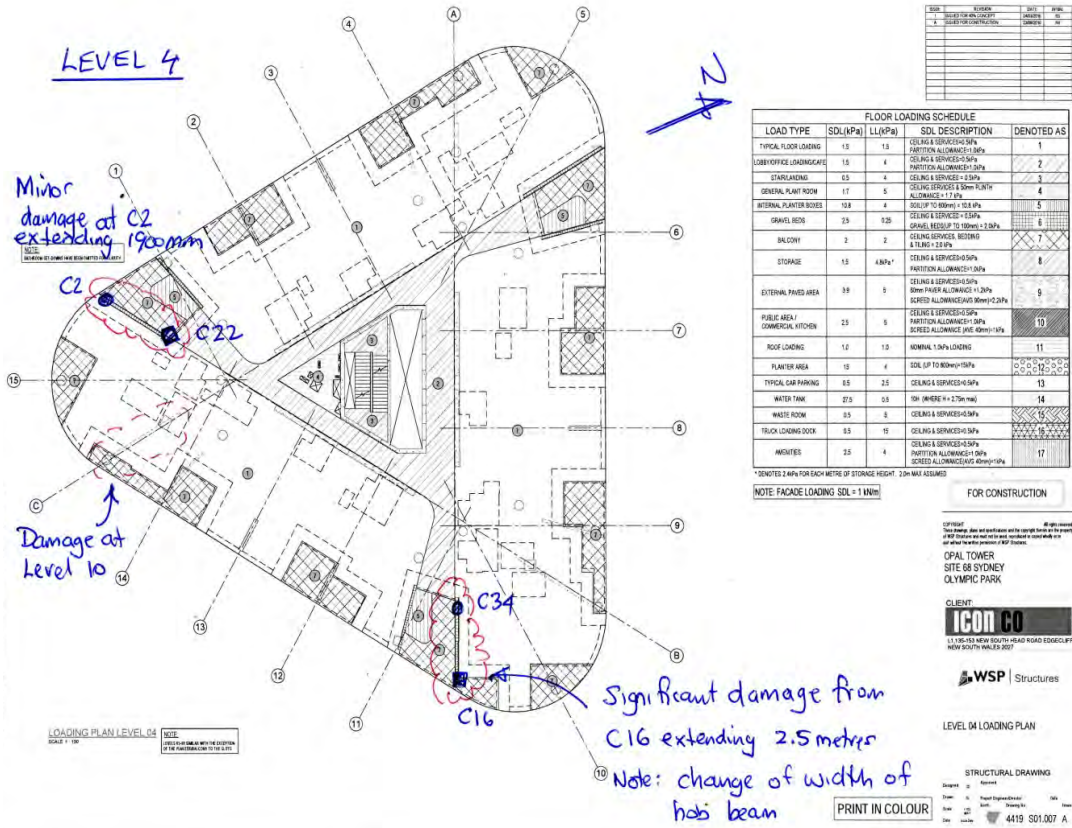


Figure 3. Plan view indicating locations of damage observed on Level 4

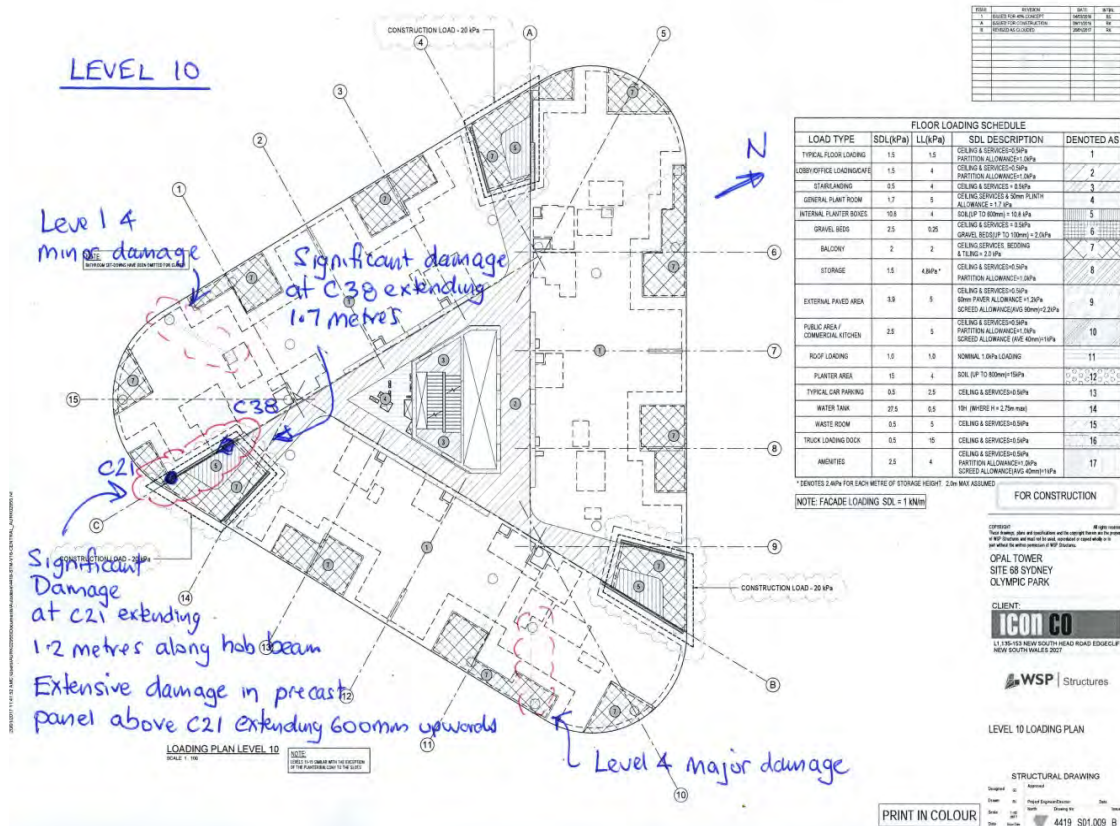


Figure 4. Plan view indicating locations of damage observed on Level 10



Figure 5. Damaged precast Panel A at Level 10 above column C21, before (top) and after (bottom) broken section of concrete removed.



Figure 6. Damaged precast Panel A at Level 10 above column C38, after some broken sections of concrete removed.



Figure 7. Photographs of the hob beam in the vicinity of column 38 at level 10, prior to pouring concrete (top photo), inside the building on 8 January 2019 after cracked concrete cover removed (middle), and outside on 8 January 2019 after the garden and waterproof covering removed (bottom). Note the positioning of the reinforcing bars, the encroachment of the column bars into the cover zone, the lack of anchorage of some horizontal bars, and the encroachment of a conduit into the concrete cover zone. The vertical dowel bars that engage with the precast concrete panels can also be seen in the top photograph.



Figure 8. Photographs of the outside (top) and inside (bottom) of panel A at Level 10 just above hob beam adjacent to column 21, indicating N20 reinforcing bars at 100 mm centres in the lower portion of the precast panel. Note the exposed grout between panel A and the hob beam and the black plastic sheath for the joining dowel (top photo, right of tape measure).



Figure 9. Photographs of the damaged floor slab adjacent to column C21 on Level 10.



Figure 10. Photographs of the damaged hob beam on Level 4 near column C34 before (top) and after (bottom) cracked concrete removed.



Figure 11. Photograph of a crack in the floor plate between Levels 3 and 4.