Structural assessment report

Project:	Housing Commission S-Type Tower
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Contents:

Executive Summary

A structural assessment has been undertaken to the housing commission flats which were constructed during the 1960's and 1970's. The housing commission flats were erected at various locations across Victoria and adopted several different standardised geometries and configurations. This report specifically studies the "S-Type" tower, so named given its shape in plan.

As part of the structural assessment, three different scenarios were explored:

- **Assessment of the building under earthquake loading equivalent to that of 33% of current design standards**
- **Assessment of the building under earthquake loading requirements as outlined in current design standards (100% of design seismic load)**
- **Assessment of the building under earthquake loading requirements as outlined in current design standards however with the north and south façade concrete pre-cast panels removed to allow installation of an apartment "extension" to facilitate increase in floor area per apartment.**

The floor structure (pre-cast concrete one-way spanning slabs) were assessed and found to generally be code compliant.

However, with respect to the lateral load resisting system for seismic conditions, design deficiencies were found to exist within the building. These deficiencies were found to be a combination of structural elements having insufficient strength compared to the expected applied design loading and inadequate minimum structural requirements for reinforcing and connection detailing of the pre-cast concrete walls.

It was found that the building satisfies current design standards for wind loading.

With respect to the design deficiencies identified as part of the analysis, rectification of these deficiencies is proposed to generally be achieved through installation of steel plate reinforcing retrofitted to the faces of the concrete shear walls. The quantity and extent of the plate reinforcing varies dependant upon the level in question and wall in question. Varying degrees of reinforcing is also required across all three of the assessments which have been undertaken within this report. A theoretical solution to strengthen the building to achieve 100% code compliance was achieved.

It has also been noted that further studies and strengthening measures may be explored as possible refinements to the current proposal dependant upon the location of the building and subsequent cost-benefit analysis (which are outside of the scope of the investigation outlined in this report):

- Replacing steel flat plate reinforcing with carbon fibre reinforcing
- Introducing additional shear walls and/or core walls adjacent to the building and nominally increasing the buildings footprint. Introduction of these additional elements may relieve the existing walls of loading and reduce the strengthening required within the existing building envelope.

1.0 Introduction

1.1 General Project Details

Sheer Force Engineering (SFE) have been engaged to undertake a conceptual study on the housing commission high-rise flats which were constructed by the government between 1960-1970s.

The buildings adopted predominantly prefabricated (pre-cast) concrete construction techniques including most of the floors and load-bearing walls. At the time of construction, the techniques and detailing adopted were at the cutting edge of pre-cast concrete construction in Australia and represents the beginnings of a typology of construction which would continue to evolve and advance to become the contemporary form of pre-cast construction used in the industry today.

The housing commission flats adopted standardised building forms and geometry which were replicated across multiple sites. The housing commission flats can be identified in the following suburbs throughout Melbourne:

- Brunswick
- Collingwood
- Prahran
- Northcote
- St. Kilda
- Carlton
- Flemington
- Richmond
- South Melbourne
- Fitzroy
- Kensington
- North Melbourne
- Williamstown

The study undertaken in this report focuses on the typical building form identified as the "S-Type" building, so called due to its apparent shape when viewed in plan. The S-Type building has been adopted at the locations highlighted in green in the location list above.

The scope of this study is to assess the S-Type building against current code requirements which include:

- **NCC 2022 Volume 1** (for structural related elements)
- **AS1170.0 :2002** – Structural Design Actions – General Principles
- **AS1170.1 :2002** – Structural Design Actions – Permanent, Imposed and other Actions
- **AS1170.2 :2021** – Structural Design Actions – Wind
- **AS1170.4 :2007** – Earthquake Actions in Australia
- **AS3600 :2018** – Concrete Structures
- **AS3826 :1998** – Strengthening Existing Buildings for Earthquake *

** AS3826 is not referenced in the NCC as it is not applicable to new buildings, but is a good basis for an assessment procedure which specifically deals with earthquake actions with respect to existing buildings.*

1.2 Documentation

SFE have been provided with existing structural drawings for the S-Type building. It is not clear if said drawings are "as-built" drawings, however they appear to be provided with appropriate detail representative of documentation which may have been referenced during the construction of the S-Type buildings. It is considered that minor modifications and changes may exist from site to site subject to specific site requirements and coordination with other engineering disciplines for each site.

The drawings have been produced by W. P. Brown & Associates and are generally dated 1963.

SFE have also been provided with an article extract from a publication called "Australian Civil Engineering and Construction" which references the buildings in question and the general type of construction techniques adopted.

2.0 Building Summary

2.1 General Building Form

The S-Type building comprises 20 suspended habitable floors and a suspended roof (total ground to roof height of building is circa 61.5m). With reference to the typical plan arrangement presented in Figure 1, notable elements of the typical floor include:

- Access corridor
- Centralised dual lift core
- Fire stairs near the extremity of each wing
- Centralised general amenity and service riser zones (including garbage chutes etc.)
- Habitable apartments

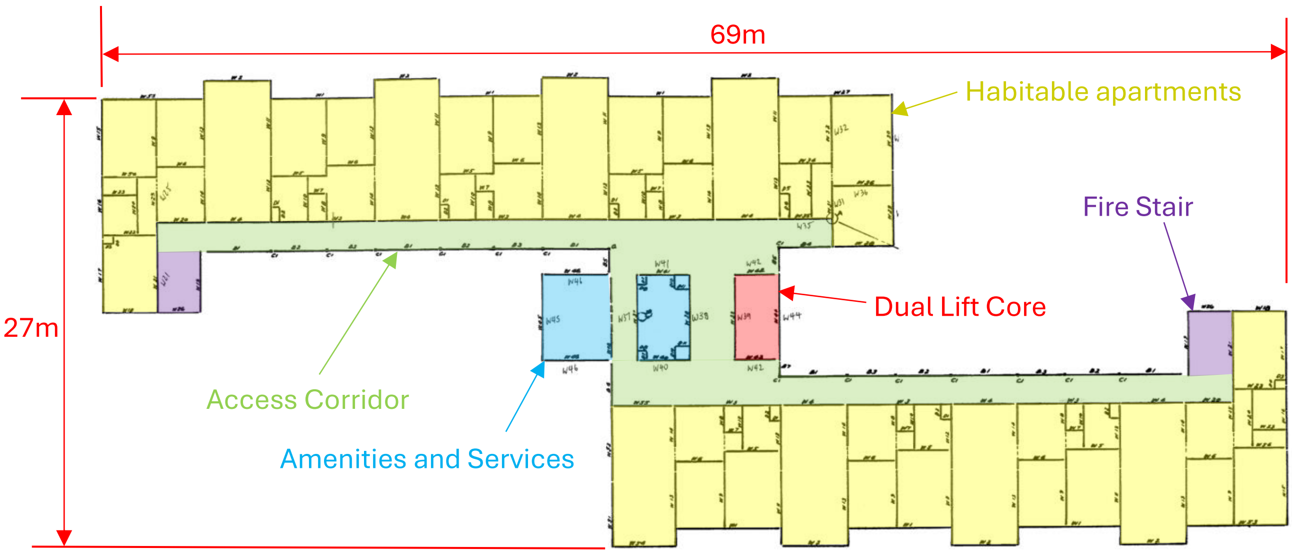

Figure 1 – Plan view, typical floor, of the S-Type Housing Commission Building.

For reference purposes, the orientation of the building plan in Figure 1 will assume north to be towards the top of the page. It is noted that given that the S-Type building is located at each respective site in different plan orientations, reference to north may change from site to site.

As indicated in Figure 1 the building is 69m wide in the east-west direction and 27m long in the north-south direction.

2.2 Structural Details

2.2.1 Typical Floor Slab Structure

The structural floor of the typical apartment floor adopts pre-cast concrete slab panels placed in a checkerboard type fashion.

The slab panels generally span in the east-west direction between "principal load-bearing walls" which run in the north-south direction. Figure 2 presents an isolated plan view of the eastern wing of the building which identifies the principal load-bearing walls as well as the pre-cast slab panel break-up.

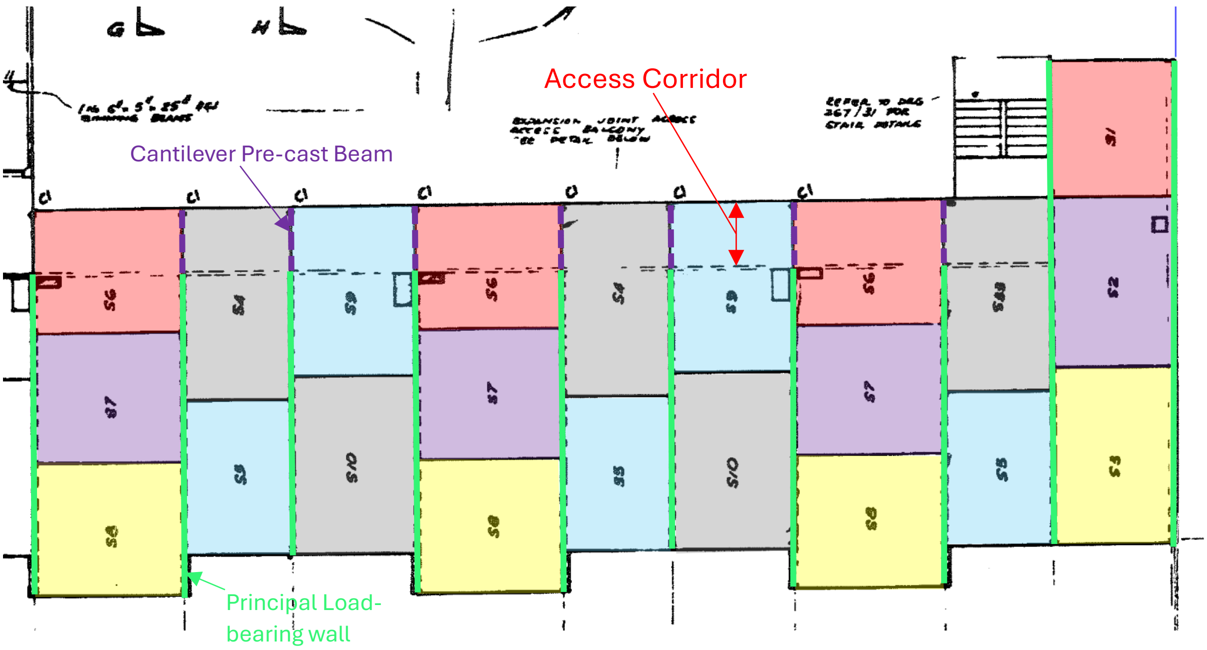

Figure 2 – Isolated plan view of east-wing identifying principal load-bearing walls and pre-cast slab segments

As indicated in Figure 2, the slab panels span between pre-cast concrete cantilevered beams over the access corridor extent.

Based on the information provided within the structural documentation, details of the pre-cast slab panels are presented below in Table 1:

Table 1 – Typical Pre-cast slab panel details adopted within the S-Type Building

Element	Imperial Value	Metric Equivalent
Slab Thickness	5”	127mm
Top Reinforcement	605 Mesh (No. 5 Bar at 6” Pich)	5.385 DIA. Bar @ 152mm
Bottom Reinforcement	306 Mesh (No. 6 Bar at 3” Pich)	4.877 DIA. Bar @ 76mm

Figure 3 shows an isolated view of a single slab panel with further detailed elements identified, which include:

- **Cast-in Steel angles:** The slab panel-to-panel connection typically comprises a lose flat plate being welded to a corresponding cast-in angle plate to each adjacent panel. The cast-in angle plates are welded to cogged tail bars which are embedded in each panel. Refer to Figure 4 for a cross-section showing details of the panel-to-panel joint.
- **Top Hairpin Bars:** Top hairpin bars are cast-in to the slab panels and protrude out the ends (in the direction of the slab span, see cross-section at edge of panel identifying this in Figure 5). The detailing of the top hairpin bars indicates that the intention is to provide connection from panel-to-panel as well as provide continuity of top reinforcement (and therefore span) over each principal load-bearing wall. The aim of which appears to be allowing the pre-cast panel to behave as a continuous in-situ type slab over each wall support location.
- **Panel Edge Rebate:** A rebate is provided at the end of each panel (generally the east and west ends) where the panels sit above load-bearing walls. This appears to allow the introduction of a grouted connection (wet stitch) to further assist the floor in behaving as a continuous in-situ floor and also allows a grouted base connection to the load-bearing wall above each respective floor. Figure 5 shows a cross-sectional view of a typical slab panel with the rebate identified.
- **Lifting Points:** Each of the slab panels has been provided with 4x lifting points to allow for lifting out of the casting bed and craning the panels into position.

To allow continuity of reinforcement (and therefore span), a lose flat plate is welded to the protruding top hairpin bars of each panel. The junction of load-bearing wall to slab panel specifies 2x stages of grouting. With reference to the cross-section presented in Figure 6, The floor panel erection/construction sequence appears to comprise the following steps:

1. Load-bearing wall under is installed
2. Packers are provided at the top of the load-bearing wall to allow future grouted joint and provide tolerance
3. The pre-cast slab panels are then lowered into position
4. The first stage of grouting is completed with includes the area between the underside of the slab panel and load-bearing wall under and the gap between the slab panel ends
5. The lose flat plates are welded to each top hairpin bar allowing continuity of reinforcement to be established
6. The pre-cast panel above is installed and temporarily propped.
7. The second stage grouting is completed which includes the area to the underside of the load-bearing wall above.

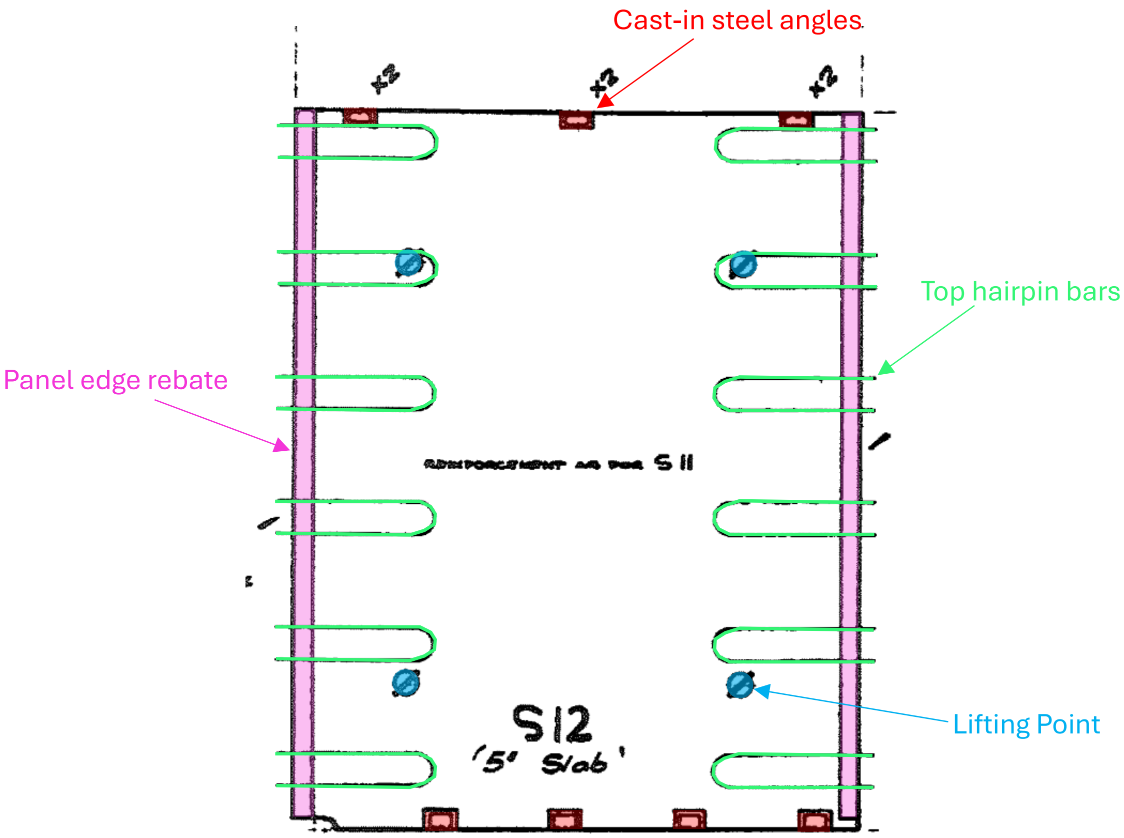

Figure 3 – Isolated plan view of a single pre-cast slab panel

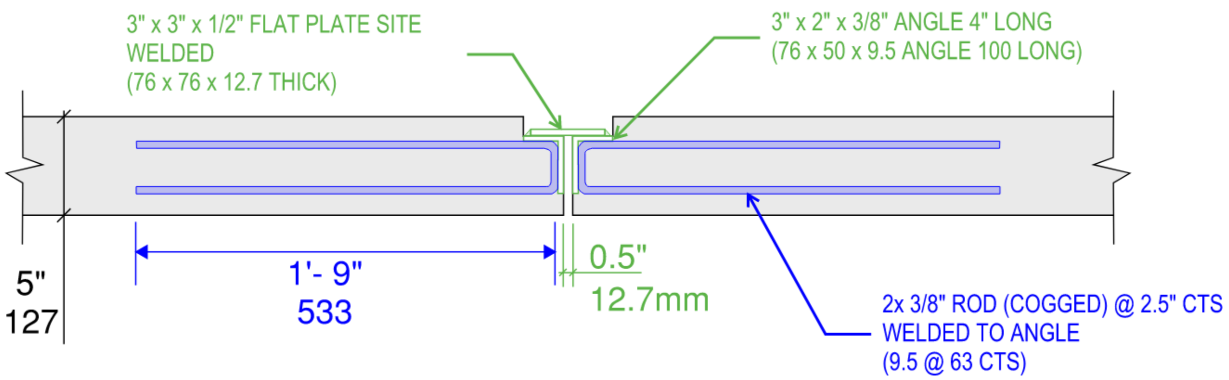

Figure 4 – Typical pre-cast panel-to-panel joint connection detail (joint parallel to direction of span)

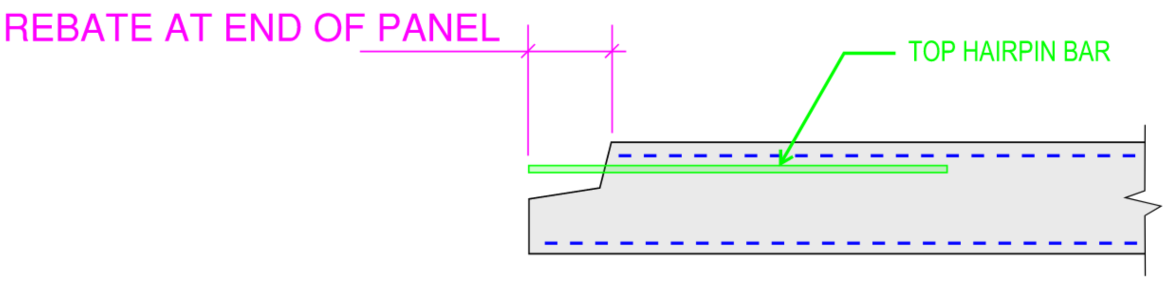

Figure 5 – Typical cross-section at end of pre-cast slab panel. Indicating protruding top hairpin bar and rebate at and of panel.

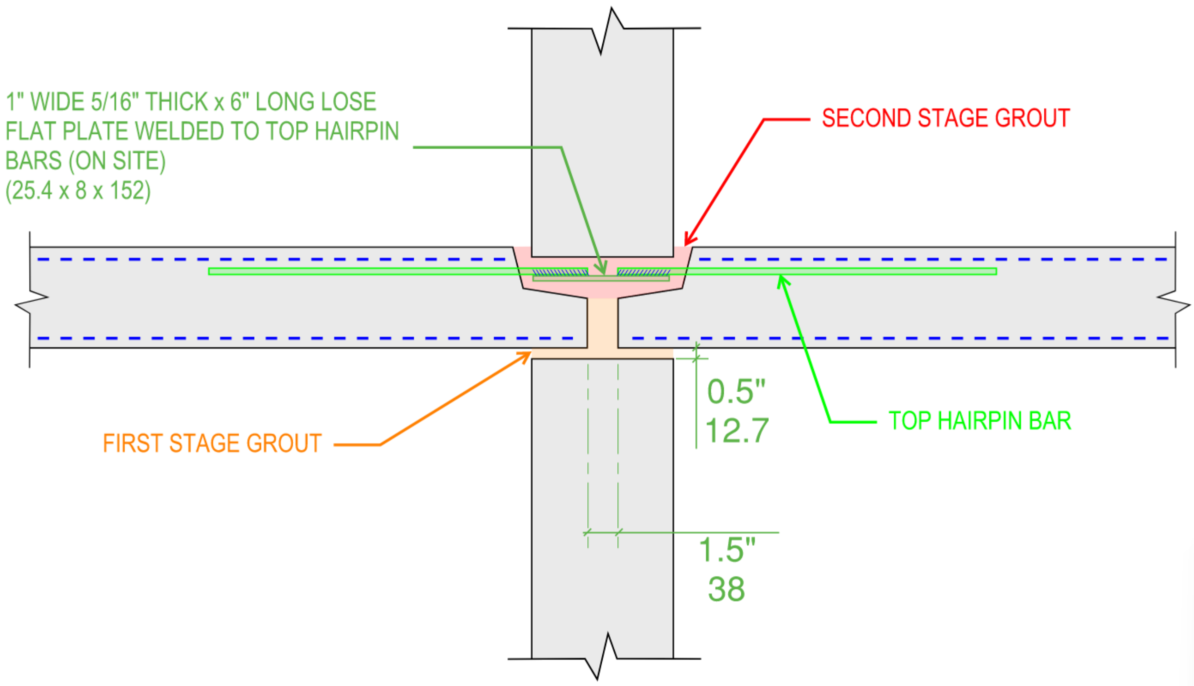

Figure 6 – Typical Panel to load-bearing wall junction cross-section.

2.2.2 Typical Load-Bearing Wall Structure

Pre-cast concrete detailing has also been adopted for the load-bearing walls. The wall panel break-up appears to have been made based upon crane lifting capacity, transportation considerations and the layouts of each apartment.

The wall panels are a single height lift and stop-start to the underside and topside of each slab (as indicated in previous Figure 6).

Figure 7 shows a typical example of an apartment pre-cast concrete party wall. This indicates the general approach to the size and break-up of each wall panel with respect to the layout of the building.

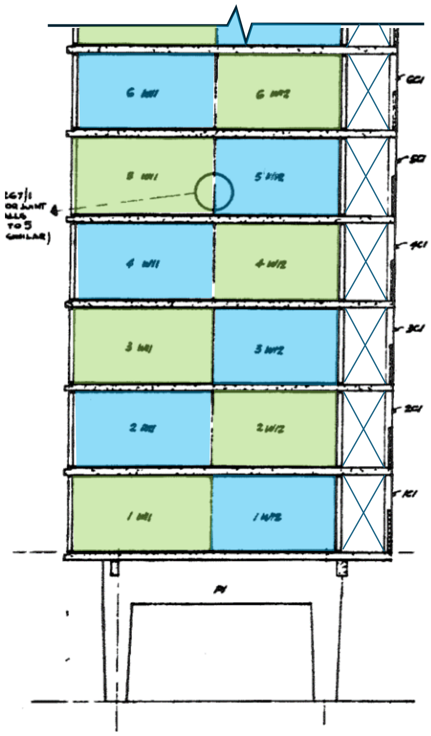

Figure 7 – Typical elevation of apartment party wall (section taken in the west wing of the building looking towards the east). Note this is a partial elevation which terminates at level 7, detailing indicated here continues to roof level)

The thickness and reinforcement for the wall panels varies depending upon the location of the wall (both in plan and elevation). Based on the information provided within the structural documentation, Table 2 provides a general summary of the wall details dependent upon the location of each wall…

Table 2 – Typical Pre-cast wall panel details adopted within the S-Type Building based on location

Plan Location	Level	Thickness (inch)	Thickness (mm)	Vertical Reinf. INCH (mm)	Vertical Reinf. Spacing (mm)[1.]	Horizontal Reinf. INCH (mm)	Horizontal Reinf. Spacing (mm)[1.]
Principal Load-bearing walls (running north-south) within each wing	1-5	7”	178	1/2”(12.7)	304	3/8”(9.5)	304
	6-12	6”	152	1/2”(12.7)	304	3/8”(9.5)	304
	13-20	4”	100	1/2”(12.7)	457	3/8”(9.5)	304
End walls of each wing	1-5	7”	178	1/2”(12.7)	304	3/8”(9.5)	304
	6-20	6”	152	1/2”(12.7)	304	3/8”(9.5)	304
North and South façade panels	1-20	4”	100	3/8”(9.5)	130	1/2”(12.7)	450
General interior walls to apartments	1/20	4”	100	3/8”(9.5)	530	3/8”(9.5)	605

1. *Generally, the bar spacing identified in this table is an average spacing across the wall panel as the spacing varies across each panel and is generally not called up as a consistent spacing*

The reinforcement identified in the structural documentation calls up 2x layers of reinforcement (this is for all wall thicknesses at all locations including the 100mm thick walls up to the 178mm thick walls).

Table 3 below shows a summary of the cover requirements specified for the wall reinforcement within the structural documentation. Figure 8 shows a typical cross-section of a standard 6 inch pre-cast wall indicating the reinforcement cover and orientation:

Table 3 – Typical reinforcement cover specification for pre-cast wall panels according to the structural documentation

Bar	Cover (INCH)	Cover (mm)
Horizontal Reinforcement	1”	25.4
Vertical Reinforcement	1 3/8”	35

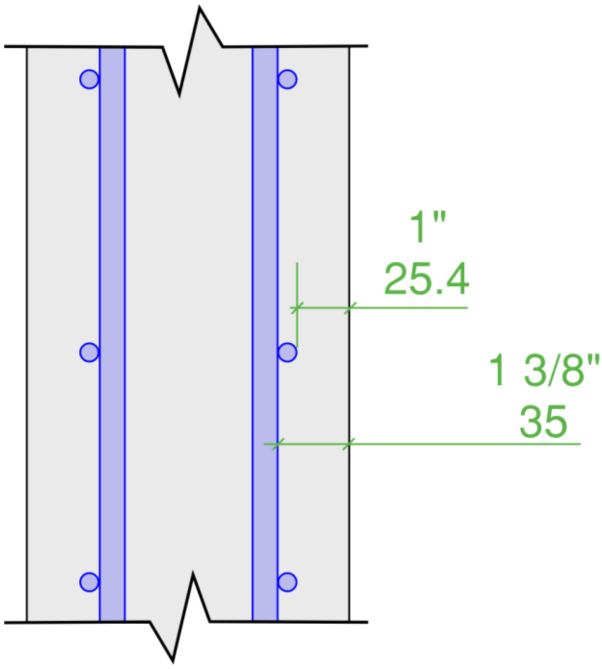

Figure 8 – Pre-cast panel wall cross-section indicating typical bar configuration and cover for 6” wall

The top of the pre-cast panels is provided with dowel bars which protrude out the top of the panel and terminates within the wet-stitch zone of the one-way pre-cast slab panels (see Figure 9 indicating this in cross-seton).

The base of the panels are embedded 1/2 inch (12.7mm) into the wet-stitch joint of the pre-cast slab panels. No reinforcement is provided at the bottom panel junction (again see Figure 9 illustrating this in cross-section). This is with the exception of select panel locations which have allowance for the lower dowel to continue through and embed a small distance (2.5 inch or 63mm) into the base of the pre-cast panel above. This arrangement is presented in the cross-section in Figure 10.

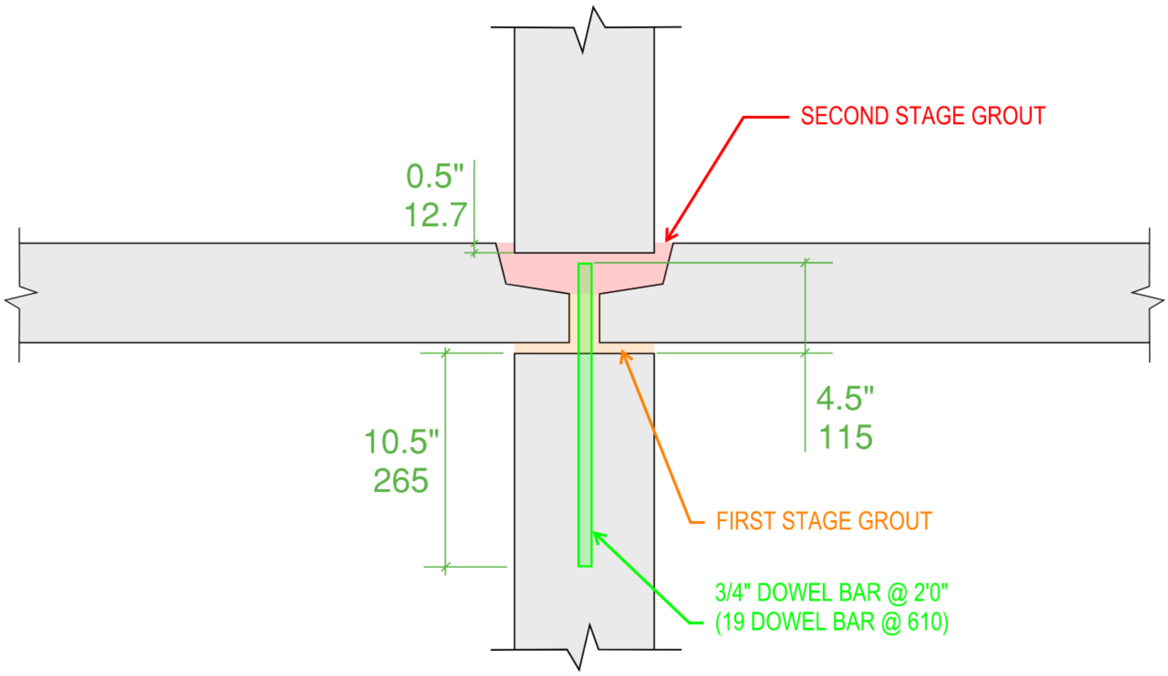

Figure 9 – Typical load-bearing wall Panel to slab panel connection detail (dowel terminates within slab depth). This detail occurs in the majority of the wall panels.

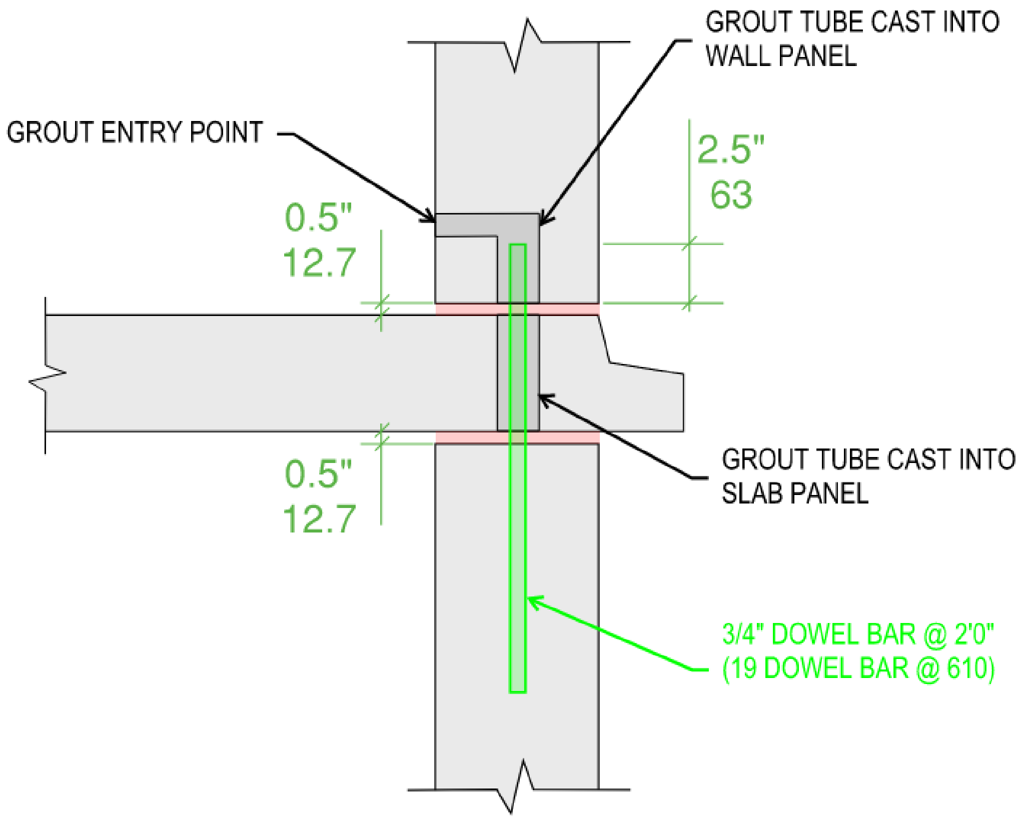

Figure 10 – Typical load-bearing wall panel to slab panel connection (with dowel continuing into panel above). This detail occurs at some select wall panel locations within the building.

The detail presented in Figure 10 (where the connecting dowel continues through to the panel above), generally occurs at:

- The end walls of each wing at the east and west extremities
- The lift core walls
- Most of the walls which house the Amenities and services areas
- 2x discrete dowels at each long façade wall panel on the north and south sides of the habitable apartment areas of the building.

Figure 11 illustrates the locations identified above in plan view for clarity.

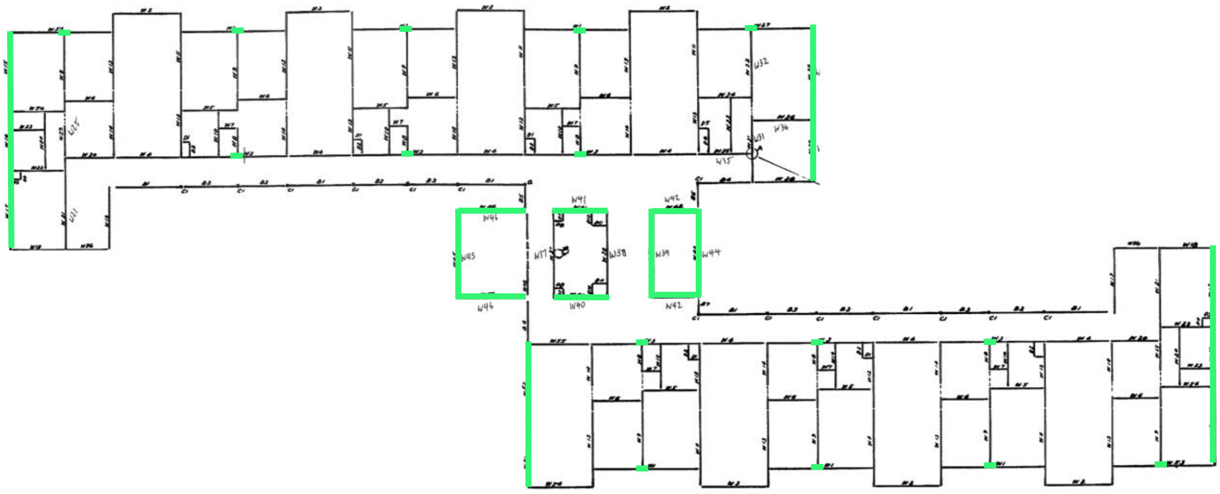

Figure 11 – Plan view of typical floor highlighting where pre-cast wall panel dowel continues from panel below and into panel above (green)

Full building height tension rods are also provided in the outermost east/west wing walls and second to last walls. These rods are embedded in their respective panels and have a lapped/splice connection at each floor level.

The documentation refers to a welded plate type arrangement (similar to that of the top reinforcement of the slab panels presented in Figure 6 although with reinforcement orientated in the vertical direction as opposed to the horizontal direction). However, the documentation marks these details are "superseded" and references an "approved C.H.P. alternative bolted connection" detail. This alternative bolted connection does not appear to be included in the documentation we have been provided with. Nevertheless, it is assumed that such an alternate detail seeks to achieve the same desired Structural Engineering objective, i.e. allow full continuity of the tension rods to be maintained for the full height of the building.

2x tension rods are provided at each end of the previously identified wall locations, each being 7/8" dia. (or circa 22mm). Figure 12 shows a plan of the typical floor identifying the location of the full height tension rods,

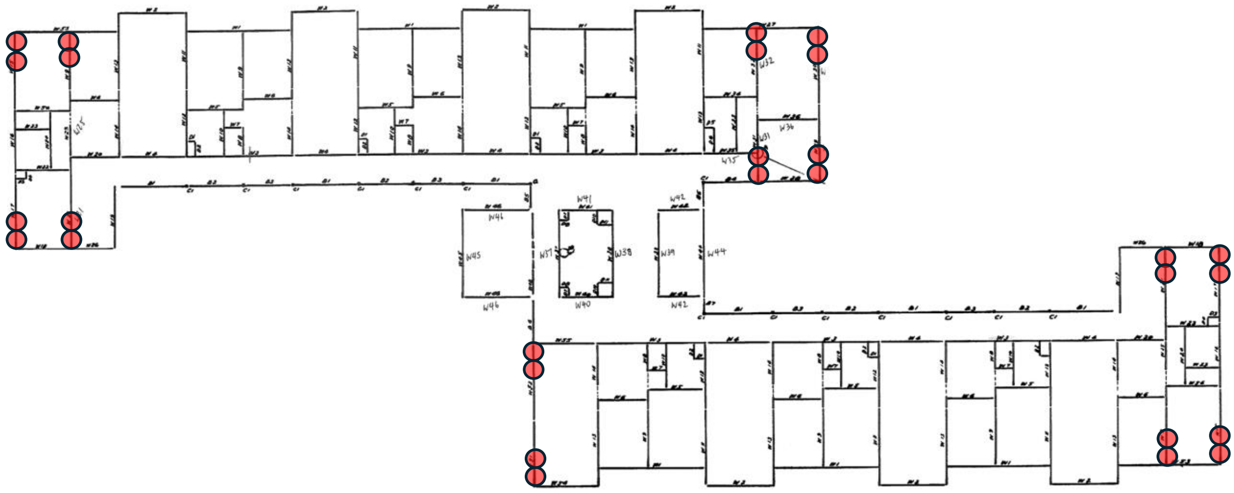

Figure 12 – Plan view of typical floor identifying location of full building height tension rods embedded within wall panels (not to scale)

Figure 13 – Elevation view of westernmost wall of west wing indicating full height tension rods within wall panels.

2.3 Level 1 Transfer Structure

The majority of the primary and secondary load-bearing pre-cast walls terminate above the level 1 floor level.

This necessitates a transfer structure across much of the extent of level 1. A series of in-situ post-tensioned concrete beams span in the north-south and east-west directions.

- North-South Beams generally 1400 deep and 600 wide
- East-west Beams generally 460 deep and 300 wide

Pre-cast panels sit directly above the transfer beams to complete the floor structure (the pattern and detailing being much the same as the typica floor above as identified in Section 2.2.1 of this report).

Figure 14 presents an isometric view of the beam and wall arrangement in isolation between ground floor and level 1 (slab panels have been hidden from view for clarity).

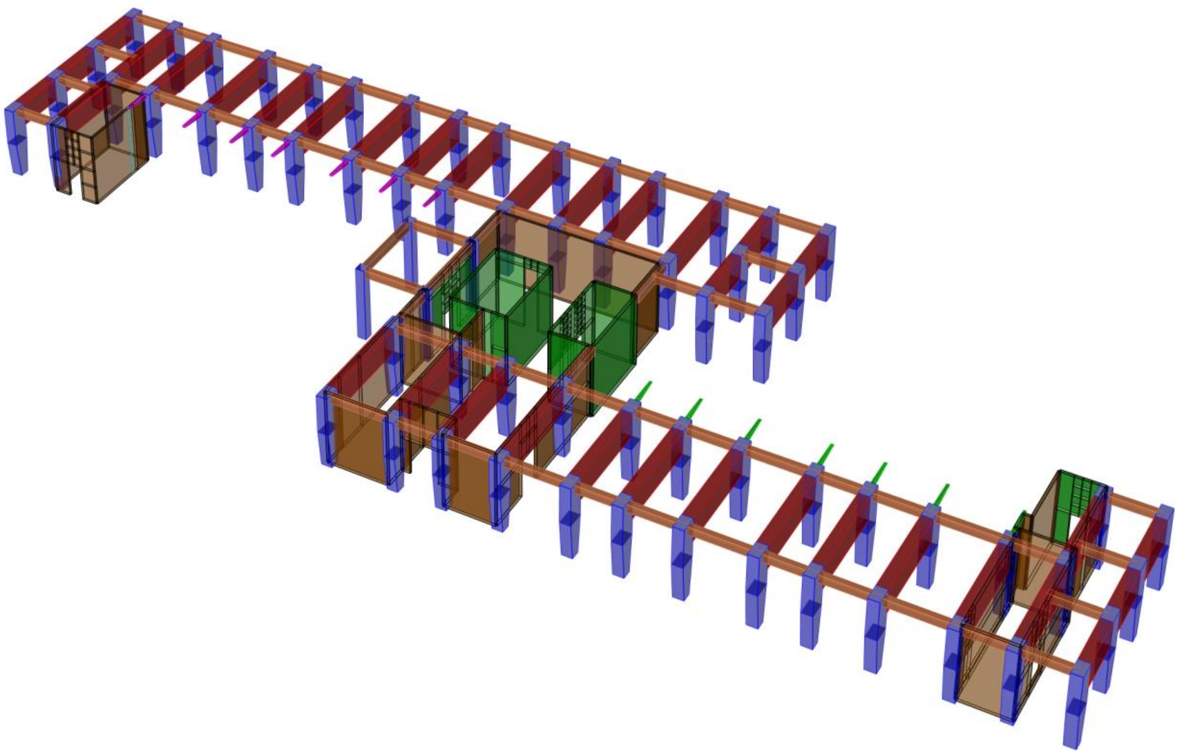

Figure 14 – Isometric view of level 1 (concrete floor panels hidden from view for clarity) Indicating walls which continue through to foundation level and transfer beams spanning in the north-south and east-west directions.

2.4 Concrete Properties

The documentation we have at hand for the concrete specification of the building indicates that a light-weight concrete mix was used throughout the entire super-structure. Table 4 provides a summary of the concrete properties which have been specified for the building:

Table 4 – Concrete property specification for the S-Type tower super structure

Concrete Property	Value (Imperial)	Value (Metric)
28 Day Compressive Strength	3,000-lb/in^2	20MPa
Density	115-lb/ft^3	18kN/m^3

The concrete density adopted represents almost 30% weight reduction when compared with standard weight concrete mixes generally used today. The concrete 28-day compressive strength of 20MPa is significantly less than standard concrete mixes used today which are generally around 40MPa on average.

3.0 General Non-Compliances with Current Code

Design standards constantly evolve as technology advances and more research is conducted into different fields of engineering.

Prior to undertaking our analysis on the S-Type tower structure, a general review of code compliance is necessary. This review identifies detailing and minimum reinforcement requirements which need to be adhered to for new structures built to comply with current design standards.

These detailing and minimum reinforcement requirements are generally required irrespective of the applied load on the structure.

3.1 Wall Reinforcement spacing

The concrete design code (AS3600) outlines the maximum allowable spacing of vertical and horizontal reinforcement within structural walls.

An extract of AS3600-2018 identifying this requirement is presented in Figure 15 below…

> **11.7.3 Spacing of reinforcement**
>
> The minimum clear distance between parallel bars, ducts and tendons shall be sufficient to ensure the concrete can be placed and compacted to conform with Clause 17.1.3 but shall be not less than $3d_b$.
>
> The maximum centre-to-centre spacing of parallel bars shall be the lesser of $2.5t_w$ and 350 mm.

Figure 15 – Extract of AS3600-2018 specifying maximum and minimum reinforcement spacing requirements for structural walls (highlights added by SFE).

As highlighted in the extract above, the maximum reinforcement spacing is the lesser of 350mm or $2.5t_w$ with t_w being the thickness of the wall in question.

Table 5 therefore provides a summary of the maximum reinforcement spacing for the typical 4", 6" and 7" walls which have been used throughout the building. Given the small dimension of the 4" thick wall, the reinforcement spacing requirements for this wall is controlled by the $2.5t_w$ requirement, whereas the 6" and 7" thick walls are governed by the 350mm maximum spacing requirement.

Table 5 – Maximum reinforcement spacing requirements based on AS3600-2018 requirements for each wall thickness adopted in the S-Type building

Wall Thickness INCH (mm)	Maximum Reinforcement Spacing (mm)
4" (100)	250
6" (150)	350
7" (175)	350

A summary of the wall thicknesses and reinforcement sizes and spacings used within each wall is provided within Appendix A of this report.

Cells highlighted in **red** for the vertical and horizontal reinforcement spacing identifies non-compliance with the maximum spacing requirements specified in AS3600-2018.

It is noted that the majority of the non-compliances exist with the "lesser" load-bearing pre-cast walls within the building and not the "principal" load-bearing walls (see previous Figure 2 identifying general location of principal load-bearing walls.

The spacing limitations outlined in AS3600-2018 are generally intended to control cracking within concrete walls and is therefore not strictly speaking a strength requirement. In some instances, the bar spacing requirement may be waved when assessing existing/older structures. Arguably, a more important metric to satisfy is the minimum reinforcement quantity requirements within the walls for the vertical and horizontal directions. This then leads to the next area of assessment...

3.2 Minimum Wall Reinforcement Requirements

The previous section explored the maximum reinforcement spacing requirements in accordance with AS2600-2018. There is also specification with respect to the minimum quantity of reinforcement required within structural walls.

While the spacing of reinforcement is a simple measurement of the distance between reinforcing bars, the reinforcement quantity is a combination of the spacing as well as the diameter of the reinforcement. It is a measure of how much reinforcement is provided, as a percentage factor, within the wall.

It may be considered as a more important requirement compared to simply spacing alone as the overall minimum reinforcement quantity requirements ensures a minimum level of robustness which needs to be maintained within the structure.

11.7 **REINFORCEMENT REQUIREMENTS FOR WALLS**

11.7.1 **Minimum reinforcement**

Walls shall have a reinforcement ratio (p_w)—

(a) in the vertical direction, of not less than the larger of 0.0025 and the value required for strength unless the design axial compressive force does not exceed the lesser of 0.03 f'_c and 2 MPa where the limit may be reduced to 0.0015; and

(b) in the horizontal direction, of not less than 0.0025, except that for a wall designed for one-way buckling [using Clause 11.4(a)] and where there is no restraint against horizontal shrinkage or thermal movements, this may be reduced to zero if the wall is less than 2.5 m wide, or to 0.0015 otherwise.

Figure 16 – Extract of AS3600-2018 specifying minimum reinforcement requirements for structural walls

The extract above identifies that the minimum reinforcement requirements for the vertical and horizontal reinforcement of walls is 0.0025 (or 0.25% of the walls concrete cross-section) which can be reduced to 0.0015 (or 0.15% of the walls concrete cross-section) based on the vertical stress in the wall for the case of vertical reinforcement and walls with length less than 2.5m which are unrestrained for the case of horizontal reinforcement.

The reinforcement ratios for each wall type in the vertical and horizontal direction is also presented in Appendix A of this report (see the second last and fourth last columns). Walls which pass the criteria are highlighted in green, while walls which fail this criterion are highlighted in red. It is evident that the majority of the walls throughout the building satisfy the minimum reinforcement requirements specified in AS3600-2018.

It is further note that the walls which do not satisfy the minimum reinforcement requirements are the lesser load-bearing walls, as opposed to the principal load-bearing walls (those identified in Figure 2).

3.3 Minimum Dowel Bar Requirements

AS3600-2018 specifies the minimum requirements for dowel connection of pre-fabricated structural walls. An extract of this requirement is presented below in Figure 17.

17.7.3 **Vertical integrity ties**

All vertical structural members except for non-loadbearing elements shall have connections at horizontal joints in accordance with the following requirements:

(a) Connections between prefabricated columns shall have a designed strength in tension not less than $f_{vt} \times A_g$, where f_{vt} is taken as 1.4 MPa and A_g is in mm^2. For columns with a larger cross sections than required for strength, a reduced effective area A_g may be used based on the cross section required but shall be not less than one-half the total area.

(b) Prefabricated wall panels shall have a minimum of two ties per panel, with a designed strength not less than 45 kN per tie.

and

(c) When design forces result in no tension at the base, the ties required by Clause 17.7.3(b) shall be anchored into a reinforced concrete floor slab-on-ground or footing.

Connection details that rely solely on friction caused by gravity loads shall not be permitted.

Figure 17 – Extract of AS3600-2018 identifying minimum dowel/tie requirements for pre-fabricated structural walls

According to the extract above, a minimum of 2x ties per panel is required, each of which requiring a design capacity of 45kN. Further, there is a specification that connection details that rely solely on friction caused by gravity loads shall not be permitted.

As identified in Section 2.2.2 and illustrated in Figure 9, the base connection for the majority of the pre-cast panels (both the principal load-bearing panels and the secondary load-bearing panels) relies on a half inch embedment (12.7mm) of the base of the panel into the wet-stich grout connection of the pre-fabricated wall panels, without the inclusion of reinforcement ties/dowels.

This does not satisfy the requirements of item (b) highlighted in Figure 17. In the event of an earthquake and the resultant rigorous building shaking, it is also plausible that the 12.7mm notch within the wet-stitch which holds the base of the panels in place may spall, crack and no longer become effective, thus relying predominantly on friction to hold the base in place from moving laterally. Therefore, strictly speaking the final paragraph highlighted in Figure 17 is not satisfied given the detailing which has been provide in the structural documentation for the S-Type building.

This would be considered a major breach and significantly reduce the overall robustness requirements of the building compared to that required in AS3600-2018. Therefore, to achieve a bass level of code-compliance, this item would require structural intervention in order to meet code requirements. Refer to Section 5.0 for proposed strengthening and structural interventions which have been developed as part of our study.

3.4 Embedment and Development of Dowel Bars

As identified in Section 2.2.2, there are select locations where dowels/ties are provided at the base of the wall panels (see Figure 10 for the detail at these locations and Figure 11 for the locations on plan where they occur).

It has been noted that the dowel bar projects into the base of the pre-cast panels only a short distance (2.5" or circa 63mm). AS3600-2018 specifies the minimum development length requirement for reinforcing bars. An extract of this requirement is presented in Figure 18 below

13.1.2.4 *Development length to develop less than the yield strength*

Where the full yield strength of the bar is not required, the development length (L_{st}) to develop a tensile stress (σ_{st}), less than the yield strength (f_{sy}), shall be calculated from—

$$L_{st} = L_{sy.t} \frac{\sigma_{st}}{f_{sy}} \qquad \text{. . . 13.1.2.4}$$

but shall be not less than—

(a) $12d_b$; or

(b) for slabs, as permitted by Clause 9.1.3.1(a)(ii).

Figure 18 – Extract of AS3600-2018 specifying the minimum development (embedment) requirements for reinforcing bar.

The minimum development (or embedment) of reinforcing bar as per the extract above is $12d_b$ with d_b being the diameter of the reinforcing bar in question. As per the detail presented in Figure 10, the dowel bar diameter is ¾" (circa 19mm) meaning that the minimum development/embedment is 12 x ¾" which is 9 inches (or 12 x 19 which is 228mm). This means that the embedment which has been specified on the structural documentation is around 30% of that which is required as per the provisions of AS3600-2018.

This means that the effectiveness of these dowel bars for shear (and tension) is reduced from that which may be calculated using the guidelines within AS3600-2018. The effectiveness, particularly during earthquake shaking, is difficult to quantify as it falls outside of the guidelines of the concrete design code. Therefore, a prudent approach would be to assume their effectiveness as being negligible for the purposes of assessing the overall buildings performance considering such a reduced embedment.

4.0 Analysis

4.1 Assessments Undertaken

The structural analysis performed on the building explores 3x possible separate scenarios with respect to the strengthening and improvements to be made to the structure. These include:

- **33% Seismic Load Assessment (no other modifications made to the building):** The structure has been assessed under a seismic load equivalent in magnitude to 33% of the code required seismic load on the building. It has been widely accepted that strengthening of existing buildings to satisfy current code requirements may become cost prohibitive, particularly when assessing the seismic compliance of older buildings which pre-date 1993 (which saw the first major revisions of the Australian Seismic design code developed). Previous studies have been undertaken which have determined that a minimum compliance of 33% may result in significant damage to the building in question, however, provides sufficient robustness to ensure life-safety to the building's occupants. This was the approach proposed in the now expired design standard AS3826 (Strengthening Existing Buildings for Earthquake). It was also the approach mandated in the Christchurch region of New Zealand following the significant earthquake event they experienced in February 2011.
- **100% Seismic Load Assessment (no other modifications made to the building):** The structure has been assessed under current seismic load requirements (100% of that specified within the Australian Seismic Design Code).
- **100% Seismic Load Assessment with modifications to the structure to allow installation of apartment extension:** A further study has been undertaken to determine the adequacy of the structure under the hypothetical scenario that an apartment extension system be installed at each apartment across the full height of the building. The extension is to provide a circa 1.0m increase in length to each apartment . The extension will provide both an increase to the usable floor area of each apartment and also enhances the thermal and climate performance of the dwellings due to the sun shading.

4.2 Desing Assumptions and Parameters

The following sections provides a summary of design parameters which have been adopted as part of our study. Table 6 provides a summary of the wind and seismic factors which have been adopted as part of our assessment of the S-Type tower.

Table 6 – Earthquake and Wind Factors adopted as part of the assessment of the S-Type tower

Factor	Value
Importance Level	3
Return Period (Ultimate Wind)	1/1000
Return Period (Serviceability Wind)	1/25
Return Period (Seismic)	1/1000
Wind Region	A5
Terrain Category	3
Combination Factor k_c	0.9
Ductility (μ)	1.0
S_p	0.77
k_p	1.3
Z	0.09

4.3 Typical Slab Assessment

The typical slab structure has been assessed given the required design loading and proportions which have been provided within the structural documentation and summarised within this report.

The typical slab/floor arrangement was generally found to be adequate and code compliant with current design standards.

This includes the assessment for removal of the north and south façade pre-cast panels and installation of the apartment extension.

4.4 Earthquake and Wind Assessment

Our assessment indicates that the building is structurally adequate under wind loading conditions when assessed based on current code requirements.

However, design deficiencies were observed with wall strength in both tension and shear action for seismic loading conditions. This was the case for both the 30% load application and 100% load application.

Figure 19 shows an example elevation of the westernmost primary load-bearing wall of the building as well as the next wall adjacent in the east direction. Regions shaded in blue represent locations within the wall which are over-stressed and require additional strengthening, while the magenta shading represents locations where the walls are structurally sufficient based on the design wall stresses.

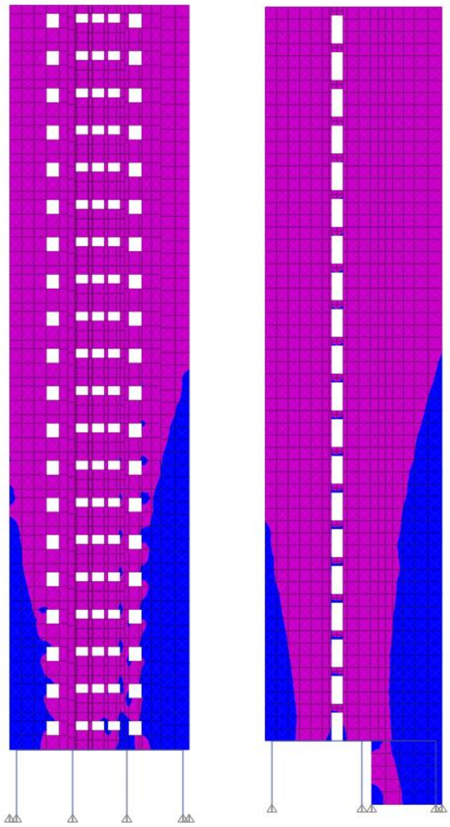

Figure 19 – Elevation of the westernmost primary load-bearing wall (left) and the next internal wall immediately to the east (right) indicating locations where the walls are overstressed based on seismic loading conditions (blue shaded areas).

5.0 Proposed Strengthening Measures

The proposed strengthening measures for the pre-cast walls aim to achieve compliance with the minimum reinforcement and connection requirements identified in Section 3.0 of this report as well as the strength deficiencies identified in Section 4.4.

In order to achieve the minimum connection requirements from wall to wall, it is proposed that a short length of steel plate be provided either side of the pre-cast panel which are connected to the panel via through-bolts. The steel plates are to continue from the top of one panel through the slab immediately above and connect to the base of the panel above. A cross-section arrangement of this connection detail is presented in Figure 20 below…

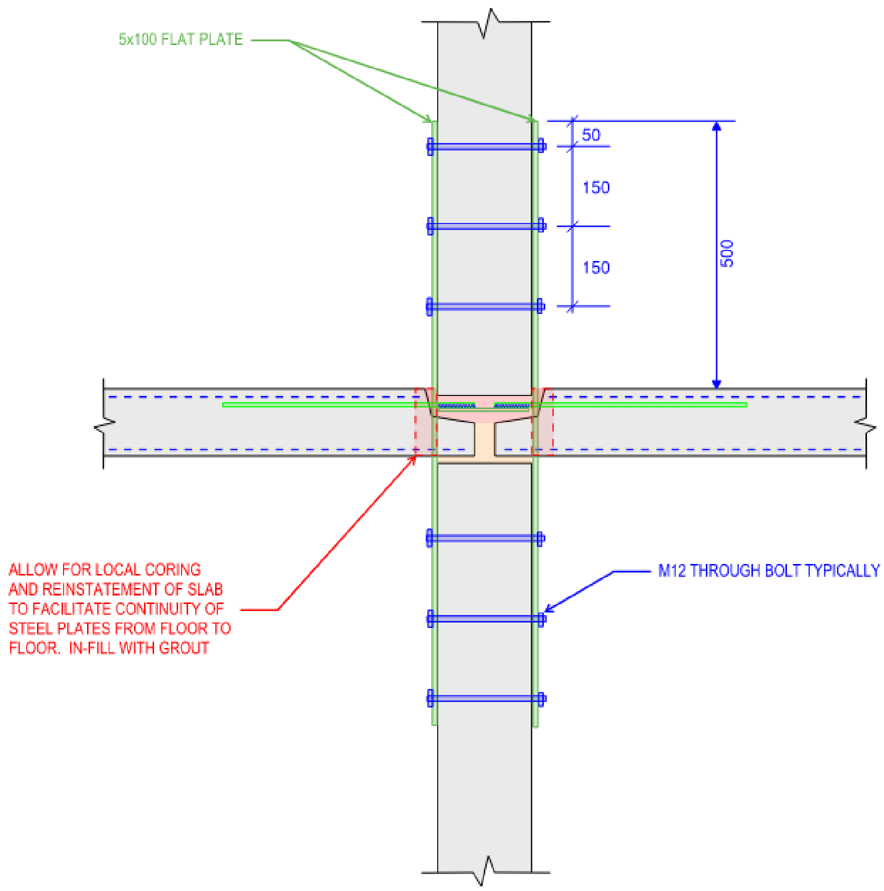

Figure 20 – Cross-section view of typical wall-to-wall connection detail with strengthening plate.

The proposed strengthening indicated within Figure 20 is required at all panel locations and one connection point is to be provided at each base corner of each panel, making 2x connection points per panel as per code requirements.

Where insufficient strength is found within the wall, the strengthening plates are to continue through the height of the wall and terminate at the required location (beyond which point the wall stresses become structurally acceptable). There are also a number of locations where horizontal plate strengthening is required due to insufficient shear capacity of the walls themselves. The typical arrangement where vertical and horizontal strengthening plates are required within the wall is presented in the cross section at Figure 21.

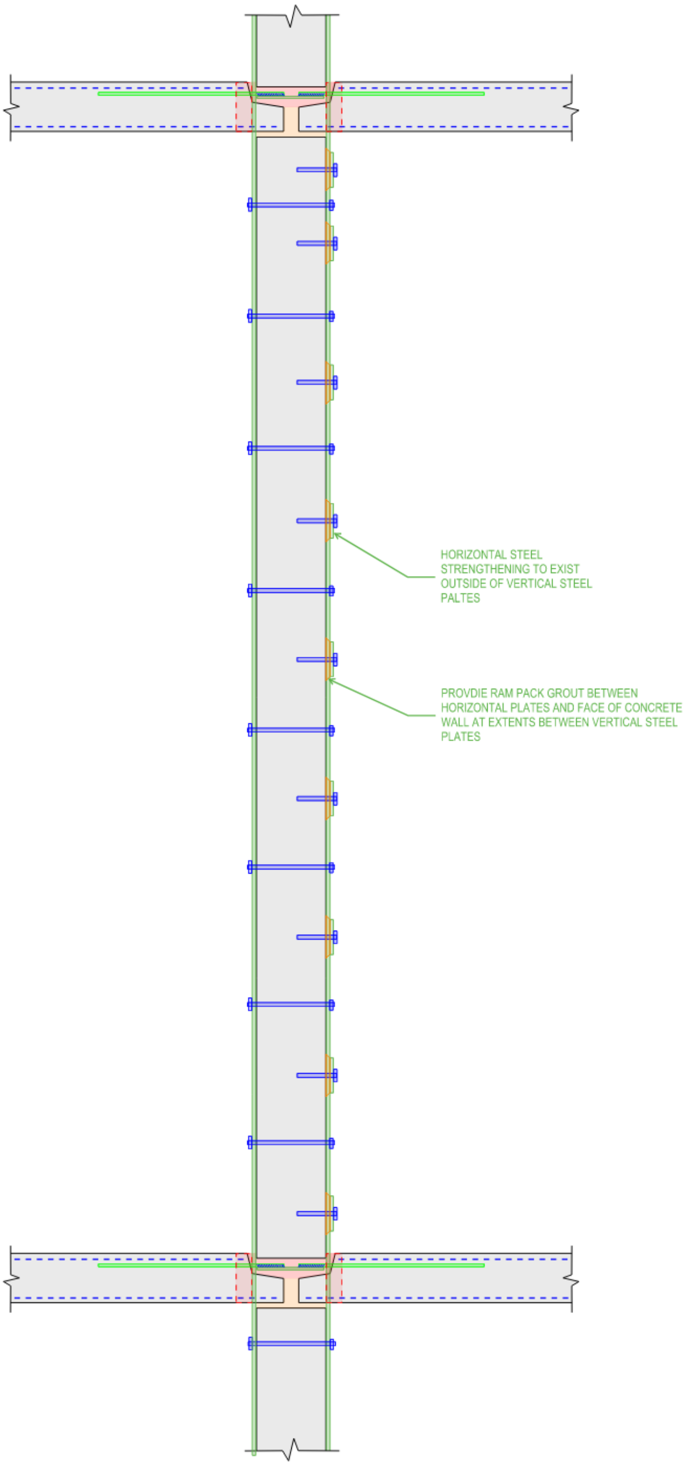

Figure 21 – Cross-section view of wall indicating arrangement where vertical and horizontal strengthening plates are required.

6.0 Possible Future Studies

The strengthening options for the housing commission tower are not restricted by the confines of the recommendations which are provided within this report and accompanying drawings.

Other studies may be undertaken with respect to achieving code compliance within the tower. This section briefly explores two possible such options which may be further investigated via separate studies.

6.1 Carbon Fibre Strengthening

Carbon fibre strengthening has become more and more popular over the last few decades. It can offer a comparable, and in some cases, superior outcome to conventional steel plate strengthening.

The location and orientation of the reinforcing requirement remains the same for the currently proposed steel plates, however the installation time may see a reduction due to the wide-spread bolting to the concrete walls not being required. Figure 22 shows an example strengthening arrangement to the base of a shear wall/column structure. The carbon fibre strands are orientated in both directions at regular intervals to achieve vertical tension capacity as well as horizontal shear capacity.

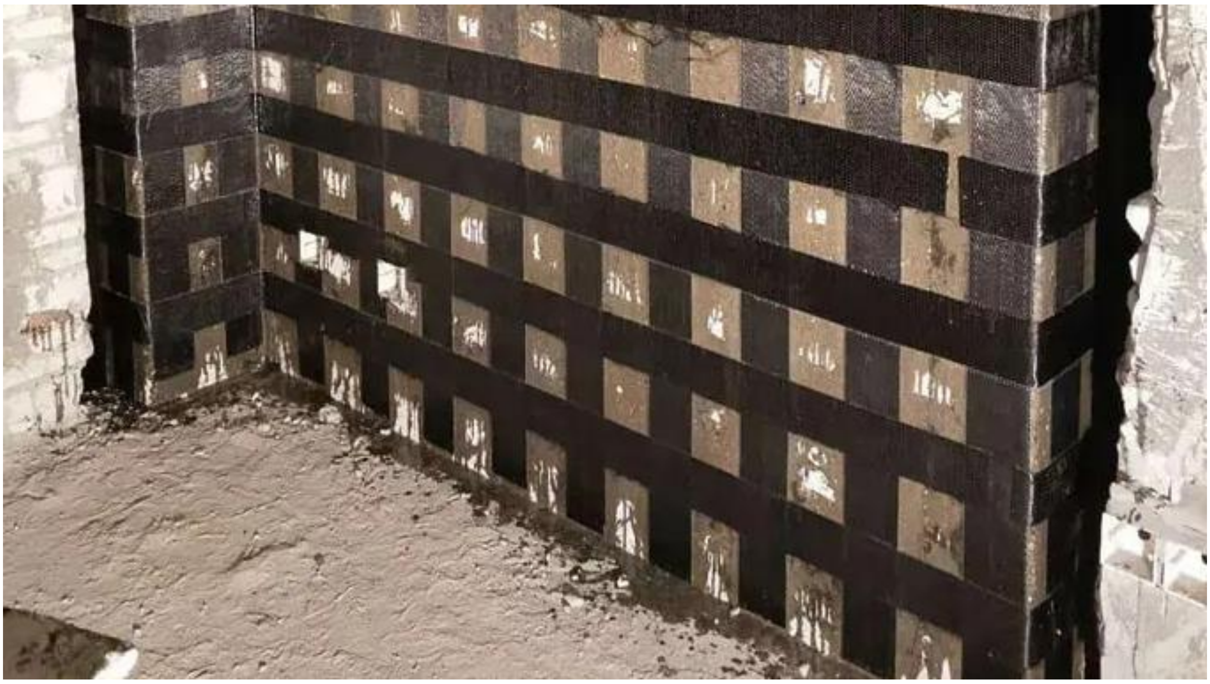

Figure 22 – Example image of a concrete shear wall strengthened through the introduction of carbon fibre reinforcing in both vertical and horizontal directions.

It is noted that design and installation of such strengthening systems are quite specialised in nature and should be assessed by a qualified and experienced operator who is familiar with carbon fibre strengthening when applied to concrete shear wall enhancement.

6.2 Introduction of Additional Shear Walls

Another option which may be explored is the introduction of additional shear walls and/or core walls to the building. It is understood that the current vertical transport solution for the building may provide insufficient quantities of lifts and unacceptable wait times. This is particularly the case when factoring in lifts which are not providing service from time-to-time due to break-down and maintenance requirements. This may be alleviated through the introduction of additional lifts and lift shaft structures.

The introduction of additional lift shafts would also provide the structural benefit of providing additional strengthening and rigidity to the building. Dependant upon the proportioning and location of the additional lift cores, their introduction may relieve the stress burden during seismic conditions on the existing shear walls which have largely been found to have insufficient reinforcement and connection detailing.

Further lateral stiffening elements may be introduced through the introduction of additional apartments to the building. The party walls to such additional apartments may act in the same manner as the aforementioned lift cores.

Introduction of any new core or shear wall element would require adequate structural connection between the new built portion and the existing building to ensure that adequate load transfer can be achieved through the two elements. A logical location for those potential new elements are at the extreme western and easter portions of the building. These locations are best suited from a planning and layout perspective and also are the locations where the wall stresses within the existing shear walls of the building were found to be the highest due to torsional effects from earthquake shaking. Figure 23 presents an indicative plan arrangement at the western wing of the building illustrating how this new extension may be achieved.

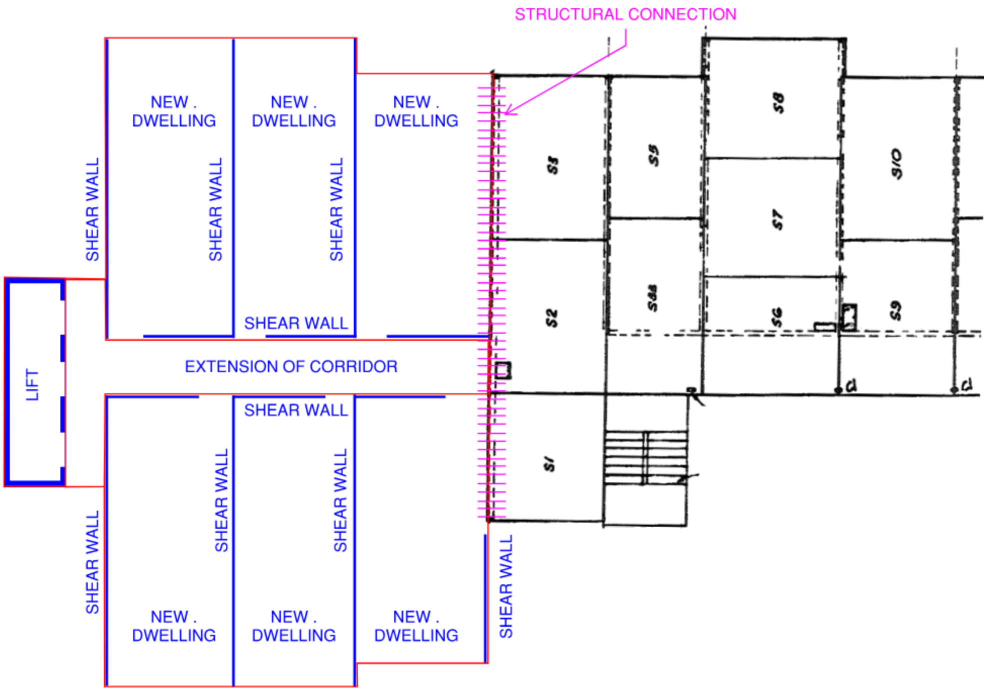

Figure 23 – Hypothetical plan arrangement at western wing of building indicating possible new dwellings and lift shaft and associated new additional structural shear wall elements.

Appendix A:

Wall thickness and reinforcement Summary

S-Type Tower Wall Thickness and Reinforcement Summary

SHEERFORCE ENGINEERING

TAG	LOCATION	WALL THICKNESS (INCH)	WALL THICKNESS (mm)	CROSS-SECTION AREA (mm^2)	REINFORCEMENT LAYERS	HORIZ BAR DIA (INCH)	HORIZ BAR DIA (mm)	HORIZ BAR SPACING (INCH)	HORIZ BAR SPACING (mm)	HORIZ BAR AREA mm^2/m	HORIZONTAL BAR AREA mm^2/mm	VERT BAR DIA (INCH)	VERT BAR DIA (mm)	VERT BAR SPACING (INCH)	VERT BAR SPACING (mm)	VERT BAR AREA mm^2/m	HORIZ BAR RATIO	MIV HORIZ BAR RATIO	VERT BAR RATIO	MIN VERT BAR RATIO
W1	UNDER OPENINGS	4	101.6	101600	2	1/2	12.7	5	130	1949	1.949	3/8	9.525	18	450	317	0.0192	0.0025	0.0031	0.0025
W1	BETWEEN OPENINGS	4	101.6	101600	2	3/8	9.525	18	450	317	0.317	3/8	9.525	18	450	317	0.0031	0.0025	0.0031	0.0025
W8	UNDER OPENINGS	4	101.6	101600	2	1/2	12.7	5	130	1949	1.949	3/8	9.525	18	450	317	0.0192	0.0025	0.0031	0.0025
W8	BETWEEN OPENINGS	4	101.6	101600	2	3/8	9.525	18	450	317	0.317	3/8	9.525	18	450	317	0.0031	0.0025	0.0031	0.0025
1W16 - 5W16	UNDER OPENINGS	7	177.8	177800	2	3/8	9.525	20	500	285	0.285	3/8	9.525	19	470	303	0.0016	0.0025	0.0017	0.0025
1W16 - 5W16	BETWEEN OPENINGS	7	177.8	177800	2	3/8	9.525	12	315	452	0.452	3/8	9.525	4	90	1583	0.0025	0.0025	0.0089	0.0025
6W16 - 20W16	UNDER OPENINGS	6	152.4	152400	2	3/8	9.525	20	500	285	0.285	3/8	9.525	19	470	303	0.0019	0.0025	0.0020	0.0025
6W16 - 20W16	BETWEEN OPENINGS	6	152.4	152400	2	3/8	9.525	12	315	452	0.452	3/8	9.525	4	90	1583	0.0030	0.0025	0.0104	0.0025
W3	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	28	700	362	0.362	3/8	9.525	25	630	226	0.0036	0.0025	0.0022	0.0025
W20	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	20	520	274	0.274	3/8	9.525	18	450	317	0.0027	0.0025	0.0031	0.0025
W4	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	19	475	533	0.533	3/8	9.525	20	510	279	0.0052	0.0025	0.0028	0.0025
W55	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	19	475	533	0.533	3/8	9.525	20	510	279	0.0052	0.0025	0.0028	0.0025
W35	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	31	790	321	0.321	3/8	9.525	32	820	174	0.0032	0.0025	0.0017	0.0025
W39	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
W42	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
W42	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
6W40 - 20W40	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0029	0.0025	0.0049	0.0025
1W41 - 5W41	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
6W41 - 20W41	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0029	0.0025	0.0049	0.0025
W44	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
W45	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	13	320	445	0.445	1/2	12.7	13	340	745	0.0025	0.0025	0.0042	0.0025
W46	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	9	240	594	0.594	3/8	9.525	12	311	458	0.0039	0.0025	0.0030	0.0025
1W8 - 5W8	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W8 - 12W8	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W8 -20W8	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W9 - 5W9	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W9 - 12W9	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W9 -20W9	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W11 - 5W11	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W11 - 12W11	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W11 -20W11	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W12 - 5W12	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W12 - 12W12	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W12 -20W12	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W13 - 5W13	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W13 - 12W13	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W13 -20W13	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W814- 5W14	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W14 - 12W14	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W14 -20W14	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W21 - 5W21	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W21 - 12W21	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W21 -20W21	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W25 - 5W25	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W25 - 12W25	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W25 -20W25	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W31 - 5W31	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W31 - 12W31	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W31 -20W31	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W32 - 5W32	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W32 - 12W32	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W32 -20W32	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W38 - 5W38	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W38 - 12W38	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
13W38 -20W38	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	18	457.2	554	0.0046	0.0025	0.0055	0.0025
1W15 - 5W15	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W15 - 20W15	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
1W17 - 5W17	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W17 - 20W17	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
1W29 - 5W29	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W29 - 20W29	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
1W30 - 5W30	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W30 - 20W30	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
1W51 - 5W51	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W51 - 20W51	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
1W52 - 5W52	AVERAGE ALL AREAS	7	177.8	177800	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0026	0.0025	0.0047	0.0025
6W52 - 20W52	AVERAGE ALL AREAS	6	152.4	152400	2	3/8	9.525	12	304.8	468	0.468	1/2	12.7	12	304.8	831	0.0031	0.0025	0.0055	0.0025
W2	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W6	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W5	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W56	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W34	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W23	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W10	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W18	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W28	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W24	AVERAGE ALL AREAS	4	101.6	101600	2	3/8	9.525	22	560	254	0.254	3/8	9.525	21	540	264	0.0025	0.0025	0.0026	0.0025
W53	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0071	0.0025	0.0071	0.0025
W22	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0071	0.0025	0.0071	0.0025
W19	AVERAGE ALL AREAS	7	177.8	177800	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0041	0.0025	0.0041	0.0025
W26	AVERAGE ALL AREAS	6	152.4	152400	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0047	0.0025	0.0047	0.0025
W7	AVERAGE ALL AREAS	7	177.8	177800	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0041	0.0025	0.0041	0.0025
W37	AVERAGE ALL AREAS	7	177.8	177800	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0041	0.0025	0.0041	0.0025
W27	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0071	0.0025	0.0071	0.0025
W50	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0071	0.0025	0.0071	0.0025
W36	AVERAGE ALL AREAS	4	101.6	101600	2	1/2	12.7	14	350	724	0.724	1/2	12.7	14	350	724	0.0071	0.0025	0.0071	0.0025