1.0 Project Feasibility

Before more detailed steps towards this project can take place, as an initial feasibility requirement of the project, cost estimates were prepared to ensure the client was satisfied to proceed with the project, given they would have an initial budget in mind stay within. An approximate estimate and a preliminary estimate have been prepared here.

1.1 Approximate Estimate

The approximate estimate gives the initial idea of the cost estimate. The approximate estimate has been based off a project in Adelaide, South Australia, called 'The Square,' which is a 4-storey modular complex, fully finished as of 2022 (inclusive of surrounding retail and other buildings), with a specific site area of $1390m^2$. Within the costing report of 2016, it was found that the per square metre cost of this specific project, at that time was $1700/sqm. This price has been adjusted to May 2023 prices according to Rawlinson's Construction Cost Guide. It is assumed that the price will be adjusted further by 20% (not included in Rawlinson's) to account for price differences between Perth and Rottnest Island.

Description	Raw Cost of Project from 2016 Report	Price Adjustment From 2016 to 31 Dec 2022 in Adelaide	Price Adjustment From Adelaide to Perth, 31 December	Price Adjustment From 31 Dec 2022 to 31 May in Perth	Price Adjustment From Perth to Rottnest, 31 December
Price/m^2	$1,700	$2,119	$2,107	$2,177	$2,541
Factor Adjustments	Nil	$\frac{131.31}{105.35}$	$\frac{130.58}{131.31}$	$\frac{103.33}{100}$	$\frac{120}{100}$

A further estimate was taken from Rawlinson's Construction Cost Guide. The Rottnest Island Development project is to have a dual function:

- The first two floors will serve as a Bike Shop, and
- The upper three floors will serve as residential accommodation.

Prices for both have been determined below.

Description 13.0. Residential 13.2. MULTI UNIT - LOW DENSITY 13.2.2. APARTMENTS - maximum three stories, one- or two-bedroom units, excluding balconies, no lift 13.2.2.2. Medium standard finish	Raw Cost of Project from Rawlinson's Construction Cost Guide Clause 13.2.2.2, 31 Dec 2022 Perth	Price Adjustment From 31 Dec 2022 to 31 May in Perth	Price Adjustment From Perth to Rottnest, 31 December
Price/m^2	$2,235	$2,310	$2,771
Factor Adjustments	Nil	$\frac{103.33}{100}$	$\frac{120}{100}$

Description 14.0 RETAIL 14.1 Suburban 14.1.1. NEIGHBOURHOOD SHOPS - standard shell construction including shopfronts, plasterboard ceilings, electrical service to board, cold water supply to fixture point only and drainage. No fittings, hot water, air-conditioning, sprinklers, or malls: 14.1.1.2. Two-Storey	Raw Cost of Project from Rawlinson's Construction Cost Guide Clause 14.1.1.2, 31 Dec 2022 Perth	Price Adjustment From 31 Dec 2022 to 31 May in Perth	Price Adjustment From Perth to Rottnest, 31 December
Price/m^2	$1,100	$1,137	$1364
Factor Adjustments	Nil	$\frac{103.33}{100}$	$\frac{120}{100}$

The figures determined for the approximate costs for both the retail and residential multi-unit complex have been added together, as this is representative of the structure to be constructed on the Rottnest Site. The overall approximate cost of these two building prices together is: $4135.

Following on from this, the two figures attained will be averaged against each other. The price figure determined by the Rawlinson Construction Cost Guide has been deemed to hold greater value as it is more indicative of the full project to occur. Therefore, the Rawlinson figure holds a weighting of twice that of the 'Square' project.

The cost outcome of this project from weighing the average has come to \$3604/m^2

The site area where the structure lies is a 46*20 = 920m^2.

The Approximate Estimate therefore comes in as 3604*920 = $3,315,680

1.2 Preliminary Cost

A further, more detailed estimate can be derived through performing a preliminary estimate. This estimate gives a breakdown of the costing for the components within the project, and which areas attribute to the greatest proportion of costs. In performing this cost estimate, the following criteria were considered:

- The Estimate was undertaken following a review of the approximate estimate.
- The cost breakdowns were derived from the 2023 Rawlinson's Cost Guide, for multi-unit apartments and retail buildings.
- The price was adjusted from prices in Sydney, to prices at Rottnest Island, by converting to Perth prices and adjusting by 20% for Rottnest Island.
- The rates for both the retail and unit apartment buildings have excluded elevator services and so have been added to the preliminary cost.

Elemental Weighting	Sub-Elemental Weighting	Description	Unit	Quantity	Rate 3-Storey Building (Sydney)	Rate 2-Storey Retail Shop (Sydney)	Rate (Rottnest Island)	Total	Reference
Preliminaries									
8.99%	8.99%	Preliminaries	sqm	920	$274.25	$97.50	$406.30	$373,799.25	Rawlinsons 2023, Pg 40 & 42
Substructure									
4.72%	4.72%	Substructure	sqm	920	$112.25	$83.00	$213.40	$196,326.31	Rawlinsons 2023, Pg 40 & 42
Superstructure									
41.37%	0.89%	Columns	sqm	920	$0.00	$36.75	$40.17	$36,952.58	Rawlinsons 2023, Pg 40 & 42
	10.95%	Upper Floors	sqm	920	$263.25	$189.75	$495.11	$455,497.14	Rawlinsons 2023, Pg 40 & 42
	1.57%	Staircase	sqm	920	$50.00	$15.00	$71.04	$65,358.31	Rawlinsons 2023, Pg 40 & 42
	6.12%	Roof	sqm	920	$131.50	$121.50	$276.52	$254,394.65	Rawlinsons 2023, Pg 40 & 42
	9.57%	External Walls	sqm	920	$396.00	$0.00	$432.81	$398,182.93	Rawlinsons 2023, Pg 40 & 42
	0.91%	External Doors	sqm	920	$37.75	$0.00	$41.26	$37,958.09	Rawlinsons 2023, Pg 40 & 42
	5.02%	Windows	sqm	920	$0.00	$207.50	$226.79	$208,643.83	Rawlinsons 2023, Pg 40 & 42
	4.92%	Internal Walls	sqm	920	$150.25	$53.25	$222.41	$204,621.78	Rawlinsons 2023, Pg 40 & 42

	0.27%	Internal Screens	sqm	920	$11.00	$0.00	$12.02	$11,060.64	Rawlinsons 2023, Pg 40 & 42
	1.16%	Internal Doors	sqm	920	$41.25	$6.75	$52.46	$48,264.60	Rawlinsons 2023, Pg 40 & 42
Finishes									
8.91%	2.29%	Wall	sqm	920	$84.50	$10.25	$103.56	$95,272.30	Rawlinsons 2023, Pg 40 & 42
	2.75%	Floor	sqm	920	$94.00	$19.75	$124.32	$114,377.04	Rawlinsons 2023, Pg 40 & 42
	3.87%	Ceiling	sqm	920	$87.00	$73.00	$174.87	$160,881.99	Rawlinsons 2023, Pg 40 & 42
Fitments									
3.84%	3.84%	Fitments	sqm	920	$159.00	$0.00	$173.78	$159,876.48	Rawlinsons 2023, Pg 40 & 42
Services									
29.49%	10.55%	Plumbing	sqm	920	$374.25	$62.35	$477.18	$439,006.74	Rawlinsons 2023, Pg 40 & 42
	1.21%	Mechanical	sqm	920	$50.25	$0.00	$54.92	$50,527.00	Rawlinsons 2023, Pg 40 & 42
	0.42%	Fire	sqm	920	$11.25	$6.00	$18.85	$17,345.09	Rawlinsons 2023, Pg 40 & 42
	4.53%	Electrical	sqm	920	$121.75	$65.75	$204.93	$188,533.58	Rawlinsons 2023, Pg 40 & 42
	12.77%	Transportation (Accounting for Elevator Shaft)	no.	2	$128,000.00	$115,000.00	$265,586.44	$531,172.88	Rawlinsons 2023, Pg 264
External Services									
0.50%	0.50%	External Services	sqm	920	$11.25	$9.50	$22.68	$20,864.38	Rawlinsons 2023, Pg 40 & 42
Contingency									
2.19%	2.19%	Contingency	sqm	920	$63.25	$27.25	$98.91	$90,998.88	Rawlinsons 2023, Pg 40 & 42
100.00%		GST EXCL.						$4,159,916.49	
		GST INCL.						$4,575,908.14	

The elemental breakdown of the preliminary cost is shown below. The superstructure is the element that represents the largest portion of the cost breakdown, at 41.37% of the total price. This element has been further broken down into the sub-elemental components, with walls and floors holding the greatest values.

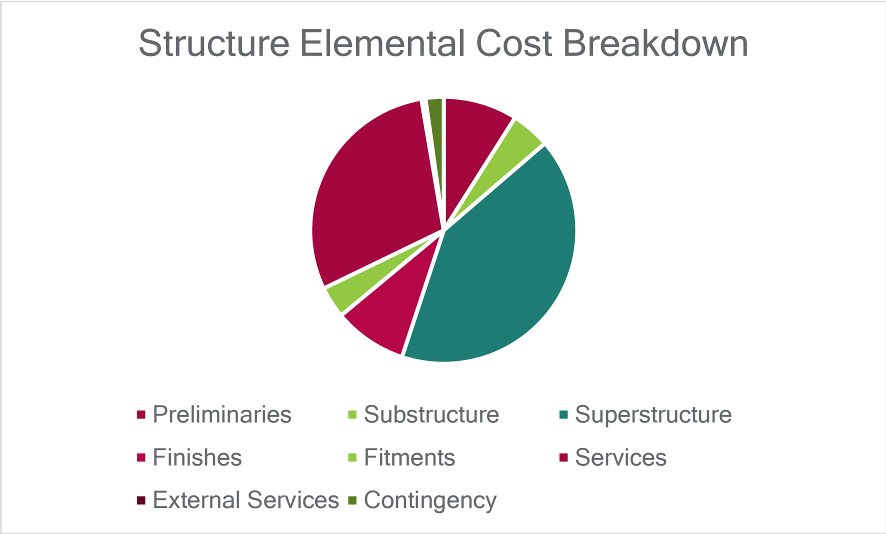


Figure 1: Structure Elemental Cost Breakdown

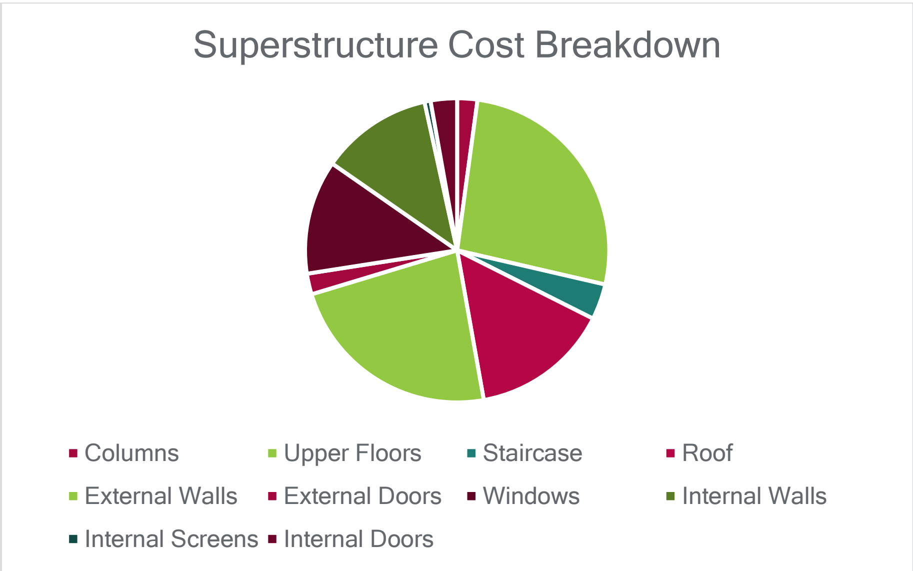


Figure 2: Superstructure Sub-Elemental Cost Breakdown

The preliminary estimate came in as $4,159,916, compared to the approximate estimate derived earlier $3,315,680. This is accepted as it is more in depth for the elements and includes more items than what was taken in the approximate estimate.

2.3.6 Methodology Breakdown

The following tables in this section have broken down the project into smaller tasks. By doing this the duration of each task is determined. Leading to a close approximate of the duration of the project. This allows an efficient way of planning the project to find the shortest period of time in which construction may be completed. The entirety of the project may be broken into three main sections. The preliminary works, the main construction works and the end works.

Once all tasks are broken down the data will then be input into Microsoft Projects to produce a Gantt Chart. A visual diagram that will show the duration of each task. On this calendar the works is laid out to be completed during 8 hours a day from Monday to Friday.

Preliminary Works		
Task	**Duration (Days)**	**Description**
Delivery of Barge	1	Prior to any works the barge will need to arrive in order to transport plants and other equipment to the site. Delivery of the barge should be expected in one day. The barge service used is Pelagic Marine services.
Mobilisation of plant and site offices	10	All plants, site offices and site amenities are to be transported using the barge. These items will be loaded onto the two prime movers and placed onto the barge. 10 days should be allowed for the transportation of the site offices, plants and crane. In addition, the mobile crane will be used for the lifting and placement of the site offices and other site amenities.
Work Induction Training	1	All workers and staff are to be given a full work induction training before any work can commence. This training should include topics of occupational health and safety, standard site safety and emergency protocols.
General Site Clearance	2	An excavator of 1.02m3 bucket size, average haul distance of 20m, moving at an average of 5km/hr will be used. In terms of unloading and loading times and soil loosening, it has an efficiency of 40%. (60mins*1.02*60) / (20/1.39/0.4) = 102.08 m3/hr With a total area of 2000m2 and an assumed vegetation depth of 0.5m, we have approximately 1000m3 volume of rubbish and vegetation to be cleared. 1000/102.08 = 9.7 -> 10 hours

		Labour required should be 1 operator and 1 garbage truck Plant required – garbage truck and excavator. Therefore 2 days should be given.
Site Survey	3	3 days should be given for a site survey, as it is the expected time for a surveyor to survey the site and produce a report.
Fencing of the Site	1	Fences are to be transported to the site and assembled around the boundaries of the site. 1 day should be given for this task to be completed.
Initial Site inspections	1	An initial site inspection entails the inspection of the fencing and other important and safety aspects prior to commencement of construction.

Table 5: Methodology Breakdown

Main Construction works

Excavation and Earthworks		
Task	**Duration (Days)**	**Description**
Topsoil Removal	2.5	0.3m of top soil to be excavated for a site of 2000m2. A loader will be required for this work. With a bucket capacity of 0.25m3 (Caterpillar Handbook) a haul distance assumed at 15m to the truck, moving at an average of 5km/hr and an efficiency of 40%. (60*0.25*60)/(15/1.39/0.4) = 33.36 m3/hr Total top soil volume = 600m3 600/33.36 = 18 hours 2.5 days should be given to the removal of the topsoil. Labour required – 1 operator and 1 truck driver Plant required – 1 loader and 1 semi-truck
Site leveling	0.5	Site to be levelled with 100m3 of soil to be cut/filled. An excavator of 0.75m3 bucket size, average haul distance of 10m, moving at an average of 5km/hr will be used. In terms of unloading and loading times and soil loosening, it has an efficiency of 40%. (60*0.75*60)/(10/1.39/0.4) = 150 m3/hr Total top soil = 600m3 600/150 = 3.99hrs -> 4hrs Half a day should be given for the levelling of the soil. Labour required – 1 operator, 1 truck driver Plant required – 1 grader and 1 semi-truck
Excavation for footing and elevator shaft foundation	5	The overall footing area for excavation was calculated to be 1056m2.

		Allow for the depth of excavation to be 0.115m. 0.155*1056 = 163.68m3 The overall excavation volume will therefore be 163.68m3 The rate of labourer hrs/m3 excavated was deemed to be 0.2hr/m3 from the Rawlinson's 2022. 163.68*0.2 = 32.736 hours. 5 days should be allowed for the excavation for footing and elevator shaft foundations. Labour required – 1 operator, 2 truck drivers Plant required – 1 excavator, 1 semi-truck, 1 Moxy truck.
Excavation for drainage soakwell and services	2	Service and drainage works will be out-sourced to a sub-contractor. Excavation volume will be estimated to be 50m3. The rate of labourer hrs/m3 excavated was deemed to be 0.2hr/m3 from the Rawlinson's 2022. 0.2*50 = 10 hours 2 days should be allowed for the excavation for drainage soakwell and services.

Table 6: Excavation and Works

Services		
Task	**Duration (Days)**	**Description**
Installation of services and drainage soakwell	7	Installation of services and drainage soakwell will be completed by subcontractors. 7 days will be assigned for this task to be completed.
Overall Backfill and Compact	11	It has been determined by Rawlinson's that compaction is 0.08 labourer hours/m2. Combined area to be compacted is 1056m2. 0.08*1056 = 84.48 hrs for 1 labourer 11 days should be assigned for this job. Labour required – 1 operator

		Plant required – 1 compactor

Table 7: Services

Foundation		
Task	**Duration (Days)**	**Description**
Reo Fixing	6	The reinforcement to be fixed is Y12. From Rawlinson's it was found that 17 hours/t. The weight/m = 0.92 0.92*2.4*2*60*9 = 2.385t 2.385*17 = 40.545hrs 6 days should be given for the completion of this task. Labour required – 2 steel fixers
Baseplate Installation	2	60 Baseplates to be installed. Labour required – 5 carpenters
Concrete Pouring	8	The volume of the concrete footing total is 61.12m3. A factor of 0.9*61.12m3 = 55 hours 8 days should be assigned for the completion of the task. Labour required – 3 concreters Plant required – 1 vibrator, 1 concrete mixer and 1 concrete pump
Concrete Curing	28	Allow at least 28 days for curing after concrete pouring is complete. This is to achieve 28-day compressive strength.
Touch ups and finishes to foundation	1	Before modules are installed allow a day to clear up the area and keeps the foundation in good condition.

Table 8: Foundation

Task	Duration (Days)	Description
	Ground floor	
Transport Pre-fabricated material	2	Allow 2 days for the materials to be transported to the site.
Assembly of Portal frame in minor direction	15	Allow 15 days for the assembly of the portal frames. Labour required – 2 concreters
Lifting of Portal frames onto footing	3	The rate of labourer hours per tonne for lifting steel framed structures was found to be 0.5hrs/t. Found in the Estimating Building Costs Table 10.3. The self-weight of the second floor was calculated to be 39t. 39*0.5 = 19.5 hrs Allow 3 days for the installation of first floor modules. Labour required – 3 Riggers Plant required – 1 Crane and 2 Flatbed trucks
Connections onto footing	1	Allow 1 day for all connections to be made.
Finishes	5	Allow 5 days for the finishes to the steel structure. Including first floor joist, flooring and wall stud.

Table 9: Ground Floor

Task	Duration (Days)	Description
	Module Installation Preparation	
Crane mobilisation to site	1	Allow 1 day for the crane to be transported to the site.
	Module Delivery and Placement	
Transportation of module	10	3 Modules per day will be transported on site. Plant required – 1 semitrailer and 1 flatbed truck

Second floor modules installation	3	The rate of labourer hours per tonne for lifting steel framed structures was found to be 0.5hrs/t. Found in the Estimating Building Costs Table 10.3. The self-weight of the second floor was calculated to be 43.2t. 43.2*0.5 = 21.6 hrs Allow 3 days for the installation of first floor modules. Labour required – 3 Riggers Plant required – 1 Crane and 2 Flatbed trucks
Connections after second floor modules installation	5	Allow 5 days for all connections to be made.
Transportation of module	10	3 Modules per day will be transported on site. Plant required – 1 semitrailer and 1 flatbed truck
Third floor modules installation	3	The rate of labourer hours per tonne for lifting steel framed structures was found to be 0.5hrs/t. Found in the Estimating Building Costs Table 10.3. The self-weight of the third floor was calculated to be 43.2t. 43.2*0.5 = 21.6 hrs Allow 3 days for the installation of first floor modules. Labour required – 3 Riggers Plant required – 1 Crane and 2 Flatbed trucks
Connections after third floor modules installation	5	Allow 5 days for all connections to be made.
Transportation of module	5	3 Modules per day will be transported on site. Plant required – 1 semitrailer and 1 flatbed truck
Fourth floor modules installation	2	The rate of labourer hours per tonne for lifting steel framed structures was found to be 0.5hrs/t. Found in the Estimating Building Costs Table 10.3.

		The self-weight of the fourth floor was calculated to be 30t. 30*0.5 = 15 hrs Allow 2 days for the installation of first floor modules. Labour required – 3 Riggers Plant required – 1 Crane and 2 Flatbed trucks
Connections after fourth floor modules installation	5	Allow 5 days for all connections to be made.
Roofing Installation		
Roof structure installation	0.5	Only the roof structure to be lifted by the crane and placed on the 2nd Floor Module. Therefore, allow half a day maximum for only one lifting to be completed by the crane
Connect roofing to the second floor	5	Allow 5 days for all connections to be made.

Table 10: Modular Construction

3.0 Project Costing

A detailed costing estimate for the project has been prepared in accordance with the Civil Engineering Standard of Method of Measurements 4 (CESMM4). In line with the item codes, the costs have been derived for each element of construction.

3.2. Representative Rate Derivation

A representative rate derivation has been determined based on rates found in Rawlinson's and other online sources, representing the cost per unit for an element of the construction.

3.2.1. Concrete for Footings

Bill Item: Pad Footings

Inclusive Tasks:

Placing In-Situ Concrete Footings for 30MPa Concrete with Y12 rebar concrete.

- Earthworks
- Rebar Placement
- Formwork Placement
- Concrete Pouring

Footing Element	Description	Unit	Quantity	Labour	Plant	Material	Total	Cost per m^3
Earthworks	Excavator	$/ cum	121.1*0.4 = 48.44 hr		$100/hr		$4844.00	$70.08
	Backhoe Excavator Operator	$/hr	121.1*0.4 = 48.44 hr	$74/hr			$3584.56	$51.86
Rebar	Materials: Steel Bars 9Y12, 9 bars per footing	$/t	2.385 t			$3,850/t	$9,182.25	$132.84
	Labourer: Steel Fixers	$/hr	17 hrs/t * 2.385 = 40.545 hours	$83.25/ hr			$3,375.37	$48.83
Formwork	Materials: Rough Finish Formwork	$/sqm	172.8 sqm			$20.70/ sqm	$3,576.96	51.75
	Tradesman Formwork (Placement)	$/hr	0.6 * 1.125 * 172.8 = 116.64 hours	$83.25/ hr			$9710.28	$140.48
Concrete Pour	Materials:	$/ cum	69.12 cum			$233/ cum	$16,104.96	$233.00
	Labourer For Concrete Pour for Footings (Column Footing)	$/hr	0.9 * 69.12 = 62.208 hours	$74/hr			$4,603.39	$66.60
	Concrete Mixer (Dry Hire)	$/Day	7		$140/ day		$980.00	$14.18
	Vibrator – Immersion Vibe Shaft and Head	$/Day	7		$100/ day		$700.00	$10.13
	Concrete Pump (Dry Hire)	$/ hour	56 hours		$60/ hour		$3,360	$48.61
Total								$868.36/ m^3

3.1 Bill of Quantities

A detailed take-offs and subsequent bill of quantities has been derived for the Rottnest Island dual purpose Pedal and Flipper and residential unit development. The table below represents the cost breakdown per CESMM4 classes is detailed below. The detailed breakdown of the BOQ is outlined in Appendix B.

Relevant CESMM4 Class Items for Development Project	Cost
Class A - GENERAL ITEMS	$ 1,277,414.81
Class B - GROUND INVESTIGATION	$ 5,885.00
Class D - DEMOLITION AND SITE CLEARANCE	$ 72,759.50
Class E - EARTHWORKS	$ 11,102.94
Class F - IN-SITU CONCRETE	$ 63,748.61
Class G - CONCRETE ANXILLARIES	$ 28,379.37
Class M - STRUCTURAL METALWORK	$ 2,088,848.30
Class N MISCELLANEOUS METALWORK	$ 785,723.25
Class X WATERPROOF	$ 61,090.71
Class Z: SIMPLE BUILDING WORKS INCIDENTAL TO CIVIL ENGINEERING WORKS	$ 1,979,570.42
GST EXCLUSIVE AMOUNT	$ 6,374,522.90
ESTIMATED GST PAYABLE (10%)	$ 637,452.29
TOTAL TENDER PRICE	$ 7,011,975.19

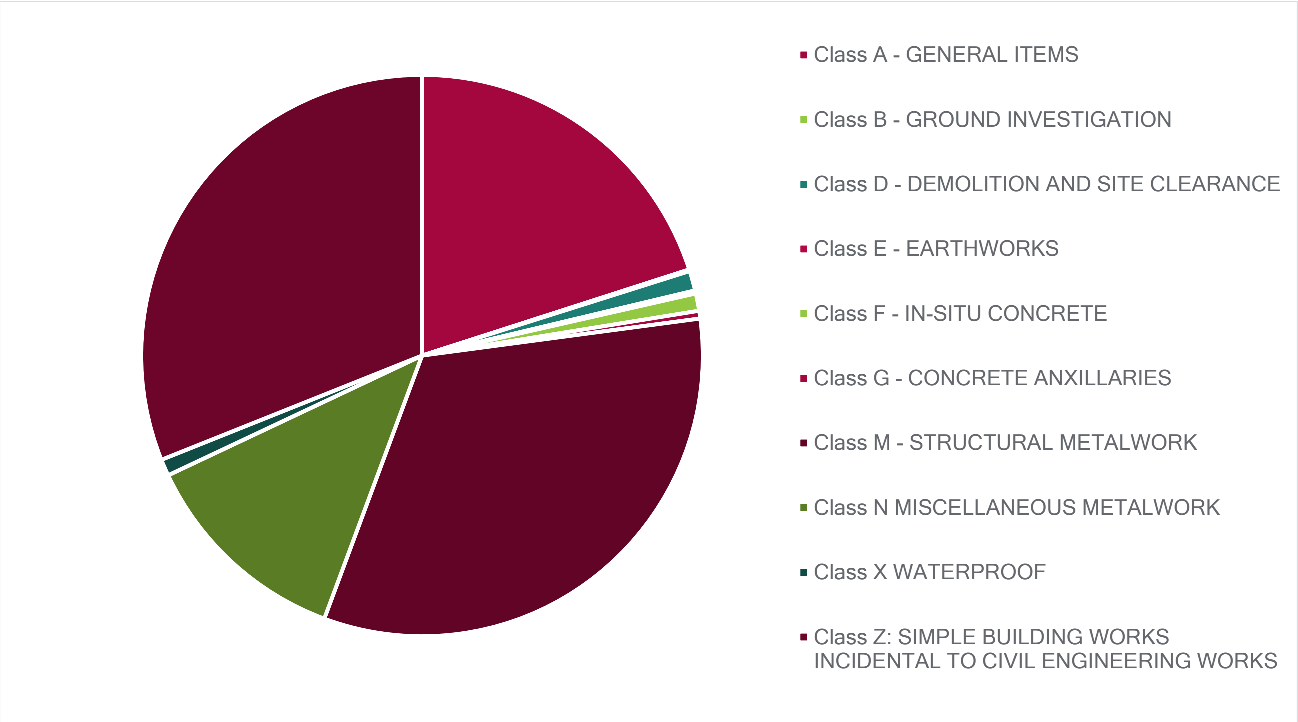


Figure 3: Cost Breakdown - Detailed Estimate

The overall cost for this project, excluding GST came to $6,374,522.90. The prices determined for the costing of this project was taken from the Rawlinson's 2023 Construction Cost Guide. The prices are relevant to Perth costs as acquisition of materials will be done locally and shipped over there.

An increase of 10% for profits, contingencies and risks were added to the overall project. Including GST too, the total is equal to $7,011,975.19. This price is significantly higher than that of the preliminary cost and approximate cost. This indicates that the initial estimates were a good baseline but was not able to fully cover the costs of construction on an island off the coast of Perth, as opposed to on the mainland itself.

The portions of the project where the greatest costs were found are in the structural steel elements, the simple building works incidental to civil engineering and the general items. This was expected, as the steel products dominate the design and are represent a large proportion of costs. General items incurred large numbers due to extended period for construction required and due to the necessity of transport by barge off-shore.

8.0 Construction Management Plan

Prior to the start of the redevelopment work, a Construction Management Plan will be created. As a minimum, this plan will include the following:

- Modelling identifying vegetative areas and individual trees to be preserved onsite, with an obligations to demarcate these areas prior to the start of work.
- Machine and equipment hygiene standards, including the requirement to be clean upon access to the site.
- Fauna protection procedures to reduce the risk of wildlife harm or fatalities (e.g., speed control restrictions, pre-work zone inspections)
- Management of waste and spillage procedures
- Site stabilization procedures to prevent erosion after soil disturbance and excavations
- Referral to RIA inspection and water quality triggers, in addition actions required if the triggers are violated.
- Information on significant heritage sites
- Process for unexpected finds in heritage areas, which include regulatory and reporting responsibilities for potential Native artefacts
- Noise mitigation procedures
- The chance detection procedure for potential contamination and hazardous materials.

An Environment Induction Pack will also be developed for the contractors to ensure that environmental features and values of the site are acknowledged, and to further clarify the environmental requirements associated with the construction process.

9.0 Traffic Management Plan

This project will involve the transportation of plant, equipment (cranes, concrete mixer) and modules and will include the use of vehicles to transport the plant, equipment and modules to site. The modules will travel from the manufacturing facility which is located 50-250 km of the site destination and the plant and equipment will also be required to be transported to site. Therefore, a thorough traffic management plan is crucial for minimizing any inherent hazards associated with placing vehicles and plant equipment on the road, as well as to plan assure the safety of traffic commuting along the road.

Traffic management to Fremantle Port	The modules will need to be transported from the manufacturing facility to Fremantle Port. The width of the module is 4m and 5m in height. The idea is to transport three of the modules at a time (totalling, 12m in width). The transportation of the vehicle must adhere to Mainroads Western Australia Guidelines for Transporting Multiple Items on an Oversize or Over-mass Vehicle. Section 2.1 of this guideline states that a vehicle carrying an oversize large indivisible item may carry additional oversize large indivisible items, provided: The items form part of a modular load, for example are constructed as a module or are packaged together for transport; and given the length, measured from the front of the vehicle combination to the rear of the load doesn't go over 19 metres. Per section 6 of Class 1 RAV Oversize Period Permit Operating Conditions: A RAV that we have assumed to use is a prime mover towing a semi-trailer. Per MRWA specifications the maximum length of this is to be 40m, maximum width is to be 5.5m and maximum height it to be 5.5m. Assuming that the modules are manufactured at the Fleetwood Australia warehouse near Perth Airport, the modules will have to be transported through the area to the Pelagic Barge Services located at, 20 Rous Head Rd, North Fremantle WA 6159. The truck is anticipated to occupy a lane and a half of Leach Highway, therefore requiring pilot vehicles. Pilot requirement for the modular transport includes 1 pilot vehicle for non-central zones and 2 pilot vehicles for central zones. The crane used for construction is assumed to be a mobile crane that has the ability to drive along Leach Highway and reach the port.

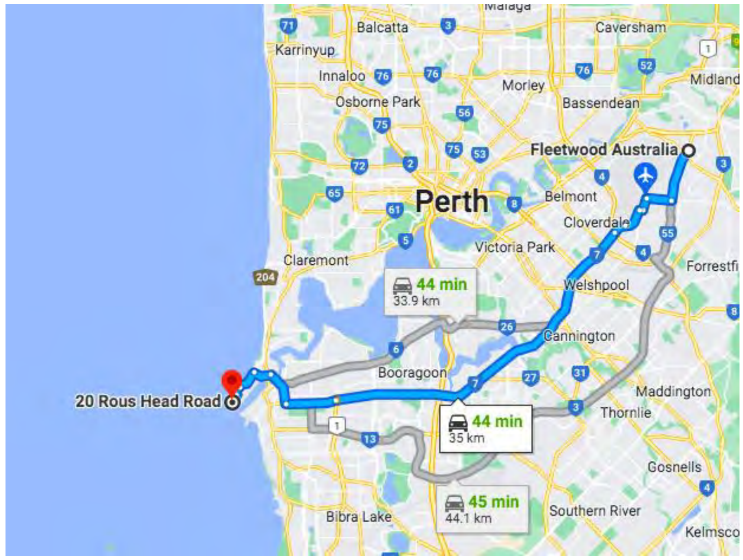


Figure 1: Transport from Fleetwood to Pelagic Barge Services

Figure 6: Fremantle Port Precinct

Class 1 RAV Oversize Period Permit Operating Conditions describes peak hours as 7.30 am to 9.00 am and 4.30 pm to 6.00 pm on a Monday, Tuesday, Wednesday, Thursday or Friday, other than a public holiday.

A RAV of our dimensions must not travel at night except within the Fremantle Port Precinct and Henderson Industrial Estate Area

A RAV our dimensions must not travel in the Metropolitan Area during peak hours except in the Henderson Industrial Estate Area. Therefore, the best approach would be to transport the modules prior to peak hours at around 5am to ensure enough time is given for transport time.

Leach Highway is a fairly busy road with high traffic density during peak hours. It is important to note that as Leach highway nears Fremantle, the density of

<table>
<tr>
<td></td>
<td>trucks also increase as most are headed towards distributing centres near the port, therefore, it is important to ensure that our transportation takes into account that other large trucks will be encountered. However, Leach Highway towards Fremantle is a four-lane highway creating some leeway for transportation and lanes for emergency stopping where required.</td>
</tr>
<tr>
<td>Traffic management from Fremantle Port to Rottnest Island port</td>
<td>From Fremantle Port to Rottnest Island, Pelagic Barge services is to be engaged transporting all material to Rottnest Island.

Previously, anything constructed on Rottnest Island has been carried by the Pelagic Rottnest Barge Service. They use a roll-on / roll-off landing barge and carry trucks, construction equipment and food.
The barges are 20 in length and 5 metres in width.
<table>
<tr><td>Gates open – 20 Rous Head Road, North Fremantle</td><td>0515 hours</td></tr>
<tr><td>Same day perishable deliveries and vehicles to arrive by</td><td>0600 hours</td></tr>
<tr><td>Gates closed for same day delivery</td><td>0600 hours</td></tr>
<tr><td>Scheduled vessel departure</td><td>0645 hours</td></tr>
<tr><td>Scheduled arrival – Thomsons Bay</td><td>0830 hours</td></tr>
<tr><td>Last time for arrival of vehicles and returning goods at the Thomsons Bay wharf</td><td>1030 hours</td></tr>
<tr><td>Departure (Thomsons Bay, depending on volumes, returning freight)</td><td>1100-1200 hours</td></tr>
<tr><td>Gates close – 16 Mews Road, Fremantle</td><td>1600 hours</td></tr>
</table>
Figure 7: Pelagic travel schedule

Therefore, the trucks holding the modules will be driven onto the barge and equipment will be directly loaded onto the barge and transported to Rottnest Island.

The truck carrying modules and equipment must be ready to be loaded onto the barge by 6am, therefore will need to leave the manufacturing centre by 5am the same day.</td>
</tr>
<tr>
<td>Traffic management from Rottnest Island Port to Peddle and Flipper proposal area</td>
<td>There is only a small road, therefore care must be taken to manage traffic effectively in and around this area. When required and the construction material is arriving on site, to minimise disruptions to the island traffic, Henderson Avenue to Brand Way and then Welch Road leading to the back of the Peddle and Flipper will be utilised rather than the main road of Bedford Avenue, shown below in the figure.

A Traffic Management Plan will be created in conjunction with input by the local Rottnest Island Council to ensure that the traffic and construction works are undertaken in a safe way and within a safe environment. This Traffic Management plan, permits and objectives will is required to be approved by the council prior to work commencement.</td>
</tr>
</table>

<table>
<tr>
<td>Traffic management during construction</td>
<td>
As the equipment used for construction are large such as the crane (will be lifting large modules 930kg) and truck.
Closing off Welch Road for the duration of construction will be a safe option to reduce the opportunity for traffic incidents to occur. Welch Road is a relatively short road that connects Brand Way to Bedford Avenue. It is important to note that no other shops or facilities are accessed via Welch Road.
Brand Way after the intersection of Henderson Avenue will also be closed off in the instances the modules are transported to the site and large trucks and cranes move towards the site. The activities will consist of delivery of barge, mobilisation of plant and site offices, work induction training, general site clearance, site survey, fencing oof the site and initial site inspection which will be undertaken from Monday 15th of May to Wednesday 7th of June. This will last a total of 23 including weekends.
Major disruptions to traffic is not expected to occur as traffic can be redirected through Watjil Place and regular traffic can move through Bedford Avenue. This will allow regular access to facilities, shops and amenities.
Cyclists, pedestrians and foot traffic will be redirected through the diversion by traffic operators and detour signs.
Road barriers and detour sign will be implemented to redirect all traffic and a speed limit around construction areas of 40km/hr will be utilised to ensure that traffic speeds slow down as they approach the development area and take the detour
Traffic operators will be employed to ensure that the cranes, trucks and escort vehicles are directed and guided to the right area.
All the facilities, shops and amenities within closed and construction area within close proximity will be notified of the works.
The construction area will be fenced off to reduce the opportunity for external traffic to enter the construction area.


</td>
</tr>
</table>

2.4 SPACEGASS Concept Arrangement

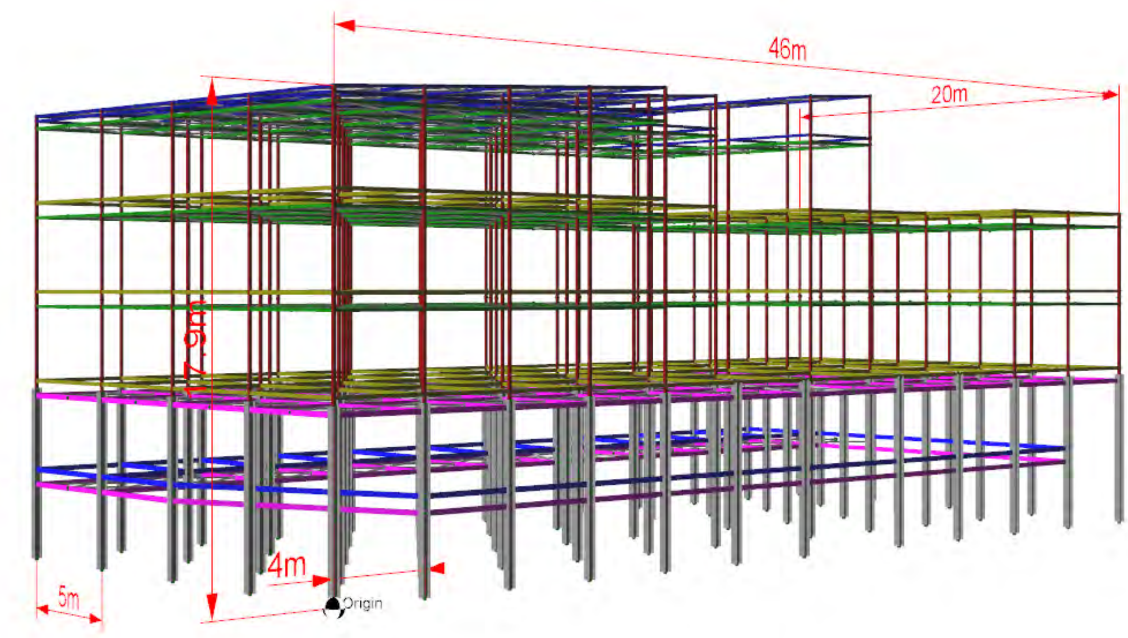


Figure 10: Beam and Column Arrangement East

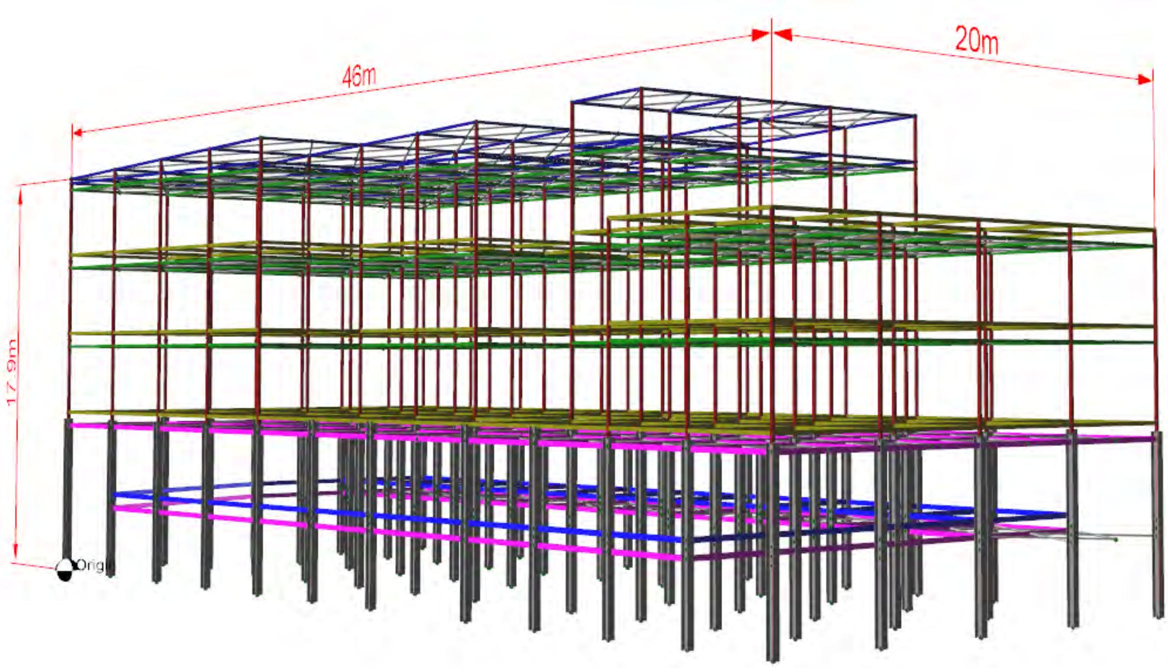


Figure 11: Beam and Column Arrangement West

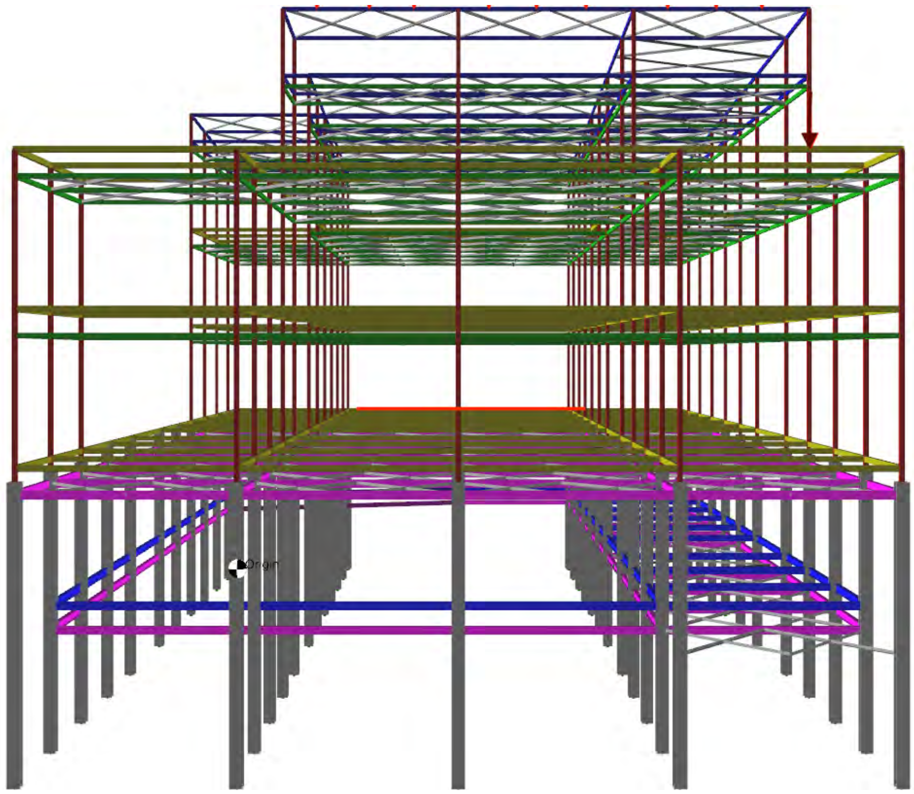


Figure 12: Beam and Column Arrangement Side Profile

Through collaboration within the design team and with the architect, the team have developed an efficient structural design.

As per the client project brief, the aim was to create a practical yet innovative design that worked towards achieving net zero. Therefore, the team incorporated pre-fabricated modules into the design which allow for a more straightforward assembly, construction and disassembly when required. The module sizes were duplicated where possible to ensure that transportation difficulties, costs, and installation difficulties will be minimised. This method of straight-forward construction will mean that once the modules are used for their required plans, they can be disassembled and moved to the next project without any complications creating a more sustainable design.

The design of a regular column grid layout is an important contributor to future flexibility. This gives for more flexibility in future planning rather than being constrained by irregular arrangements designed for specific planning issues.

Further, the residential module has been developed to enable for simpler modular movement since the size has been confined to maximum transport requirements that the maximum height of module can be 3.9m and the maximum transportable length can be 15m. Given these requirements, and as per our design, there is the ability to multiple modules per truck per journey. This sizing module also creates ease of loading and travel on the barge to Rottnest Island.

The structural steel system was modelled using SpaceGass to assess the elements of the residential complex. As the structure is placed under respective loads iteratively, the performance of the structure can be assessed. Figure 1, Figure 2 and Figure 3 above show the full structural building system including cavity for services, roofing, and bracing which are critical elements of design.

2.6 Load Paths

A break down the load system is provided below:

The roof carries three loads consisting of permanent loading of self-weight (G), wind loading (W) and imposed loading (Q). Both the permanent load and imposed load act in the negative Y-axis direction whilst the wind loading acts perpendicular to the roof sheeting.

For the structure, it was determined that the load path travels from the roof to columns and beams to columns down to the footing, as show below.

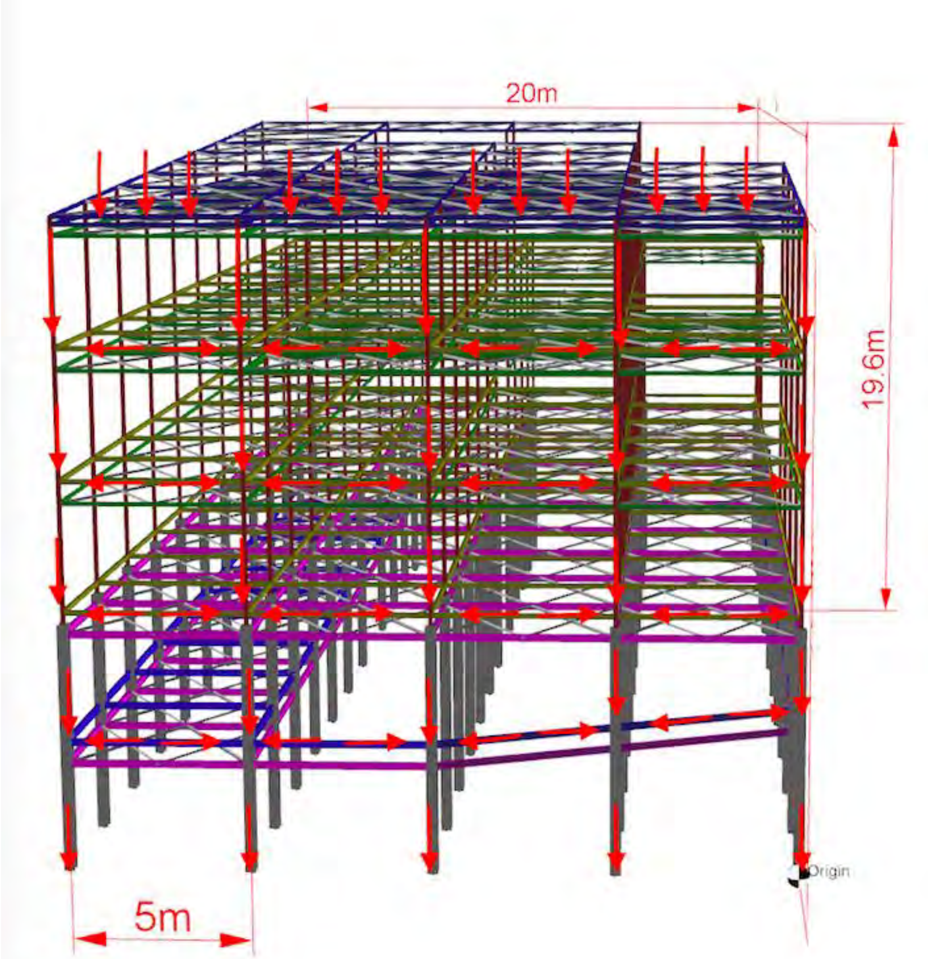


Figure 13: Load Path

2.7 Materials Selection

Module 1: Roof		
Component	**Material**	**Justification**
Roof Cladding	Colourbond Trimdek 0.48 mm BMT	- Sufficient to carry Wind Loading - Meets Requirement for Minimum Slope > 2 Degrees
Purlins (Single Span)	Zed Section Z10015 (1 Row Bridging)	- Sufficient to carry loads across span to roof bearers.
Roof Bracing	Equal Angle Section (75 * 6)	- Secures in tension easily. - Industry Standard
Roof Beam Framing	Rectangular Hollow Section (125*75*5)	- Same as Ceilings in Residential Modules (Repeatability)
Columns	Square Hollow Section (125*6)	- Same as columns of Residential Modules, to keep same connection between modules.
Insulation	Pink Sound break Insulation	- Industry Standard - Meets Requirements for FRL 90/90/90. - Meets Acoustic Requirements.
External Panelling	BlueChip NATURION Non-Combustible Natural Finish Panel 8mm Thick Panels	- According to Architects Specifications – due to natural aesthetic. - Non-Combustible, meeting NCC fire resistance specifications. - Resistant to UV Radiation - Minimal Maintenance

Modules 2: Residential Module		
Component	Material	Justification
Ceiling		
Ceiling Beams	Rectangular Hollow Section (125*75*5)	- Sufficient Capacity to Carry Loads. - Sufficient Size to Fit Ceiling Installations
Ceiling Internal Cladding	Fyrchek Cladding, 13mm thickness	- Meets Fire Resistance Requirements. - Meets Acoustic Resistance
Ceiling Joists	C Purlins	- Industry Standard.
Ceiling Bracing	Equal Angle Section (75 * 6)	- Typical Bracing Element
Insulation	Pink Sound break Insulation	- Industry Standard - Meets Requirements for FRL 90/90/90. - Meets Acoustic Requirements.
Wall		
Column	Square Hollow Section (125*6)	- Sufficient load capacity - Can easily connect upper modules to lower modules.
Internal Wall Structure	Rondo Stud and Track Wall	- Meets Acoustic Requirements - Meets FRL 90/90/90 Requirements
Bracing	Rectangular Hollow Section (100*50*4) Welded to Frame	- Fits within walls - Good strength in both compression and tension.
Internal Cladding	Fyrchek Cladding, 13mm thickness	- Meets Fire Resistance Requirements. - Meets Acoustic Resistance

External Cladding	BlueChip NATURION Non-Combustible Natural Finish Panel 8mm Thick Panels	- According to Architects Specifications – due to natural aesthetic. - Non-Combustible, meeting NCC fire resistance specifications. - Resistant to UV Radiation - Minimal Maintenance
Insulation	Pink Sound break Insulation	- Industry Standard - Meets Requirements for FRL 90/90/90. - Meets Acoustic Requirements.
Floors		
Base Floor	Hebel Flooring	- Lightweight concrete. - Good strength - Fire Resistance - Acoustic Design
Floor Bearers	Rectangular Hollow Section (200*100*4 RHS) Welded to Frame	- Sufficient Capacity - Sufficient space for floor bearer and joist system.
Joists	Stramit Residential Flooring	- Industry Standard

Prefabricated Commercial Floors: Pedal and Flipper		
Component	Material	Justification
Ceiling		
Ceiling Beams	200 UB 25.4 Universal Beams	- Same as for flooring - Requires less procurement of different materials.
Ceiling Internal Cladding	Fyrchek Cladding, 13mm thickness	- Meets Fire Resistance Requirements. - Meets Acoustic Resistance
Ceiling Joists	C Purlins	- Industry Standard.
Ceiling Bracing	Equal Angle Section (75 * 6)	- Secures in tension easily.
Wall		
Column	Universal Column 310 UC 39	- Sufficient Load Capacity - Size of column allows for a 400mm*400mm plate to be welded on to allow - Open Section means that services can be placed along the length of the column and hidden with cladding.
Internal Wall Structure	Rondo Stud and Track Wall Structure for all internal	- Meets Acoustic Requirements - Meets FRL 90/90/90 Requirements
Bracing	Rectangular Hollow Section (100*50*4) Welded to Frame	- Fits within walls - Good strength in both compression and tension.
Internal Cladding	Fyrchek Cladding, 13mm thickness	- Meets Fire Resistance Requirements. - Meets Acoustic Resistance
External Panelling	BlueChip NATURION Non-Combustible Natural Finish Panel 8mm Thick Panels	- According to Architects Specifications – due to natural aesthetic. - Non-Combustible, meeting NCC fire resistance specifications. - Resistant to UV Radiation - Minimal Maintenance

Insulation	Pink Sound break Insulation	- Industry Standard - Meets Requirements for FRL 90/90/90. - Meets Acoustic Requirements.
Floors		
Footings	30 MPa Concrete	- Provides foundational strength.
Ground Floor Slab	100 mm Slab with SL62 mesh	- Reduces moment induced in footing.
First Floor Flooring	Hebel Power Slab	- Good Strength for large spans. - Good for heavier loads - Good Fire rating - Good Acoustic Rating
First Floor Joists	C Purlins	- Industry Standard.

Green Star

It is best practice that the proposed project should be compared to the 'Green Star' rating system developed by the Green Building Council of Australia (GBCA).
This rating is an independent verification that the project is sustainable.
Obtaining Green Star accreditation shows leadership, innovation, environmental care, and social responsibility.

A 'Green Star' accredited project can result in (Green Building Council Australia, 2020):

- Lower operating costs
- Use 66% less electricity than average Australian buildings
- Use 51% less potable water than the average Australian building built to meet minimum industry requirements
- Produce 55% fewer greenhouse gas emissions than average Australian buildings

The following elements can be reviewed to ensure that the design is more sustainable:

Life-cycle impacts - concrete

19C.1.1 Portland Cement Reduction: Up to three points are awarded if the Portland cement content is lowered by 40%, as assessed by mass across all concrete utilised in the project, when compared to a standard reference case.

19C.1.2 Water Reduction: Up to half a point is awarded if the mix water for all concrete used in the project comprises at least 50% captured or recycled water.

19C.1.3 Aggregates Reduction: Up to half a point is awarded where
a) At least 40% of the coarse aggregate used in the concrete is crushed slag aggregate or an alternate source. Assuming that the usage of such materials does not result in an increase of more than five kilos of Portland cement per cubic metre of concrete;

b) At least 25% of fine aggregate used in the concrete is manufactured sand or an alternative source. Assuming that the usage of such materials does not result in an increase of more than five kilos of Portland cement per cubic metre of concrete.

Life-cycle impacts – steel

19C.2.1 Reduced Mass of Steel Framing: Two points are available when the amount of steel frame used is reduced in comparison to the standard practice case situation.

19C.2.2 Reduced Mass of Steel Reinforcement: Two points are available when the amount of steel reinforcement used in concrete slabs is reduced in comparison to the standard practice case situation.

To provide an example, the BHP Billiton skyscraper at 480 Queen Street in Brisbane is one such structure that has received no less than six Green Star certifications.
A substantial component of the project was built with 350 Grade Universal Beams from OneSteel, which were not only strong enough to carry the weight of this massive structure, but also utilised recycled steel when possible.

A sustainable option to explore for the elevator would be the implementation of modular elevators. The Schindler 5500 is a single modular system that can be used in residential and commercial spaces. The Schindler 5500 modular elevator system is a greener option as it keeps energy consumption to a minimum by sing power more efficiently. This is done by incorporating regenerative drives, carefully selected materials, and an ecologically sound solution to produce a greener product to use.

Implementing modular elevators has other benefits such as ease of installation as the module will be able to be accessed externally and implemented with lower risks.

Traditional elevators are constructed by using a hoist way as part of the building with steel, wood, or concrete blocks. Each component is lowered or physically carried into the vertical spaces through sometimes dangerous shaft openings or hatches and then assembled one piece at a time.

With modular elevator construction the process turns the elevator horizontal in a high-quality factory environment which allows for safe and quick construction. In this way the hoist way is a hard and durable steel frame. To achieve fire rating, it is wrapped in drywall. The horizontal shaft will then have an elevator pre-installed inside, which includes, elevator car, rails, wiring and a drive system. This will mean they are complete units ready for setup, adjustment, and testing.

3.0 Steel Design

3.1 Design Life

This structure has been designed to last a life of 50 years. All elements have been designed in accordance with the relevant Australian Standards. For steel design, AS4100 – Steel Structures – has been used.

3.2 Building Classifications.

This structure is comprised of two main sections. The ground floor and first floor are commercially used areas where the main Pedal and Flipper can be found along with other shops and restaurants. Floors 2, 3 and 4 will be used extensively for residential use purposes. This is seen as both a low-rise, multi-residential building.

Importance Level

The importance level, in accordance with AS1170.0, for this building is 2 due to the purpose for which it is used for.

For this importance level, the annual probability of exceedance that must be designed for is:

- 1:500 years for Wind ULS
- 1:25 years for Wind SLS

Fire Rating

Any structure built must recognise the risk to the building and to the individuals within the building if a fire were to break out within any part of the structure. Furthermore, this location of this structure in Rottnest Island has a Bushfire Attack Level (BAL) of 12.5, so management of this risk is key. A Fire Resistance Level (FRL) is that which is prescribed by the National Construction Code (NCC) guidelines, to maintain structural adequacy, integrity, and insulation to any part of a structure for a given time, to allow for all individuals in the building to evacuate the building.

The building will be designed based on the classification of the building into its relevant constituent sections, described in the NCC.

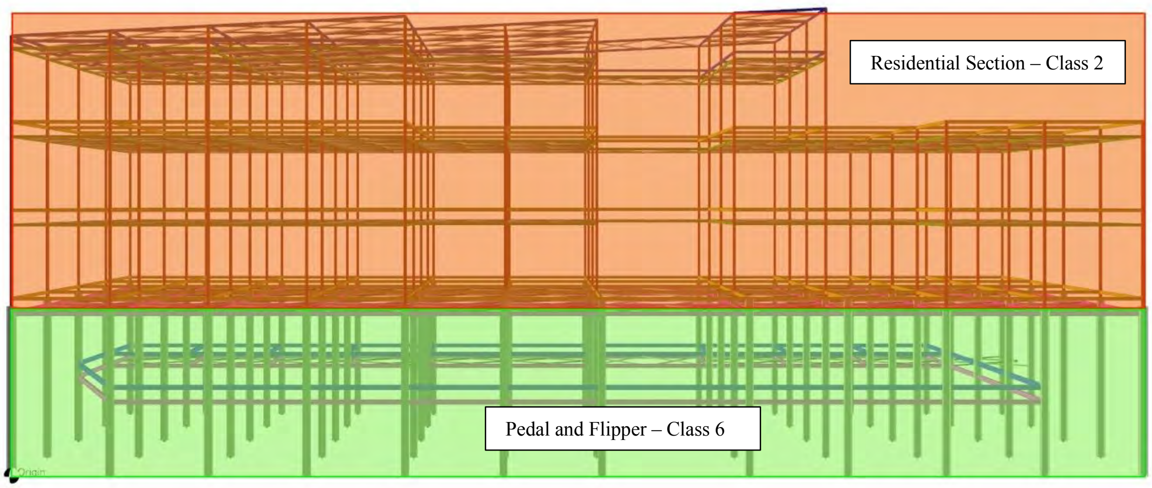
Building Class – Pedal and Flipper (Floors Ground and 1)	6	From BCA Clause A6.6
Building Class – Residential (Floors 2, 3, 4)	2	From BCA Clause A6.6
Construction Type	A	From BCA Table C1.1
Fire Resistance Level	90/90/90	From BCA Table 3

Structural elements within this building will be designed in accordance with the 90/90/90 FRL, maintaining structural adequacy, insulation, and integrity for at least 90 minutes. The main structure to be considered is made of structural steel. The use of in-situ concrete is limited to the footings and ground slab.

The steel members will need to be treated for fire resistance. According to the Australian Steel Institute, some ways of protecting steel to produce the resistance to fire required can be done by:

- Using spray insulations, like vermiculite (Australian Steel Institute, n.d.)
- Providing Fire-Rated Boarding/Cladding Around Structural Elements, like Fyrchek Cladding.
- Paint a thin film of Intumescent Coatings.

These are all viable options which may be used to ensure that steel is adequately fire resistant. All sub-elements within the structure are to be designed to meet this similar standard. Materials used should also be non-combustible products.



3.3 Structural Loading

This structure has been designed to carry loads in accordance with AS1170 series of standards. The load types considered for this structure are:

- Permanent Loads (G) - AS1170.1
- Imposed Loads (Q) - AS1170.1
- Wind Loads (W) - AS1170.2

Permanent Loads

Loads caused by the self-weight of permanent components of the building are known as permanent loads. Following the determination of permanent structural and non-structural elements, the permanent loads could be determined.

Imposed Loads

Loads caused by non-permanent objects placed within the structure. Due to the variety of uses for each of the areas of the building, the loads applied to the structure also vary. The description of imposed loads is found in the table below.

Area	Area Description	Matching Code Description from AS1170	Imposed Load
1	Pedal & Flipper Bike Collection, Gift Shop, etc.	D Shopping Areas	4.0 kPa
2	Receptions/Lobbies	C3 Areas without Obstacles for Moving	4.0 kPa
3	Elevator/Stairwell (For Public and Residential Use)	C3 Areas without Obstacles for Moving	4.0 kPa
4	Restaurants	C1 Areas with tables	2.0 kPa
5	Storage (For Storage Height = 2.5m)	E General Storage other than those specified.	6.0 kPa
6	Residential	A1 Self-contained Dwellings	1.5 kPa
7	Roofs Used for Floor Type Activity	A2 Balconies and Roofs used for floor type activities, with community access	4.0 kPa

Wind Loads
Wind Load Parameters

Region Classification: A1
Terrain Category = 2
Building Height, at Max Point = 19.6 m – (5 Storey Building)
Regional Wind Speed (V_R) = 45 m/s
Climate Change Multiplier (M_c) = 1.0
Wind Direction Multiplier (M_d) = 1.0 (Max Case)
Terrain/Height Multiplier ($M_{z.cat}$) = 1.08
Shielding Multiplier (M_s) = 1.0
Topographic Multiplier (M_t) = 1.0

Max Cases for Wind Loading

V_u = 48.60 m/s $\quad$ p_{ult} = 1.417
V_s = 39.96 m/s $\quad$ p_{serv} = 0.958

Assumed:
- Frictional Drag Forces Neglected.
- Assumed Shielding Value of 1.0.
- Assumed Topographic Value of 1.0.
- Pressure on side walls was not deemed critical, so not considered.

Wind Pressure Breakdown:

Internal Pressures:

p_{ult}	p_{serv}
-0.340 kPa	-0.230 kPa
0 kPa	0 kPa

External Pressures:

Windward Wall

p_{ult}	**p_{serv}**
0.794 kPa	0.537 kPa

Leeward Wall (0 Degrees)

p_{ult}	**p_{serv}**
-0.567 kPa	-0.3832 kPa

Leeward Wall (0 Degrees)

p_{ult}	**p_{serv}**
-0.249 kPa	-0.169 kPa

	p_{ult}		**p_{serv}**	
	Upwind	Downwind	Upwind	Downwind
0 – 9.325(m)	-1.474	-0.680	-0.996	-0.460
9.325 – 18.65(m)	-0.794	-0.340	-0.536	-0.230
18.65 – 20(m)	-0.794	-0.340	-0.536	-0.230

	p_{ult}		**p_{serv}**	
	Upwind	Downwind	Upwind	Downwind
0 – 9.325(m)	-1.020	-0.453	-0.690	-0.307
9.325 – 18.65(m)	-1.020	-0.453	-0.690	-0.307
18.65 – 37.3(m)	-0.567	0.000	-0.383	0.000

*Note Negative Pressures Denote Pressures Acting Away from the Wall (Suction)

3.4 SpaceGass Design

*Seismic Loading for this Design has been neglected.

The following decisions were made when undertaking the design for SpaceGass.

- Bracing in the ceilings for 75*6 EA members were assumed to be pin connected and only holding tension, using FFFFRR.
- Bracing using 100*50*4 RHS in module are to be welded in place and carry both tension and compression.
- Wind loads were applied acting perpendicularly to members
- All other fixed connections were using member end fixity, FFFFFF.
- The base of all columns was designed as being pinned, due to adequate design for moment bearing connection at base is hard. Node notation is FFFRRR (carrying no moments in any direction).
- One part of the structure was examined, due to it being considered as a critical section. The results were carried forward to apply to other parts of the structure too. Connections were designed to meet the results found in the SpaceGass Folder.
- Self-weights were designed within SpaceGass itself, with additional loads considered from elements not designed for structural integrity, but still have a permanent action.

3.5 Results

Steel Member Results

Members were designed through the SpaceGass model using an iterative process. The members were inputted to have the section properties within the software, and then added the loads to the building. The members chosen were as a result of both functionality and design capacity, without exceeding costs.

There were two sections examined within the structure, the residential modules and the prefabricated pedal and flipper structure. The max results found from the software have been summarised in the following tables. These results have further been used when designing for connections between members, to achieve the desired connection.

Residential Modules		
Elements	Critical Force	Design Action
	Bending Moments	
125*6 SHS Column	16.32 kNm	1.2G + $W_{u, down}$
200*100*4 Beam Bearers	27.24 kNm	1.2G + 1.5 Q
	Shear Forces	
125*6 SHS Column	17.79 kN	1.2G + $W_{u, down}$
200*100*4 Beam Bearers	44.3 kN	1.2G + 1.5 Q
	Axial Forces	
125*6 SHS Column	218.46 kN	1.2G + 1.5 Q
200*100*4 Beam Bearers	32.7 kN	1.2G + 1.5 Q

Prefabricated Pedal and Flipper		
Elements	Critical Force	Design Action
	Bending Moments	
310 UC 96.8 Column	56.99 kNm	1.2G + $W_{u, down}$
200 UB 25.4 Bearer Beams	16.44 kNm	1.2G + 1.5 Q
	Shear Forces	
310 UC 96.8 Column	23 kN	1.2G + $W_{u, down}$
200 UB 25.4 Bearer Beams	19.19 kN	1.2G + 1.5 Q
	Axial Forces	
310 UC 96.8 Column	269.38 kN	1.2G + 1.5 Q
200 UB 25.4 Bearer Beams	15.01 kN	1.2G + 1.5 Q

3.6 Connections

For the connections in this design, module connections have been designed to maximise efficiency of construction, modularity, cost, and level of difficulty. Connections come in the form of bolting or welds. The welds have been designed using general purpose welds due to these welds being less costly and requiring less scrutiny, due to the design factor of 0.6. Welds are ideally between 6mm and 8mm, as this is taken as the optimum size to ensure good welding can occur. Bolts have been designed to take both shear and tension forces. The bolts to be used are snug fitted bolts as these bolts can be easily removable, allowing for ease of taking apart in the event of moving the structure from site A to site B. All welded connections are to be done off-site in manufacturing factory. Base connection has been taken as a pinned connection too.

Connection	Element 1	Element 2	Element 1 Connection	Element 2 Connection	Unifying Connector
Base Plate Connection	Pad Footing	310 UC 96.8 Column	M20 4.6/s bolts integrated into Footing	Weld Base Plate to bottom of UC Column.	Holes in base plate are fitted around bolt dimensions and using bolts fix the connection.
UB Beam Connection to UC Column (along major axis)	310 UC 96.8 Column	200 UB 25.4 Column	Bolted connection to Flange of UC Column	Weld Base Plate to bottom of UC Column	8 8.8/S M20 Bolts connects beam connected to column
UB Beam Connection to UC Column (along minor axis)	310 UC 96.8 Column	200 UB 25.4 Column	Welded Stiffeners to Flange of UC Column, for bolt connection to UB beam.	Flanges of Beams to be bolted to Stiffeners on UC column	8 8.8/S M20 bolts using 8.8/S M20 Bolts.
Residential Modules to UC Columns.	310 UC 96.8 Column	125*6 SHS Column	Welded 6mm E43xx GP weld 400*400mm Plate to the top of UC Column	Welded 200*200mm Plate to the base of SHS Column	Use 8.8/S M20 Bolts
Residential Column to Residential Beams	125*6 SHS Column	200*100*6 RHS Column	Welded Connection	Welded Connection	6mm E55xx GP WeldDesigned for shear and moment
Residential Modules to Residential Modules	125*6 SHS Column	125*6 SHS Column	Welded 5mm base plate to Column, then 200mm 100*4 SHS Male Connection, with two 24mm diameter holes.	Two 24 diameter holes through the 125*6 SHS column at top, with 6mm E43xx GP welded nuts for M20 bolts	Use of M20 4.6/S bolts
Roof Module to Residential Module	125*6 SHS Column	125*6 SHS Column	Welded 5mm base plate to Column, then 200mm 100*4 SHS Male Connection, with two	Two 24 diameter holes through the 125*6 SHS column at top, with 6mm E43xx GP welded	Use of M20 4.6/S bolts

			24mm diameter holes.	nuts for M20 bolts	
Connection Between Columns Together	125*6 SHS Column	125*6 SHS Column	Welded 3mm Right Angle plate, using E43xx 6mm Weld	Welded 3mm Right Angle plate, using E43xx 6mm Weld	Use of M20 4.6/S bolts

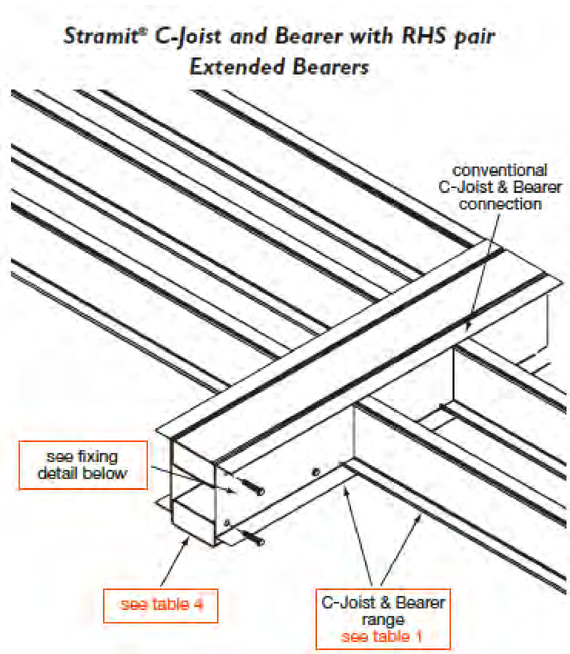
Internal Wall

Using Rondo Steel Studs

- Maximum Spacing Between Studs, 600 mm
- For Wall Lined Both Sides with 13mm Plasterboard (Fyrchek)
- Wall Height = 3000mm
- Use 0.50 BMT Studs at 500mm spacing
- Number of Noggings = 0, due to height of wall < 4.4m

3.7 Floor Design

- For Residential Apartments, Q = 1.5kPa
- Using this, Flooring is Designed According to Stramit C-Joist – Bearer Connection
- Screw a C-channel bearer to the inside of the 200*100*4 RHS beam at the base, on either of the longer 5m sides
- Use B23524 Bearers – 7.04 kg/m
- For 5 m Bearer Span, Joists required are J28319 – 6.37 kg/m
- Spacing between joists, 400 mm.



3.8 Staircase design/ installation

Design standards:

The staircase design shall be designed and governed per AS 1657 and the National Construction Code (NCC).

AS 1657 requirements:

Clause 4.1:

- Stairway width: greater than 600mm
- Pitch: 26.5 degrees < pitch < 45 degrees

Clause 4.4.4

- Rises: maximum 18 rises without landing

  maximum 36 rises without change in direction
- Clearance: maximum overhead clearance = 2m

National Construction Code (NCC) requirements:

Part 3.9.1.2

- Maximum Going (G) = 355mm
- Minimum Going (G) = 240mm
- Maximum Riser (R) = 190mm
- Maximum Riser (R) = 115mm
- Maximum Slope Relationship (2R + G) = 700mm
- Minimum Slope Relationship (2R + G) = 550mm

Part 3.9.1.3

- For greater than 3 rises or 570mm there is a 750mm landing requirement

Part 3.9.2

- A continuous handrail must be provided along the side of stairway, ramp or the like

Part 3.9.2.4

- Handrails must adhere to the requirement listed in this section

Design summary

- Minimum Going (G) = 240mm
- Maximum Riser (R) = 190mm
- Minimum Slope Relationship (2R + G) = 550mm
- Landing elevation = 1800mm
- Landing Length = 1500mm

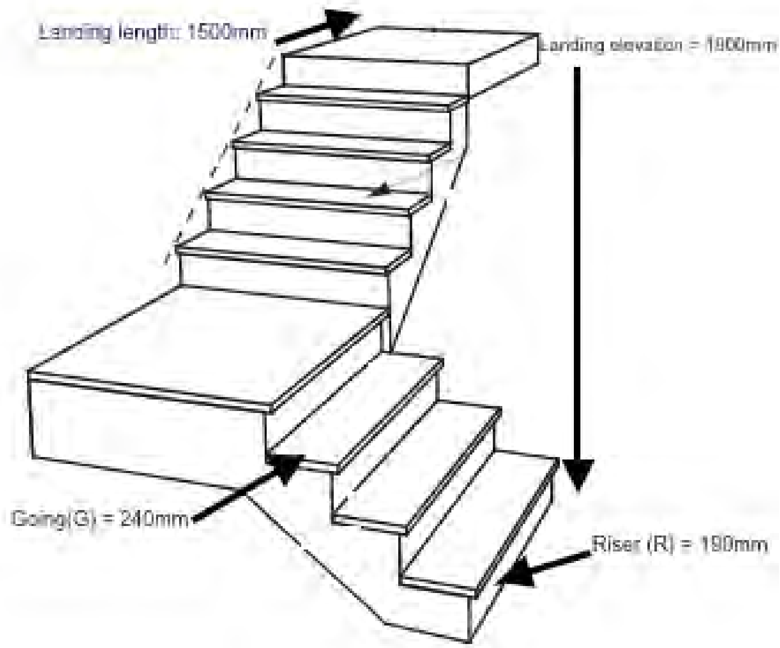


Figure 14: Staircase concept design

4.0 Concrete Design

4.1 Durability

The following analysis of the footing design is in accordance with AS3600. The concrete requires a durability that will survive the design life of 50 years.

Site classification			
Surface location	Soil Class	Min f'c	Cover
In ground	A	30MPa	75mm

4.2 Geotechnical recommendations

For the primary ground support, a concrete pad footing was selected. Ground support is required in structures as the loads are transferred towards the columns, which are transferred to the ground. It should be noted that walls are non-load bearing hence the need for columns and ground support. Due to the lack of geotechnical information in the design brief, in regards to the soil of the site, numerous design assumptions were made. In accordance to the AS2870 and AS3600, the dimensions and the amount of steel reinforcement required were determined.

The design brief, provided by the client, details the assumptions of values needed to calculate the design of the footing. The assumptions are as follows:

- Angle of friction, $\varphi = 30$
- Effective cohesion, $c' = 0$ kPa
- Bulk density, $\gamma_{soil} = 12$ kN/m3
- Yield stress, $f_{sy} =$ 400kPa

4.3 Footing design summary

- Length = 2400mm
- Breadth = 2198mm
- Thickness = 200mm
- Reinforcement = 9 x 12mm Bars @ 280mm spacing

The table below details a summary of the concrete pad footing design. Detailed calculations have been provided in Appendix G Pad footing calculations

Member	Specification	Dimensions	Reference	Drawing Reference
Concrete Pad footing	30 MPa Concrete	2400mm by 2400mm	AS2870 and AS3600	C001
Steel reinforcement	9 Y12	12mm diameter at 280mm spacing	AS3600	C002

Appendix A – Wind Action Calculations

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1. Design Wind Speed

$$V_{sit,\beta} = V_R M_c M_d (M_{z.cat} M_s M_t)$$

$$Regional\ Location: Rottnest\ Island$$
$$\therefore Region\ A1\ (Figure\ 3.1, AS\ 1170.2)$$

$$(1)\ \boldsymbol{Regional\ Wind\ Speed\ (V_R)}$$
$$Structure\ Importance\ Level: 2\ (AS\ 1170.0)$$
$$Design\ Life: 50\ years$$
$$Annual\ Probability\ of\ Exceedance\ for\ ULS: 1/500\ (Table\ 3.3, AS\ 1170.0)$$
$$\therefore For\ 1/500\ in\ Region\ A,$$
$$V_R^{ult} = V_{500} = 45 m.s^{-1} (Table\ 3.1, AS\ 1170.2)$$
$$Annual\ Probability\ of\ Exceedance\ for\ SLS: 1/25\ (Appendix\ C, AS\ 1170.0)$$
$$\therefore For\ 1/25\ in\ Region\ A,$$
$$V_R^{serv} = V_{25} = 37 m.s^{-1} (Table\ 3.1, AS\ 1170.2)$$

$$(2)\ \boldsymbol{Climate\ Change\ Multiplier\ (M_c)}$$
$$For\ Region\ A1$$
$$M_c = 1.0$$

$$(3)\ \boldsymbol{Wind\ Direction\ Multiplier\ (M_d)}$$
$$The\ Factors\ for\ Each\ Cardinal\ Direction\ in\ Region\ A1\ are\ listed\ in\ this\ table:$$
$$(Taken\ from\ Table\ 3.2, AS1170.2)$$

Cardinal Direction	Region A1 Factors
N	0.90
NE	0.85
E	0.85
SE	0.80
S	0.80
SW	0.95
W	1.00
NW	0.95

$$Westerly\ Direction\ is\ Critical\ Direction$$
$$\therefore use\ M_d = 1.00$$

$$(4)\ \boldsymbol{Terrain/Height\ Multiplier\ (M_{z.cat})}$$
$$Terrain\ Category\ 2\ (TC2)$$
$$Height\ of\ Building\ is\ 19.6m\ at\ highest\ point$$
$$M_{z.cat} = 1.08\ (Table\ 4.1, AS1170.2)$$

$$(5)\ \boldsymbol{Shielding\ Multiplier\ (M_s)}$$
Use a conservative Figure in the Event that there is redevelopment of the area and shielding is removed:
$$M_s = 1.0\ (Table\ 4.3, AS1170.2)$$

$$(6)\ \boldsymbol{Topographic\ Multiplier}\ (\boldsymbol{M_t})$$
$$Assumed\ that\ area\ is\ almost\ flat,$$
$$\therefore M_t = 1.0$$

$$V_{sit,\beta} = V_R M_c M_d (M_{z.cat} M_s M_t)$$

Cardinal Direction	V_R(m.s-1)	M_c	M_c	$M_{z.cat}$	M_s	M_t	$V_{des,\theta-ULS}$ (m.s-1)
N	45.00	1.00	0.90	1.08	1.00	1.00	43.74
NE	45.00	1.00	0.85	1.08	1.00	1.00	41.31
E	45.00	1.00	0.85	1.08	1.00	1.00	41.31
SE	45.00	1.00	0.80	1.08	1.00	1.00	38.88
S	45.00	1.00	0.80	1.08	1.00	1.00	38.88
SW	45.00	1.00	0.95	1.08	1.00	1.00	46.17
W	45.00	1.00	1.00	1.08	1.00	1.00	48.60
NW	45.00	1.00	0.95	1.08	1.00	1.00	46.17
Any Direction	45.00	1.00	1.00	1.08	1.00	1.00	48.60

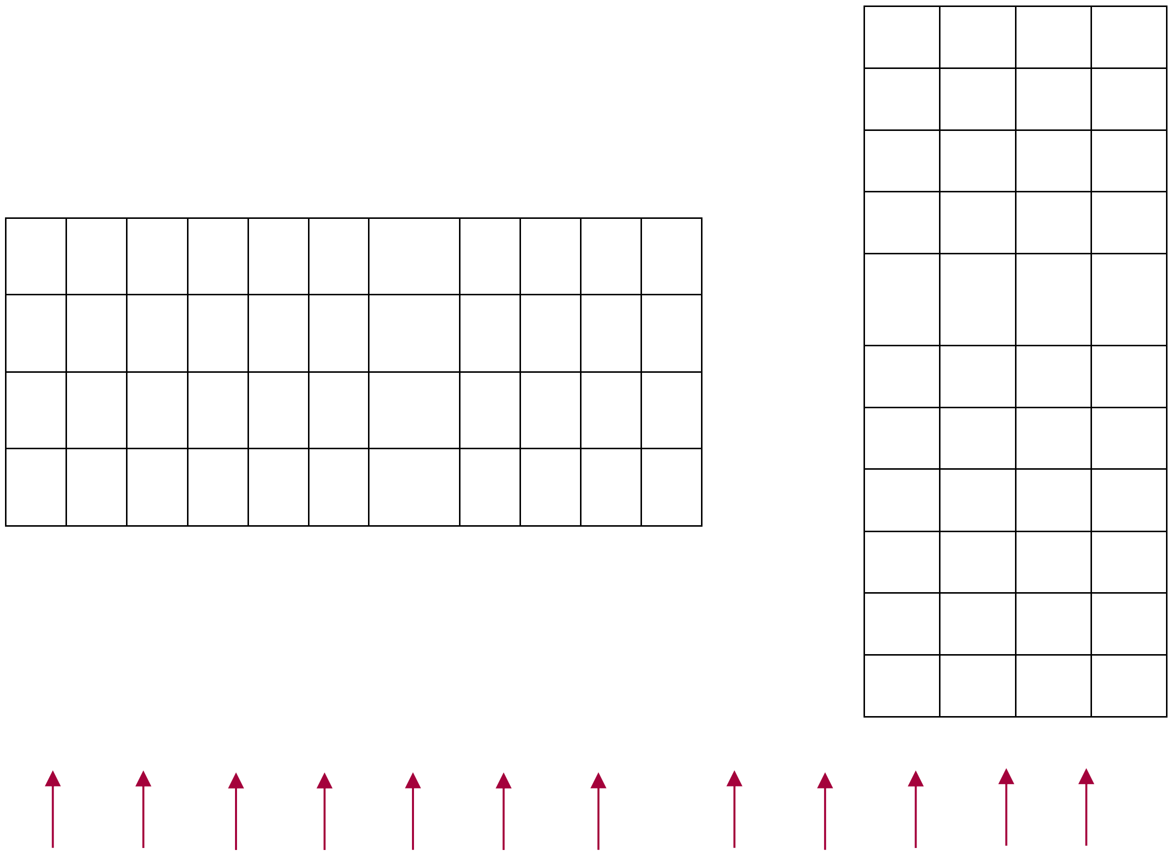
Cardinal Direction	V_R(m.s-1)	M_c	M_c	$M_{z.cat}$	M_s	M_t	$V_{des,\theta-ULS}$ (m.s-1)
N	37.00	1.00	0.90	1.08	1.00	1.00	35.96
NE	37.00	1.00	0.85	1.08	1.00	1.00	33.97
E	37.00	1.00	0.85	1.08	1.00	1.00	33.97
SE	37.00	1.00	0.80	1.08	1.00	1.00	31.97
S	37.00	1.00	0.80	1.08	1.00	1.00	31.97
SW	37.00	1.00	0.95	1.08	1.00	1.00	37.96
W	37.00	1.00	1.00	1.08	1.00	1.00	39.96
NW	37.00	1.00	0.95	1.08	1.00	1.00	37.96
Any Direction	37.00	1.00	1.00	1.08	1.00	1.00	39.96

2. Design Wind Pressure

$$p = (0.5\rho_{air})\left[V_{des,\theta}\right]^2 C_{shp} C_{dyn}$$
$$\rho_{air} = 1.2 kg.m^{-3}$$
$$C_{dyn} = 1.0\ (for\ structural\ elements\ with\ natural\ frequency < 1.0\ Hz)$$
$$C_{shp}\ is\ the\ coefficient\ related\ to\ the\ shape\ of\ the\ building\ and\ aerodynamics:$$

For Internal Pressures: $C_{shp,i} = C_{p,i} \times K_{c,i}$

For External Pressures: $C_{shp,e} = C_{p,e} \times K_a \times K_{c,e} \times K_l \times K_p$



$$p_{ult} = (0.5 \times 1.2)[48.60]^2 \times 1.00 \times C_{fig} = 1.417 \times C_{shp}(kPa)$$

$$p_{serv} = (0.5 \times 1.2)[39.96]^2 \times 1.00 \times C_{fig} = 0.958 \times C_{shp}(kPa)$$

1. Internal Pressure

$$C_{shp,i} = C_{p,i} \times K_{c,i} \times K_v$$

$C_{p,i}$	$=$	-0.3 0	Table 5.1(B)
K_v	$=$	1.0	Clause 5.3.4
$K_{c,i}$	$=$	0.8	Table 5.5

$$C_{shp,i} = -0.3 \times 0.8 \times 1.0 = -0.24$$
$$C_{shp,i} = 0 \times 0.8 \times 1.0 = 0$$

$$p_{ult} = 1.417 \times -0.24 = -0.340(kPa)\ (suction)$$

$$p_{ult} = 0(kPa)$$

$$p_{serv} = 0.958 \times -0.24 = -0.230(kPa)\ (suction)$$

$$p_{serv} = 0.958 \times 0 = 0(kPa)$$

2. External Pressure
    a. 0 Degrees

$$C_{shp,e} = C_{p,e} \times K_a \times K_{c,e} \times K_l \times K_p$$

$C_{p,e}$	=	0.7	Windward	Table 5.2(A)
	=	−0.5	Leeward	Table 5.2(B)
	=	−0.65	Side Walls 0m – 18.65m	Table 5.2(C)
	=	−0.5	Side Walls 18.65m – 20m	
K_a	=	0.93	Roof	Table 5.4
	=	1.0	Windward	
	=	1.0	Leeward	
	=	0.97	Sidewall	
$K_{c,e}$	=	0.8		Table 5.5
K_l	=	1.0	(Assumed)	
K_p	=	1.0	(Assumed)	Table 5.8

a. 90 Degrees

$C_{p,e}$	=	0.7	Windward	Table 5.2(A)
	=	−0.22	Leeward	Table 5.2(B)
	=	−0.65	Side Walls 0m – 18.65m	Table 5.2(C)
	=	−0.5	Side Walls 18.65m – 37.3m	
	=	−0.3	Side Walls 37.3m – 46m	
K_a	=	0.93	Roof	Table 5.4
	=	1.0	Windward	
	=	1.0	Leeward	
	=	0.97	Sidewall	
$K_{c,e}$	=	0.8		Table 5.5
K_l	=	1.0	(Assumed)	
K_p	=	1.0	(Assumed)	Table 5.8

Windward Wall

$$C_{shp} = 0.7 \times 1.0 \times 0.8 \times 1.0 \times 1.0 = 0.56$$

$$p_{ult} = 1.417 \times 0.56 = 0.794\ (kPa)$$

$$p_{serv} = 0.958 \times 0.56 = 0.537\ \ (kPa)$$

Leeward Wall (0 Degrees)

$$C_{shp,e} = -0.5 \times 1.0 \times 0.8 \times 1.0 \times 1.0 = -0.40$$

$$p_{ult} = 1.417 \times -0.4 = -0.567\ (kPa)$$
$$p_{serv} = 0.958 \times -0.4 = -0.3832\ (kPa)$$

Leeward Wall (90 Degrees)

$$C_{shp,e} = -0.22 \times 1.0 \times 0.8 \times 1.0 \times 1.0 = -0.176$$

$$p_{ult} = 1.417 \times -0.176 = -0.249\ (kPa)$$
$$p_{serv} = 0.958 \times -0.176 = -0.169\ (kPa)$$

Roof $\boldsymbol{C_{p,e}}(\mathbf{0}°)$				
		Upwind	**Downwind**	
	0 – 9.325(m)	-1.3	-0.6	
(h/d ≈1)	**9.325 – 18.65(m)**	-0.7	-0.3	**Table 5.3(A)**
	18.65 – 20(m)	-0.7	-0.3	

Roof $\boldsymbol{C_{p,e}}(\mathbf{90}°)$				
		Upwind	**Downwind**	
	0 – 9.325(m)	-0.9	-0.4	
(h/d = 2.3)	**9.325 – 18.65(m)**	-0.9	-0.4	**Table 5.3(A)**
	18.65 – 37.3(m)	-0.5	0.0	
	37.3 - 46(m)	-0.3	0.1	

$$C_{shp,e} = C_{p,e} \times 1.0 \times 0.8 \times 1.0 \times 1.0 = 0.8 \times C_{p,e}$$
$$p_{ult} = 1.417 \times 0.8 \times C_{p,e} =$$
$$p_{serv} = 0.958 \times 0.8 \times C_{p,e} =$$

	p_{ult} (kPa)		p_{serv} (kPa)	
	Upwind	**Downwind**	**Upwind**	**Downwind**
0 – 9.325(m)	-1.474	-0.680	-0.996	-0.460
9.325 – 18.65(m)	-0.794	-0.340	-0.536	-0.230
18.65 – 20(m)	-0.794	-0.340	-0.536	-0.230

	p_{ult} (kPa)		p_{serv} (kPa)	
	Upwind	**Downwind**	**Upwind**	**Downwind**
0 – 9.325(m)	-1.020	-0.453	-0.690	-0.307
9.325 – 18.65(m)	-1.020	-0.453	-0.690	-0.307
18.65 – 37.3(m)	-0.567	0.000	-0.383	0.000
37.3 - 46(m)	-0.340	0.113	-0.230	0.077

Appendix B – Steel Member Design Calculations

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Preliminary Member Selection:

Serviceability Limit States (AS1170.0, C1):

Columns	= Height/500	(Height = 6900 mm)	$\Delta = 13.8$mm
	= Height/500	(Height = 12700 mm)	$\Delta = 25.4$mm
Bearers	= Span/300	(Span = 5000mm)	$\Delta = 16.7$mm
Floor Perimeter Beams	= Span/300	(Span = 4000mm)	$\Delta = 13.3$mm
Roof Beam	= Span/300	(Span = 4005mm)	$\Delta = 13.4$mm

Prefabricated Pedal and Flipper Area

Column – 310 UC 96.8

Deflection					
Max Moment	M^*	=	56.99 kNm		
Section Capacity,	ϕM_{sx}	=	422 kNm	$\therefore OK$	(DCT, Table 5.3-4(A))
Max Axial Compression/Tension Force	N^*	=	925 kN		
Section Capacity	ϕN_s	=	3340 kN	$\therefore OK$	(DCT, Table 6-7)
	ϕN_{cx}	=	2759 kN	$\therefore OK$	(DCT, Table 6-7(A))
	ϕN_{cy}	=	1842 kN	$\therefore OK$	(DCT, Table 6-7(B))
	n	=	$N^*/\phi N_s$		
		=	0.2769		
Moment Capacity	ϕM_{rx}	=	422 (1-n)	$\therefore OK$	(DCT, Table 8-4)
		=	305 kN < 422 kN		
	ϕM_{ry}	=	187 (1-n)	$\therefore OK$	(DCT, Table 8-4)
		=	135.21<187	[for both Tension and Compression]	
Shear Force	V*	=	23 kN		
	ϕV	=	527 kN > V*	$\therefore OK$	(DCT, Table 5.3-4(A))
In-plane/Out of Plane Buckling		=	$M^*/\phi M_{sx}$		
		=	$+ N^*/\phi N_s$ 0.411 < 1	$\therefore OK$	

Beam – 200 UB 25.4

Deflection			11.5mm < 16.7mm	$\therefore OK$	
Max Moment	M^*	=	16.44 kNm		
Section Capacity,	ϕM_{sx}	=	74.6 kNm	$\therefore OK$	(DCT, Table 5.3-3(A))
Max Axial Compression/Tension Force	N^*	=	15.01 kN		

Section Capacity	ϕN_s	=	1047 kN	$\therefore OK$	(DCT, Table 6-6)
	ϕN_{cx}	=	932 kN	$\therefore OK$	(DCT, Table 6-6(A))
	ϕN_{cy}	=	193 kN	$\therefore OK$	(DCT, Table 6-6(B))
	n	=	$N^*/\phi N_s$		
		=	0.014		
Moment Capacity	ϕM_{rx}	=	74.6 (1-n)	$\therefore OK$	(DCT, Table 8-3)
		=	73.5 kN < 74.6 kN		
	ϕM_{ry}	=	19.8 (1-n)	$\therefore OK$	(DCT, Table 8-3)
		=	19.5 < 19.8	[for both Tension and Compression]	
Shear Force	V*	=	19.19 kN		
	ϕV	=	204 kN > V*	$\therefore OK$	(DCT, Table 5.3-3(A))
In-plane/Out of Plane Buckling		=	$M^*/\phi M_{sx}$		
		=	$+ N^*/\phi N_s$	$\therefore OK$	
			0.2347 < 1		

Column –125 × 6 SHS

Deflection			21.76mm < 25.4 mm		
Max Moment	M^*	=	16.32 kNm		
Section Capacity,	ϕM_{sx}	=	48.6 kNm	$\therefore OK$	DCT 2, Table 5.2-4(2)
Max Axial Compression/Tension Force	N^*	=	218.46 kN		
Section Capacity	ϕN_s	=	1110 kN	$\therefore OK$	(DCT, Table 6-7)
	ϕN_{cx}	=	680 kN	$\therefore OK$	(DCT, Table 6-7(A))
	n	=	$N^*/\phi N_s$		
		=	0.1968		
Moment Capacity	ϕM_{rx}	=	57.3 (1-n)	$\therefore OK$	(DCT, Table 8-4)
		=	46.02 kN < 48.6 kN	[for both Tension and Compression]	
Shear Force	V*	=	17.79 kN		
	ϕV	=	527 kN > V*	$\therefore OK$	DCT 2, Table 5.2-4(2)
In-plane/Out of Plane Buckling		=	$M^*/\phi M_{sx} + N^*/\phi N_s$		
		=	0.532	$\therefore OK$	
Tension On SHS Beam Section When Lifting Each Module	N^*	=	25.7 kN		
For Outer Female SHS Section – 125*6 SHS (Assume 24mm Dia Holes)	ϕN_t	=	1110kN	$\therefore OK$	DCT 2, Table 7-6(1)
			Or		
		=	$\dfrac{(2730 - 2 \times 6 \times 24) \times 1050}{2370}$		
		=	939.23 kN		

For Outer Male SHS Section – 100*4 SHS (Assume 24mm Dia Holes)	ϕN_t	=	493 kN	$\therefore OK$	DCT 2, Table 7-6(1)
			Or		
		=	$\frac{(2730 - 2 \times 6 \times 24) \times 1050}{2370}$		
		=	939.23 kN		

Beam –$\mathbf{200 \times 100 \times 4}$ *RHS*

Deflection			9.87mm < 16.7mm mm		
Max Moment	M^*	=	27.24 kNm		
Section Capacity,	ϕM_{sx}	=	58.4 kNm	$\therefore OK$	DCT 2, Table 5.2-2 (2) (A)
Max Axial Compression/Tension Force	N^*	=	32.7 kN		
Section Capacity	ϕN_s	=	688 kN	$\therefore OK$	DCT 2, Table 6-4(2)
	ϕN_{cx}	=	512 kN	$\therefore OK$	DCT 2, Table 6-4(2)(A)
	ϕN_{cy}	=	255 kN	$\therefore OK$	DCT 2, Table 6-4(2)(B)
	n	=	$N^*/\phi N_s$		
		=	0.0475		
Moment Capacity	ϕM_{rx}	=	58.4 (1-n)	$\therefore OK$	DCT 2, Table 8-4(2)
		=	55.63 < 58.4 kN		
	ϕM_{ry}	=	23.5 (1-n)	$\therefore OK$	DCT 2, Table 8-4(2)
		=	22.384 < 23.5	[for both Tension and Compression]	
Shear Force	V*	=	23 kN		
	ϕV	=	355 kN > V*	$\therefore OK$	DCT 2, Table 5.2-2(2)(A)
In-plane/Out of Plane Buckling		=	$M^*/\phi M_{sx}$		
		=	$+ N^*/\phi N_s$	$\therefore OK$	
			0.514 < 1		

Appendix C – Pad Footing Design Calculations

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	Justin Lim				
		Pad footing design			
	Code	**Calculations**	**value**	**Unit**	**Section**
		Column load	269.43	kN	
		Moment	27.1	kN/m	
		Bearing capacity (SWL of pad footing)	150	kPa	
		Ultimate bearing capacity, qu	450	kPa	
		Factor of safety	3		
		Depth of the footing	1.5	m	Assumed
		depth of water table from the ground	2	m	
		Soil cohesion, c	0	kPa	Assumed
		Friction angle, ϕ	30		Assumed
	AS2870	Allowable bearing pressure qall			
		qall =qu/FS	50	kPa	
		Assume a square footing			
		qall= Pg/ A - ud			
		50 = (1.1 X 269.43) / B^2 - 0			
		B	2.44	m	
		Design for bearing capacity using Meyerhof method			
		Check for eccentricity			
		eB = M/P			
		eB = 27.1/269.43	0.100583		
		B/6 = 2.44/6	0.4		
		Check for moment eccentricity is ok!			
		Bearing capacity factors			
		For site class A ϕ= 30	30		

$\phi°$	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.49	1.6	0.1
10	8.34	2.5	0.4
15	10.97	3.9	1.1
20	14.83	6.4	2.9
25	20.71	10.7	6.8
26	22.25	11.8	8.0
28	25.79	14.7	11.2
30	30.13	18.4	15.7

		Nc	30.13		
		Nq	18.4		
		Nγ	15.7		
		Shape factors			
		B' = B= 2eB= 2.4- (2 x 0.1005827)	2.2	m	
		L' = L	2.4	m	

$$N_\phi = \tan^2(45° + \phi°/2)$$

		N Ø =tan^2 (45 + 30/2)	3		

Factor	Meyerhof
s_c	$1+0.2N_\phi\left[\frac{B}{L}\right]$
s_q	$1+0.1N_\phi\left[\frac{B}{L}\right]$ and 1 for $\phi=0$
s_γ	Same as s_q

		Sc	1.55		
		Sq	1.27		
		Sγ	1.27		
		Depth factors			
		Assume footing depth 1.5m			

d_c	$1+0.2\sqrt{N_\phi}\left[\frac{D_f}{B}\right]$
d_q	$1+0.1\sqrt{N_\phi}\left[\frac{D_f}{B}\right]$ and 1 for $\phi=0$
d_γ	same as d_q

dc	1.24	
dq	1.12	
dγ	1.12	

Load inclination factors

i_c	$\left[1-\frac{\alpha^\circ}{90}\right]^2$ for any ϕ
i_q	same as i_c for any ϕ
i_γ	$\left[1-\frac{\alpha^\circ}{\phi^\circ}\right]^2$ for $\phi>0$ and zero for $\phi=0$

ic	1	
iq	1	
ir	1	

$$q_{ult}=cN_cs_cd_ci_c+qN_qs_qd_qi_q+0.5\gamma_bBN_\gamma s_\gamma d_\gamma i_\gamma$$

$\gamma b =$ $\bar{\gamma}=\gamma'+\frac{d}{B}(\gamma-\gamma')$

Yb=	12.0	
qult=	728.07	kPa
Actual allowable stress, qall qall= qult/ FOS = 854.36/3	242.69	kPa
Applied stress qapp-max (P/B^2)(1 +6eb/B) [(1.1*269.43)/2.4^2]*[(1+6*0.1005827)/2.4]	34.38	kPa
qapp- max < qall therefore, the footing is safe, 34.377< 242.69 **Ok**		
Structural design e*= M*/P* 27.1/269.43	0.101	m
q*u-max =P*/A((1 +6eb/B) [(1.1*269.43)/2.4^2]*[(1+6*0.100582)/2.4]	34.38	kPa
q*u-min =P*/A((1 -6eb/B) [(1.1*269.43)/2.4^2]*[(1-6*0.100582)/2.4]	8.50	kPa
Shear at critical location d from centre of column qu*= (34.377+27.44767)/2	30.9	kPa
Column width = 305 mm		
Check beam shear to obtain footing thickness (34.377-8.50074)/2400 = (34.377-x)/[(2400/2)-(305/2)] x=	27.44767	kPa

Max shear in terms of d		
V* = (30.912335 * 2400 * 10^-3)*(2400/2 - 305/2 - d)		
V* = 74.1896(1047.5 - d)		
Vuc= β1 Bd (Ast/Bd * f'c)^1/3		
Vuc =0.7 * 1.1 * 2400 * d * (0.0035 * 30)^1/3	871.8298d	
V* < Vuc		
d > 82.148mm		
Assume a cover of 75mm and 20mm reo thickness		
t =82+ 75 + 20/2	167	mm
Use 200mm		
d = 115mm	115	mm
Checking punching shear strength		
Shear perimeter = (B+d) X (L+d)		
420 X 420		
V* =30.912335*10^-3 (2400 * 2400 - 700 * 700)	172.6	kN
Vud = U*d*fcv		
U = 2*(420+420)	1680	mm
d	115	mm
$f_{cv} = 0.17(1+\frac{2}{\beta_h})\sqrt{f_c'} \le 0.34\sqrt{f_c'}$		
f'c	30	Mpa
0.17(1+2/(Bh))sqrt(f'c)	2.8	Mpa
0.34sqrt(f'c)	1.86	Mpa
fcv =>	1.86	Mpa
ϕ Vud = (0.7)(1680)(115)(1.86)	251.546	kN
V* < ϕ Vud		
Punching shear is ok		
Checking tensile reinforcement for bending moment		
qu* = 30.912335		
M*= 30.912335/1000 * 2400 (((2400/2)-(305/2))^2)/2	40.7	kNm
Ast = M*/0.68dfsy		
(40.7 * 10^8)/(0.68 * 167 * 400)	89600.21	mm2
Ast-min = 0.0035bd		
Ast-min = 0.0035 * 2400 * 115	966	mm2
Ast-min governs		

No. of bars	Area, A_{st}, (mm²) of Bars of Type-Size (Grade and mm)								
	Plain Bars		Deformed Bars						
	R6	R10	Y12	Y16	Y20	Y24	Y28	Y32	Y36
1	31	80	110	200	310	450	620	800	1,020
2	62	160	220	400	620	900	1,240	1,600	2,040
3	93	240	330	600	930	1,350	1,860	2,400	3,060
4	124	320	440	800	1,240	1,800	2,480	3,200	4,080
5	155	400	550	1,000	1,550	2,250	3,100	4,000	5,100
6	186	480	660	1,200	1,860	2,700	3,720	4,800	6,120
7	217	560	770	1,400	2,170	3,150	4,340	5,600	7,140
8	248	640	880	1,600	2,480	3,600	4,960	6,400	8,160
9	279	720	990	1,800	2,790	4,050	5,580	7,200	9,180
10	310	800	1,100	2,000	3,100	4,500	6,200	8,000	10,200

Choose **9 Y12**		
Spacing = (2400- 75*2 -12)/(9-1)	279.75	mm
280mm Spacing		

Appendix D – Connection Calculations

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Design For Rigid Connection between SHS Column and RHS Beam

For Residential Modules, 200*100*4 RHS will be welded to the 125*6 SHS columns.

Max Moment	M^*	=	27.24 kNm		
Max Shear	V*	=	44.42 kN		
Find max design shear force.	v_w^*	=	$\sqrt{\left(\frac{V^*}{2*Depth_{RHS}}\right)^2+\left(\frac{M^*\times y_{RHS}}{I_{weld}}\right)^2}$		
Force Due to Shear	v_v^*	=	$\frac{V^*}{2*Depth_{RHS}}$		
		=	$\frac{44.42\times10^3}{400\times t}=111.05\ N/mm$		
Force Due to Moment	v_m^*	=	$\frac{M^*\times y_{RHS}}{I_{weld}}$		
	I_{weld}	=	$2\times(I_{RHS\ length}+I_{RHS\ width})$		
		=	$2\times(100^3\times t+\frac{200^3}{12}\times t)$		
		=	$3.333\times10^6\times t\ mm^4$		
	v_m^*	=	$\frac{27.24\times10^6\times100}{3.333\times10^6\times t}$ $=818.02\ N/mm$		
	v_w^*	=	$\sqrt{111.05^2+818.02^2}$		
		=	0.8255 kN/mm		
Choose Weld	ϕv_w	=	0.840 kN/mm – GP Weld: - t_w = 6mm, - f_{uw} = 550 MPa > 0.8255 kN/mm Use 6mm E55xx GP Weld	$\therefore OK$	DCT Table 9.9

Design For Rigid Connection between UC Column and UB Beam

The UB Beams will be Connected to the UC Column by welding a plate to the end of the UB Beam and then Bolting the Plate to the Flange of the UC Column with 8, 8.8/S bolts.

For UB Beam Welded to Plate:

Max Moment	M^*	=	16.44 kNm	
Max Shear	V*	=	19.19 kN	
Find max design shear force.	v_w^*	=	$\sqrt{\left(\frac{V^*}{2*Depth_{web}}\right)^2+\left(\frac{M^*\times y_{top\ of\ web}}{I_{weld}}\right)^2}$ $\frac{M^*\times y_{top\ of\ flange}}{I_{weld}}$	Or
Force Due to Shear	v_v^*	=	$\frac{V^*}{2*Depth_{web}}$	

		=	$\dfrac{19.19 \times 10^3}{2 \times 188}$		
		=	$51.03\ N/mm$		
Force Due to Moment	v_m^*	=	$\dfrac{M^* \times y_{top\ of\ web}}{I_{weld}}$		
	I_{weld}	=	$2 \times I_{web\ depth} + 4 \times I_{flange\ outstand} + 4 \times I_{flange\ thickness} + 2 \times I_{flange\ width}$		
		=	$t\begin{pmatrix} 2 \times \left(\frac{188^3}{12}\right) + 4 \times \left(\frac{134 - 5.8}{2}\right) \times \left(\frac{188}{2}\right)^2 \\ +4 \times 7.8 \times \left(\frac{188 + 7.8}{2}\right)^2 + 2 \times \left(\frac{203}{2}\right)^2 \times 134 \end{pmatrix}$		
		=	$6.433 \times 10^6 \times t\ mm^4$		
	v_m^*	=	$\dfrac{16.44 \times 10^6 \times 188}{6.433 \times 10^6 \times t} = 480.45\ N/mm$		
	v_w^*	=	$\sqrt{(51.03)^2 + (480.45)^2}$	Or	
		=	483.15 N/mm		
	v_w^*	=	$\dfrac{M^* \times y_{top\ of\ flange}}{I_{weld}}$		
		=	$\dfrac{16.44 \times 10^6 \times 203}{6.433 \times 10^6 \times t}$		
		=	518.78 N/mm = 0.519kN/mm		
	ϕv_w	=	0.657 kN/mm – GP Weld: - t_w = 6mm, - f_{uw} = 430 MPa > 0.519 kN/mm Use E43xx GP Weld	$\therefore OK$	DCT Table 9.9

For Plate Bolted to UC Column Flange

Design Shear Force Per Bolt	V*	=	19.19 kN Or		
		=	$\phi 0.15 \times V_B = 0.15 \times 204 = 30.6\ kN$ Use $\frac{30.6}{8} = 3.825\ kN$		
Design Moment	M	=	$\max(\phi 0.5 \times M_B, M^*)$ (using grade 300 steel) $\phi 0.5 \times M_B = \phi 0.5 \times f_y \times Z_{e,UB}$		
		=	$0.9 \times 0.5 \times 300 \times 259 \times 10^3$		
		=	37.296 kNm > M* = 16.44 kNm		
			$\therefore Design\ for\ 37.296$		

Max Load Per Bolt	N*	=	$\frac{MyA}{I}$	
		=	$\frac{37.296 \times 10^6 \times 150 \times A}{100 \times 10^3 \times A}$	
		=	55.944 kN	

Plate to Have 8 bolts, 4 Rows of 2 Bolts.
Vertical pacing between Rows is 100 mm
Horizontal Spacing between Bolts in rows is 150 mm.
Horizontal and Vertical Edge Spacing 50 mm from edge to nearest bolt.

Shear Design Capacity	ϕV_f	=	$\phi \times 0.62 \times f_{uf} \times k_{rd} \times k_r \times A_c$	AS4100 Clause 9.2.2.1
	ϕ	=	0.8	
	K_{rd}	=	1	
	K_r	=	1 (Distance Between Furthest Flanges = 300mm)	
	F_{uf}	=	830 MPa (8.8/S Bolts)	
	A_c	=	Core Area to be determined based on bolt sizing	
	ϕV_f	=	$0.8 \times 0.62 \times 1 \times 1 \times 830 \times A_c$	
		=	411.68 A_C	
Tension Design Capacity	ϕN_{tf}	=	$\phi \times f_{uf} \times A_s$	AS4100 Clause 9.2.2.2
	ϕ	=	0.8	
	f_{uf}	=	830	
	A_s	=	Tensile Stress Area to be determined based on bolt sizing	
			Bolt Prying Effect to be considered = 1.3	
			$\frac{0.8 \times 830 \times A_s}{1.3}$	
			= 510.77 A_c	
Bolts in Combined Shear			$\left(\frac{V^*}{\phi V_{tf}}\right)^2 + \left(\frac{N^*}{\phi N_{tf}}\right)^2 \leq 1.0$	AS 4100, Clause 9.2.2.3

Bolt	$A_c\ (mm)$	$A_s\ (mm)$	$A_o\ (mm)$	ϕV_f	ϕN_{tf}	$\left(\frac{V^*}{\phi V_{tf}}\right)^2 + \left(\frac{N^*}{\phi N_{tf}}\right)^2$
M16	144	157	201	59.3	80.2	0.4909
M20	225	245	314	92.6	125.1	0.2016
M24	324	353	452	133.4	180.3	0.0971

Can choose any bolt sizes therefore, as they all satisfy the conditions.
For this use 8.8/S M20 bolts as there is sufficient capacity

Thickness of Butt Plate	M^*_{plate}	=	$F_1 \times \frac{e}{2}$	9.2.2.3
	F_1	=	55.994 (Max Tensile Force)	
	e	=	50 mm (Distance from UB Flange to Outer Bolts)	
	M^*_{plate}	=		

$$55.994 \times 10^{-3} \times \frac{5}{2} = 1.3986$$

$$\phi M_{plate} = \phi \times f_y \times Z_e$$

$$0.9 \times 300 \times \frac{125}{6} \times t^2$$

$$\phi M_{plate} > M^*_{plate}$$

$$t > \sqrt{\frac{1.3986 \times 10^6}{0.9 \times 300 \times \frac{125}{6}}} = 15.8\ mm$$

Prying Effects – Thickness min = 1.25 Bolt Area

$$1.25 \times 20 = 25 \text{ mm}$$

$$\therefore \text{use 25 mm plate, Grade 300}$$

Check Spacing Requirements:

Min Edge Distance $1.5 \times d_f = 1.5 \times 20 = 30\ mm$

Min Gauge Distance $2.5 \times d_f = 2.5 \times 20 = 50\ mm$

$$\therefore requirements\ satisfied$$

Checking Bearing of Plate

$$V_b^* \leq \phi 3.2 \times d_f \times t_p \times f_{up}$$

$$= 0.9 \times 3.2 \times 20 \times 25 \times 440 = 704\ kN$$

As 4100, 9.2.2.4(1)

$$V_b^* = 2.4 \text{ kN per bolt}$$

$$\therefore Sufficient$$

$$a_e = 3.2 \times d_f$$

$$= 64 \text{ mm} > 50 \text{ mm}$$

$$\therefore Need\ to\ be\ Checked$$

$$V_b^* \leq \phi a_e \times t_p \times f_{up}$$

$$= 0.9 \times 50 \times 25 \times 440 = 495\ kN$$

AS 4100, 9.2.2.4(2)

$$\therefore Sufficient$$

Determine Whether Stiffeners Required

$$M_y = f_y \times Z$$

$$= 320 \times 232 \times 10^3$$

$$= 74.24 \text{ kNm} > 16.44 \text{ kNm}$$

$$\sigma_{max} = \frac{16.44 \times 10^6}{232 \times 10^3}$$

$$= 70.86 \text{ MPa}$$

$$\sigma_{ave} = 70.86 \times \frac{101.5 - \frac{7.8}{2}}{101.5}$$

$$= 68.137$$

$$F_f = 68.137 \times 7.8 \times 134$$

$$= 71.217 \text{ kN}$$

$$\sigma_{web} = \frac{70.86}{2} \times \frac{101.5 - \frac{7.8}{2}}{101.5} \times \frac{188}{2} \times 5.8$$

$$= 18.574 \text{ kN}$$

Yield Capacity of Column	b_{bf}	=	$7.8 + (15.4 + 25) \times 2 \times 2.5$ =209.8 mm
	ϕR_{by}	=	$0.9 \times 1.25 \times 209.8 \times 5.8 \times 320$ $= 438.06$ kN > Flange Force $\therefore$ *no stiffeners required* Stiffeners Will still be included for Connection of secondary 200 UB 25.4 Beam, to create 2-way portal frame.

Plates to be welded to the flanges of the 310 UC Columns
Using 6mm E43xx GP Weld for Both Plates
Stiffener Connections

310 UC 96.8 Flange Outstand Width	$b_{outstand}$	=	$\frac{305 - 9.9}{2} = 147.66\ mm$
Web Depth	d_{web}	=	277 mm

Design for Connection Between SHS Columns

The SHS Columns of the Modules are Pin Connected to Each other, by Telescoping Smaller 100*4 SHS sections into the Larger 125*6 SHS Columns and Bolting together.

Base of 125*6 SHS Welded with 5 mm plate and then 100*4 mm SHS Welded on Top.

Max Moment	M^*	=	0 kN
Max Shear	V*	=	17.79 kN Through Entire Connected Section
Find max design shear force.	v_w^*	=	$\frac{V^*}{2 * Depth_{SHS}}$
Force Due to Shear	v_v^*	=	$\frac{17.79 \times 10^3}{2 \times 100} = 88.95\ N/mm$ 0.657 kN/mm – GP Weld: - t_w = 6mm, - f_{uw} = 430 MPa > 0.519 kN/mm Use E43xx GP Weld

Use Same Weld Size for Both SHS and Plate Welds.

For Bolt Connections
Assumed to be using 2 M20 Bolts to Fix Modules Together – Not Designed to Carry Loads

Design for Connection Between UC Column and SHS Columns

The UC Column will be connected by welding a plate to the end of the UB Beam and then Bolting the Plate to Base Plates, Welded to the Base of SHS Columns

For UC Column Welded Plate:

The UC Column will be connected by welding a plate to the end of the UB Beam and then Bolting the Plate to Base Plates, Welded to the Base of SHS Columns

For UC Column Welded Plate:

Max Moment $M^* = 50.44$ kNm

Max Shear V* = 14.55 kN

Find max design shear force. $v_w^* = \sqrt{\left(\frac{V^*}{2 * Depth_{web}}\right)^2 + \left(\frac{M^* \times y_{top\ of\ web}}{I_{weld}}\right)^2}$ Or $\frac{M^* \times y_{top\ of\ flange}}{I_{weld}}$

Force Due to Shear $v_v^* = \frac{V^*}{2 * Depth_{web}}$

$$= \frac{14.55 \times 10^3}{2 \times 277}$$

$$= 26.26\ N/mm$$

Force Due to Moment $v_m^* = \frac{M^* \times y_{top\ of\ web}}{I_{weld}}$

$$I_{weld} = 2 \times I_{web\ depth} + 4 \times I_{flange\ outstand} + 4 \times I_{flange\ thickness} + 2 \times I_{flange\ width}$$

$$= t\left(\begin{array}{c} 2 \times \left(\frac{277^3}{12}\right) + 4 \times \left(\frac{305 - 9.9}{2}\right) \times \left(\frac{277}{2}\right)^2 \\ +4 \times 15.4 \times \left(\frac{277 - 15.4}{2}\right)^2 + 2 \times \left(\frac{308}{2}\right)^2 \times 305 \end{array}\right)$$

$$= 30.38 \times 10^6 \times t\ mm^4$$

$$v_m^* = \frac{50.44 \times 10^6 \times 277}{30.38 \times 10^6 \times t} = 459.84\ N/mm$$

$$v_w^* = \sqrt{(26.26)^2 + (459.84)^2}$$

$$= 460.59\ \text{N/mm}$$

Or

$$v_w^* = \frac{M^* \times y_{top\ of\ flange}}{I_{weld}}$$

$$= \frac{50.44 \times 10^6 \times 308}{30.38 \times 10^6 \times t}$$

$$= 511.3\ \text{N/mm} = 0.511\ \text{kN/mm}$$

ϕv_w = 0.657 kN/mm – GP Weld:
- t_w = 6mm,
- f_{uw} = 430 MPa

> 0.519 kN/mm

Use E43xx GP Weld

$\therefore OK$ DCT Table 9.9

Column to Footing Connection

	Justin Lim				
		Column to Footing Connection			
	Code	**Calculations**	**value**	**Unit**	**Section**
		Column load - Nc*	269.43	kN	
		Moment - Mx*	27.1	kN/m	
		The applied load and moment are equivalent to an axial load of 269.43kN acting at an eccentricity			
		e = Mx*/Nc*	102.8	mm	
		102.8mm from the centreline of the column			
		Assume 4M20 bolts			Assumed
		Try 350 x 350 base plate - grade 300 steel			
		offset distance from CL columns to bolts			
		350-270	80	mm	
		Bolt spacing < 2(offset + bolt diameter)			
		Try 20mm bolts			
		2(80 + 20) =	200	mm	
		Bolt spacing < 200mm			
		Use 120mm spacing			
		Plastic analysis and effective area design			
		310UC96.8			
		Column Depth	305	mm	
		Flange thickness	15.4	mm	
		Take moment about centroid of the columns compression flange			
		Ntb* = [(27.1 * 10^3) - (269.43 * 0.5) * (305-15.4)]/[(350 + 0.5(305-15.4)]	-24	kN	
		Use 4 M20 bolts			
	DCT Table 9.3	M20 ϕ Ntb	163	kN	
		(ϕ Ntb)(2)	326	kN	
		ϕ Ntb > Ntb*			
		Check is OK			
		Compressive load on concrete under compression flange			
		24 + 269.43	293.43	kN	
		Design bearing strength of grout and concrete under base plate = 0.51 f'c			
		Assumed concrete strength f'c	30	MPa	
		(0.51)(30)	15.3	MPa	
		Effective area of baseplate required			
		(293.43 * 10^3)/15.3	19178.43	mm2	
		Length of effective strip = 19178.4/500	38.36	mm	
		Plate thickness			
		$t_i \geq \sqrt{\frac{4\ M_c^*}{\phi\ b_i\ f_y}}$			
		Mcf* = (15.3)(500)*[(38.36-15.4)/2]^2/2	0.504098	kNm	
		Design cantilever moment at ther edge of the columns tension flange			
		Mtf* = (24)(225-305/2) * 10^3	1.74	kNm	
		Mc* = MAX[0.5, 1.74]	1.74	kNm	
	AS4100 Table 2.1	ti ≥ sqrt[(4 * 1.74 * 10^6) / (0.9 * 500 * 300)	7.18	mm	
		Minimum 7.1mm thick plate			
		Choose 30mm thickness			
		Column to base plate weld design			
		Column's tension flange weld:			
		$N_{tf}^* = \frac{(1-k_{mw})M^*}{d_c - t_{fc}} - \frac{(1-k_c)N_c^*}{2}$ *(tension flange design force)*			
		kmw = lwx/lx			

		kw = web area / total cross sectional area				
		kmw = (9.9 *(305 - 2 * 15.4)^3/12) / (223 * 10^6)	0.0762			
		kw = (305 - 2*15.4)/12372	0.022163			
		Ntf* = [(1-0.076)*27.1*10^3]/(305-15.4) - [(1-0.022)*(269.4)]/2	45.27	kN		
		Lw = total run of fillet weld around the tension flange				
		Lw = 305 + (305-9.58)	600.42	mm		
		Vz* = Ntf*/Lw				
		Vz* = 45.27/600	0.07545	kN/mm		
		tt = tw/sqrt(2)				
		tt = 9.9/sqrt(2)	7			
		ϕVw = ϕ * 0.6 * fuw * tt				
		ϕVw = (0.6)(0.6)(410)(7)(10^3)	0.626	kN/mm		
		ϕVw > Vz*				
		Therefore, ok				
		GP welds will be used				
		6mm CFW GP Welds to be used				