Structural Concept Design

Rev-A

Mona Vale Surf Life Saving Club (SLSC)

30th August 2018

Warren and Mahoney

171328

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Structural
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1.0 Executive Summary

Taylor Thomson Whitting has been commissioned to provide Structural, Civil & Traffic Engineering consultancy services for Mona Vale Surf Life Saving Club redevelopment through the project architects Warren and Mahoney.

The design is proposed to be split into 2 stages;

- Stage 1: Up to completion of Concept Design.
- Stage 2: DA, Design Development and Construction (subject to Council approval for the project to proceed to stage 2).

Stage 1 of the project is likely to include the following;

- Review of the previous studies which have been undertaken on the site to ascertain their applicability to the current project requirements.
- Development of strategies for decanting.
- Development of new building structures as a result of evaluation of the future requirements.
- Review of the service requirements against the project budgets and development of a priority list of the project works.
- Review of the existing civil infrastructure across the site.
- Site Civil works including new stormwater systems and detention, and connection into existing systems.
- Possible new car parking throughout the site and on adjacent roads.
- Assisting Services Consultants with development of plant areas and reticulation strategies.
- Future proofing the new development.

Stage 2 of the project is likely to include the requirements to;

- Finalisation of design documentation for the new works.
- Preparation of tender drawings and specifications.
- Briefing of tendering contractors where necessary.
- Assisting in post tender reviews.
- Responding to queries during construction.
- Site inspection works the following items are some of the key areas which may be able to provide benefit to the project.

This report outlines the Stage 1 structural concept designs and is guided by the architectural design intent with consideration to the project specific requirements and constraints drawn from previous advice provided to other consultants. The table below provides a summary of the items considered and advise for consideration as the project develops through stage 1.

Images within this document have typically been taken from the information provided to TTW by Warren and Mahoney and as such sources have not been identified. Where external images have been used references are provided.

Item	Current Advice/Constraints	TTW Advice
Materials	Non-Corrosive Materials	Wood – Glulam, CLT, External Plywood Concrete (B2 Exposure) f'c min 40MPa Stainless steel fittings SAF 2205 Duplex. Keep fittings inside the glazing line if possible.
Foundations	Conventional footings inside the acceptable risk line. 250kPa bearing capacity for shallow footings (1.5m deep) 800kPa on bored piers (2.5m deep) Ref: JK Geotechnics report ref:28092ZRrpt 13/02/2015)	Trench boxes to 1.5m required for strip footings. Preferred option is 600mm diameter shallow bored piers to 2.5m depth. Boring tubes required to support loose Sandside walls. Spacing to be determined based on load distribution.
Ground Level Slab	No advice provided for slab on ground. Further geotechnical advice required to assess the local sand as suitable for a slab on ground.	A 150mm slab on ground should be achievable within the footprint of the structure. A thicker slab may be required or works to the subgrade may be to increase the CBR. Works to be advised by geotechnical engineer.
Walls	Robust ground level walls required to support the level 1 slabs and transfer structures Level 1 walls to be lightweight to minimise foundation loads and to provide lateral stability to the roof structure.	140 Reinforced masonry walls typically. Reinforced concrete walls within wall thickness where higher loads are transferred. 200 Reinforced concrete wall adjacent to the feature cantilevered stairs.
Suspended Slab	Large spans required to provide open space within central storage area. Level 1 framing does not align with ground level structure therefore suspended slabs are required to transfer Level 1 framing loads. (Consider potential to align structure to remove transfer.)	205 precast hollowcore panels + 60 avg. structural screed spanning reinforced concrete walls. Central section: 300 Post-tensioned banded slab with 450x2400 band beams 400 Post-tensioned flat plate. North Section: 350 reinforced concrete flat slab (if 450 circular column can be positioned) All precast hollowcore panels will need to be supplied interstate.
Roof	Roof framing must large spans over the central function room and southern members lounge. Framing must also achieve portal overhangs.	Glulam columns proposed along facade line. Option 1 – 380x130 GL13 glulam beams at 4.8m c/c with 200x45 hySpan secondary beams at 1.2m c/c and 25 plywood topping. Option 2 – 380x130 GL13 glulam beams with 130 CLT in the secondary direction Option 3 – 300 solid CLT panels

Feature Stairs	Cantilevered concrete stair treads (1250mm long with 245mm going)	Vibration and deflection criteria indicate a minimum tread depth of 75mm required. The exposure classification may require the depth to increase if an off-form concrete solution is to be pursued. Precast concrete treads are recommended to achieve a high-quality finish. Typically, slender cantilevered treads are achieved with steel structural elements 'clad' in finished stone/concrete. If this is

2.0 Site History

Mona Vale Surf Life Saving Club (SLSC) was formed in 1922. There were originally two club houses on the current site. The current club house which was built by the members and gifted to the local community in 1969. It is now considered a council asset which is leased back to the club.

The current club house consists of a brick façade and internal walls. Existing footings are expected to be conventional strip footings and isolated pad footings. This assumption would be consistent with the preliminary geotechnical investigation report (JK Geotechnics Ref: 28092ZRrpt 2015).

Figure 1: Satellite Image of Mona Vale SLSC General Position

Figure 2: Existing Mona Vale SLSC

Figure 3: Existing Mona Vale SLSC Elevations

3.0 Proposed Redevelopment

The proposed redevelopment illustrated below is based on the preferred architectural option presented in the concept options presentation by Warren and Mahoney. It is noted that the presented designs are a work in progress and have been used as a point of reference for the development of structural concepts.

Works include the demolition of the existing structure and the construction of a new 2 storey club house. The preferred concept divides the structure into 3 distinct sections on plan.

At ground level, the central section houses the life saving store and surf sports store. This section requires large clear spans to enable boats and other large equipment to be manoeuvred within the space. The North and South sections require greater division of space to house the public changing and WC facilities in addition to a café, nippers shop/store, gym, canteen and ancillary offices and stores.

The first floor contains a large function room above the central storage area, a member's lounge with kitchen and meeting room in the southern section and a restaurant and kitchen above the North section. The first floor also includes external balconies on the north and central sections. Based on an overlay of the ground level and the first-floor plans, the balconies represent substantial cantilevered areas with limited structural support points to enable them to perform.

The open spaces required in the first-floor limits structural support points for the roof to within the façade line along the East elevation. As the first-floor façade line does not coincide with the structural walls below a transfer structure will be required.

Figure 4: Built Form Architectural Concept (Concept Options, Warren and Mahoney, 2017)

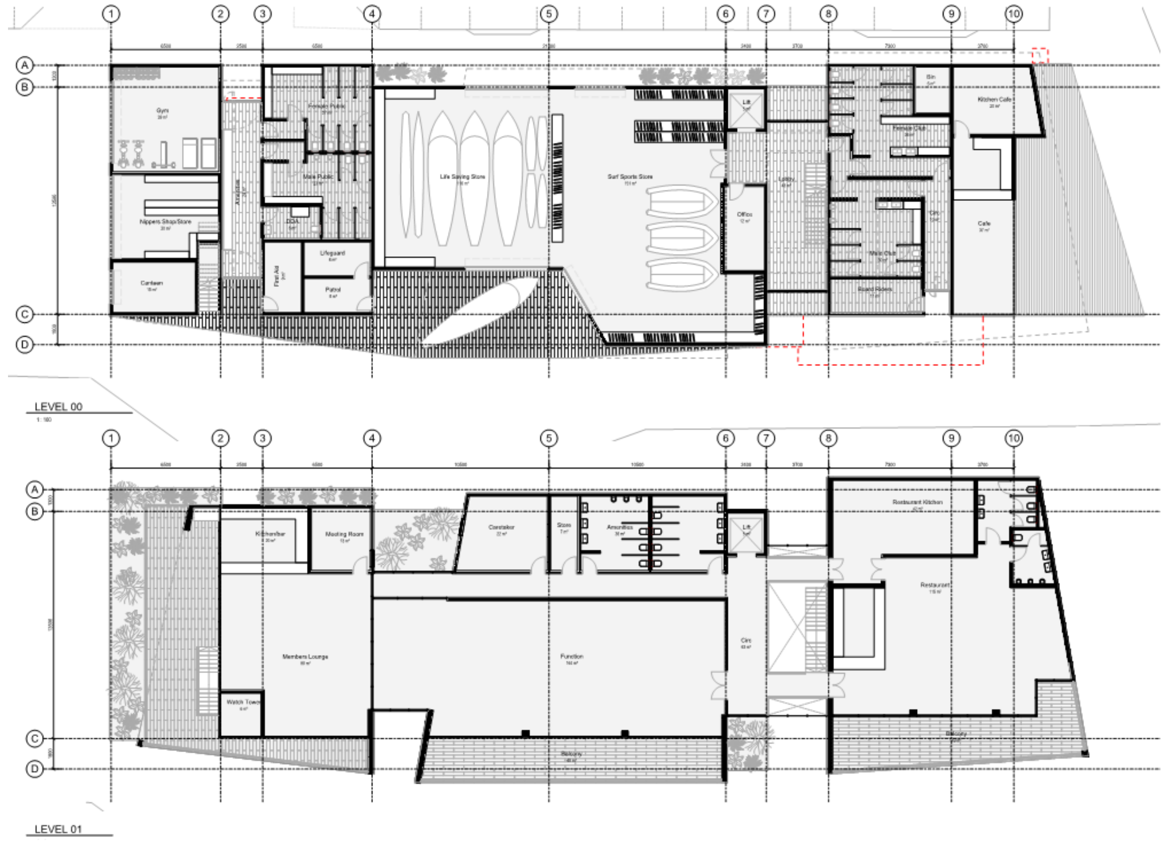


Figure 5: Architectural Floor Plans

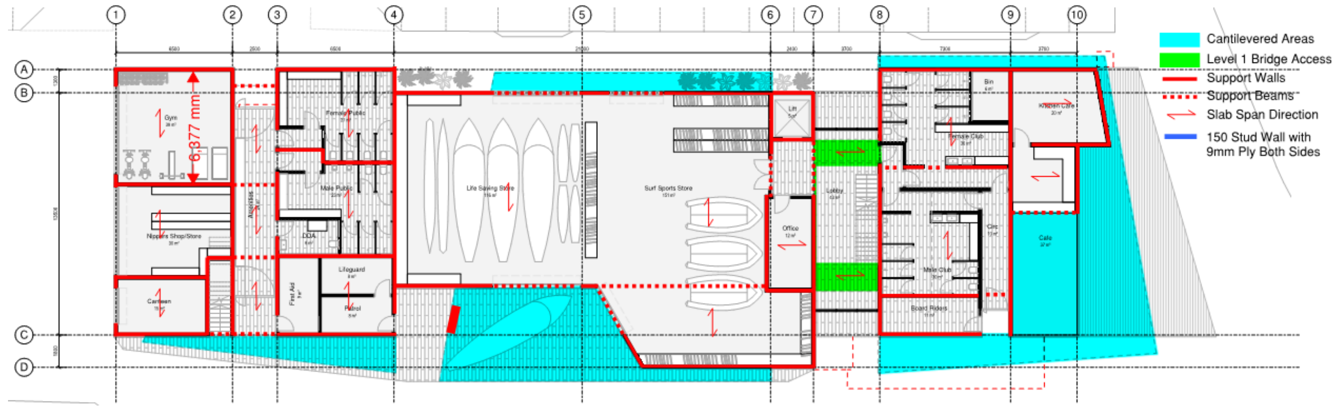


Figure 6: Preliminary Structural Framing

4.0 Site Constraints

4.1 The Site and Surrounds

The site is located along Surfview Road, Mona Vale. There are no neighbouring structures in the vicinity of the proposed works. It is noted that sand dunes, dune vegetation and local trees are to be protected during the works. There appears to be a clear grassed area available to the south. This area could be utilised for on-site storage and set down areas and may be beneficial if precast concrete components require on site storage prior to lifting.

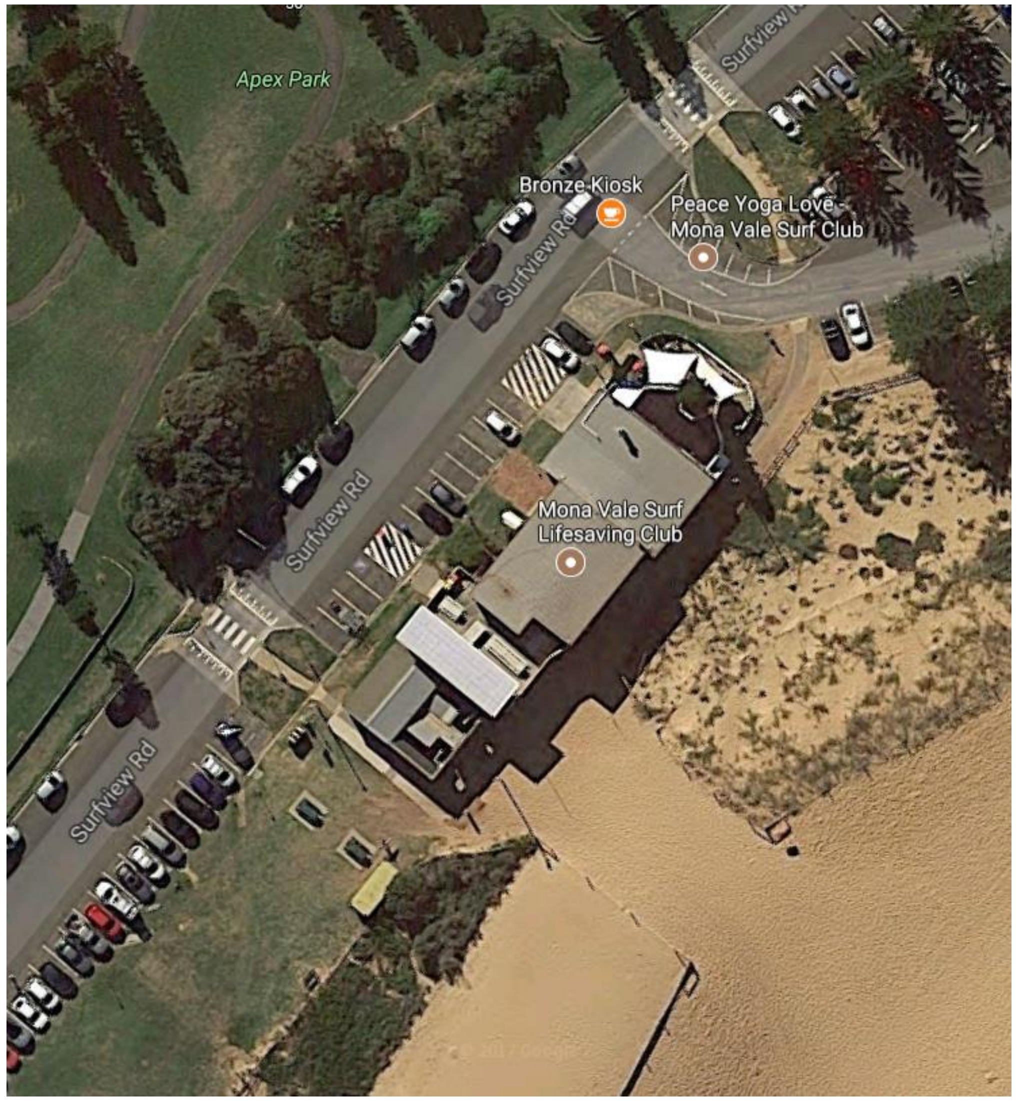


Figure 7: Satellite Image of Mona Vale SLSC Clubhouse

4.2 Coastal Erosion

A comprehensive coastal erosion report has been provided to TTW to aid in the preparation of concept designs. The primary structural output of the report was the definition of the “acceptable risk” lines detailing the extent at which conventional or piled foundations would be required. Based on the geometry of the proposed structure it appears that the new club house is planned to be within the dimensions of the existing building. This would locate the new proposed structure within the region specified as an acceptable risk on conventional foundations. Conventional foundations include shallow strip footings, isolated pad footings and raft slabs.

Based on the size and use of the structure it is likely that conventional footings will be suitable for the project. There can be considerable savings gained through the use of conventional foundations over piled foundations.

If required, a combination of piled and conventional footings can be considered. There is added complexity involved when a structure is based on a variety of foundation types. Particular attention is required to locate separation joints within the structure and allow for differential settlements based on geotechnical advice. As a general rule differential settlements should be limited to 1mm for every 1m between foundations.

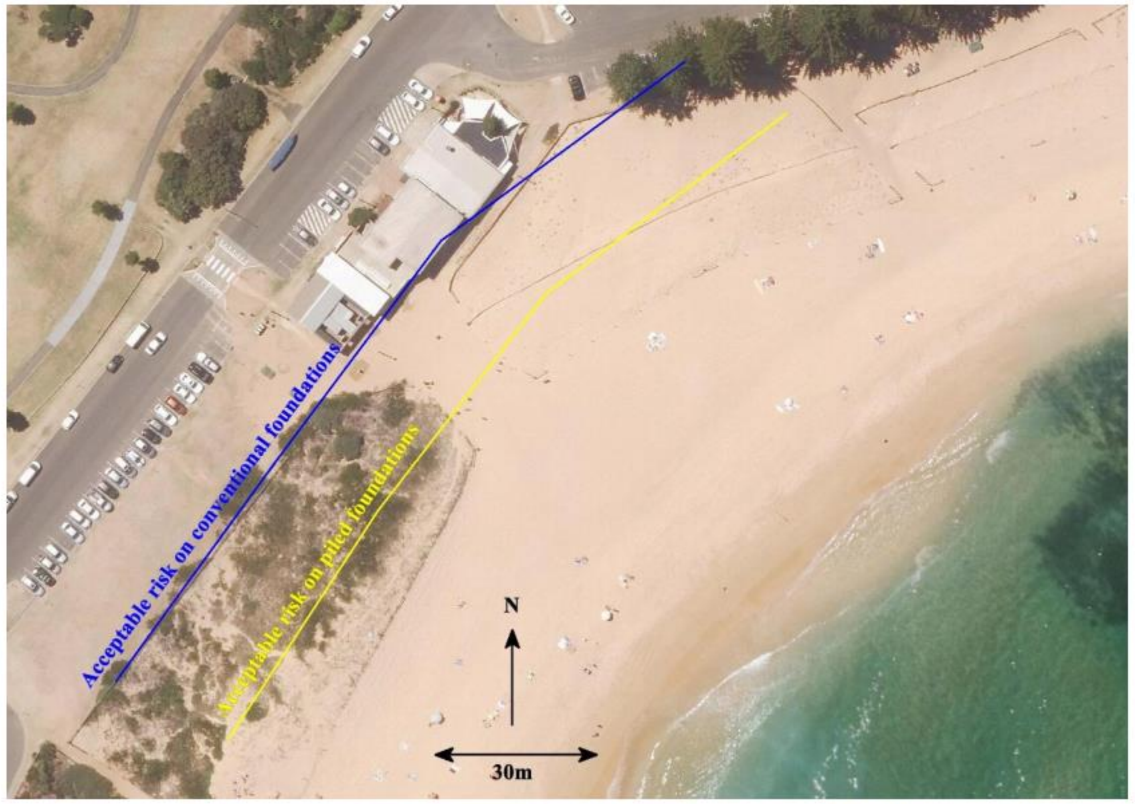


Figure ES1: Acceptable risk lines for new development for a 100-year design life (at 2117) at Mona Vale SLSC (aerial photograph taken 2014)

Figure 8: Coastal Erosion "Acceptable Risk" lines for foundations

4.3 Services

From a review of the available services drawings, it appears that the main services to the existing structure are located along Surfview Road. Existing services within the site for the existing club house appear to be documented. All existing services within the new structural footprint should be isolated and removed from footing locations

4.4 Ground Conditions

A preliminary geotechnical investigation report by JK Geotechnics provides advice on the geotechnical properties on proposed site. The report identified natural sands to an extent of 9.6m. The first 1.5m was assessed as loose, improving to medium dense below 1.5m.

An allowable bearing pressure of 250kPa was recommended for pad (at least 1m width) and strip footings (at least 0.5m width) founded within medium dense sand at a max. Depth of 1.5m.

Reinforced concrete piers have been advised as alternative and may provide a good option that would avoid shoring requirements of a strip footing. An allowable bearing pressure of 800kPa at a depth of at least 2.5m has been advised.

5.0 Project Works

5.1 Materials

Reinforced Concrete, Exposure Classification: B2 => f'c min = 40MPa; 7 day curing min and 25MPa at stripping; special class as per AS1379, min cover = 45mm off-form

Timber; connection details critical; Treatment and maintenance requirements.

Stainless Steel (SAF2205 Duplex) recommended for extreme exposure conditions.

Masonry

5.2 Foundations

The allowable bearing pressure of 250kPa at 1.5m depth advised for shallow strip and isolated pad footings is expected to be suitable for a structure of this type and size. To achieve the required depth in loose sand a shoring system such as trench boxes or a 1:2 temporary batter would be required. The shoring/batter requirement can be a source of additional costs to what would otherwise be an economic foundation option.

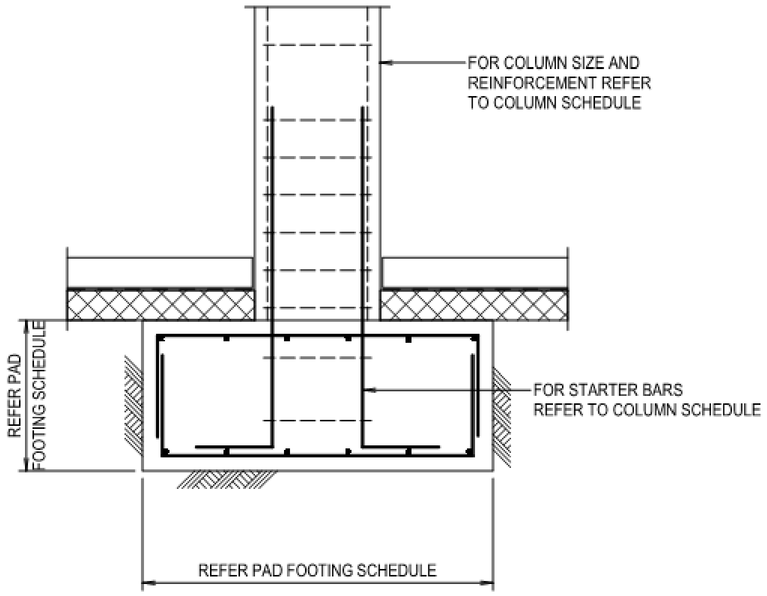


Figure 9: Typical Pad Footing

Reinforced concrete piers have been advised as an alternative. Bored piers need to be installed into medium dense sand and founded at least 2.5m below surface level. A detailed analysis of the loads applied to the structure would allow the spacing of piers to be varied in order to optimise the quantity of piers required by decreasing the spacing where loads are concentrated and increasing in lighter loaded areas. This option would require a capping beam near ground level to span between piers and support the structural walls and columns.

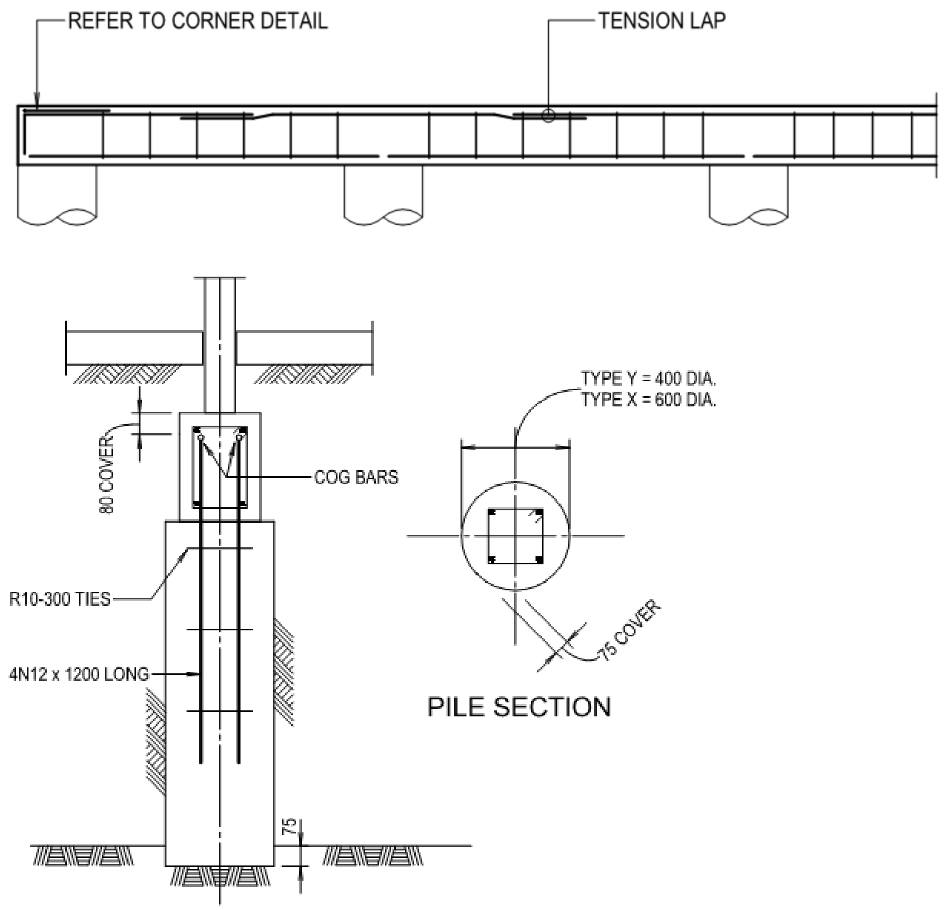


Figure 10: Typical Footing Piers

In order to provide a comparison of bored piers vs a strip footing a comparison of the allowable bearing capacities for a 600mm wide strip footing and a 600mm diameter pier is included below.

600mm dia. pier allowable load = 226kN

600mm wide strip footing allowable load = 150kN/m

Spacing of piers required for equal allowable load = 226/150 = 1.5m

Therefore, 600mm diameter piers at 1.5m spacing (centre to centre) are equal to a 600mm wide strip footing over the same length.

For the purpose of preliminary cost estimation, 600mm diameter piles at 1.5m c/c under all ground level walls is recommended. As the design progresses the locations and spacing of these piers can be refined and optimised based on the design loads.

5.3 Walls

Precast concrete walls can achieve a level of quality control on the finished surface of the lower level walls. This level of control reduces the risk of repair works being required due to insufficient cover to reinforcements and simplifies the construction sequence. Internal slabs may need to be complete prior to erection of precast walls to provide propping points for the panels until the level 1 floors are in place to stabilise the wall panels. Precast wall panels can be formed in a range of sizes and can include openings. if required. To gain the full benefit of precast walls a rationalised approach should be taken to standardise the size of panels as much as possible to allow the same form to be used multiple times.

Figure 11: Precast Wall Panels Examples

140 reinforced masonry is recommended as the typical wall type in the ground level. Masonry is a robust material non-corrosive material capable of withstanding the exposed coastal conditions. The surface can be painted or rendered to the required finish. Cover to reinforcement is provided by a combination of the masonry block edge thickness and the concrete infill. Additionally, a 140 masonry wall provides a suitable support structure for the precast hollowcore panels.

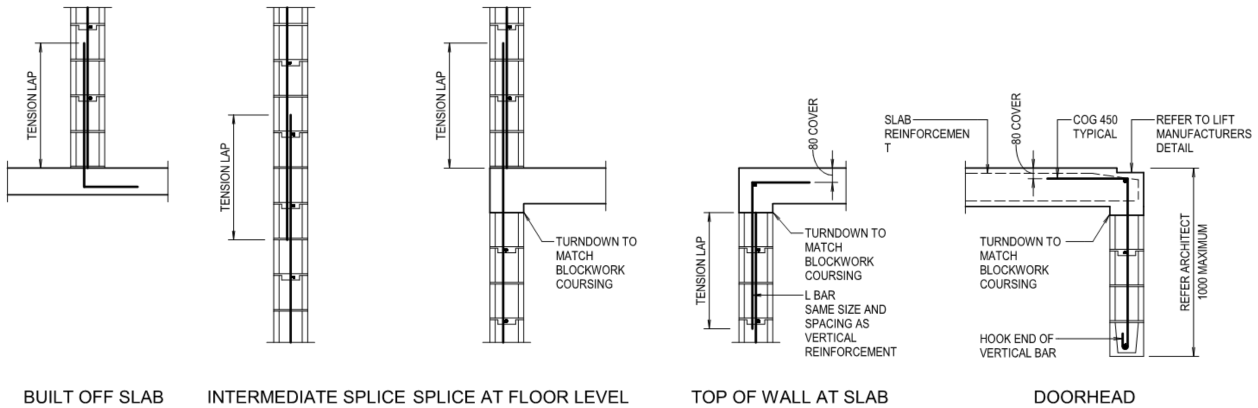


Figure 12: Typical Reinforced Blockwork Details

150 timber stud walls are proposed in the upper level to support the roof structure and provide lateral stability to the upper level of the building. The intent is to keep the upper section as light as possible to reduce the transfer loads carried by the Level 1 slabs. Studs can be closer spaced if required to support point loads from primary roof beams.

5.4 Suspended Slab

5.4.1 Pre-Cast Hollowcore Floor Panels

In the north and south sections of the building a precast slab solution may be an effective solution. Based on the spans required in these sections a 205 precast hollowcore slab would be suitable. Hollowcore slabs include an average depth of 60mm structural screed to tie the slab diaphragm together and to level the slab resulting in a total structural depth of 265mm. The panels are craned into position and provide an immediately available platform to begin construction of the upper storey. The main advantage of this product is the savings in time versus the time required to form the slab soffit in off form concrete solutions.

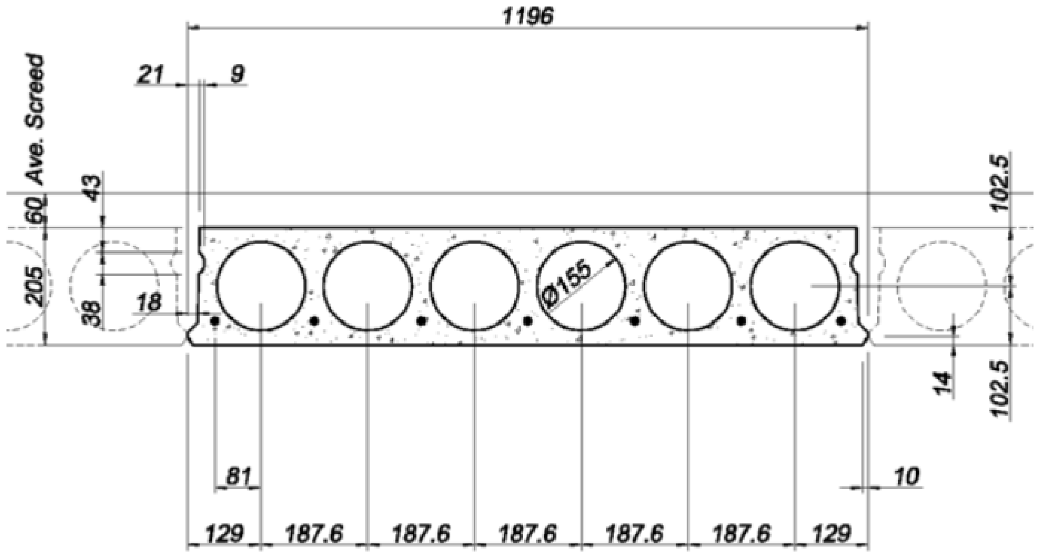


Figure 13: 205 Hollowcore Slab Panel Section

Pre-cast concrete offers a superior finish to off form concrete due to the quality of concrete used for early stressing of panels and the quality control that can be achieved in a factory setting. Strand cover can be specified to achieve the required cover for coastal exposure condition B2 accordance with AS3600. At panel ends a reinforced concrete edge beam is typically cast to provide cover to exposed tendons on the precast panels.

To provide the cantilevered portion of the southern section, continuous top reinforcement can be provided within the structural topping. To achieve this the cantilevered section must be propped at the external edge until the topping achieves sufficient strength engage the precast section.

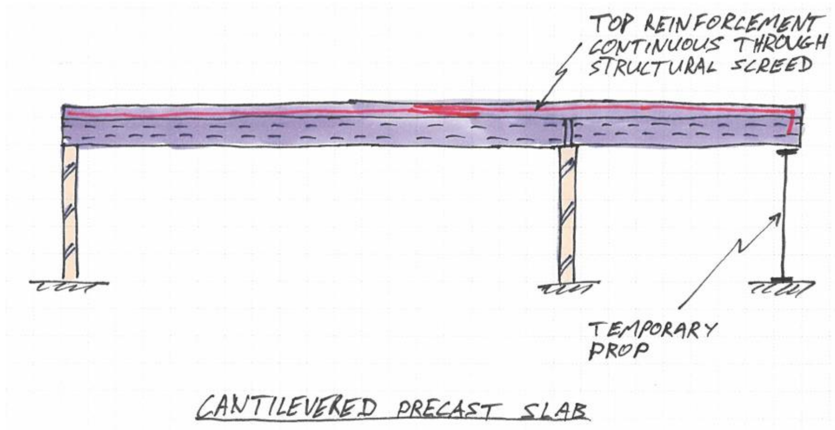


Figure 14: Cantilevered Hollowcore Slab

Service openings can be included in hollowcore slabs. Small openings can be readily provided however large opening require trimmer beams to effectively remove a section of the slab panel and transfer this load to the adjacent panels. Figure 15 below illustrate typical opening arrangements for hollowcore slabs.

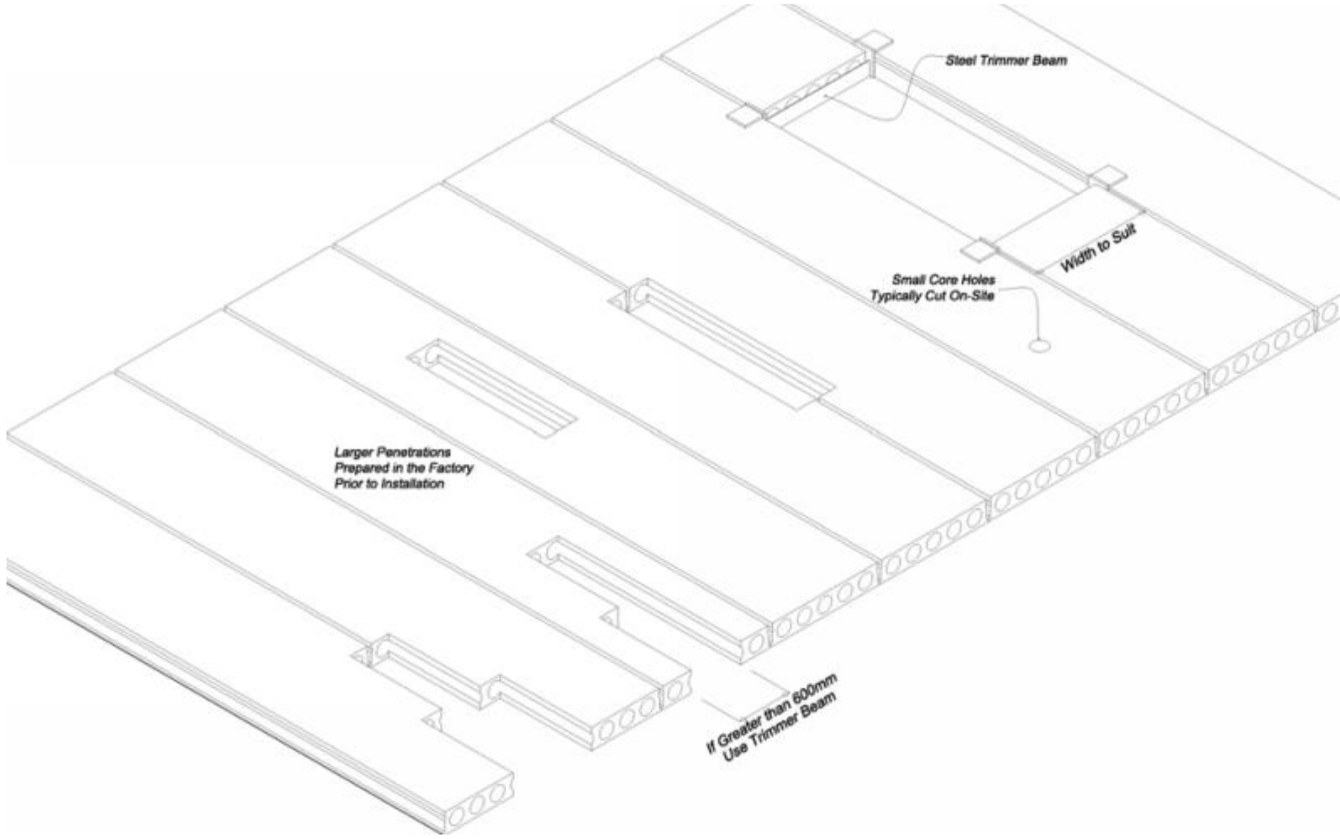


Figure 15: Typical Penetration Arrangements in Hollowcore Slabs

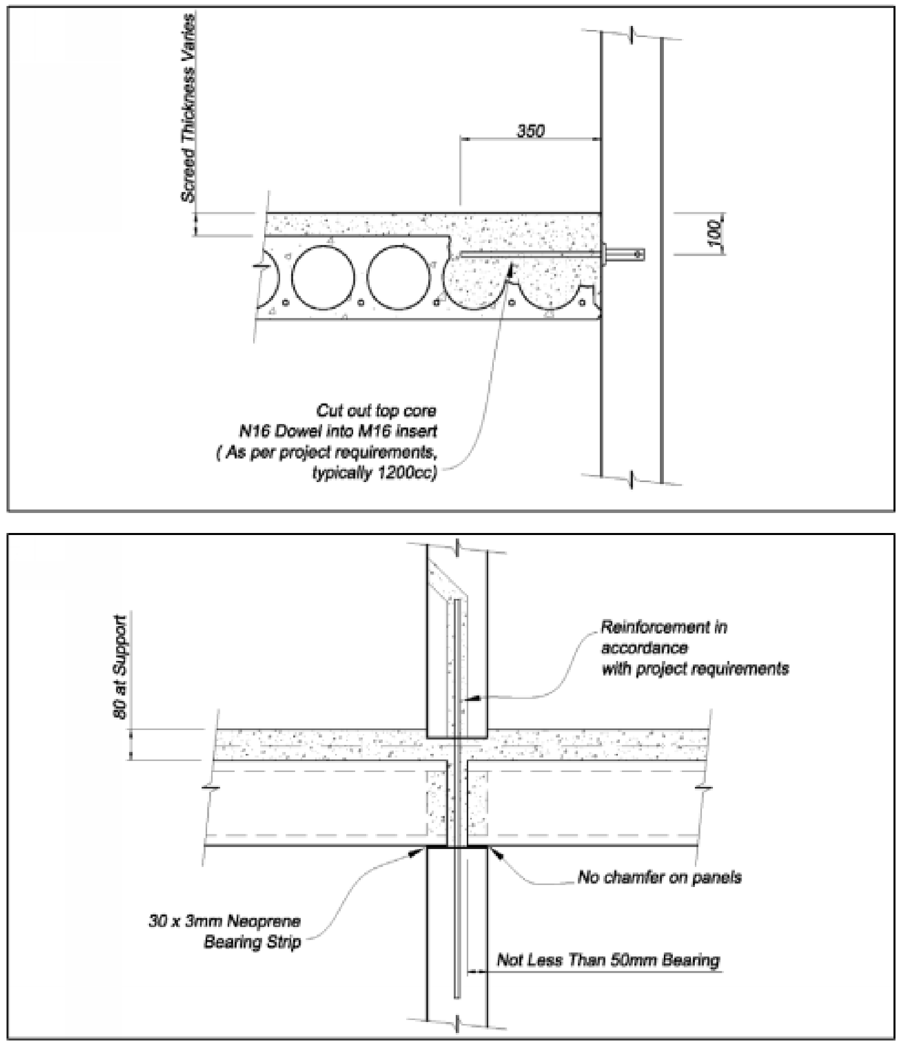


Figure 16: Hollowcore Floor to Wall Typical Details (Hollowcore Concrete Pty Ltd)

5.4.2 Post-tensioned (PT) banded slab

A PT banded slab has been considered for the central section where large spans are required to provide a clear floor plan for the surf life saving and surf sports store. To provide support to the level 1 function room balcony and transfer loads from the roof through the level 1 façade line, a transfer cantilever is required. This cantilever is currently up to 5.3m long. To achieve this a 300mm thick slab is required with 450x2400mm band beams running North-South. See Figure 17 below for details.

A P/T banded slab requires a profiled formwork support to cast the slab soffit. This form of construction is common in the Sydney region and an efficient use of the structural thickness at the expense of additional formworks costs. P/T banded slabs are historically 3-4% cheaper than flat plate construction.

To reduce the risk of corrosion of the P/T anchor points it is proposed that the live end of the tendons is located on the landward side of the slab. The risk of exposure can be further reduced by the use of anchor pockets in the top of the slab. At the dead end of the strand, the tendon is fully cast into the slab and the level of cover can be controlled.

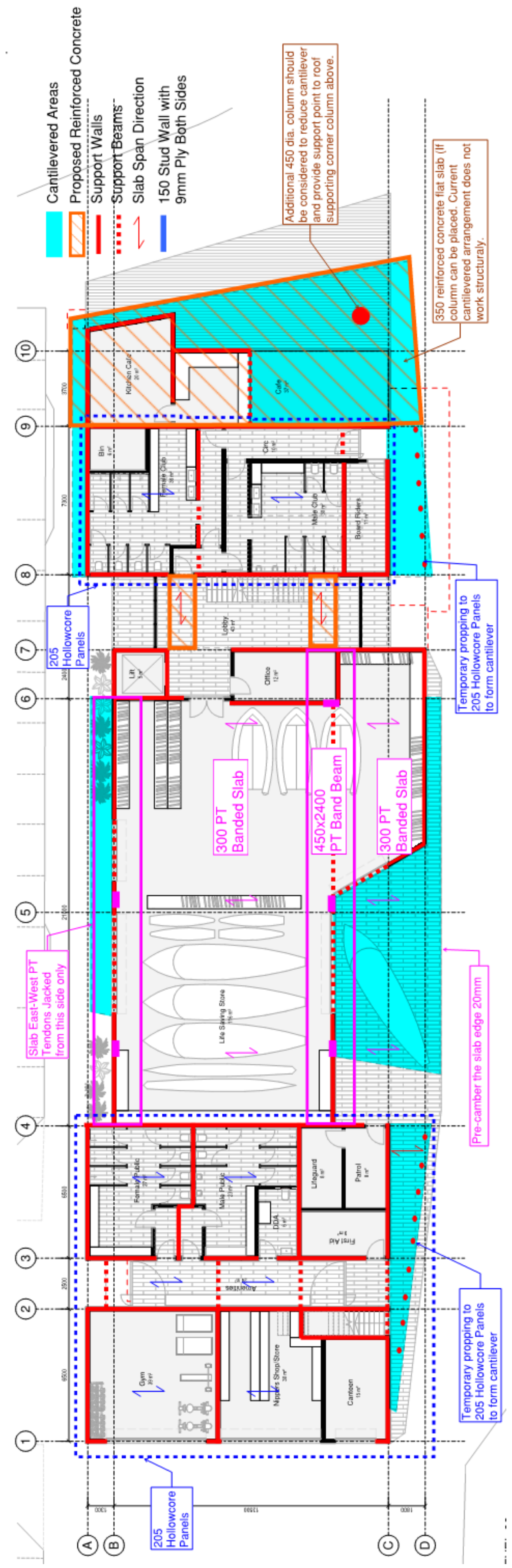
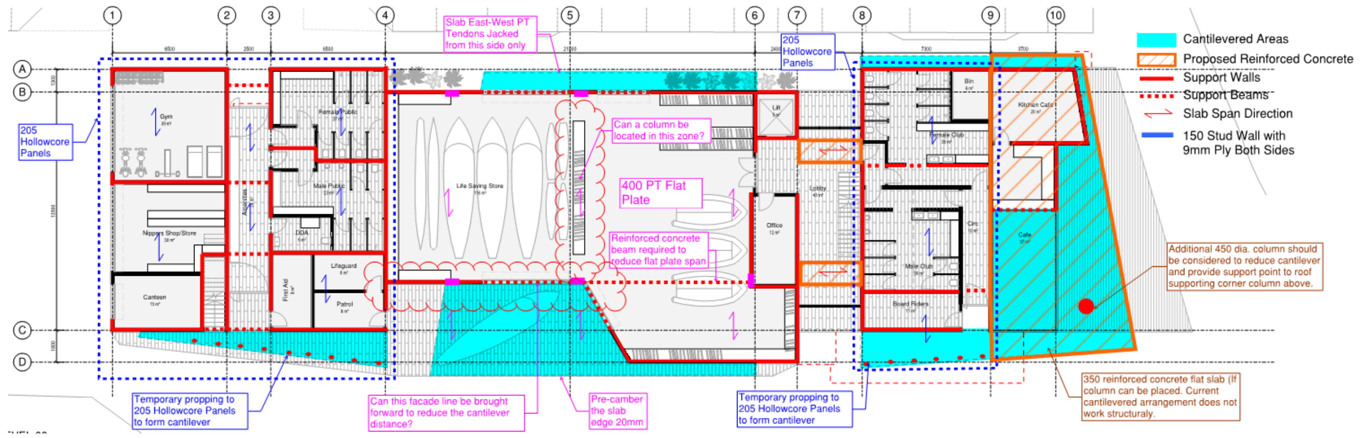


Figure 17: Level 1 P/T Banded Slab with 205 Pre-casts Hollowcore Panels

5.4.3 P/T Flat Plate

A P/T flat plate is a flat soffited P/T slab. A flat soffit can be useful for running services unobstructed in all directions under the slab. A 400mm thick flat plate would be required in order to achieve the span and cantilevers in the central section of the SLSC. A Reinforced concrete beam is required in the North South direction to reduce the total span in the northern half of the central section. See **Error! Reference source not found. Error! Reference source not found.** for details



5.4.4 Summary of Slab Options

Construction Type	Pros	Cons
Banded PT Slab	Most efficient form of PT construction Most flexibility with regards to future penetrations Minimises structure self-weight reducing foundation loads and seismic lateral loads	More complicated formwork required Deeper overall construction depth Additional coordination of services required Live ends require special attention due to coastal exposure. Can be improved by using pans at live end.
PT Flat Plate	Simple formwork Simple zone in which to run services Distributed tendons can simplify detailing	Typically 2-5% more expensive than banded slabs Less flexibility for future penetrations Increased structure self-weight Live ends require special attention due to coastal exposure. Can be improved by using pans at live end
Reinforced Concrete Beams and Precast Hollowcore Slab	Increased off site manufacture improves WHS and quality control Fastest speed of erection for structural hollowcore panels Reduced/eliminated secondary members	Coordination of services very important Heavier crane requirements Can be difficult to achieve diaphragm action

Table 1 - Floor Slab Options Pros/Cons

One of the most challenging areas to achieve structurally is the northern cantilevered section over the café. Figure 18 below highlights the current arrangement. There is a glazing line for the café that could potentially be used however this still represents a cantilever of over 4m in 2 directions and will be difficult to achieve with the desired flat soffit. A column has been indicatively indicated in Figure 17 above to align with the corner of the façade above and reduce the cantilevered section. If it is possible to locate a column in this position it may be possible to achieve a reinforced concrete flat slab 350mm thick with a 450 diameter round column. This column could be an architectural flared column.

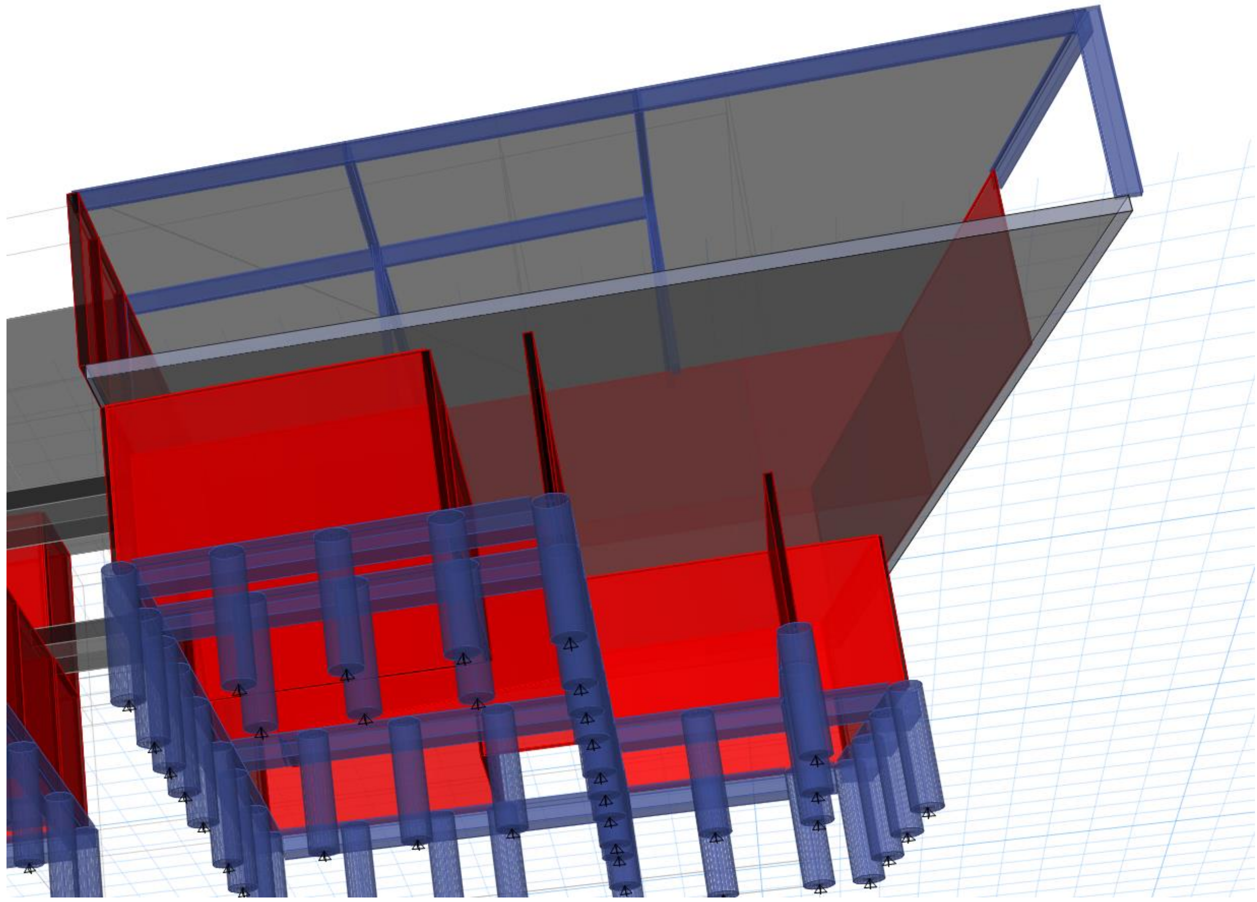


Figure 18: Northern Cantilevered Slab in Cafe

5.5 Roof

3 roof options have been explored that could deliver the clear spans and cantilevers required in the upper level. To for the front face of the portals the roof cantilevers from the façade line to the front face of the portals.

A sealed membrane surface is recommended to avoid the traditional fixings associated with standard roof sheeting. The primary reason for this is to avoid any potential corrosion of the fixings and the panels themselves. In order to apply a surface membrane a solid surface is required for application of adhesives.

Note the below options have been assessed on the basis of a non-trafficable roof and standard structural deflection criteria. A detailed analysis of the vibration effects has not been undertaken. Consideration should be given to the pitch of the roof to prevent ponding as structural deflections will compound this issue.

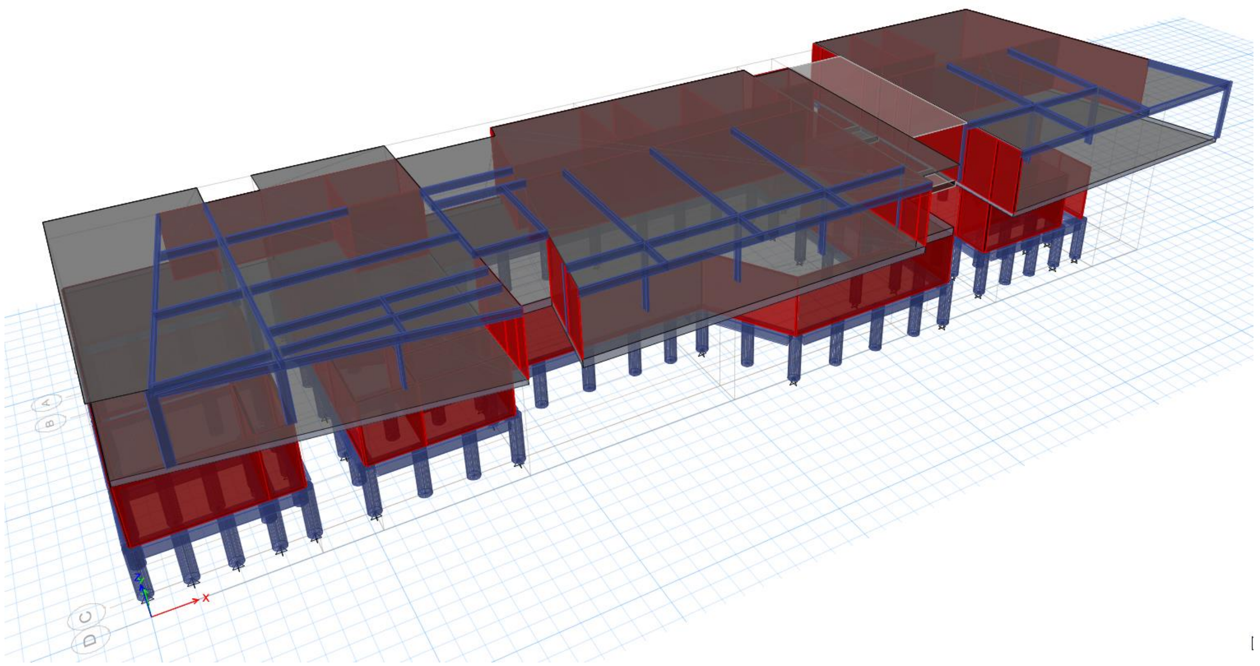


Figure 19: Structural Analysis Model

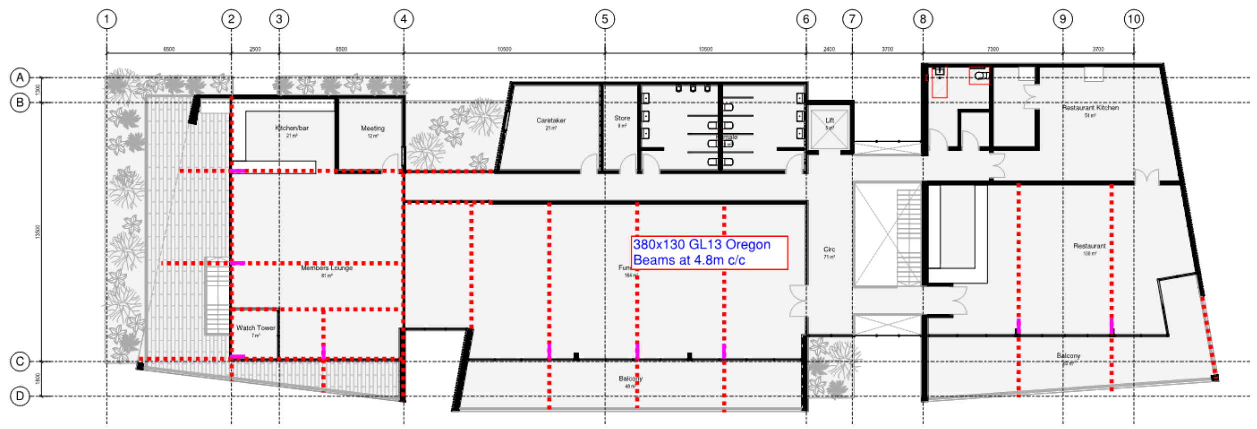


Figure 20: Primary Beam Framing

5.5.1 Option 1 - Glulam beams with hySpan secondary beams and plywood

This option is likely to be the lightest option but is also expected to be the slowest to install. 380x130 GL13 glulam beams at 4.8m c/c are used to span the primary direction and support the cantilevers of the portals. The primary beams are supported on 380x130 GL13 glulam columns which are spaced to align with at 4.8m c/c to fit with standard glazing dimensions. It is therefore the glazing dimensions that are controlling the spacing of the glulam columns and beams. To achieve the desired facia dimension the glulam beams can be tapered from the façade line as outlined in Figure 21.

The secondary direction is spanned by 200x45 hySpan beams at 1.2m c/c and sealed with 25mm sheets of external plywood to form a uniform surface for membrane application.

The total structural depth of this option is 405mm. Service penetrations can be provided through glulam beams however their size and location will need to be assessed.

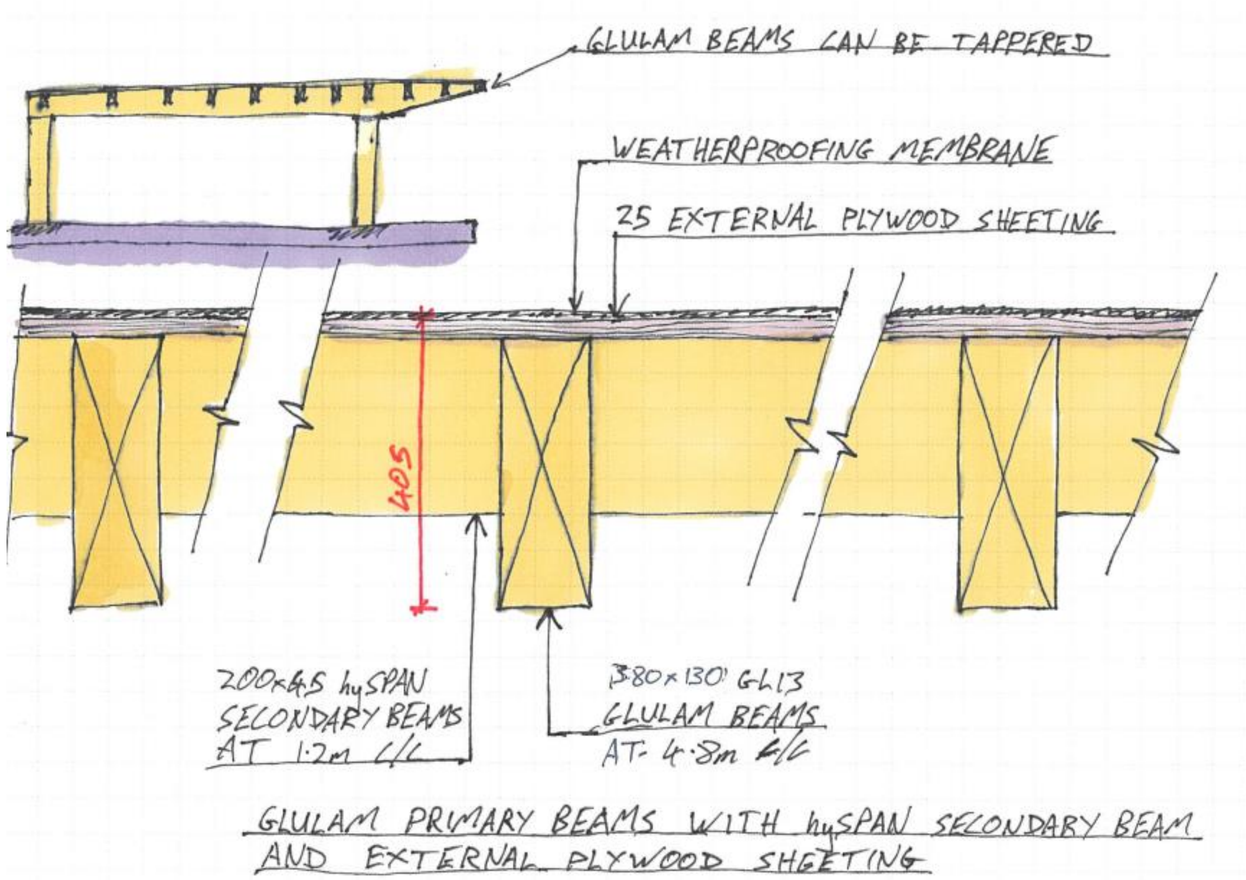


Figure 21: Option 1 - Glulam beams with hySpan Secondary Beams

5.5.2 Option 2 - Glulam beams with CLT in the Secondary Direction

Similar to option 1 with the primary span and cantilevers for the portals achieved by 380x130 GL13 glulam beams. 130 CLT panels are used to span the secondary direction and provide the surface for membrane application eliminating the installation of secondary beams and additional plywood.

The CLT panels are craned into position and follow a similar logic to the hollow core slabs on level 1. Installation is quick, and a sealed upper level can allow subsequent trades to get underway internally. It should also be noted that CLT panels are limited to 12m lengths to fit a standard shipping container and lead times for supply should be investigated.

A total structural depth of 510mm is required for this option. Again, service openings can be provided through the glulam beams but will require further assessment for the size and location of these openings.

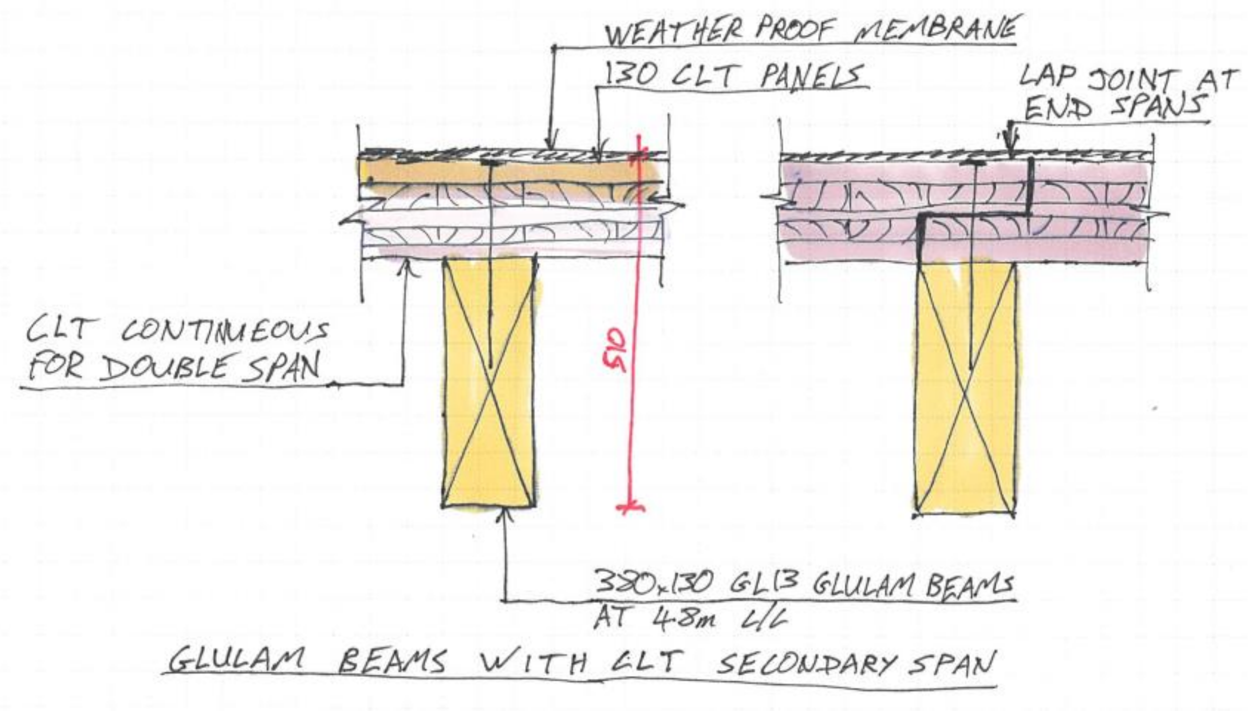


Figure 22: Glulam beams with CLT secondary spans

5.5.3 Option 3 - Solid CLT Roof

A solid CLT option was explored for the potential benefit of achieving a flat roof soffit and the cantilevers for the portals with a single thickness of structure throughout the roof. The total structural depth of this option is 300mm. However, vibration can become a concern for long span CLT panels and this will need to be assessed in more detail if this option is pursued.

The support beam and columns along the façade line may require larger sections than indicated in Figure 23 below due to the increased mass of the roof. There will also be larger point loads applied to the level 1 transfer slabs that will require further assessment. It should also be noted that CLT panels are limited to 12m lengths to fit a standard shipping container and lead times for supply should be investigated.

The stability of this option is reduced by the removal of the glulam portal frames and consideration is needed to ensure the roof structure is sufficiently stabilised by vertical wall elements. This is particularly important along the eastern façade line where North-South stability elements are limited.

Advantages of this option include:

- Easy provision of services to underside of soffit.
- Minimal structural depth.
- Continuous flat soffit in line with the architectural Intent.
- Speed of construction.

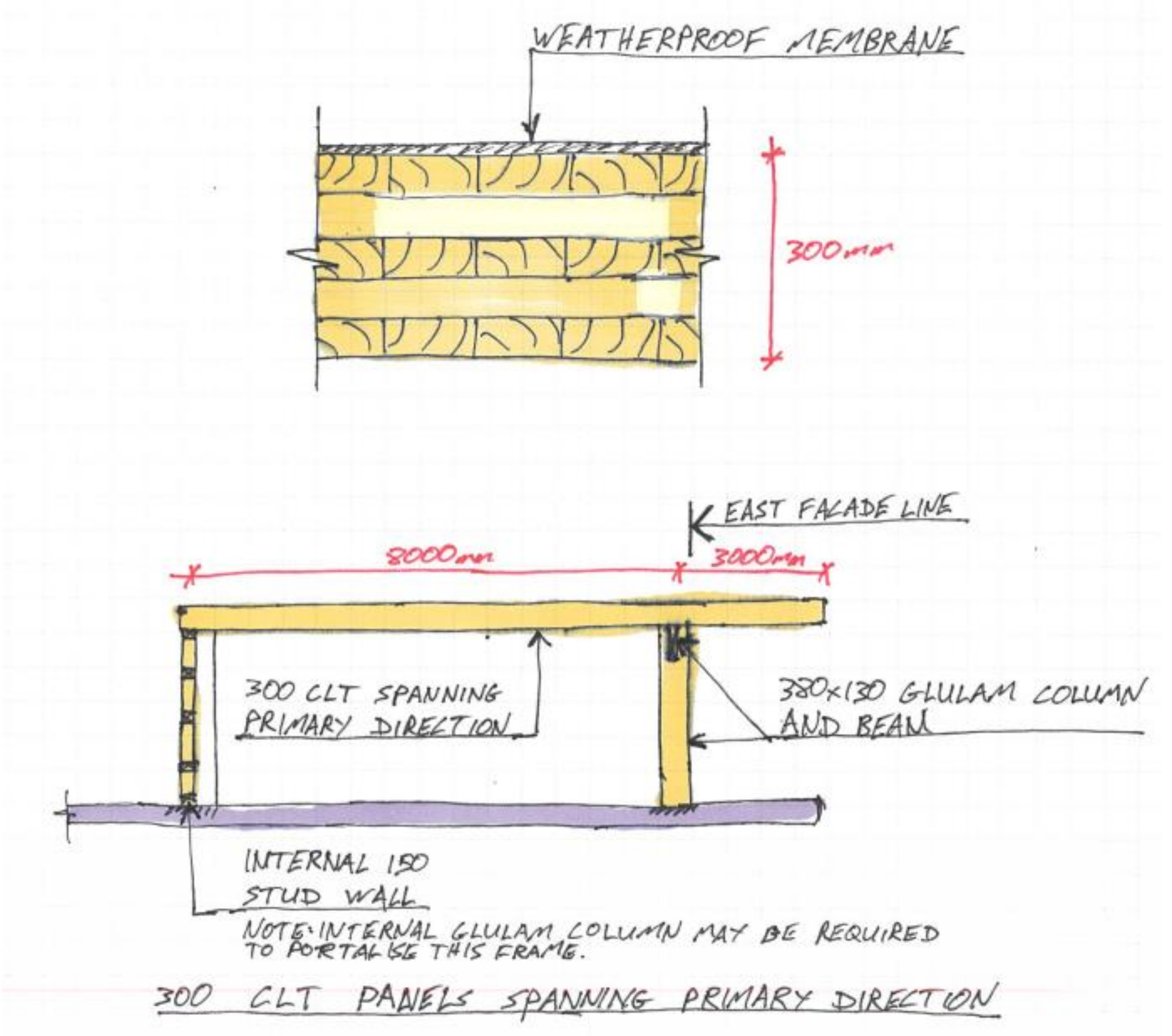


Figure 23: CLT Roof with Glulam Columns and Facade Line Support Beam

5.6 Expressed 1st Floor "Portals"

Some examples of glulam structural beams and columns are outlined below to aid in conceptualising the outlined roof framing options.

Glulam columns are proposed within the glazing line and will be required to support the roof joists and frame the façade glazing. The columns should be located to the inside face of the glazing. This will protect the columns and the stainless steel fittings that will be required at connection points. The connections can be expressed, as illustrated in Figure 24 below, or alternatively recessed into the columns and plugged.

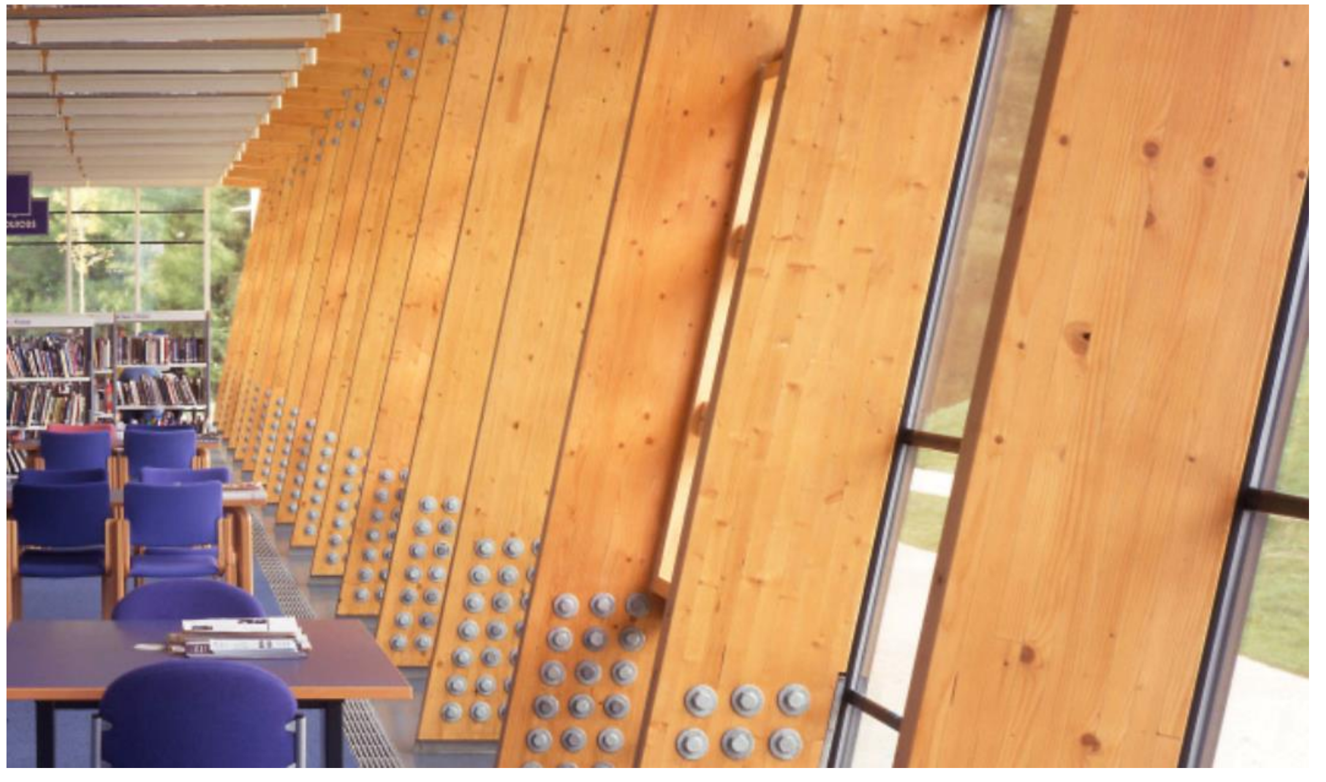


Figure 24: Glulam facade column example (March Library, Cambridgeshire, UK_Bernard Stilwell Architects)

Figure 25 illustrates a residential project that utilised glulam columns and beams to achieve a glazed façade and long span roof. The image demonstrates the capability of glulam products to achieve spans of a similar nature to the Mona Vale SLSC.

Figure 25: Hannah Residence (British Columbia, Canada, Halliwell and Smith Blue Sky Architects)

5.7 Feature Stairs

Based on the supplied architectural drawings each stair tread is estimated as 1250mm with a 245mm going. In order to support the threads a 200mm off-form reinforced concrete wall is proposed adjacent to the stairs from which the individual treads will cantilever.

The primary concern with the stair is controlling the vibration of the cantilevered threads. This will require detailed analysis to determine the final details for the project. In order to provide an initial estimate for a reinforced concrete section that could satisfy the vibration requirements a natural frequency analysis was conducted.

The result indicated that a 75mm thick tread had a natural frequency of 28Hz and would deflect 3.2mm under a 1.2kN point load at the tip of the cantilever. This would be within acceptable limits and indicates a relatively rigid thread and is a good start point to begin discussions on how the stairs are to be achieved.

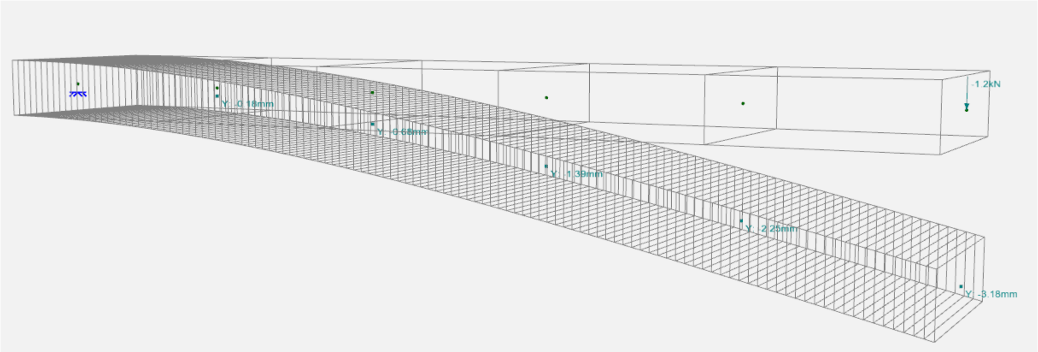


Figure 26: Tread displacement (Not to Scale)

In order to achieve a high quality finish, a precast concrete tread is recommended. The anchorage details to the supporting wall then become the focus of the engineering design and will determine to an extent the required wall thickness. Based on initial estimates and the cover requirements for a B2 exposure classification, a 200mm thick 40MPa reinforced concrete wall will be required.

Figure 27: Examples of Concrete Cantilevered Treads

There are slender cantilevered profiles available; however, these are often achieved by a steel structural member clad in a concrete/stone finish. See

Figure 28: Example of Steel Cantilevered Tread with Concrete Finish

6.0 Structural Design Principles

6.1 Design Criteria

6.1.1 Wind Loading

Wind loading at this stage has been calculated from AS1170.2 – 2011 using the parameters listed in Table 2. Given the nature of mass within the building, it is likely that seismic effects will govern at ULS and that serviceability effects due to wind are manageable with the proposed stability structure, as such wind tunnel testing is not expected to be required.

Item	Value
Building Importance Level	III (Major structures affecting crowds)
Terrain Category	1.5
Region	A2
Annual Probability of Exceedance	1:1000
Regional Wind Speed (ULS)	V_R=46m/s
Regional Wind Speed (SLS)	V_R=37m/s

Table 2 - Wind Loading Criteria

6.1.2 Earthquake Loading

Earthquake loadings shall be in accordance with AS1170.4 – 2007 and AS/NZS1170.0 – 2007.

Item	Value
Importance Level (BCA Table B1.2a):	III
Annual Probability of Exceedance	1:1000
Probability Factor (k_p):	1.3
Hazard Factor (Z):	0.08
Site sub-soil Class:	C_e
Earthquake Design Category:	II

Table 3 - Seismic Loading Criteria

The basic combinations for the ultimate limit states used in checking strength are as follows. They are based upon AS1170.0 section 4.

LOAD COMBINATION	G	Q	W_u	E_u	C
1	1.2	1.5			
2	1.2	Ψ_c	1.0		
3	1.0	Ψ_c		1.0	
4	1.2	1.5Ψ_l			
5	1.35				
6	0.9		1.0 up		

Table 4 - ULS Load Combination Factors

LOAD COMBINATION	G	Q	W_s	E_s
7	1.0			
8		Ψ_s		
9		Ψ_l		
10			1.0	
11				1.0

Table 5 - SLS Load Combination Factors

G : structure self-weight plus superimposed dead loads
Q : imposed action
W_u : ultimate wind action
W_s : serviceability wind action
E_u : ultimate earthquake action
E_s : serviceability earthquake action
Ψ_c : combination factor for imposed action
Ψ_s : short-term factor
Ψ_l : long-term factor

6.1.3 Fire Resistance

The deemed to satisfy resistance periods for *structural adequacy* can be found in Table 6.

Classification	Use	FRL (Structural Adequacy)
5	Commercial	120 minutes
6	Shops or buildings for the sale of goods by retail or the supply of services direct to the public	180 minutes
9b	Building of a public nature – an assembly building	120 minutes

Table 6 - Deemed to Satisfy FRLs for Load Bearing Elements

6.2 Performance Criteria

AS1170:0 outlines suggested deflection limits as outlined below.

Case	Deflection Limit
Total Lateral Displacement of Building	Height/500
Maximum Inter-Storey Drift per Floor	Storey Height/500

Table 7 - Lateral Displacement Performance Criteria

6.3 Structural Design Codes and Standards

All structural elements will be designed in accordance with the relevant structural standards referenced in the National Construction Code 2016.

In particular, the structural design will comply with the following Australian Standards:

- AS1170 [All Parts] Structural Design Actions
- AS3600 (2009) Concrete Structures
- AS4100 (1998) Steel Structures
- AS3700 (2011) Masonry Structures
- AS2159 (2009) Piling Design and Installation
- AS4678 (2002) Earth-retaining Structures
- AS1720.1 (2010) Timber Structures

7.0 Precast Vs Traditional Construction Techniques

Throughout this report reference is made to both precast and traditional construction techniques. Both offer advantages and disadvantages. To keep the upper level lightweight and achieve the architectural form of the structure, timber construction is a logical solution. Therefore, the focus of this chapter will be the ground floor and first floor level wall and floor systems.

7.1 Walls

Traditional reinforced masonry construction is one such traditional construction technique.

Advantages include:

- Well known construction technique in the market.
- Tenders can be competitive (depending on market conditions).
- Reinforcement cover is readily achieved by block wall thickness for corrosion protection.
- Blocks work as formwork for concrete infill.
- Materials can be easily moved by hand.
- Materials typically available locally and can be ordered and delivered as required.

Disadvantages include:

- Slower construction relative to precast.
- Concrete curing time required before floor loads can be applied.
- Quality dependent on workmanship.
- Finishes must be applied to constructed walls.

Precast tilt up wall panels of varying configurations provide an alternative to traditional walls.

Advantages include:

- Quick to erect.
- Can be immediately loaded (no curing time required).
- Quality controlled through factory fabrication.
- Can be manufactured with required openings.
- Can provide as a finished surface.

Disadvantages include:

- Crainage required for panel placement.
- Temporary restraint required (Additional WHS risk).
- Procurement times can impact construction programme.
- Errors are difficult to remedy and may cause delays if new panels are required.

7.2 Floors

Post-tensioned concrete and precast hollow core slabs are explored in this report. It is important to note that to maximise the cost benefit of either system, one system should be used throughout the floor plate. The main reasons for this are that only one trade is mobilised, reducing establishment and coordination costs and with increasing volume of work, typically better rates can be achieved.

Post-tensioned advantages include:

- Competitive D&C market.
- Large spans and relatively thin floor plates can be achieved.
- Cantilevered sections can be readily catered for in the design.

Post-tensioned disadvantages include:

- Formwork propping required to soffit of slab until curing is complete.
- Limited access for internal works until formwork is removed.
- Future penetrations require engineering assessment and advice.

Precast hollowcore slabs advantages:

- Quick to erect.
- Eliminates formwork propping allowing early access for fit-out.
- Quality controlled through factory fabrication.
- Standard details available to accommodate openings.

Precast hollowcore slabs disadvantages:

- Crainage required for erection.
- Future large penetrations can be difficult to incorporate.
- Larger down stand beams required to achieve panel spans.
- Large cantilevered sections can be difficult to achieve.
- Procurement times can impact construction programme.

A high-level matrix comparison of the outlined methods is illustrated below:

Method	Cost	Procurement Time	Erection Time	Maintenance
Walls				
Reinforced Masonry Walls	QS to advise			
Precast Tilt-Up Panels	QS to advise			
Floors				
Post-Tensioned Slab	QS to advise			
Precast Hollowcore	QS to advise			

It is important to note that to gain the maximum cost benefit from the chosen technique only one technique should be pursued. Cost associated with precast construction, such as crainage requirements, would not typically be required for post tensioned slab construction but if a combination of these systems are used cost efficiencies are lost in the mobilisation of each system, coordination of trades and inefficiencies in pricing due to the volume of each trade being reduced.

If pre-cast floors are to be pursued for the central storage section on ground level, it may become difficult to achieve the cantilevered transfer structure above the wash down bay. It will also impact on the head height in this area as a larger down stand beam would be required to support precast panels. The impacts of the final technique can be explored through design development stage of the project.