University of Southampton

Design Report
CENV6160 Architectural Engineering Project

01
Portsmouth Coastal Marketplace

This sustainable tourist attraction exemplifies the perfect combination of traditional working market programs and contemporary features, creating a strong public connection to Portsmouth's beautiful waterfront.

Portsmouth, a city located on the south coast of England, is a popular tourist destination known for its rich maritime history and stunning seafront views. However, there is a building on the seafront called “The Pyramids Centre” that has become obsolete and currently underused. The building is not designed on a human scale, and it fails to take advantage of the beautiful views out towards the sea. Rather, the building is introspective, making it uninviting to the public. Ultimately, the building makes access horizontally across the coast from the castle field to the rock gardens very difficult.

Therefore, the council recognizes the need for a new development that can attract tourists and create a new convivial space that is inclusive to everyone in Portsmouth. This report focuses on the development of a central hub, consisting of a 4850m2 deck and a 15m tall canopy roof, that improves coastal connectivity whilst taking advantage of key views.

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Figure 1 – South view of proposal

Figure 2 – West view of proposal

1.1 Introduction to the project

Centralised along the picturesque seafront of Portsmouth, there stands an obsolete building known as the Pyramids. Constructed in 1988/1989, this unique building boasts a predominantly concrete structure supported by a central steel truss roof. Over the years, the space has been put to good use, most recently undergoing a £4 million investment to transform it from a swimming pool into a vibrant children's play area. However, the Pyramids Centre is near the end of its design life (demolition due 2030). The maintenance costs associated with the building are high, and substantial investment would be required to elevate its standard to a more desirable level, especially if it is to continue serving its current leisure purposes. Therefore, the city council recognises the need for a new development that ensures sustainability and guarantees long-lasting profitability.

The existing play centre serves as a "weatherproof" haven for families, but the vision for the redevelopment extends far beyond its current offerings. With its annual influx of approximately 11 million tourists, Portsmouth seeks to create a focal point along its iconic seafront that will act as a magnetic attraction for visitors throughout the year. A thriving hub accessed from all sides of the city is the key to achieving this ambitious goal.

To meet the demands of both sustainability and profitability, the council envisions a groundbreaking redevelopment project for the Pyramids. The aim is to create a space that captivates tourists and provides an exceptional experience for locals and visitors alike, offering a diverse range of activities and amenities that can be enjoyed seasonally.

This project will revitalise the seafront and serve as a catalyst for economic growth, positioning Portsmouth as a premier destination along the British coastline. In this proposal, the report will explore the visionary plans for the redevelopment by embracing the principles of sustainability, accessibility, and adaptability. The project aims to improve the seafront image of Portsmouth while preserving its rich heritage and natural beauty.

Ultimately, the proposal will focus on providing key views of the seafront, especially where the current building has failed to utilise them. In summary, this report will cover the design of a raised deck with an elegant 15m tall canopy roof, designed for multi-use, tailored to the council's needs. The height, size, volume, proportion, and arrangement of the building are determined based on their potential effects on the overall ambience of Southsea Castle, the conservation area (rock garden and castle field), and the surrounding townscape and landscape. The council outlined that Paying particular attention to the potential impact on the Special Protection Area (SPA) and the population of Brent geese and Solent waders is essential.

1.2 Project brief

To replace the outdated 'Pyramid's Centre' with an adaptable and iconic structure that accommodate a mix of uses.

1.3 Assumptions

The project assumes that the existing foundation will not affect the soil properties or conditions on site. Using the existing structure for development has not been considered due to lack of engineering and architectural drawings of the 'Pyramids Centre.'

Figure 3 – View of top deck

1.4 Objectives of the Project

Sustainability objectives

- To use sustainable materials that lower the embodied carbon of the project
- To consider strategies that will maximise the buildings use
- To develop strategies that reduce the use of concrete in foundations.
- To ensure that the building is easily maintained in order to maximise its design life.
- To explore the use of renewable energy

General objectives

- To utilise key views of the seafront
- To be conscious of surrounding building's views
- Enhance the ease of movement across the seafront
- Provide a multi-use space, that can be enjoyed seasonally
- Provide iconic architecture that seeks to honor the site's past
- Ensure that accessibility is at the forefront of the design

1.5 Project Goals

Short Term

- Attract a diverse range of vendors to create a vibrant marketplace atmosphere.
- Showcase local talent to enrich the cultural experience.

Intermediate Term

- Create a welcoming and inclusive community atmosphere for visitors, vendors, and residents to connect and engage.

Long Term

- Create a bustling and iconic central hub that attracts visitors to boost the local economy.
- To create a versatile environment that can be easily modified to accommodate evolving needs and objectives, ensuring the space remains relevant, functional, and aligned with the council's vision and goals for the community.

2.1 Site Analysis

SWOT ANALYSIS

Strengths

- The coast of Portsmouth offers stunning natural beauty with its picturesque beaches, cliffs, and panoramic views of the sea, attracting both locals and tourists.
- The coast benefits from established tourism infrastructure, including hotels, restaurants, shops, and leisure facilities, supporting a vibrant tourism industry.

Weaknesses

- Like many coastal destinations, Portsmouth's coast may experience seasonal fluctuations in visitor numbers, with higher demand during summer months and potential lower footfall in off-peak seasons.
- The capacity of the beaches along the coast may be limited during peak periods, leading to overcrowding and potential dissatisfaction among visitors.

Opportunity's

- There is an opportunity to provide a centralised hub that attracts visitors and locals from far.
- Embracing sustainable tourism practices and promoting eco-friendly initiatives can attract environmentally conscious tourists and enhance the reputation of Portsmouth's coast as a responsible destination.

Threats

- The coast of Portsmouth is vulnerable to the impacts of climate change, including rising sea levels, which can lead to erosion, flooding, and infrastructure damage.
- Portsmouth faces competition from other coastal destinations both within the UK and internationally, which may impact its ability to attract tourists and generate revenue.

Figure 4 – Site Analysis Map

SITE ADDRESS
Clarence Esplanade, Southsea, Portsmouth, Southsea PO5 3ST

TOTAL AREA
Site -16,700 m2
The Pyramid Centre – 6,900 m2

LOCATION
Latitude 50.779232, -1.085072

KEY
- Site Boundary
- Parking Spaces
1 - 132 Spaces
2 - 141 Spaces
- Road Access
- Sun Path Summer Solstice
- Bore Hole Data:
0.4 m Topsoil
2.5 m Gravely Sand
6.6m Stiff Blue Clay

A – The Castle
B – Castle Field
C – Rock Garden

Figure 5 – View 1

Figure 6 – View 2

Figure 7 – View 3

Figure 8 – Map of UK

The proposed site is in South Sea Portsmouth. Site analysis, Figure 4, has provided information for the design of the redevelopment, from wind speeds for wind loading, to the sun path for pv panel organization. The result of the site analysis will be outlined later in the report.

The Pyramids Centre is a large facility with multiple levels and spacious interior areas. It houses various amenities and facilities, including swimming pools, a gym, a leisure center, conference rooms, and event spaces. The site of the Pyramids Centre also has direct access to the adjacent Southsea Beach. The Pyramids Centre is surrounded by a mix of natural and built environments. On one side, it faces the open sea, providing a stunning view of the sea.

On the other side, Southsea Common offers expansive green spaces (B and C, Figure 4), providing a peaceful and picturesque setting. While parking is mentioned as a weakness, the site does have parking facilities available nearby. Visitors can find parking spaces in the vicinity, although availability may be limited during peak times. However, this will be analysed, as Portsmouth Council has started to encourage facilities with limited parking to encourage greener travel.

Currently the site is blocked due to overgrown bushes, mounds and lack of paths on the west side of the site, as shown in Figure 6. The aging infrastructure, Figure 5, has led to functional issues, decreased aesthetic appeal, and potential safety concerns for visitors. Thus, the 16,700 m2 site isn't currently used to its maximum potential.

2.2 Urban Studies

- University
- Open/Green Space
- Residential
- Commercial
- Shopping
- Naval
- Site

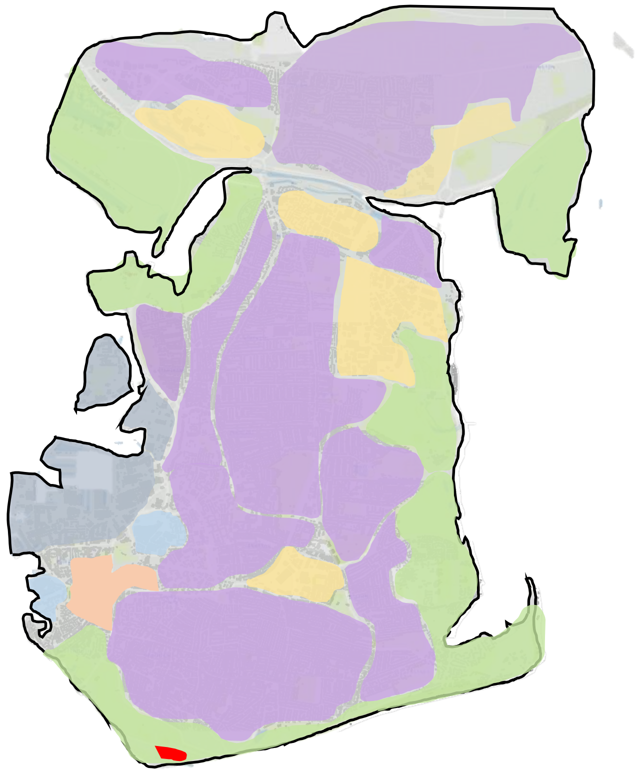


Figure 9 – Portsmouth City Zones (Group Work)

- Gunwharf Quays
- The Spinnaker Tower
- Clarence Amusements
- Southsea Castle
- The Rose Gardens
- Site
- Southsea Rock Garden
- South Parade Pier
- Canoe Lake
- Fort Cumberland
- Southsea Model Village
- Blue Reef Aquarium

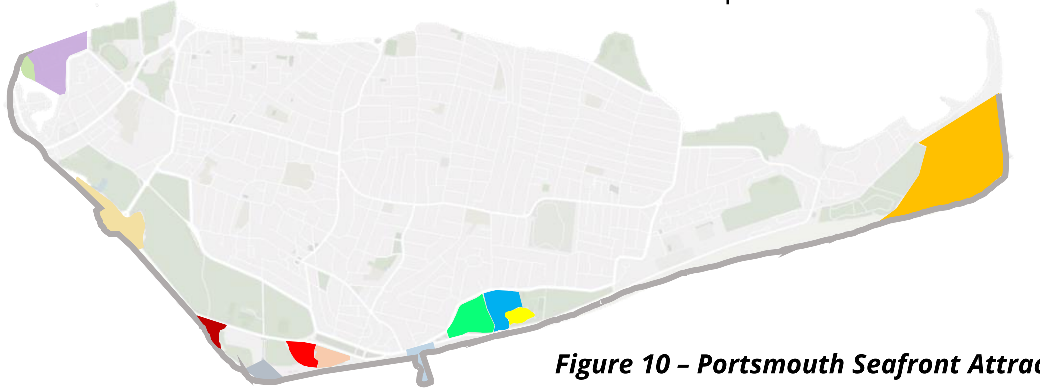


Figure 10 – Portsmouth Seafront Attractions (Group Work)

PESTLE ANALYSIS

P-Political
The site is focalised at the south point of the seafront therefore a redevelopment will improve the quality of the seafront public realm, thus bringing an improvement to surrounding housing and facilities.

E-Economics
The surrounding seafront attractions, shown in figure 10, can draw in a variety of visitors to the seafront and ultimately to the devlopment. The collaboration with these facilities will boost the local economy.

S-Sociological
As reported by a local resident, there are areas in Portsmouth that have residents' northwest that never go to the seafront due to the divide in social class. This proposal aims to bring people of all social status and all ages to a place where they can feel safe and included.

T-Technological
The deck of the structure is designed to undergo a variety of loading conditions, providing a space that can be used for all purposes.

L-Legal
The seafront masterplan has addressed the need to redevelop the pyramids and is in partnership with the owners to make this happen in a way that is viable to all involved.

E – Environmental
Incorporating eco-friendly materials, utilizing renewable energy sources, and minimizing waste generation align with environmental goals. Integrating green spaces or landscaping within the market area can enhance its aesthetic appeal and contribute to a pleasant atmosphere.

2.3 Planning – Frontage 4

Figure 11 – Frontage 4 (Image from Council website)

Frontage 4 has been designed with the unique heritage of the Southsea Castle area in mind. The primary defense consists of a rock revetment and a small concrete wall to reduce the impact of waves. The landscaping plan incorporates timber seating, as well as features to promote biodiversity such as wildflower areas, bee posts, and tide pools. There will also be boulder play areas and sound tubes for children to enjoy.

With improved lighting, the Southsea Castle area will become a more relaxing, exciting, and enjoyable space for people to spend time. The reports proposal aims to provide an open space for the masterplan to incorporate similar elements around the structure. Ideally the demolition of the pyramid centre can happen once the construction of frontage 4 ends, to avoid collision in plannings.

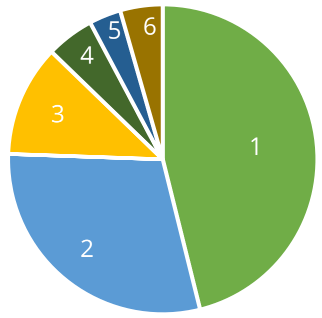


Figure 12 – Proportion of entrys

Seafront masterplan consultation

1 - Scope for an element of residential development if required as an enabling use – 88 entries
2 - Opportunity to have a building or collection of buildings which could accommodate a mix of uses, 53 entries
3 - Building or buildings should have strong frontages onto the promenade and Clarence Esplanade, 21 entries
4 - Public space around the Pyramids buildings should be enhanced as part of any development, including consideration of how it integrates with new sea defenses, 9 entries
5 - Creation of a physical and/or visual 'green link' between Castle Fields and Rock Gardens, 6 entries
6 - Something else, 8 entries

The respondents had general comments in response to comment number**:**
2 - Do not want over-development, keep open space. Do not want a hotel. Need to remain as public facilities which are low cost and accessible to all. Not needed. Demolish The Pyramids, in bad condition. Wrong location.
4 - The Pyramids should be demolished. Leave the area as it is, as open as possible. Do not destroy the rock gardens. Make it accessible for wheelchair-users. Concern about local wildlife. Develop the area in line with the rest of Southsea.

Overall, the proposal will consider the publics opinion and will try to keep as much open space as possible. The design will also be accessible with no areas on site where wheelchair-users can not access. In summary, it is important to consider incorporating a green link between the Rock Gardens and Castle Fields in the development project, both physically and visually. Additionally, the Rock Gardens themselves should be integrated into any development plans to improve pathways and enhance passive surveillance.

2.4 Historical Background

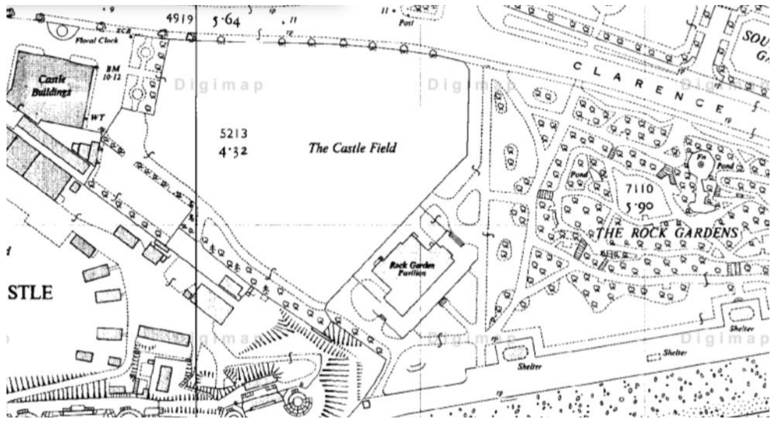


Figure 13 – Map of Site 1950

Figure 14 – Rock Garden Pavilion (Google Images)

Rock Garden Pavilion
In the 1950's the rock garden pavilion was a popular destination for concerts and events. The proposal aims to pay homage to the castle field pavilion by creating a fun and inviting space that connects itself to the rock garden like the original pavilion successfully achieved.

2.5 Site Selection

Figure 16, shows the movement of building layouts on site. With the pyramid and seafront vegetation currently only allowing 9% of the building's facade (highlighted in yellow) to see the sea. In order to persuade the residents that the intervention has a positive impact on their views, the percentage either needs to stay the same or increase. Figure 15, summarises the analysis and shows areas limited to building heights. The current intervention increases the residents view of the sea with only the overhang of the structure in the 'red zone'. Therefore, the building has a positive impact on the resident's view of the sea.

Figure 15 – View Analysis Summary

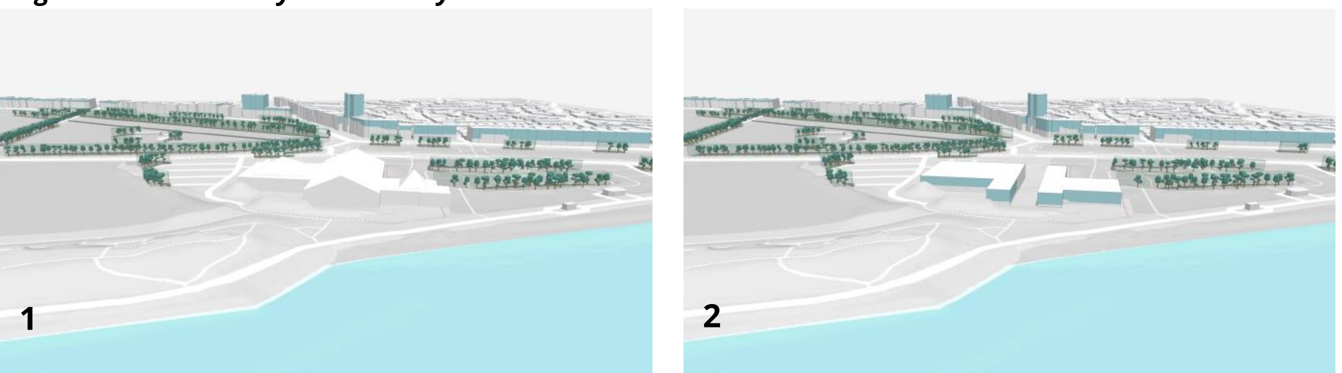
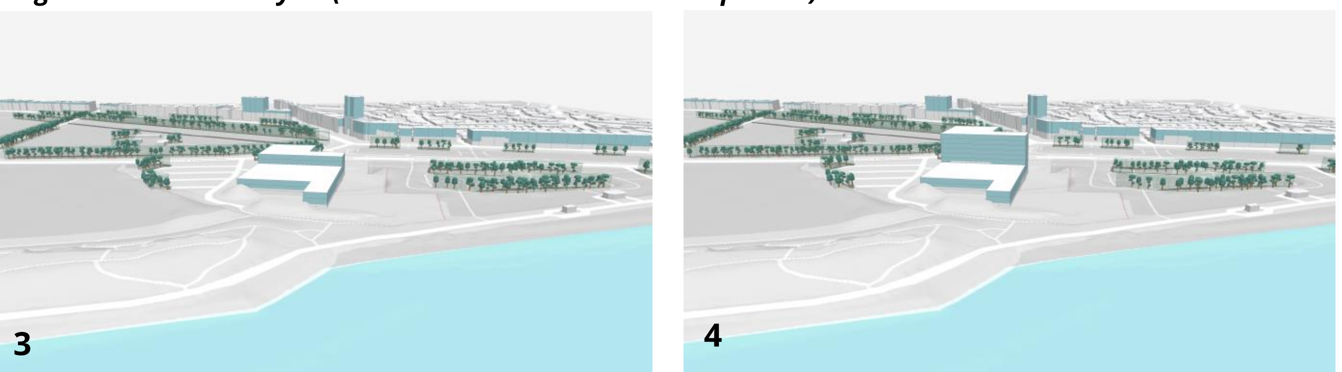


Figure 16 – View Analysis (Personal Contribution to Group Work)



- Site Boundary - The 'PC'

- Resident's Façade Under Analysis

- Area not affecting surrounding building's view of sea

- Area affecting surrounding building's view of sea

- Area limited to 9m height to not affect surrounding building's view of sea

1 - Height – 13 m, Buildings' façade that can view the sea – 9%

2 - Height – 9 m, Buildings' façade that can view the sea – 10%

3 - Height – 15 m, Buildings' façade that can view the sea– 10%

4 - Height – 27 m, Buildings' façade that can view the sea– 10%

2.6 Climate Conditions

Average temperatures and precipitation

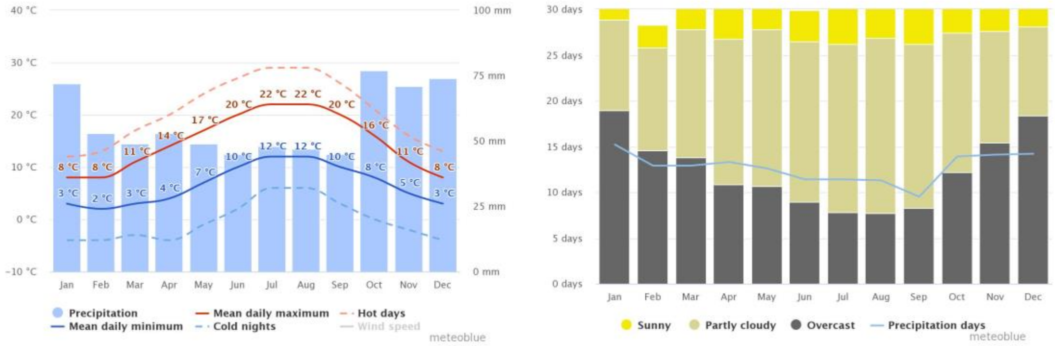


Figure 17 – Average Temperature and Precipitation

Portsmouth's temperature varies from an average of 22 degrees in Jul/Aug to 2 degrees in Dec/Jan. Thus, the building's enclosed spaces will be insulated, ventilated and well shaded in order to maintain a comfortable environment. Pipes and gutters are designed to withstand 75 mm of precipitation averaged from October.

Wind speed and Direction

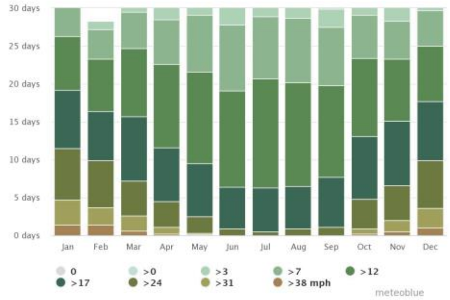


Figure 18 – Wind Speed

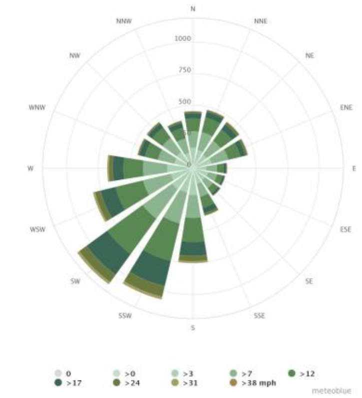


Figure 19 – Wind Direction

Wind speeds are found to be greater than 17 m/s in December, prominently in a SW direction. However, after analysing historical logs of wind speed in Portsmouth a design speed of 22 m/s is chosen to calculate wind loading on the proposed structure. The wind load calculated in Appendix A is **0.57** kN/m2. It is assumed that this load can act in all directions of the structure.

2.7 Geology

Figure 20 – Geology Map (Digimap)

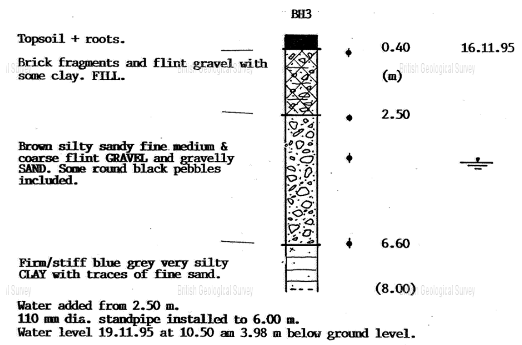


Figure 21 – Borehole data (Red dot)

Figure 20, taken from Digimaps, gives the Lithological Description: Glauconitic silty sands and sandy silts.

The borehole, summarised in Figure 4 Section 2.1, is shown in detail in Figure 21. These stratum values are used to calculated the foundation of the structure in Appendix A.

Figure 22 – Typical Cathedral Columns

Figure 23 – Nine-Square Grid House

Figure 24 – IQL Pavilion London

Figure 25 – Stansted Airport

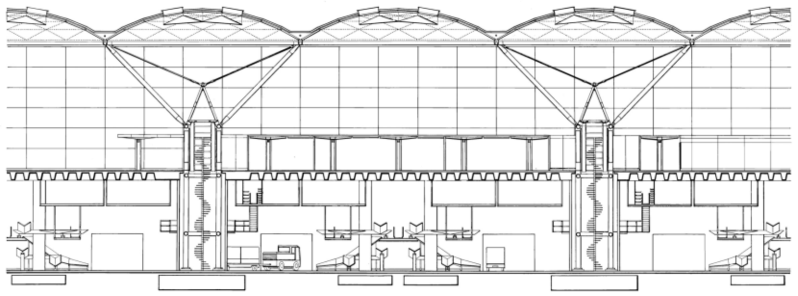


Figure 26 – Sydney Fish Market

Figure 27 – Sharjah Library

3.1 Precedent Studies

Stansted Airport – Foster and Partners
Stansted Airport showcases a sweeping roof structure supported by exposed steel trusses. Airports have a system where services are hidden underground rather than on the roof, seen in Figure 25, which results in a slenderer roof with less dead load. Taking inspiration from this system, the proposal will feature services and plant for the canopy, hidden in rooms under the large deck

Sharjah Library – Foster and Partners
The Sharjah Library design shows off a slender roof, sustainable design features, and a soaring central atrium that maximizes natural light. The roof appears to be very thin due to its tapering edge, however as Figure 27 shows, the centre of the roof is considerably larger than the edge. The proposal will use this method of tapering the roof edge to provide a more elegant and impressive look, whilst keeping the centre thicker to provide the drainage slope.

Sydney Fish Market – 3XN
Sydney fish market features large sweeping stairs that act as seating. The proposal will incorporate large stairs that are inviting and provide multi-use for enjoying the sea views.
The inner space is also used as a large open fish market, which has inspired one of the suggested uses for the proposal.

Nine-Square Grid House – Shigeru Ban Architects
Nine-Square Grid House successfully uses glulam arches to provide a tall open space to be enjoyed by the users. The proposal will incorporate large glulam arches that feature the same structural principles and sustainability goals as this precedent.

Coal Drops Yard, Kings Cross – Heatherwick Studio
Coal Drops Yard successfully creates an exciting space, where visitors can easily navigate the shops, restaurants and cafes. Instead of having a central walk space with shops on edge like coal drops yard, the proposal will instead have outward faced walk spaces with central shop spaces lined up in a similar way to the precedent.

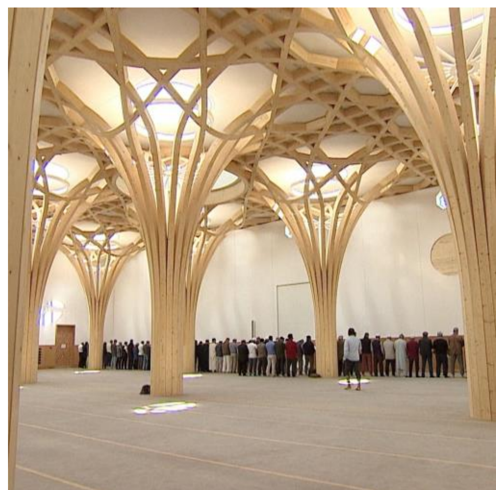
Cathedral Arches
Cathedral Arches have inspired the elegant form of the proposed glulam column.

IQL Pavilion London – ACME
Like the Sydney fish market, the centralised stairs provide a primary space to relax and enjoy the views, with efficient access up to a top deck. This project is entirely made of glulam and is an exemplar project of sustainable design.

Cambridge Mosque - Marks Barfield Architects
Cambridge mosque follows the same arch structural principles as nine-square grid house. Like the grid house, the structure system will be analysed in Section 4.1 , to grasp a greater understanding of the structure that provides an open and impressive space. As well as construction techniques and design for manufacturing.

Figure 28 – Coal Drops Yard

Figure 29 – Cambridge Mosque



3.1 Site Response

Figure 30 – Vehicular Circulation

Vehicular Circulation

The site placement has considered the flow of delivery trucks around the site. The entrance to the site is directly on clarence esplanade and circulates efficiently up to the entrance of shops below the deck, which doesn't impact too much of the site.

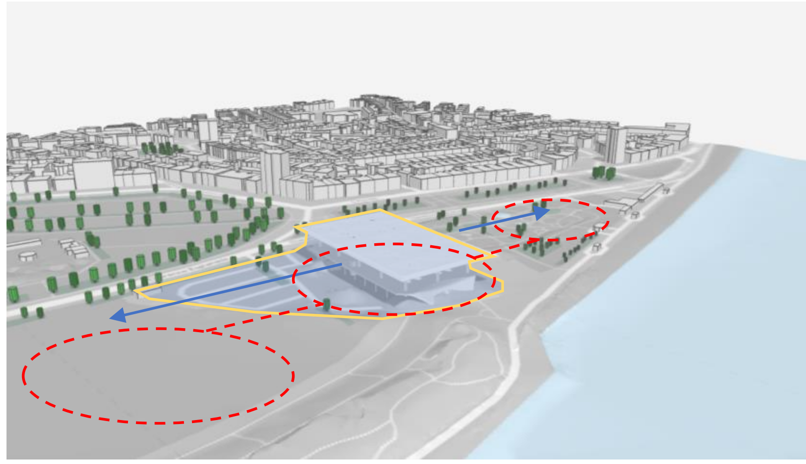


Figure 31 – Visual Permeability and Node-Tying

Visual Permeability and Node-Tying

The open nature of the canopy and future use of the open deck will leave gaps open for connecting the castle field and rock gardens. The proposal is central to these key destinations outlined in red. The structure ties them together effectively by keeping the building space open, green with landscaping and connected with paths.

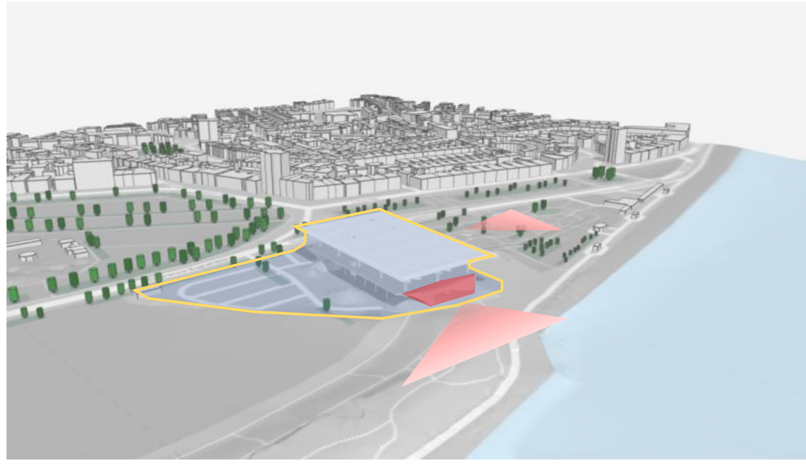


Figure 32 – Expansive outlooks

Expansive outlooks

The 5m tall deck will provide views towards the see over the 5m tall mound, and also into the rock gardens. The deck is a good location to set bars and restaurant for people to relax and enjoy the view whilst eating and drinking.

Figure 33 – People Attract People

People attract People

Often people passing by are curious to explore a facility if it is being used by many people. The open nature of the design will show off the number of people in and around the building, thus attracting more people from the road in Figure 30 or from both the rock garden and castle field.

3.2 Design Intention

After investigating the sizes of restaurants, cafes and shops in successful open market areas like Camden Town, Brighton Open Market and West Quay Southampton, the following accommodation was proposed. Sizes of proposed areas were scaled from floor plans and were estimated according to number of visitors the pavilion is expected to host. With the scaled floor areas the following rooms were calculated. The overarching spatial requirement was to not have a floor plan that imposed a larger surface area impact than the pyramids. The first key design rule was to design the floor areas around 6900 m2. The following building functions were fit into 5800 m2 surface impact on the ground, which is less than the benchmark.

Table 1 – Accommodation Schedule

Function	**Description**	**Proposed area (m2)**
Ground Floor + Exterior Deck		13,000.00
Rentable Space Type 1	This space can be used for cafes due to being closer to the unloading zone.	710.00
Rentable Space Type 2	This space can be used for established boutiques or shops that sign long term contracts with the council.	568.00
Rentable Space Storage	Un-insulated storage for the rentable spaces with direct access to car park	143.00
Food Hall	Large atrium space with plenty of ventilation, that facilitates many temporary food stalls, seating and a permanent central bar.	990.00
Food Hall Storage	Enough storage for multiple food stands to store equipment and food, hidden under the ampethetre type stairs	65.00
Plant	Plant is also hidden under the ampethetre stairs, an area that would struggle to find plenty of natural light if used for public access.	195.00
Staff Parking	Holds 5 spaces, with 2 electric chargers. Enough space for executive visitors, security and general service managers. Plenty of parking off-site for staff and visitors	60.00
Unloading and Loading	4 Bays, 2 for disabled parking with easy access to lifts, and 2 for delivery vans.	48.00
Female Toilet		55.00
Male Toilet		55.00
Lift 1 and 2		13.00
Level 1		4,850.00
Open Deck	An adaptable space that can accomdate a variety of uses, explored later in the report.	4,850.00
Roof		6,860.00
PV Panels	408 Panels	652.80
Access Hatch	2 Access Hatches	2.40
Skylights	32 skylights	433.36

3.3 Initial Concept

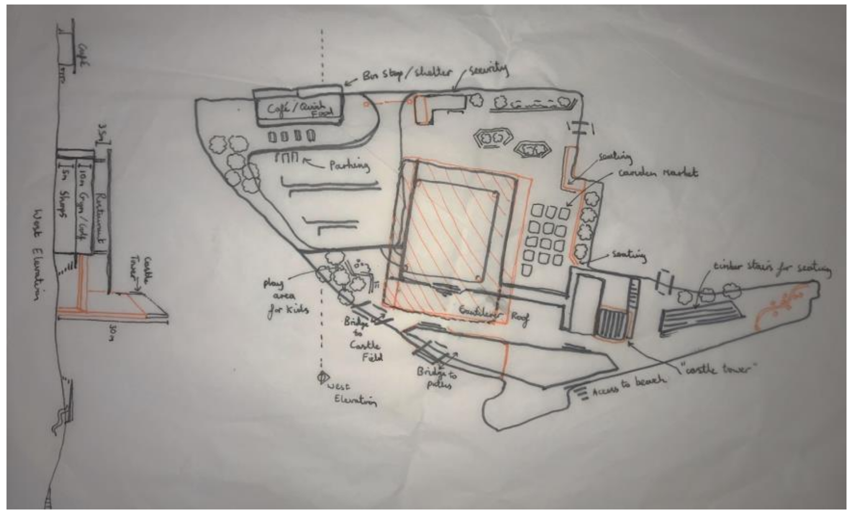


Figure 34 – Initial Concept Sketch

Figure 35 – Initial Concept Massing

Figure 36 – Initial Concept site placement

Concept

This concept utilised the existing foundation of the pyramid centre. The idea was to use the current concrete slabs and foundation to provide an existing structure for refurbishment. The centre steel pyramid roof was going to be dismantled and the left and right wing of the structure was left intact. The issue with this design was that the council struggled to find existing structural drawings for the building and so an accurate structural analysis would be too difficult to complete.

However, various design aspects were still brought through to the final proposal. Firstly, the design utilised a top deck that was the height of the concrete walls and foundation, with a canopy over the top to provide shelter. This feature is very successful as the adaptable deck utilises the future use of the building ensuring that it will be used through a variety of facility trends. Overall, the deck providing key views and a space for sheltered multi-use was the feature that was brought forward from this initial concept.

The second feature was a 30 m tall tower to take advantage of key views of the sea. However, after discussion with the council this feature was removed. The tower would have competed with the current spinnaker tower, thus may not have guaranteed a lot of use, unless the concept provided a feature or view the spinnaker tower didn't. The tower would have just remained too conspicuous on the seafront with not much of a unique selling point. Additionally, the bridge design connecting the tower to the pavilion was also removed.

Finally, the site placement was analysed, and although it was in-line with the view analysis boundaries, it wasn't located in an area that provided quick access from the road. Therefore, it was decided to rotate and elongate the viewing deck.

The council enjoyed the idea of having an impressive frontage to the building that enticed visitors in and so the first development made sure to feature large stairs for seating and direct access.

4.1 Design Strategies

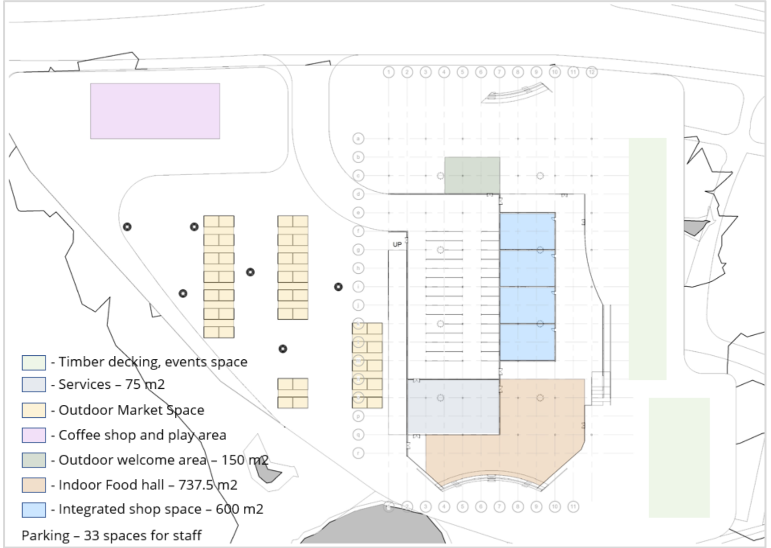


Figure 37 – Initial Ground Floor Layout

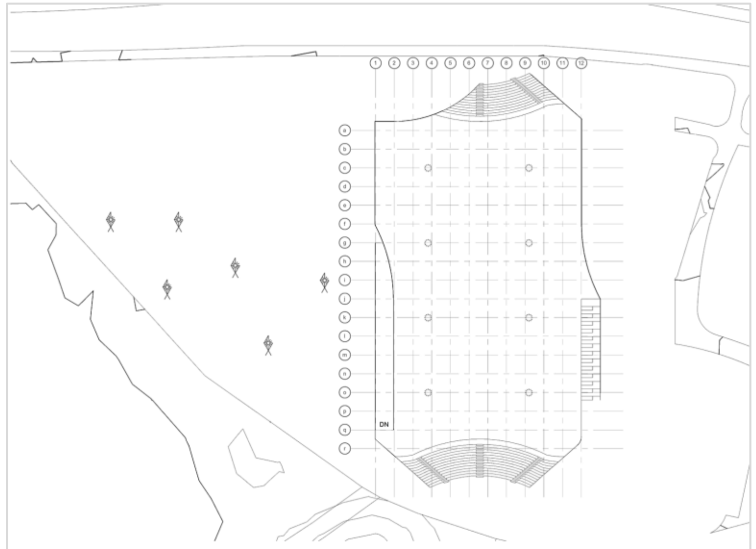


Figure 38 – Initial Level 1 Layout

Development 1

Concept 1 contained a 33 space carpark under a large deck for multiuse. The timber deck extended across the whole site with the west space used for small pavilions, constructed of the same 'tree columns' used for the canopy roof.

After consulting with supervisors, it was decided that the addition of the smaller canopies didn't have a significant use apart from sheltering the market stands from rain. Therefore, these were removed for the next devlopment, as a floor overhang could achieve the same shelter but would integrate with the structure more efficiently, with less wasted space. The north stairs had an obscure shape, which wasn't compelling as the shape didn't have a purpose apart from opening up the east entrance to an underpass. This north stair underpass would have not received a lot of natural or direct lighting, with headspace limited directly under the stairs too. Therefore, this space needed to be adapted.

There lacked loading zones, with the road access moving directly into the covered car park. After discussion with the council, it was decided that their future projects should encourage sustainable travel to their destinations. Therefore the parking was significantly reduced in size.

The large indoor food hall also had wasted space under the stairs and also a large bite out of the space to accommodate a large room for services. The storage and plant room was shifted to under the south stairs, (like the north stairs).

A large ramp was placed on the west face, however the space requirement didn't allow the slope to have rest stops (stated in design standard). Additionally, the stairs on the west side were going to seclude wheelchair uses or people who found walking difficult. Therefore, both the ramp and stairs were removed and 2 elegant glass lifts were placed instead.

The fundamental issue to this design was that the top canopy did not function well with the ground level layout. In order to reduce the use of concrete, the roof structure would have to not be tied to the deck structure, as different load conditions would require different foundation strategies. Tying the building together would require more concrete. Therefore, the 'tree columns; could not pass through an un-insulated room due to the large airgap. Thus, the design focused on creating an efficient roof system first, then focused on shifting the insulated floor layout around the tree columns design second. This column design is explored in the next section.

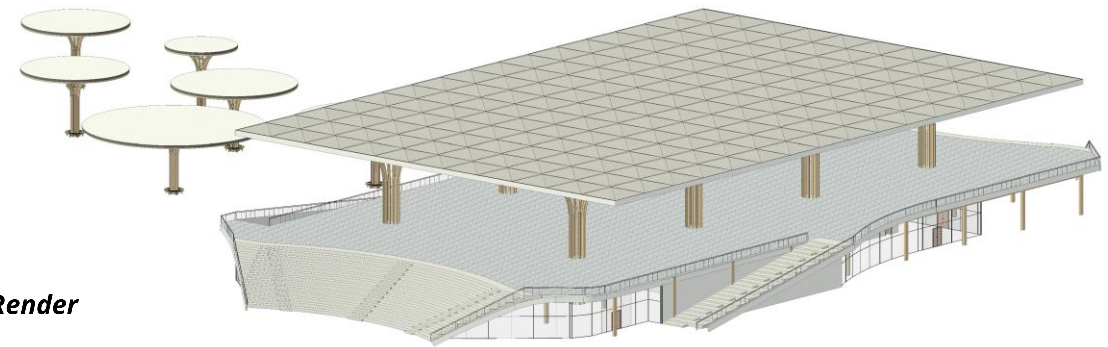
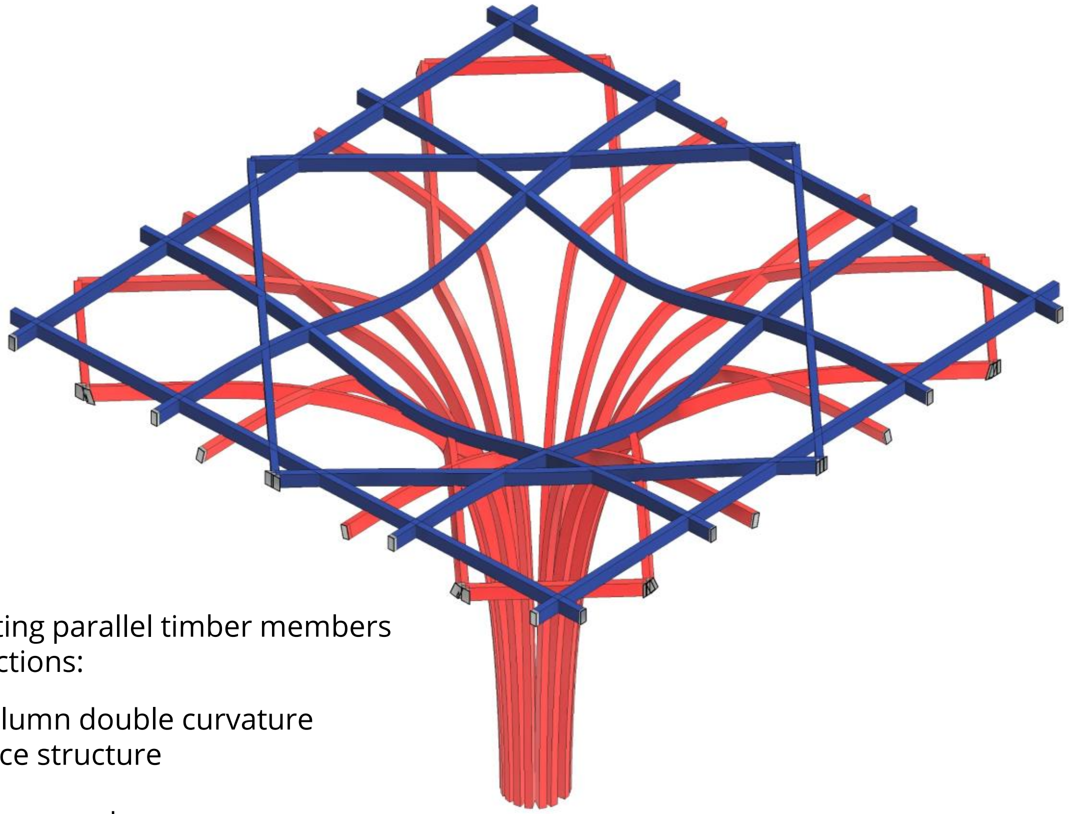


Figure 39 – 3D Render

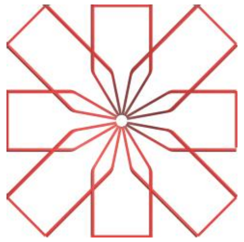
4.1 Design Strategies Continued

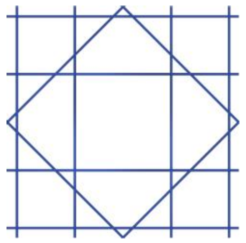


Intersecting parallel timber members in 3 directions:

- Column double curvature lattice structure

- Crisscross beam structure (Primary Load bearers distributing to columns)





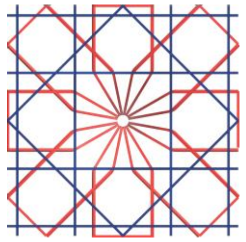
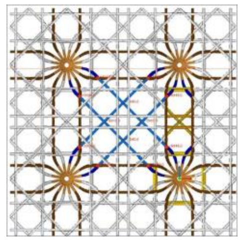


Figure 40 – D1 tree column design and structural plans

Development 2

The structure holding the canopy roof was explored in detail. Initially the Cambridge Mosque structure was recreated in grasshopper. The diagram to the right shows the order of the members; the 'tree' columns are pale brown, the primary roof beams are dark brown, the secondaries are blue, and everything else is effectively there to provide restraint. After completing FEA analysis, it was clear which members could be simplified in order to have a structure that worked more efficiently for the purpose of spanning 30m.



Max bending moment = 654 kNm

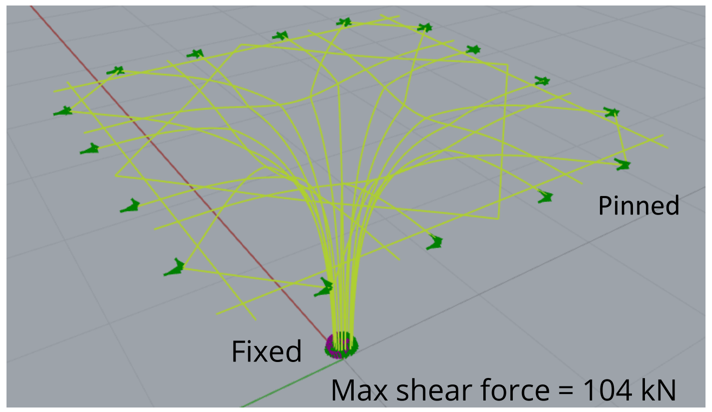
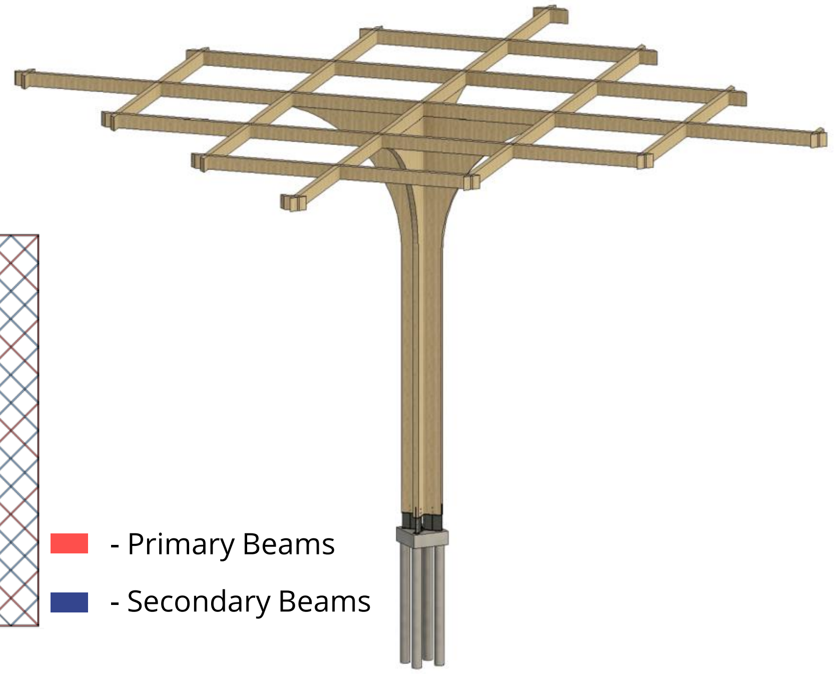


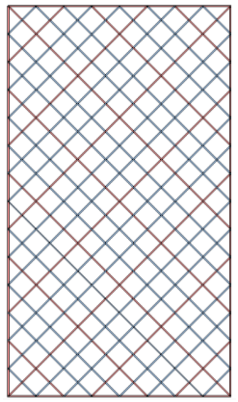
Figure 41 – Grasshopper structural analysis for D1

The column cantilevered out of the ground and the primary beams were then pinned at their ends, where the connected to the adjacent tree column. The thin members of the Cambridge Mosque structure meant that tapering them 5 m out (in order to achieve a regular grid that spanned the lower deck) at 15 m tall, created too large of a stress on the column double curvature lattice (shown in red). The maximum bending moment was 654 kNm, for a 150 x 300 member. The structure failed, as it's complicated members that tied it together didn't effectively work with the primary beams to span such distances.

Development 3

After further recreating the Nine-Square Grid House in grasshopper to get an understanding of how to utilise a truss system. The third and final devlopment was complete after a simple yet affective decision.





- Primary Beams

- Secondary Beams

Figure 42 – D2 tree column design

The Nine-Square Grid House had the roof and the columns structure formed as an "uninterrupted continuity". There were no distinct vertical columns and horizontal beams to be found in the free from structure. The third devlopment was inspired by the Kagome pattern. The primary and secondary beams were flattened onto one plane for multiple reasons. Firstly, manufacturing and assembling the parts would be a lot easier on site. Secondly, the beam members will be a lot cheaper as less members would need to be bent into the flowing form of the previous iterations. Finally, attaching the beams members to the splaying column work with gravity and rests on top of the column, rather than having to connect it through crisscross connections. Overall the design is more simple to construct, doesn't waste material and allows for the use of larger members to span further.

The structure system works in the same was as the Cambridge mosque with the primary beams acting in 4 directions away from the column, and the secondary beams tying together the primary beams. This will be covered in more detail in the structural section of the report.

Devlopment 4

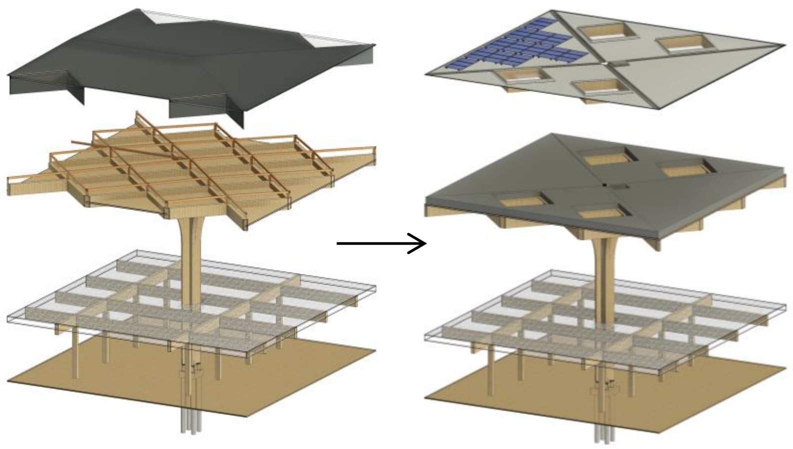


Figure 43 – Roof Development exploded diagram

Another significant development was the decision to use XPS insulation cut to fall, rather than a timber joist structure, to hold up the slope for the drainage. The drainage also was developed from having a large 4 way slope to the edge of the roof, to 15 'reversed pyramids' draining water to pipes running discreetly down the side of the column. The nature of this design is explored later in the report.

4.2 Energy Conservation Features

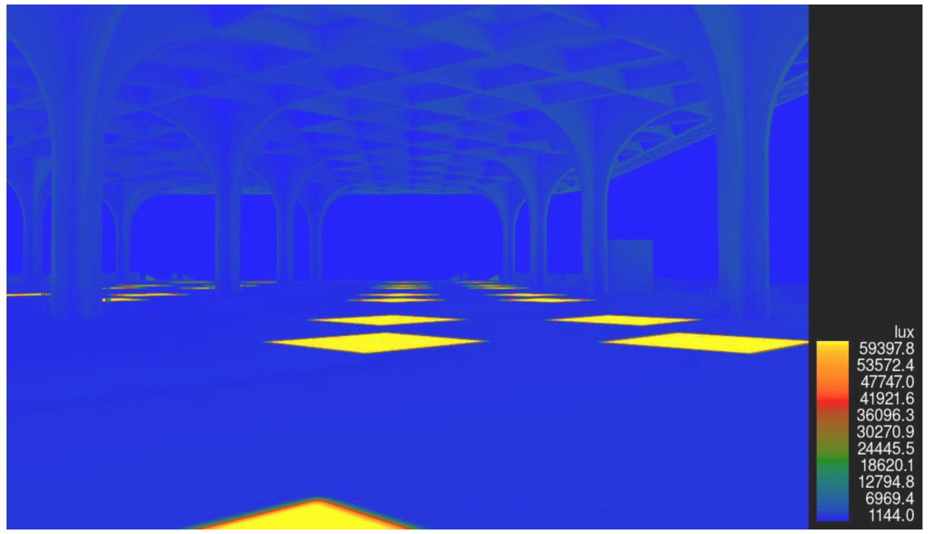


Figure 44 – Top Deck Illuminance Study

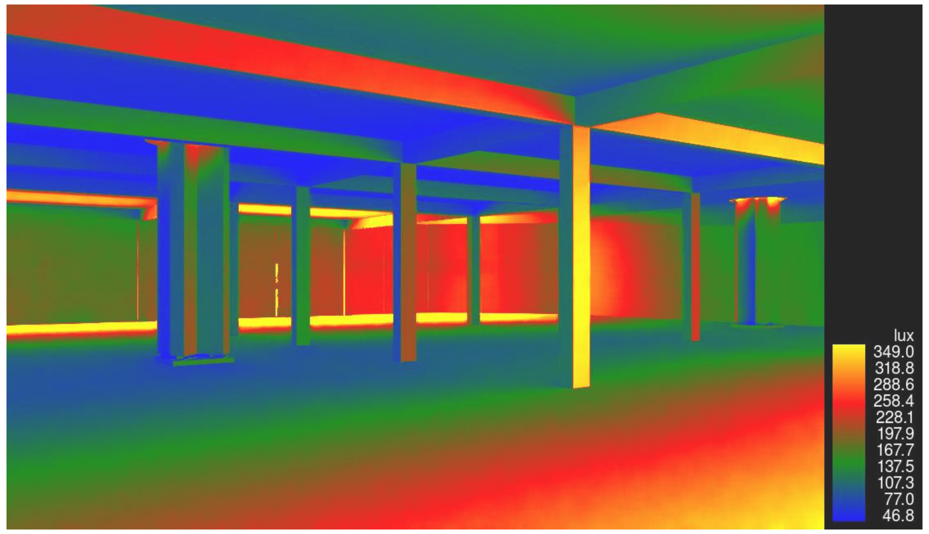


Figure 45 – Food Hall Illuminance Study

Ground Source Heat Pumps

A ground source heat pump (GSHP) is an energy-efficient heating system for large commercial buildings. It harnesses the heat energy from the ground, providing 3 to 4 kilowatts (kW) of heat for every 1 kW of electricity it consumes. This remarkable efficiency surpasses that of other heating systems. By utilizing the freely available heat energy from the ground, a GSHP significantly reduces energy consumption and lowers the building's carbon footprint.

The heat generated from the GSHP will be directly pumped into underfloor heating. Coils will be wrapped on top of the insulation layer and then screed is compressed around the coils, this will generate heat from the floors into the room above. The heating fluid is then circulated back into the GSHP ground loop to generate more heat. According to 'Pure Renewables' the payback on a GSHP should be around year eight or nine. Therefore, this system will be generating free heat for 84% of the proposals design life.

Illuminance Study

Adequate lighting and comfortable lux levels contribute to visual comfort. When lighting is too dim or too bright, it can strain the eyes, leading to fatigue, eyestrain, headaches, and difficulty focusing. By maintaining comfortable lux levels, individuals can engage in activities without undue visual discomfort.

CIBSE recommends general lighting in markets to be between 500 – 1000 lux and general lighting in food production areas to be a maximum of 500 lux. Figure 44 shows that the general shaded area during 12:00 on June 20th (Summer) is at a maximum of 1144 lux. Most of the light is provided by the skylights above.

Figure 45 shows that the food hall has a central lux of 46.6-167.7. The perimeter is exposed to the glazing therefore experiences 349.0 lux, which is at more of a comfortable level. To increase the central level to 350 lux, it would be important to provide LED lighting or to increase the diameter around the canopy columns to let in more light from the top deck.

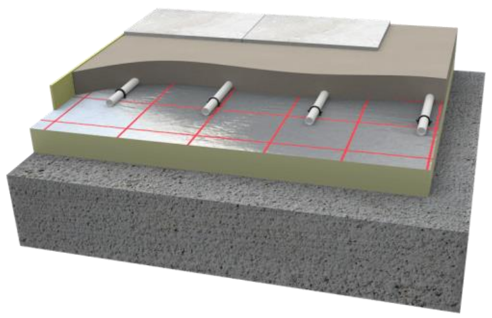


Figure 46 – Underfloor heating

Figure 47 – GSHP coils

Solar PV Output

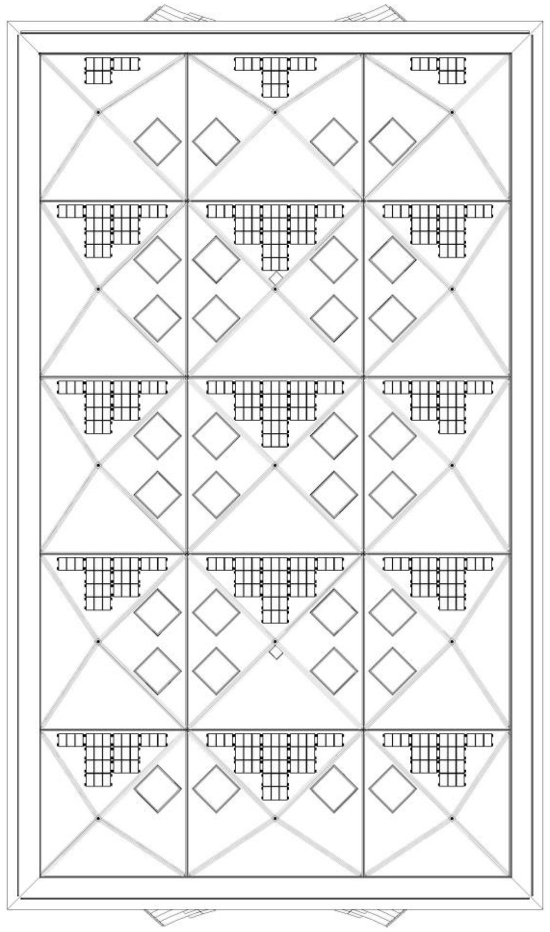


Figure 48 – Roof Plan (Not to scale)

Figure 49 – PV Panels placed at 35 deg

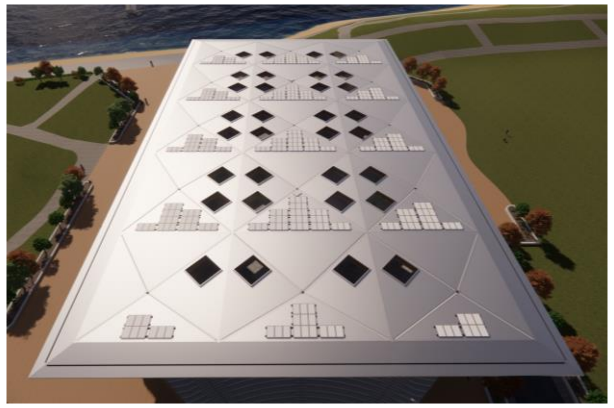


Figure 50 – Render of Roof

The roof houses 408 'Renugen' panels that have 300 Wp of power each. The total roof usage is 11%, as they are placed only on the optimal slope surface, as shown in Figure 48. They are inclined at 35° and have a southern orientation. The installation of PV panels is essential in providing renewable energy which will considerably drop the carbon impact of the proposal. The cost of implementing the whole 122.4 kW system will cost roughly £122,000. To justify this initial cost the following calculation is done:

850 kWh per year of electricity is generated for each kW of installed panels. Therefore 850 x 122.4 =104,040 kwh per year, which saves £19,000 per year. A 3.5 kW system saves £4000 per year through the Smart Export Guarantee scheme. Therefore, a 2.34 kW system is assumed to save around £23,000 per year. Therefore, the system will pay for itself after 4.5 years.

The estimate from the UK Government for a building providing shops is 84.2 kwh/m2. The proposal has a total net area of 9,700 m2. Thus, is expected to use 816,700 kwh per year. Therefore, after the system pays for itself, it will save 8% of the electricity bills per year.

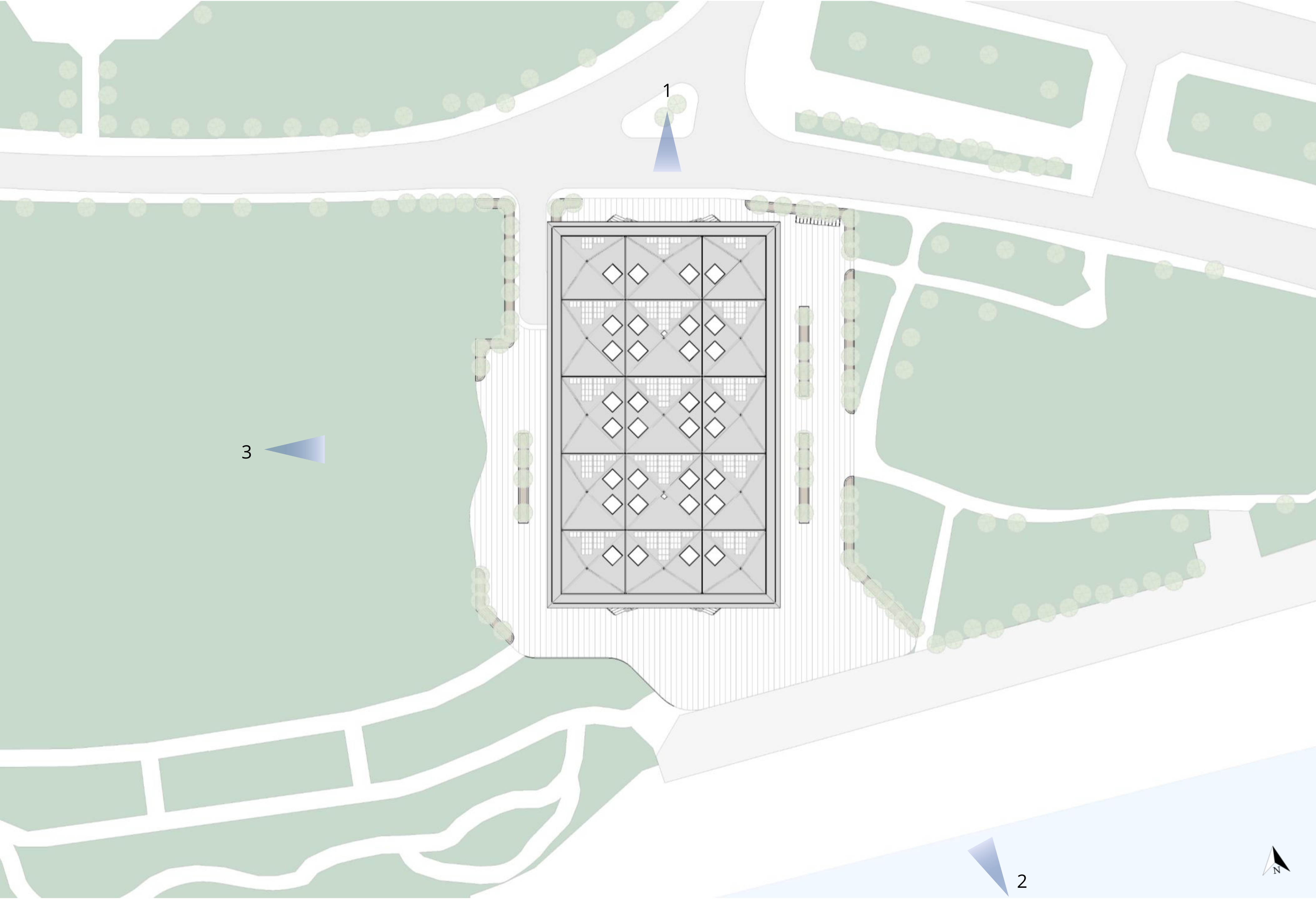
Thermal Insulation and reduction of Thermal Bridging

The rentable cafes, shops and restaurants are fully insulated. With 300 mm of Sheep's Wool for the decking floor and walls, and 150mm of XPS insulation for the concrete ground floor. Insulating a space is the number one method for saving money on energy bills. It not only creates an environment that is warm and comforting, but it also will work effectively in keeping the heat generated from the underfloor heating in the room. Double glazing is used throughout the whole proposal, as this keeps in heat a lot more effectively than single glazing.

The proposal will make sure that all thermal bridges are addressed, by either keeping insulation flush with structural elements, or by adding exterior insulation to corners of walls or the perimeter of doors and windows. This will eliminate unnecessary heat loss.

5.1 Final Proposal

Figure 51 – Site Plan (Not to scale)



Proposal Summary

The seafront building in Portsmouth is an impressive structure with a 5-meter-tall top deck, finished with marble tiles, offering panoramic views of the sea. It features stairs on the north and south sides for easy access. A large glulam canopy roof stretches across the deck, providing shelter and adding visual appeal. Surrounding the building is a timber decking with landscaped areas that guide visitors to key spots like a rock garden and Castle Hill. This versatile building serves the council's purposes while offering a remarkable seafront experience.

Location on Site

The structure is placed with the north facing stairs parallel to the road line. The castle field has been extended, where the old car park used to be. The lower timber decking opens to the sea frontage for easy access to the beach. All current paths have been made easier to access with landscaping. The structure is orientated so that the journey from the clarence esplanade is full of activities from the moment the user steps of the pavement.

Landscaping

Various plant beds were constructed for trees and bushes and are situated around the site to provide seating for visitors. The plant beds are constructed from limestone, the same material as the north and south stairs. The open timber decking will be filled with seat, play activities and portable boutiques and outdoor food stalls. The south stair face the larger portion of this deck where various artists can host concerts, with great viewing from the seating on the amphitheater style stairs.

Connectivity

The buildings large west and east walkways provide easy and well-lit access to the beach and connects the surrounding greenspace together. The well-lit space provides a safe environment for families to enjoy at night. Plenty of bike racks have been placed on the northeast section of the timber decking, to encourage cycling to the attraction.

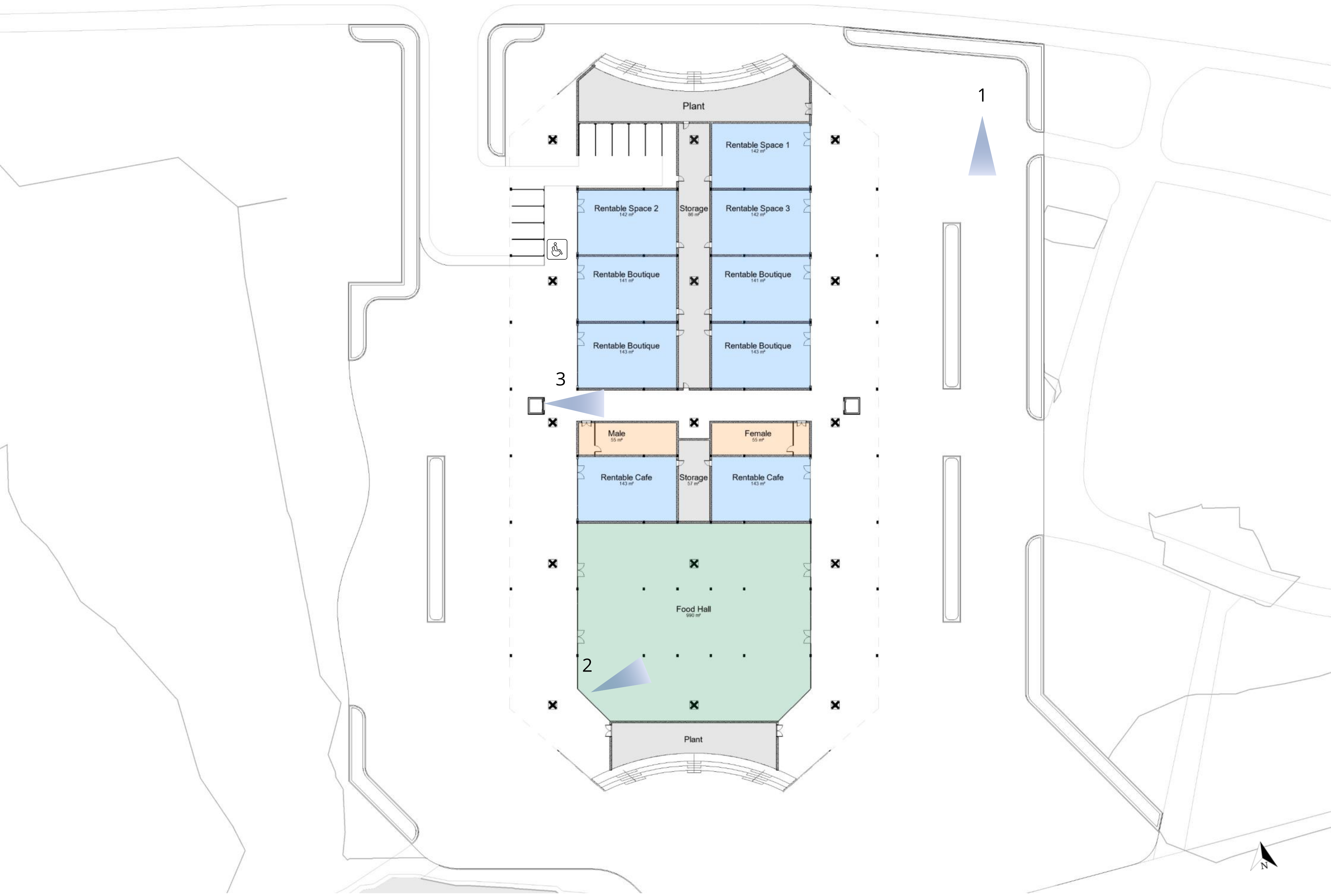
Figure 52 – View 1 of north face

Figure 53 – View 2 from sea

Figure 54 – View 3 from West

5.2 Performance Scenario

Figure 55 – Ground Floor Layout (Not to scale)



User Experience

Visitors can either climb the front stairs or work their way around the perimeter of the building, which hosts a variety of inviting spaces used for coffee shops, shops and art rooms. The layout is simple, so it is easy to navigate. Halfway down the building is access to the toilets, shown in figure 58, which will be signposted. The left side of the building keeps the carpark under the top deck, which will be well-lit and covered with security cameras. Multiple bays will host electric charging and spaces for more as the use of electric cars rises in the future. Disabled Parking on the Left next to the unloading bays will provide easy access to the lifts, illustrated in Figure 55.

Finally, as the visitors move closer to the seafront, a large glazed façade will invite visitors into a large 990 m2 food hall. It housed plenty of seating, shown in Figure 57, and plenty of food stalls. The flooring is black marble which gives the space a modern environment whilst nicely contrasting the more rustic look of the pine façade. Overall, the area will feel inclusive as the variety of food provided and easy access will welcome all social classes of Portsmouth.

The food hall will be well lit with lighting provided by the large 'tree columns' and LED roof lights placed at 2.5 m intervals. Thus, allowing the space to be kept open in the evenings.

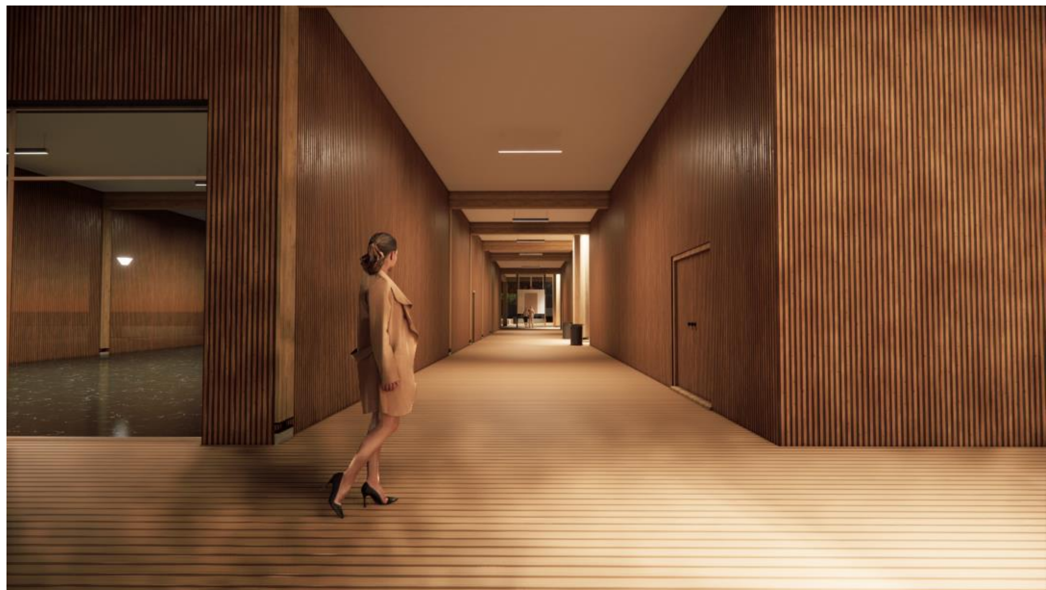
As mentioned before, the plant has been tactically placed in areas with limited access due to a lack of headspace room. The storage has been centralised for quick access from the rentable spaces. The exterior tree columns have circular bench seating around them, resembling ones often seen around trees.

Overall, the bottom space is an integrated space that provides various standard, yet essential features expected in a market space. As the ground floor covers standard hospitality, it gives an opportunity to host creative facilities on the top deck, like a tennis tournament with temporary tennis courts or placing modules that create a unique space for accessing rooftop bars or sky gardens.

Figure 56 – View 1 of East Walkway

Figure 57 – View 2 of Food Hall

Figure 58 – View 3 of Toilets and Glass Lift



5.3 Top Deck – User Experience

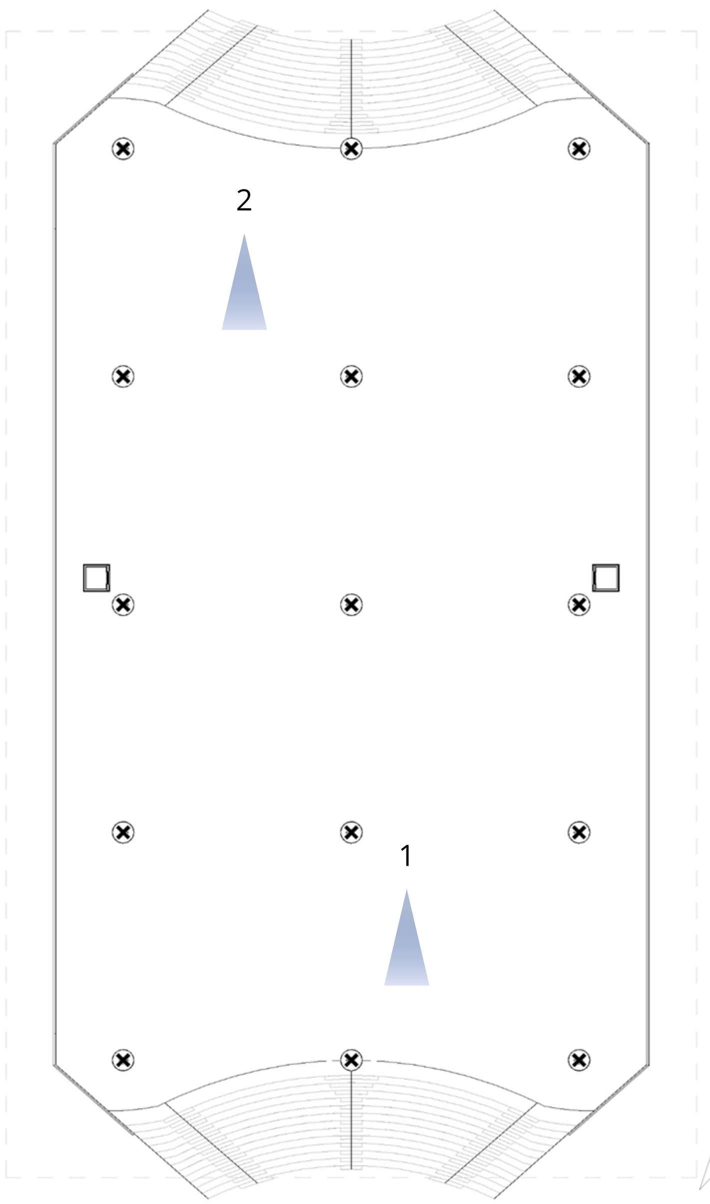


Figure 59 – Level 1 (Not to scale)

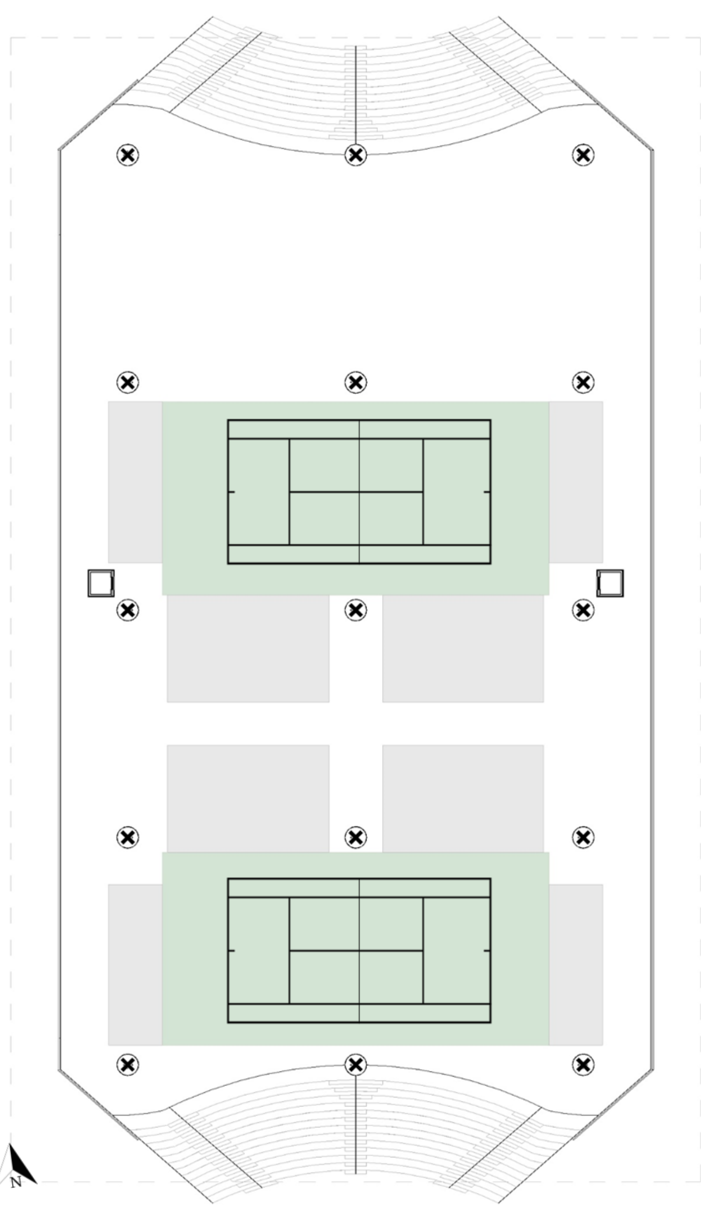


Figure 60 – Tennis Layout

Adaptable Top Deck (Level 1)

The top deck boasts a 4850 m2 floor than can host a variety of uses, as the council desires. Firstly, tennis courts and stands (gray Figure 60) fit perfectly between the columns, thus the space would be a great location for a temporary tennis tournament in the summer.

Secondly, modular food markets and restaurants/bars with multiple floors can be placed on the top deck as seen in Figure 64. A top floor garden/restaurant could be an interesting proposition too. The additional modular structure would not be able to bolt into the floor, therefore would need to be self supporting. The council could investigate recycling shipping containers in order to create these modular/self supporting structures. The restaurant/bar would be placed on the south face 4 m above the deck height, to provide the best views for drinking and socialising.

Thirdly, the council could decide to leave the deck open to the local vendors to set up their temporary food stalls and boutiques.

Finally, the open space would be a perfect location for a large concert, figure 65, however safety measures would need to be in place in order to maintain a safe environment for everyone.

Overall, the top deck has been structurally designed to withstand the largest load the council could decide on.

Figure 63 – Restaurant Garden Space (Google Images)

Figure 64 – Sydney Fish Market 3XN (Google Images)

Figure 61 – View 1 of sea

Figure 62 – View 2 of roof structure

Figure 65 – Festival or Concert (Google Images)

5.4 Roof Design

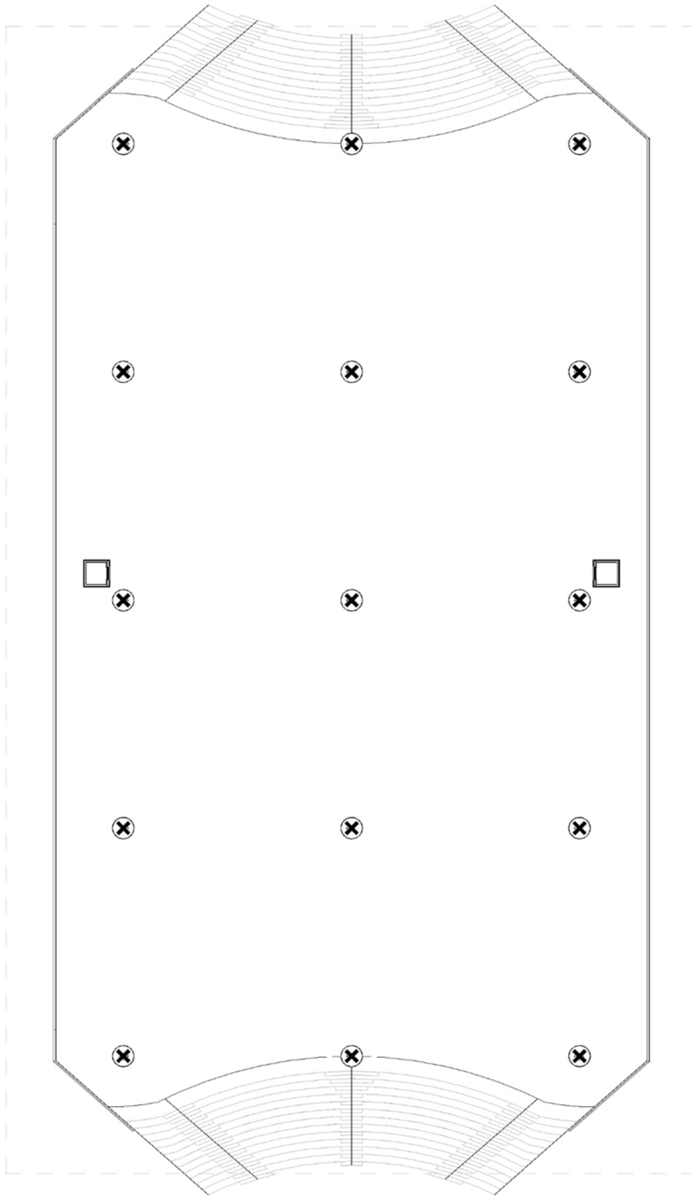


Figure 66 – Level 1 (Not to scale)

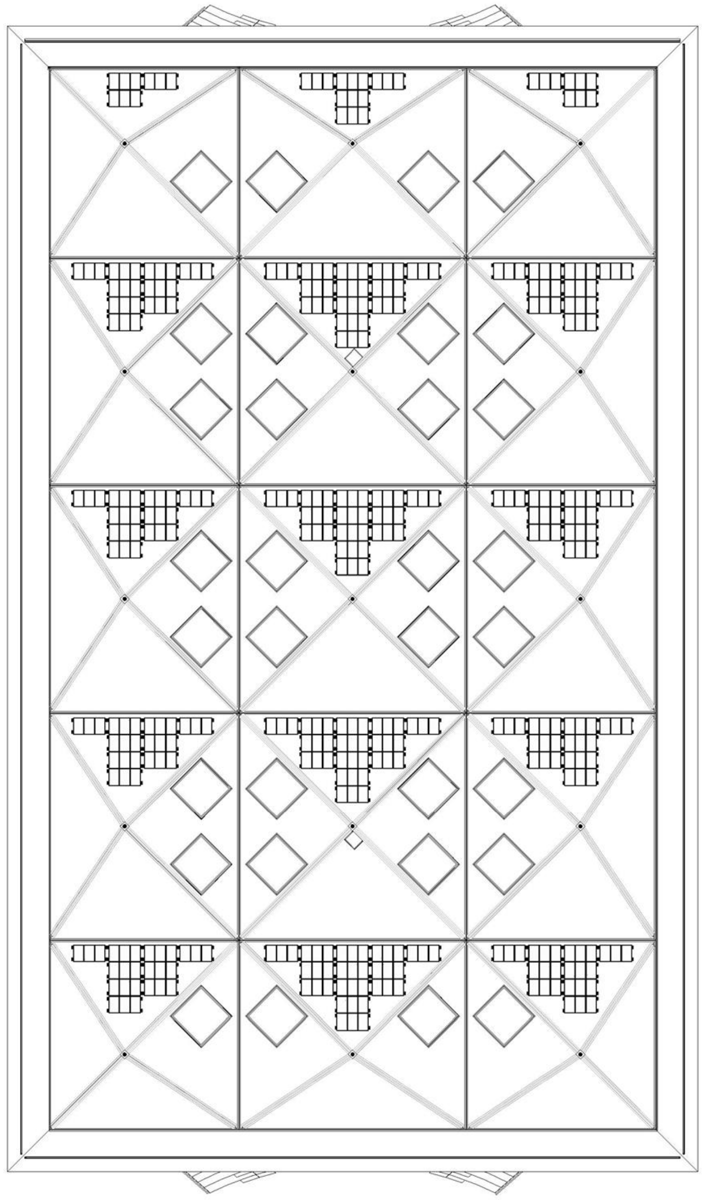


Figure 67 – Roof Plan (Not to scale)

Roof

The roof has been designed to slope at 1.5 degrees towards the 15 tree columns. With valley gutters, figure 71, directing the rain to the central drain. The pipes run directly down the column and are only facing inwards, so that they are not initially seen from the outside. The solar panels are inclined at 35 degrees, which is the optimal angle for the sun path.

The roof is made up of XPS insulation cut to fall. This is to hold up the kalzip finish and to also provide acoustic insulation, with a sound reduction of 15 – 25 db. Acoustical insulation is needed in order to prevent resonance and echoing during large rainfall.

32 skylights are placed and supported by the clt slab, providing plenty of natural light, as mentioned previously in Section 3.5.

Access to the rooftop for maintenance is easy, with 2 columns that have attachments bolted into them for a temporary ladder that leads up to a roof hatch. The roof is easy to maintain and clean due to the aluminium standing seem finish.

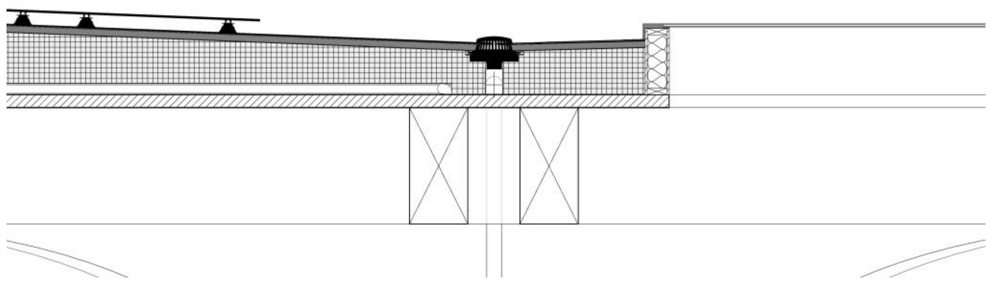


Figure 70 – Central Drainage Detail (Reference Appendix B – Detail A2

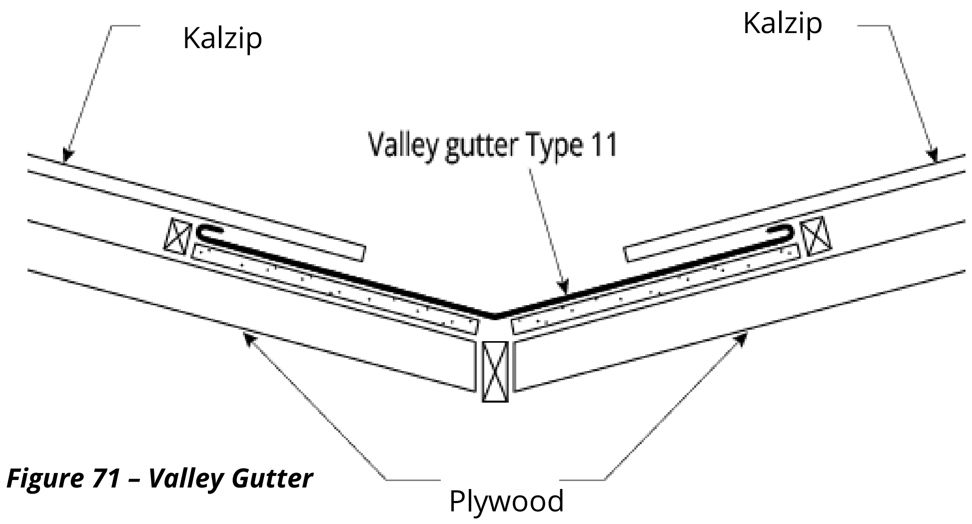


Figure 71 – Valley Gutter

1

Figure 68 – Roof Render

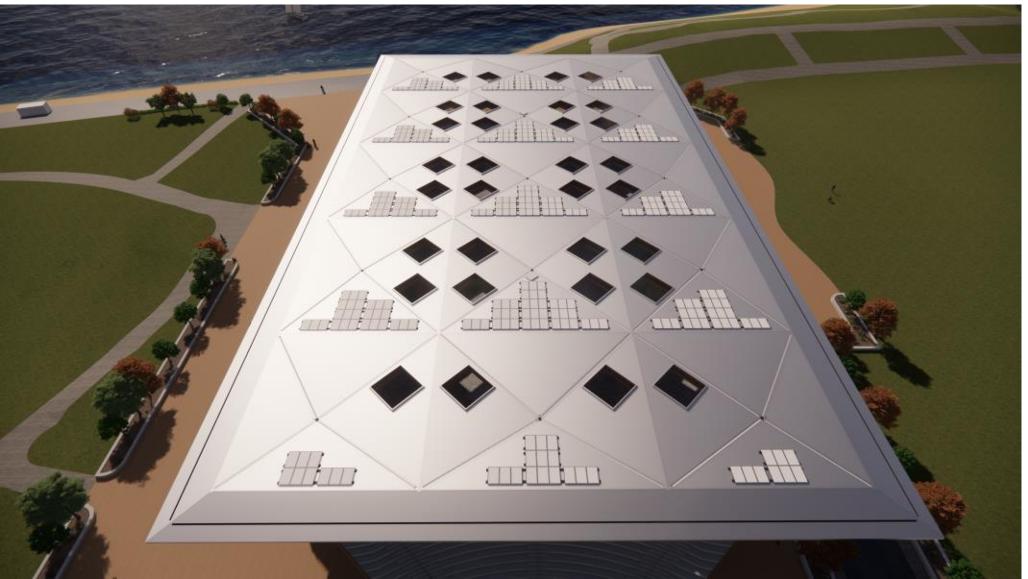
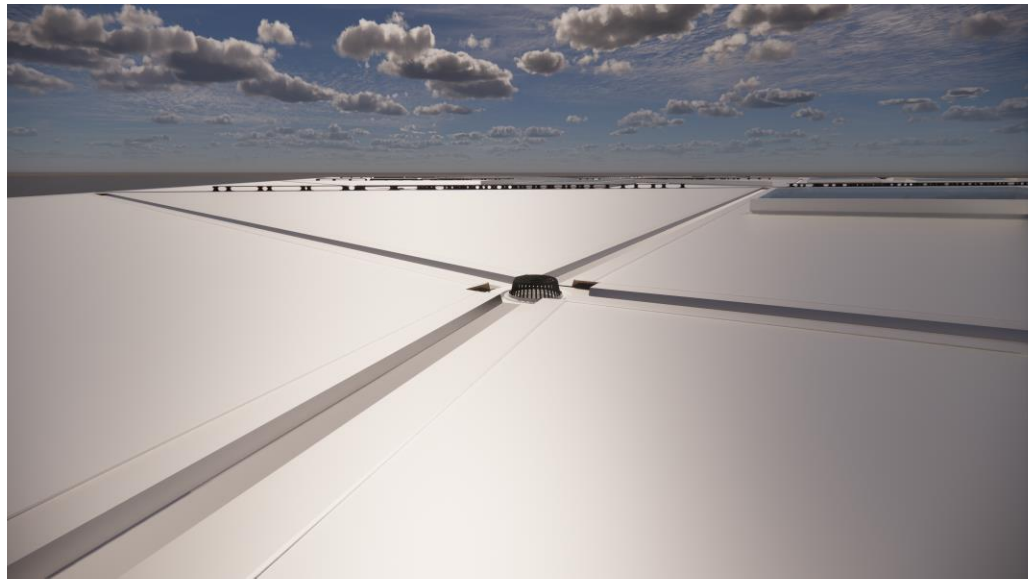


Figure 69 – View 1 South Face of Structure

Figure 72 – Valley Gutter and Drainage Render



5.5 Elevations

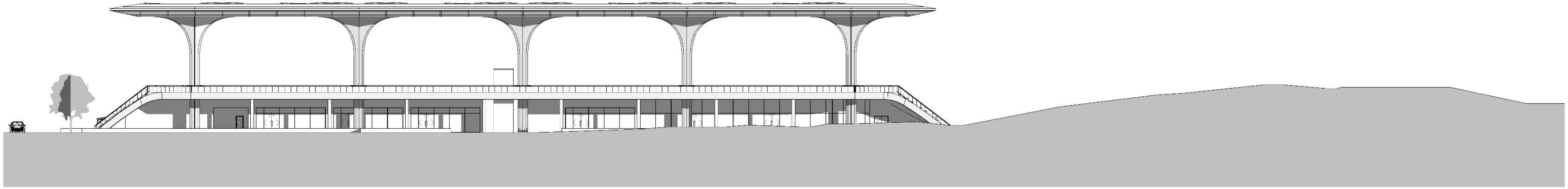


Figure 73 – West Elevation Scale 1:500 @A3

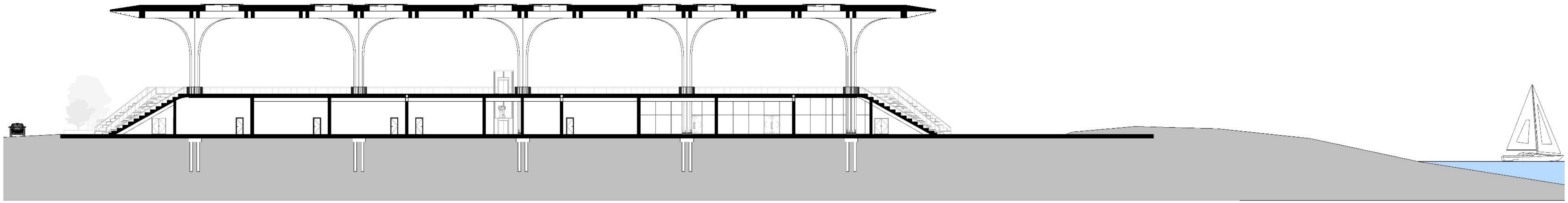


Figure 74 – Section A Scale 1:500 @A3 (See Appendix B Level 1)

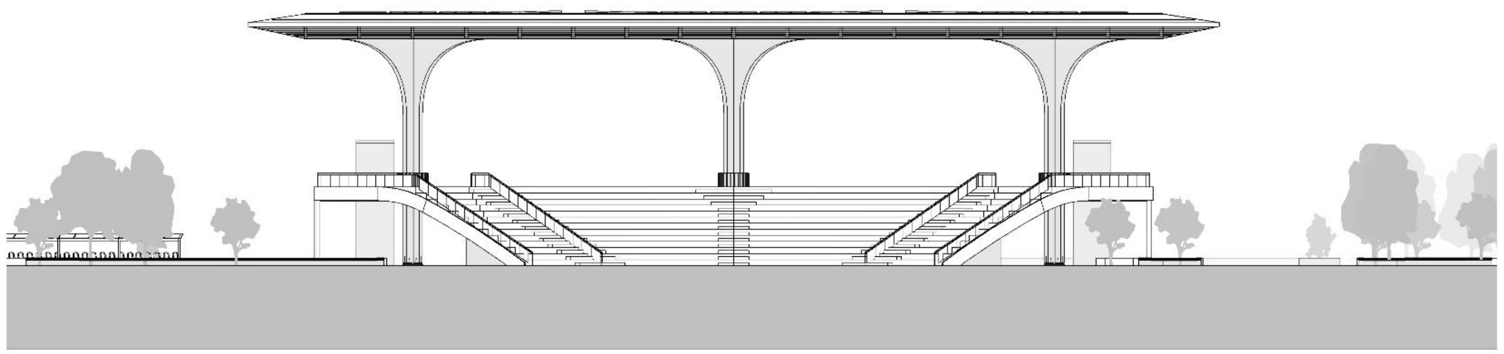


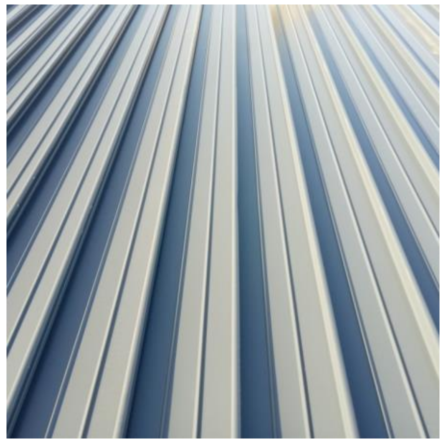
Figure 75 – North Elevation Scale 1:500 @A3

Elevation Design

The height of the building is 15 m with the lower deck at 5 m tall. The deck has been designed so that viewers can see over the mound towards the sea, as shown in Figure 73. The deck also provides a better view of the castle. The 10 m of space to the canopy roof, allows for the development of a 2-storey structure, at 3.5 m a floor. The elevation positively stands out on the coast with its captivating design inspired by grand cathedrals. Drawing inspiration from high ceilings, elegant columns, and gothic arches, the pavilion creates an atmosphere that is both aesthetically pleasing and captivating. It aims to elevate the seafront experience, offering something more beautiful and enticing than typical steel buildings.
When evening falls, the pavilion transforms into a mesmerising monument illuminated by vibrant lights. The unique design and captivating illumination make the structure a standout feature on the seafront.

5.6 Material Resources and Design for Manufacture

Figure 76 – Kalzip (Google Images)



Kalzip (Standing Seam)

"The Kalzip Deck Roof System is the further development of the handcrafted standing seam roof into an industrially pre-produced, quickly assembled roof system." It is made of aluminum, after weathering a coat of aluminum oxide is formed and hardens on to the surface. Unlike rust, it doesn't flake, therefore is a perfect material for the seafront. This material is also very easy to clean.

Figure 77 – CLT Roof Slab

CLT Roof Slab

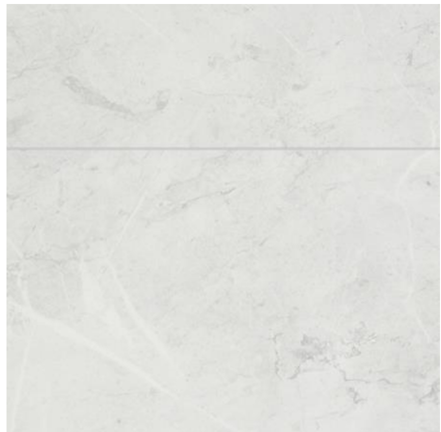
2.5 x 5m slabs are prefabricated and placed by crane on site. They are screwed with a metal bracket on site. The further use of timber will significantly offset the total embodied carbon of the proposal. Like glulam, CLT will need to have a protective coating, applied by Portsmouth Council. Even though this sounds like a lot of effort, the finish of glulam over time will outweigh the rusting look of steel.

Figure 78 – Glulam

Glulam

Glulam is a (flexible in shape) material made by gluing together laminates of timber that have been accurately planed. Glulam is not only regenerative but also is easy to manufacture, with the steel knife connections already attached to the members from factory. To maintain a neat finish, the members will need to be coated with a protective layer every year.

Figure 79 – Marble Tile



Marble Tile

The top deck is finished with marble tiles. The top deck is going to drain towards the perimeter of the deck, so this smooth surface is perfect for mopping or cleaning on wet days. The look is also premium and gives the building a modern feel. A marble floor will last 25 years or more and will not corrode or fade.

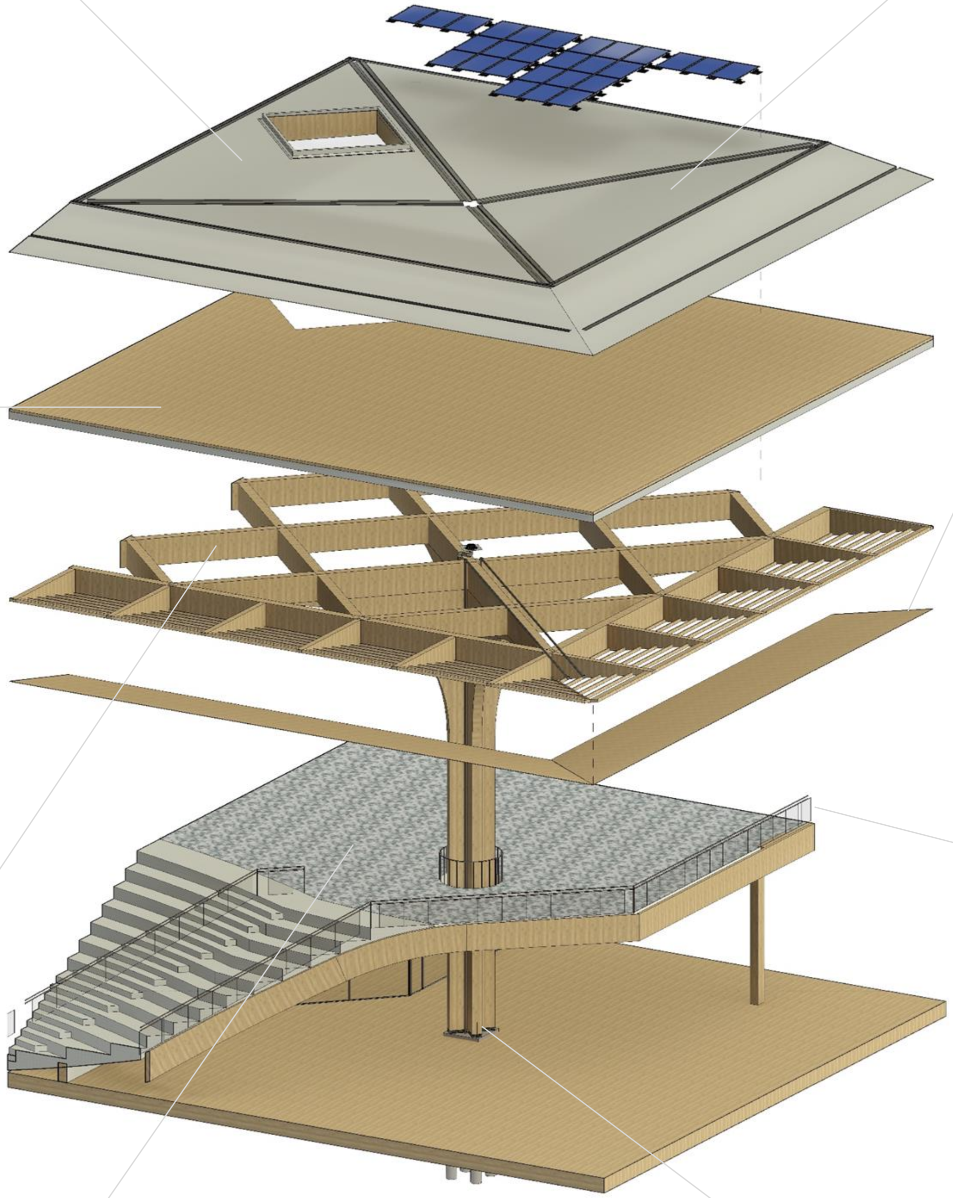


Figure 80 – Exploded diagram of a single tree column

Figure 81 – XPS Insulation cut to fall

XPS Insulation cut to fall

The XPS insulation is cut to fall in order to provide a minimum slope of 1.5°. The weight of the the roof and maintenance team, are distributed into 18 mm of plywood that is protected by a waterproof membrane. The XPS insulation then takes the compression forces and distributes them into the CLT Slab. This method of structurally holding a roof slope is easier to manufacture than many small timber joist frames.

Figure 82 – Marine Grade Plywood

Marine-Grade Plywood

As the proposal is coastal, the design wanted to incorporate material that are often used on boats. They have been proven to be effective against direct contact against sea water, therefore will be appropriate in this setting. The plywood will be exposed therefore will need to have a protective coat. Osmo Wood Protector is ideal for treating wood in high moisture areas.

Figure 83 – Glass Railings

Timber and Glass Balustrade

Glass railings have many benefits, they are easy to clean, they provided unobstructed views to the seafront and surrounding greenspace. Ultimately, glass railings are easy to install, yet effective at providing maximum security.

Figure 84 – Steel Knife Plate

Steel Knife Plate Connector

Steel knife plates are used to connect the glulam frame, and the column to the concrete finch. This connection is discrete and doesn't expose the steel connection on the exterior like other manufacturers may design for. As mentioned before the prefabricated design makes manufacturing very efficient on site.

5.7 Structural Design

Exploded 3D Structural Diagram

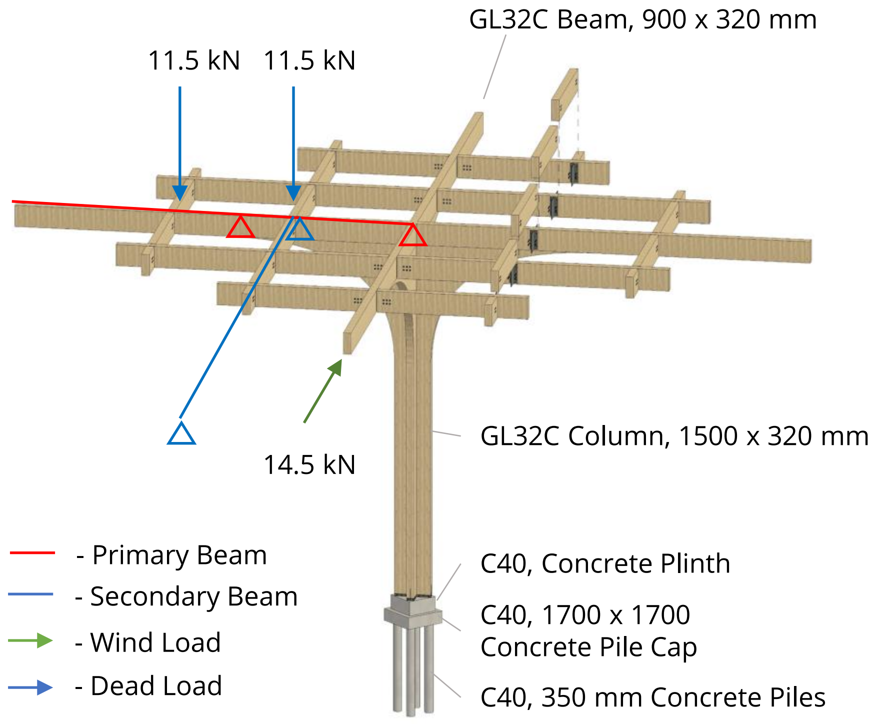


Figure 85 – Designed Tree Column Structure System

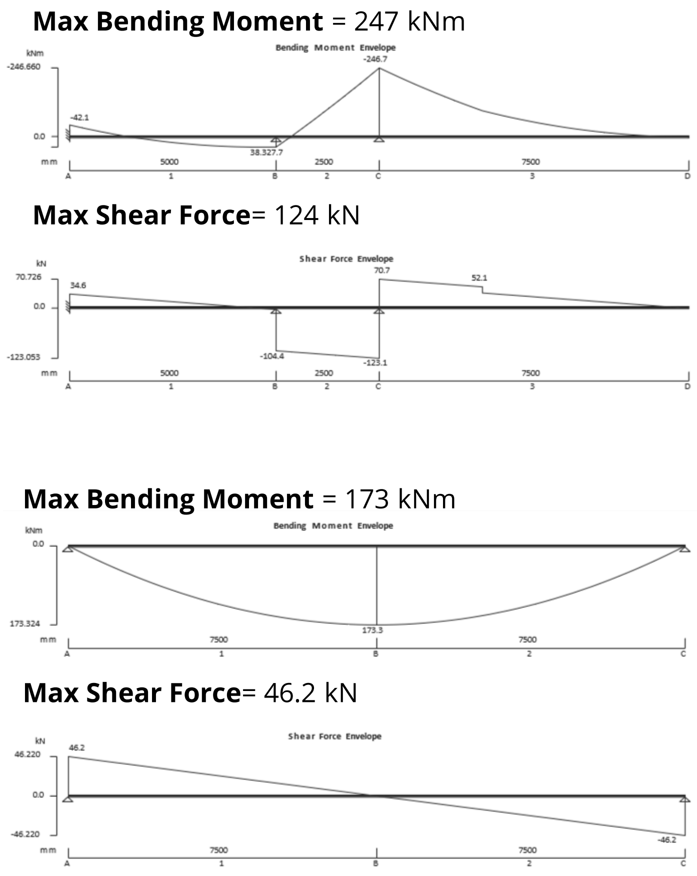
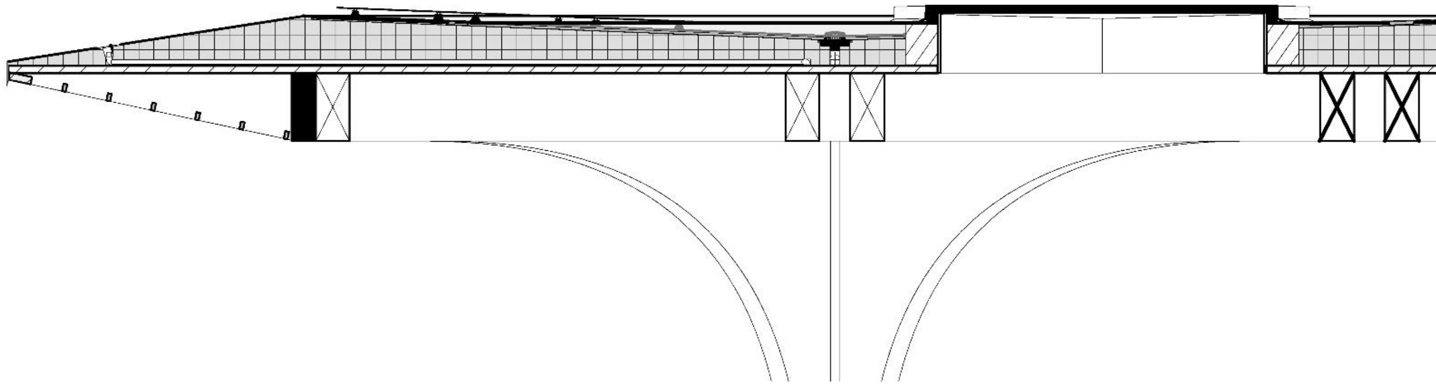


Figure 86 - Bending moment and shear force for beams (See Appendix A)



Structure System

The canopy structure has been assumed to split into 15 individual structural systems, relevant to each tree column. The central column is under the largest strain as a system therefore has been analysed in Figure 85. The column is assumed to cantilever out of the ground, fixed at the pile cap. With a glulam beam system resting on the top of the flaring column. The primary beam acts as a 15 m cantilever beam that is simply supported at the end of the columns flare and directly at the center point of the column at the connection.

The dead and live load is distributed down from the roof into the CLT slab then into the secondary beams that distribute the largest load onto the primary beam. The wind load distributes its load from the edge face of the roof, down from the beams into the column. This system could work more efficiently if the whole structure was designed as a global structure, however the assumption is a strong start for further refinement. The structural design of all of the system's elements are summarized in the table below:

Table 2 – Structural design summary

Element	Sizing	Bending Utilisation	Shear/Axial Utilisation	Deflection Utilisation
CL, XLAM-100-3	100 m\m	0.12	0.03	0.811
Primary Beam, GL32C	320 x 900 mm	0.28	0.45	0.96
Secondary Beam	320 x 900 mm	0.22	0.18	0.93
Column	1500 x 320 mm	0.12	0.05	0.96
Pile Cap	1700 x 1700 x 500 mm	0.2	0.86	n/a
Pile	320 mm diameter	0.25	n/a	0.11

In detail structural calculations are found in Appendix A.

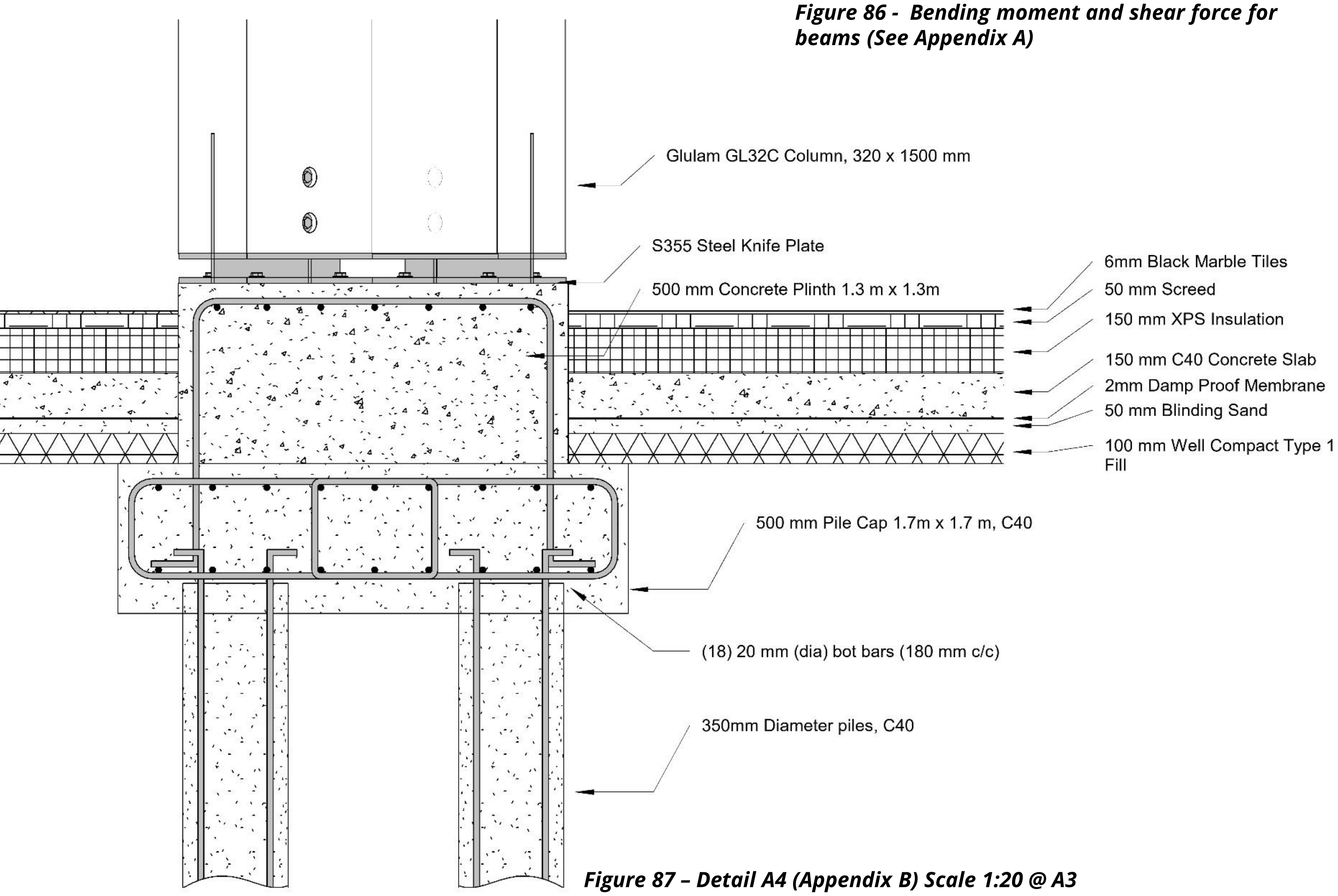


Figure 87 – Detail A4 (Appendix B) Scale 1:20 @ A3

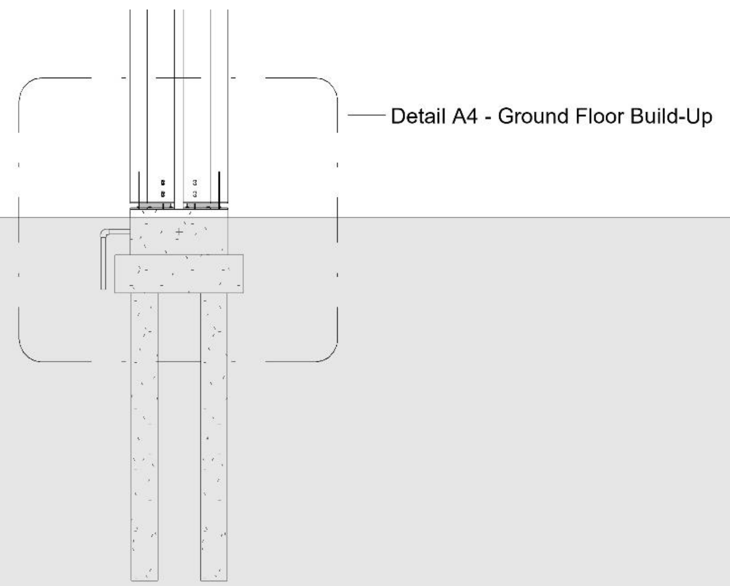


Figure 88 – Detail A (Appendix B) Scale 1:100 @A3

5.8 Carbon Calculation

Material	Material Type	Material Specification	Structural Element	Description	Component Lifespan [years]	Temporary Works	% of Temp Works Wasted	Volume [m³] or Mass [kg]?	Material Quantity [m³, kg]	Reinforcement [kg/m³]	Element Embodied Carbon [tCO₂e]	A1-A3	A4	A5w	B4	C2-C4	D	Biogenic Carbon
Timber	Manufactured_structural_timber	UK Glulam 100% FSC/PEFC	2.1 Frame	Canopy + Deck	60	No	100%	Volume [m3]	1,292		287	170	98	3	0	1012	-318	-996
Concrete	Insitu	UK C40/50 (25% GGBS)	1.1 Foundations (incl. pile caps)	Foundation	60	No	100%	Volume [m3]	462	General Slab (110)	231	192	7	12	0	21	18	0
Steel	Plate		2.1 Frame	Connections	60	No	100%	Volume [m3]	7		146	141	2	1	0	1	-67	0
Timber	Manufactured_structural_timber	UK CLT 100% FSC/PEFC	1.1 Lowest floor/slab	CLT	60	No	100%	Volume [m3]	2,100		432	244	157	4	0	1628	-512	-1601
Other	Aluminium	Sheet - European - 31% recycled content	2.3 Roof	Kalzip	60	No	100%	Volume [m3]	7		134	129	3	1	0	0	-60	0
Custom_EPD	Other_EPD	XPS Insulation	2.3 Roof	XPS Insulation	40	No	100%	Volume [m3]	3,210		160	56	2	8	80	13	45	0

Table 3 – The structural carbon tool input

Total Embodied Carbon [tCO_2e]	3148
Total Sequestered Carbon [tCO_2e]	2597
Net Embodied Carbon [tCO_2e]	551

Table 4 – Embodied Carbon output

Structure System
1.The seafront building in Portsmouth has undergone a detailed carbon footprint analysis, providing valuable insights into its environmental impact. The findings demonstrate a clear commitment to sustainability and adherence to performance targets set by renowned organizations such as RIBA and SCORS. The total embodied carbon of the building is measured at 3148 tonnes of CO2 equivalent (tCO2e). This figure encompasses the carbon emissions associated with the construction materials, manufacturing processes, transportation, and assembly of the building components.

Positively, the building demonstrates a significant amount of sequestered carbon. The total sequestered carbon measures 2597 tCO2e, indicating the amount of carbon dioxide that has been absorbed and stored within the building's materials or surrounding environment. This sequestered carbon helps offset a portion of the building's total carbon emissions, contributing to its overall environmental sustainability.

The net embodied carbon represents the difference between the total embodied carbon and the total sequestered carbon. In this case, the building has a net embodied carbon of 551 tCO2e. To provide perspective, the carbon equivalent of the building's footprint is similar to emissions from 405 average family cars running for a year.

Scheme performance against targets:

SCORS: 137 kgCO2e/m2 Scheme meets target

RIBA: 144 kgCO2e/m2 Scheme meets target

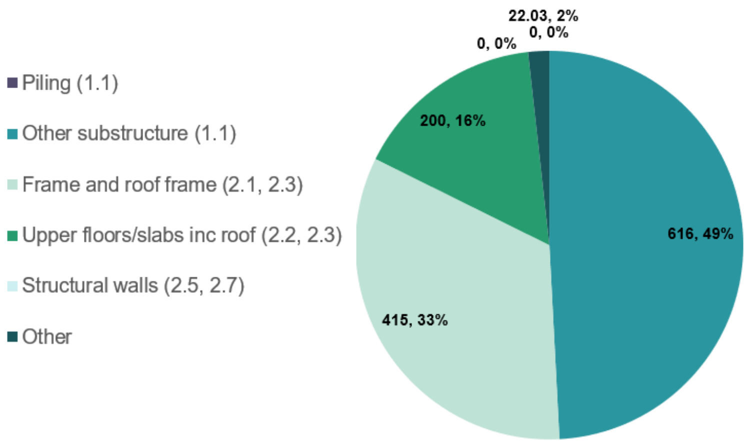


Figure 89 – Element emission breakdown

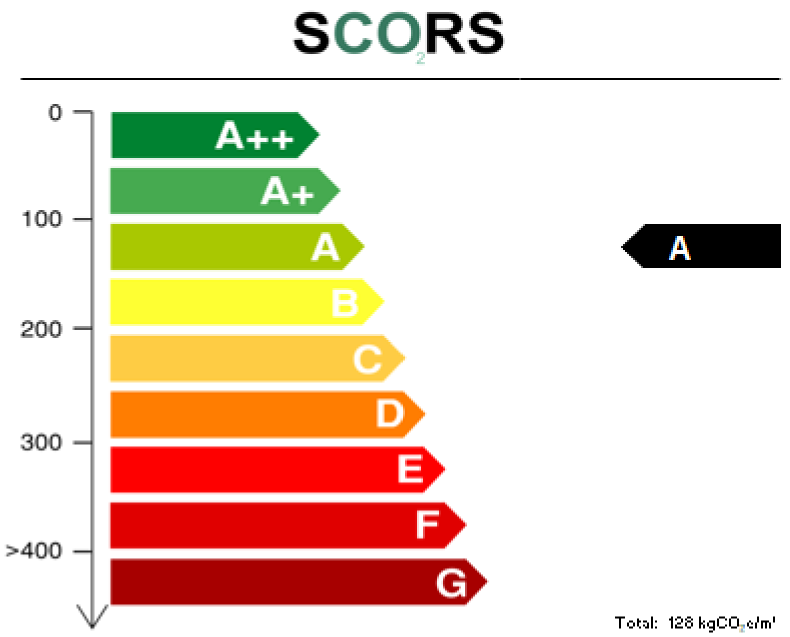
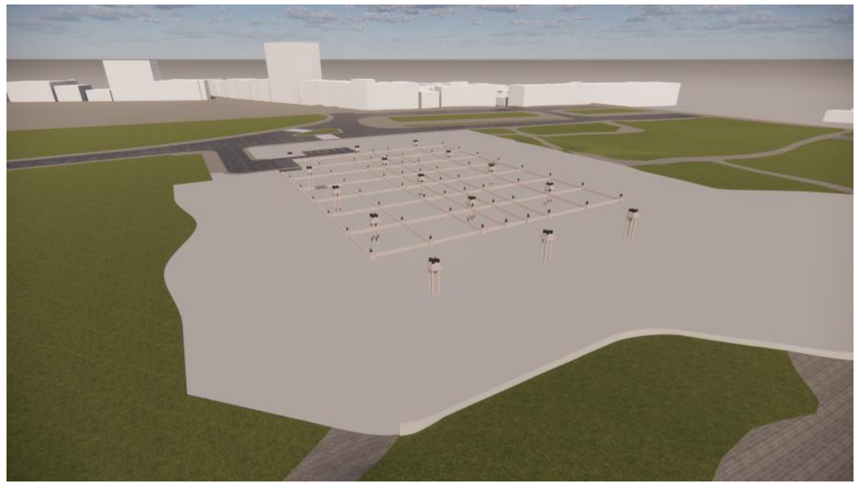


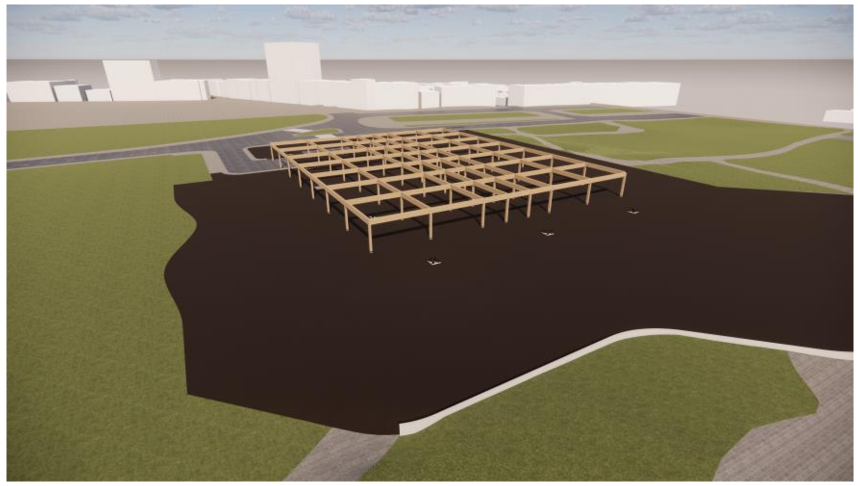
Figure 90 – SCORS performance rating

5.9 Construction Process

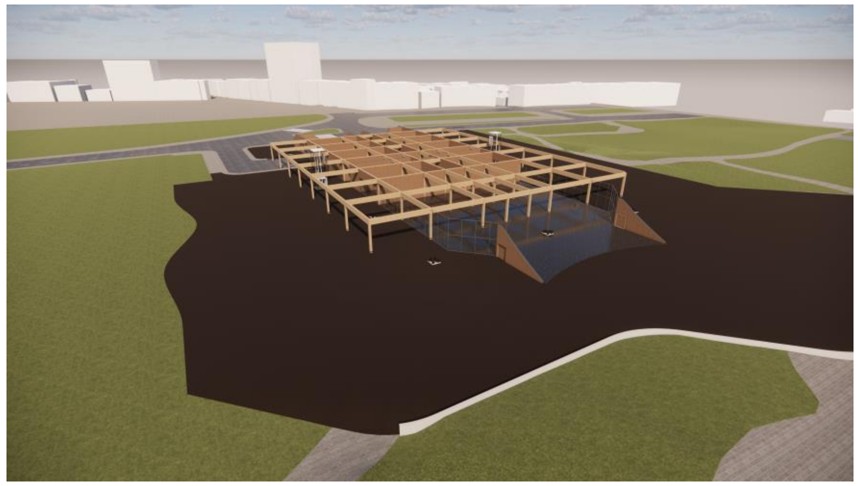
Figure 91 – 9 Construction Stages



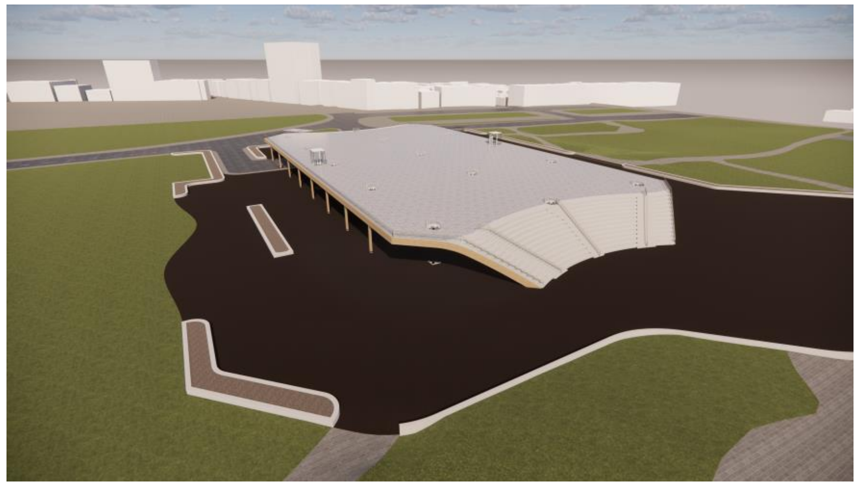
Stage 1 – Surveying, setting out and construction of concrete piles for canopy and pad and strip footing for deck structure.



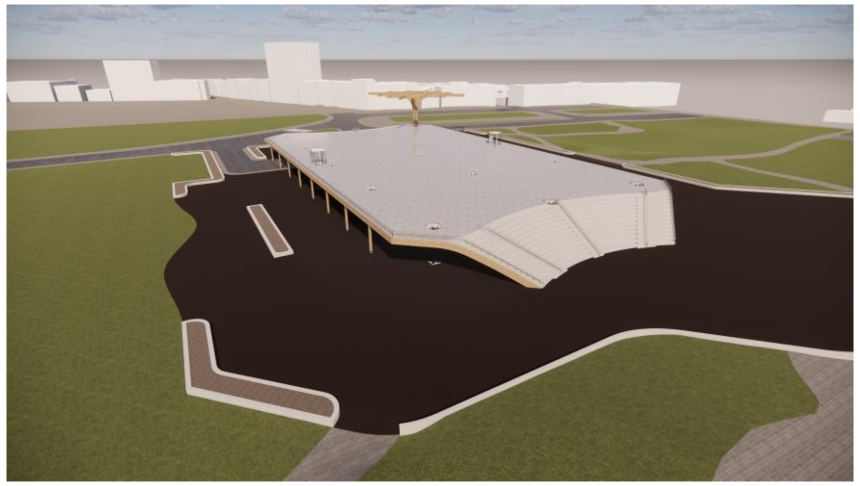
Stage 2 – Construction of Glulam Column and Beam Structure for Level 1 deck. Framing is bolted to knife plates attached to the pad foundation.



Stage 3 – Construction of Insulated Walls, Doors and insulated concrete floor, finished with black marble tiles.



Stage 4 – Construction of steel and glass lift. CLT slabs are lifted onto the beam structure, north and south stairs constructed on site and are bolted to the existing glulam structure. Plant beds constructed.



Stage 5 – Tree Column glulam structure is constructed one column at a time and tied together with steel plates.

Stage 6 – External Overhang structure (for slandering and draining the roof) is attached to the primary structure.

Stage 7 – CLT Slabs are lifted and screwed into the existing glulam structure. Timber ground decking is constructed.

Stage 8 – XPS Insulation is cut to fall and screwed into the CLT. Pipes, drains and skylights are constructed.

Stage 9 – Roof finish and solar panels placed. Landscaping and trees are placed. Open to public.

5.10 Project Review

Figure 92 – South View late afternoon

Figure 93 – South View Evening

Review and Conclusion

The redevelopment project for the Pyramids in Portsmouth has successfully achieved its goals of revitalising the seafront and creating a vibrant hub that attracts tourists throughout the year. By embracing sustainability, accessibility, and adaptability, the project has transformed an obsolete building into a thriving space that showcases iconic architecture, utilises key seafront views, and provides a multi-use environment.

Sustainability was at the heart of the project; the project passed performance targets with methods to reduce concrete and material choice to sequester carbon. The top deck's adaptability allows the building to remain relevant and well-used over time.

Some features should be refined if the proposal were to be developed further. For example, the top deck doesn't have a detailed drainage strategy. This would be important as rain can still fall past the 5m overhang. Additionally, the proposal could include renders showing the exact use opportunities for the deck so that the council can better understand the building's potential.

To conclude, the proposal utilises existing spaces and improves the flow and user experience around the structure. Overall, the building was designed holistically, with the structure and architecture designed in iteration collectively. Ultimately the structure will attract diverse vendors and create a welcoming community atmosphere. With its long-term vision of becoming a bustling central hub, the redevelopment will position Portsmouth's seafront as a premier destination, boosting the local economy and preserving its natural beauty.

Appendix A - Structural Calculations

WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex

Building data

Type of roof	Flat
Length of building	L = **20000** mm
Width of building	W = **20000** mm
Height to eaves	H = **1650** mm
Eaves type	Sharp
Total height	h = **1650** mm

Basic values

Location	Portsmouth
Wind speed velocity (FigureNA.1)	$v_{b,map}$ = **22.0** m/s
Distance to shore	L_{shore} = **0.10** km
Altitude above sea level	A_{alt} = **3.3**m
Altitude factor	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.003**
Fundamental basic wind velocity	$v_{b,0} = v_{b,map} \times c_{alt}$ = **22.1** m/s
Direction factor	c_{dir} = **1.00**
Season factor	c_{season} = **1.00**
Shape parameter K	K = **0.2**
Exponent n	n = **0.5**
Air density	ρ = **1.226** kg/m^3
Probability factor	$c_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n$ = **1.00**
Basic wind velocity (Exp. 4.1)	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **22.1** m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times v_b^2$ = **0.299** kN/m^2

Orography

Orography factor not significant	c_o = 1.0
Terrain category	Sea
Displacement height (sheltering effect excluded)	h_{dis} = 0mm

The velocity pressure for the windward face of the building with a 0 and 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 and 90 deg and roof

Reference height (at which q is sought)	z = **1650**mm
Displacement height (sheltering effects excluded)	h_{dis} = **0** mm
Exposure factor (Figure NA.7)	c_e = **1.91**
Peak velocity pressure	$q_p = c_e \times q_b$ = **0.57** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**
Height of element	h_{part} = **1650** mm
Size factor (Table NA.3)	c_s = **0.907**
Dynamic factor (Figure NA.9)	c_d = **1.000**
Structural factor	$c_sc_d = c_s \times c_d$ = **0.907**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	$q_{p,i}$ = **0.57** kN/m^2

Pressures and forces

Net pressure	$p = c_sc_d \times q_p \times c_{pe} - q_{p,i} \times c_{pi}$
Net force	$F_w = p_w \times A_{ref}$

Overall loading

Equiv leeward net force for overall section	$F_l = F_{w,wE}$ = **-8.9** kN
Net windward force for overall section	$F_w = F_{w,wD}$ = **8.2** kN
Lack of correlation (cl.7.2.2(3) – Note)	f_{corr} = **0.85** as h/W is 0.083
Overall loading overall section	$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **14.5** kN

SNOW LOAD

0.25 kN/m2 (EN 1991-1-3)

LIVE LOAD on ROOF Category H

0.4 kN/m2 (EN 1991-1-3) Roofs not accessible except for normal maintenance and repair.

DEAD LOAD on CLT ROOF SLAB

PV Panels 1 x 1.6 m:
Load = 0.04 kN/m2

Kalzip (Standing Seam) 1.2mm:
Load = 0.028 kN/m2

18mm plywood:
Load = 0.062 kN/m2

XPS Insulation Cut to Fall, Average of 465 mm:
Load = 0.13 kN/m2

Waterproofing Membrane 1.1mm:
Load = Negligible

Aluminium and Double-Glazed Skylight 3.68 x 3.68 m:
Load = 0.01 kN/m2

Total = 0.28 kN/m2

DEAD LOAD on BEAM SYSTEM

Above Total Load = 0.28 kN/m2

CLT Roof Slab XLAM-100-3 (100mm):

= 0.23 kN/m2

Total = 0.51 kN/m2

Distributed Load on Primary Beam:
(0.51x 104)/15= 3.5 kN/m

Distributed Load on Secondary Beam:
(0.51x75x)/15=2.6 kN/m

POINT LOAD on COLUMN

Dead Load = 452 kN
Live Load = 249.7 kN

Point LOAD on PILE CAP

Dead Load = 634 kN
Live Load = 249.7 kN

Point LOAD per PILE

Dead Load = 158 kN
Live Load = 62.4 kN

CLT Roof

<u>CROSS-LAMINATED TIMBER FLOOR PANEL DESIGN</u>

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Panel details

Length of panel	L = **4680** mm
Width of panel	W = **5000** mm

Panel loads

Self weight of panel	$s_{wt} = \rho_{mean} \times g_{acc} \times h$ = **0.51** kN/m^2
Total weight of panel	$s_{wt\_total} = s_{wt} \times L \times W$ = **11.933** kN
Permanent load	w_G = **0.34** kN/m^2
Imposed load	w_Q = **0.65** kN/m^2

Partial factors on actions

Permanent unfavourable actions - Table A.3	γ_G = **1.35**
Variable unfavourable actions - Table A.3	γ_Q = **1.50**

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3	γ_M = **1.250**

Member details

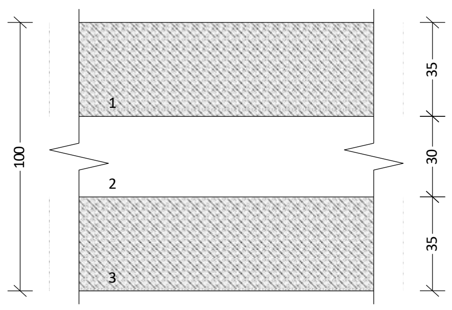
Load duration - cl.2.3.1.2	Long-term
Service class - cl.2.3.1.3	1

Cross-laminated timber section details

Manufacturer	X-LAM Alliance
Section	XLAM-100-3
Build-up	35l - 30w - 35l

Material details

Timber strength class - EN 338:2016 Table1	C50
Mean rolling shear modulus	$G_{r.mean}$ = **50** N/mm^2



XLAM-100-3 cross-laminated timber section

Net cross-sectional area, $A_{y,net}$, 700 cm^2/m

Net cross-sectional area, $A_{z,net}$, 300 cm^2/m

Net section modulus, $W_{y,net}$, 1621.7 cm^3/m

Net section modulus, $W_{z,net}$, 150 cm^3/m

Effective section modulus, $W_{y,eff}$, 1513.3 cm^3/m

Effective section modulus, $W_{z,eff}$, 150 cm^3/m

Net second moment of area, $I_{y,net}$, 8108.3 cm^4/m

Net second moment of area, $I_{z,net}$, 225 cm^4/m

Effective second moment of area, $I_{y,eff}$, 7566.7 cm^4/m

Effective second moment of area, $I_{z,eff}$, 225 cm^4/m

Net radius of inertia, $i_{y,net}$, 3.4 cm

Net radius of inertia, $i_{z,net}$, 0.9 cm

Effective radius of inertia, $i_{y,eff}$, 3.3 cm

Effective radius of inertia, $i_{z,eff}$, 0.9 cm

Timber strength class C50

Characteristic bending strength, $f_{m.k}$, 50 N/mm^2

Characteristic shear strength, $f_{v.k}$, 4 N/mm^2

Characteristic compression strength parallel to grain, $f_{c.0.k}$, 30 N/mm^2

Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 3 N/mm^2

Characteristic tension strength parallel to grain, $f_{t.0.k}$, 33.5 N/mm^2

Mean modulus of elasticity, $E_{0.mean}$, 16000 N/mm^2

Fifth percentile modulus of elasticity, $E_{0.05}$, 10700 N/mm^2

Shear modulus of elasticity, G_{mean}, 1000 N/mm^2

Mean rolling shear modulus, $G_{r.mean}$, 50 N/mm^2

Characteristic density, ρ_k, 430 kg/m^3

Mean density, ρ_{mean}, 520 kg/m^3

<u>Design section s1 - Check design at mid-span of panel</u>

Span details

Longitudinal span	$L_{e,y} = L - L_{b,y}$ = **4580** mm
Longitudinal bearing	$L_{b,y}$ = **100** mm

Analysis results

Design bending moment	$M_{y,d} = (\gamma_G \times (w_G + s_{wt}) + \gamma_Q \times w_Q) \times L_{e,y}^2 / 8$ = **5.565** kNm/m
Maximum deflection	$\delta_y = 5 \times ((w_G + s_{wt}) + w_Q) \times L_{e,y}^4 \times (1 + k_{def}) / (384 \times E_{0.mean} \times I_{y,eff})$ = **11.357** mm

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.7**
Deformation factor - Table 3.2	k_{def} = **0.6**

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **5.565** kNm/m
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_{y,net}$ = **3.432** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **28** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.123**

PASS - Design bending strength exceeds design bending stress

Deflection check

Maximum deflection	δ_y = **11.357** mm
Allowable deflection	$\delta_{y,Allowable}$ = min($L_{e,y}$ / 200, 14 mm) = **14** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.811**

PASS - Allowable deflection limit exceeds deflection

<u>Design section s2 - Check design at support of panel</u>

Span details

Longitudinal span	$L_{e,y} = L - L_{b,y}$ = **4580** mm
Longitudinal bearing	$L_{b,y}$ = **100** mm

Analysis results

Design perpendicular compression	$F_{c,y,90,d} = (\gamma_G \times (w_G + s_{wt}) + \gamma_Q \times w_Q) \times L_{e,y} / 2$ = **4.86** kN/m
Design shear force	$F_{y,d} = (\gamma_G \times (w_G + s_{wt}) + \gamma_Q \times w_Q) \times L_{e,y} / 2$ = **4.86** kN/m

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.7**
Deformation factor - Table 3.2	k_{def} = **0.6**

Check compression perpendicular to the grain - Section 6.1.5

Design perpendicular compression	$F_{c,y,90,d}$ = **4.86** kN/m
Design compressive stress	$\sigma_{c,y,90,d} = F_{c,y,90,d} / L_{b,y}$ = **0.049** N/mm^2
Design compressive strength	$f_{c,y,90,d} = k_{mod} \times f_{c.90.k} / \gamma_M$ = **1.68** N/mm^2
	$\sigma_{c,y,90,d} / f_{c,y,90,d}$ = **0.029**

PASS - Design perpendicular compressive strength exceeds design perpendicular compressive stress

Check shear force - Section 6.1.7

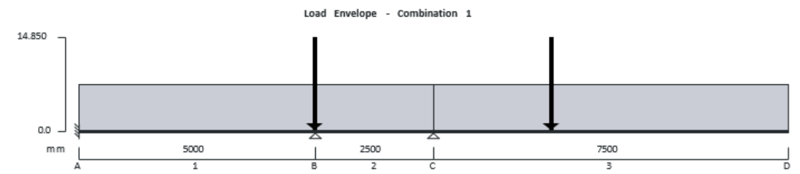
Design shear force	$F_{y,d}$ = **4.86** kN/m
Design shear stress	$\tau_{y,d} = F_{y,d} \times S_{R,y,net} / I_{y,net}$ = **0.068** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **2.24** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.03**

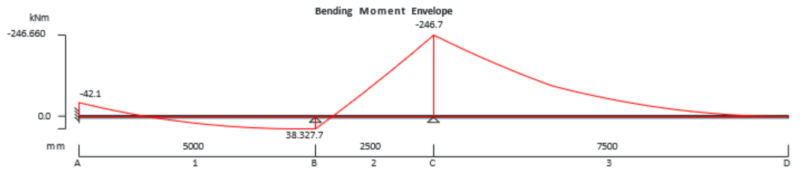
PASS - Design shear strength exceeds design shear stress

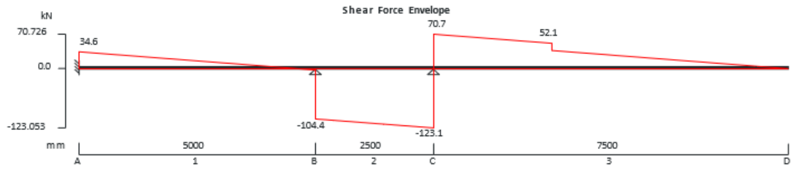
Primary Beam

GLULAM BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1







Analysis results

Maximum moment M_{max} = **38.167** kNm M_{min} = **-246.660** kNm

Design moment M = max(abs(M_{max}),abs(M_{min})) = **246.660** kNm

Maximum shear F_{max} = **70.726** kN F_{min} = **-123.053** kN

Design shear F = max(abs(F_{max}),abs(F_{min})) = **123.053** kN

Total load on beam W_{tot} = **141.452** kN

Glulam section details

Breadth of glulam section b = **320** mm

Depth of glulam section h = **900** mm

Number of glulam sections in member N = **1**

Overall breadth of glulam member $b_b = N \times b$ = **320** mm

User defined glulam properties

Characteristic bending strength $f_{m.g.k}$ = **36** N/mm²

Characteristic tensile strength parallel $f_{t.0.g.k}$ = **26** N/mm²

Characteristic tensile strength perpendicular $f_{t.90.g.k}$ = **0.5** N/mm²

Characteristic compressive strength parallel $f_{c.0.g.k}$ = **31** N/mm²

Characteristic compressive strength perpendicular $f_{c.90.g.k}$ = **3.6** N/mm²

Characteristic shear strength $f_{v.g.k}$ = **3.8** N/mm²

Mean modulus of elasticity parallel $E_{0.g.mean}$ = **20000** N/mm²

5% modulus of elasticity parallel $E_{0.g.05}$ = **18000** N/mm²

Mean modulus of elasticity perpendicular $E_{90.g.mean}$ = **490** N/mm²

Mean shear modulus $G_{g.mean}$ = **910** N/mm²

Characteristic density $\rho_{g.k}$ = **400** kg/m³

Member details

Load duration - cl.2.3.1.2 **Long-term**

Service class of timber - cl.2.3.1.3 **1**

Length of span 1 L_{s1} = **5000** mm

Length of span 2 L_{s2} = **2500** mm

Length of span 3 L_{s3} = **7500** mm

Length of bearing L_b = **175** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **288000** mm²

Section modulus $W_y = N \times b \times h^2 / 6$ = **43200000** mm³

$W_z = h \times (N \times b)^2 / 6$ = **15360000** mm³

Second moment of area $I_y = N \times b \times h^3 / 12$ = **19440000000** mm⁴

$I_z = h \times (N \times b)^3 / 12$ = **2457600000** mm⁴

Radius of gyration $r_y = \sqrt{(I_y / A)}$ = **259.8** mm

$r_z = \sqrt{(I_z / A)}$ = **92.4** mm

γ = **0.257**

Torsional moment of inertia $I_{tor} = \gamma \times \max(h, N \times b) \times \min(h, N \times b)^3$ = **7573708800** mm⁴

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.250**

Modification factors

Modification factor for load duration and moisture content - Table 3.1 k_{mod} = **0.700**

Deformation factor for service classes - Table 3.2 k_{def} = **0.600**

Depth factor for bending - exp.3.2 $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.2 $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.670**

Load configuration factor - exp.6.4 $k_{c.90}$ = **1.750**

System strength factor - cl.6.6 k_{sys} = **1.000**

Effective length - Table 6.1 $L_{ef} = 0.8 \times L_{s3}$ = **6000** mm

Critical bending stress - exp.6.31 $\sigma_{m.crit} = \pi \times \sqrt{[E_{0.g.05} \times I_z \times G_{0.g.05} \times I_{tor}]} / (L_{ef} \times W_y)$ = **235.308** N/mm²

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.g.k} / \sigma_{m.crit}]}$ = **0.391**

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{C\_max} / (N \times b \times (L_b + 2 \times \min(L_b, 30\text{ mm})))$ = **2.577** N/mm²

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.g.k} / \gamma_M$ = **3.528** N/mm²

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.730**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = M / W_y$ = **5.710** N/mm²

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.g.k} / \gamma_M$ = **20.160** N/mm²

$\sigma_{m.d} / f_{m.d}$ = **0.283**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.957** N/mm²

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.g.k} / \gamma_M$ = **2.128** N/mm²

$\tau_d / f_{v.d}$ = **0.450**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(14\text{ mm}, 0.003 \times L_{s3})$ = **14.000** mm

Instantaneous deflection due to permanent load δ_{instG} = **7.581** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **12.129** mm

Instantaneous deflection due to variable load δ_{instQ} = **1.139** mm

Factor for quasi-permanent variable action ψ_2 = **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **1.344** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **13.474** mm

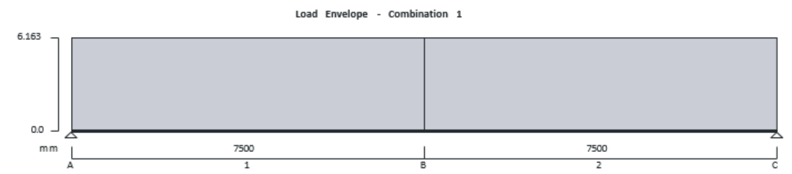
$\delta_{fin} / \delta_{lim}$ = **0.962**

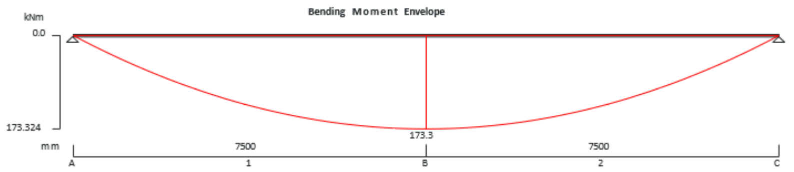
PASS - Total final deflection is less than the deflection limit

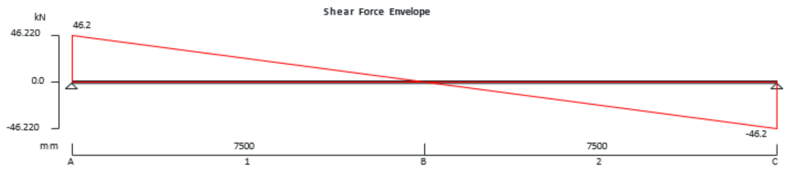
Secondary Beam

GLULAM BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1







Analysis results

Maximum moment M_{max} = **173.324** kNm M_{min} = **0.000** kNm

Design moment $M = max(abs(M_{max}),abs(M_{min}))$ = **173.324** kNm

Maximum shear F_{max} = **46.220** kN F_{min} = **-46.220** kN

Design shear $F = max(abs(F_{max}),abs(F_{min}))$ = **46.220** kN

Total load on beam W_{tot} = **92.440** kN

Glulam section details

Breadth of glulam section b = **320** mm

Depth of glulam section h = **900** mm

Number of glulam sections in member N = **1**

Overall breadth of glulam member $b_b = N \times b$ = **320** mm

User defined glulam properties

Characteristic bending strength $f_{m.g.k}$ = **32** N/mm^2

Characteristic tensile strength parallel $f_{t.0.g.k}$ = **25.6** N/mm^2

Characteristic tensile strength perpendicular $f_{t.90.g.k}$ = **0.5** N/mm^2

Characteristic compressive strength parallel $f_{c.0.g.k}$ = **32** N/mm^2

Characteristic compressive strength perpendicular $f_{c.90.g.k}$ = **2.5** N/mm^2

Characteristic shear strength $f_{v.g.k}$ = **3.5** N/mm^2

Mean modulus of elasticity parallel $E_{0.g.mean}$ = **20000** N/mm^2

5% modulus of elasticity parallel $E_{0.g.05}$ = **18000** N/mm^2

Mean modulus of elasticity perpendicular $E_{90.g.mean}$ = **300** N/mm^2

Mean shear modulus $G_{g.mean}$ = **650** N/mm^2

Characteristic density $\rho_{g.k}$ = **440** kg/m^3

Member details

Load duration - cl.2.3.1.2 **Long-term**

Service class of timber - cl.2.3.1.3 **1**

Length of span 1 L_{s1} = **7500** mm

Length of span 2 L_{s2} = **7500** mm

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **288000** mm^2

Section modulus $W_y = N \times b \times h^2 / 6$ = **43200000** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **15360000** mm^3

Second moment of area $I_y = N \times b \times h^3 / 12$ = **19440000000** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **2457600000** mm^4

Radius of gyration $r_y = \sqrt{(I_y / A)}$ = **259.8** mm

$r_z = \sqrt{(I_z / A)}$ = **92.4** mm

γ = **0.257**

Torsional moment of inertia $I_{tor} = \gamma \times max(h, N \times b) \times min(h, N \times b)^3$ = **7573708800** mm^4

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.250**

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.700**

Deformation factor for service classes - Table 3.2 k_{def} = **0.600**

Depth factor for bending - exp.3.2 $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.2 $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.670**

Load configuration factor - exp.6.4 $k_{c.90}$ = **1.750**

System strength factor - cl.6.6 k_{sys} = **1.000**

Effective length - Table 6.1 $L_{ef} = 0.8 \times L_{s2}$ = **6000** mm

Critical bending stress - exp.6.31 $\sigma_{m.crit} = \pi \times \sqrt{[E_{0.g.05} \times I_z \times G_{0.g.05} \times I_{tor}]} / (L_{ef} \times W_y)$ = **235.308** N/mm^2

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.g.k} / \sigma_{m.crit}]}$ = **0.369**

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times (L_b + min(L_b, 30 mm)))$ = **1.111** N/mm^2

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.g.k} / \gamma_M$ = **2.450** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.453**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = M / W_y$ = **4.012** N/mm^2

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.g.k} / \gamma_M$ = **17.920** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **0.224**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.359** N/mm^2

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.g.k} / \gamma_M$ = **1.960** N/mm^2

$\tau_d / f_{v.d}$ = **0.183**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = min(14 mm, 0.003 \times L_{s2})$ = **14.000** mm

Instantaneous deflection due to permanent load δ_{instG} = **7.208** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **11.532** mm

Instantaneous deflection due to variable load δ_{instQ} = **1.219** mm

Factor for quasi-permanent variable action ψ_2 = **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **1.439** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **12.971** mm

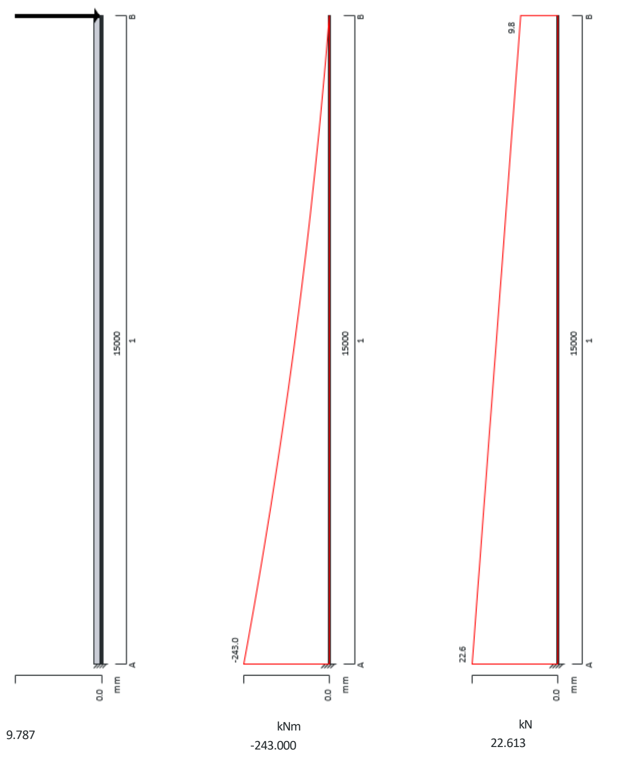
$\delta_{fin} / \delta_{lim}$ = **0.927**

PASS - Total final deflection is less than the deflection limit

Column Design

GLULAM COLUMN ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1



Load Envelope - Combination 1   Bending Moment Envelope   Shear Force Envelope

Analysis results

Maximum moment — M_{max} = **0.000** kNm M_{min} = **-243.000** kNm

Design moment — $M = \max(abs(M_{max}),abs(M_{min}))$ = **243.000** kNm

Maximum shear — F_{max} = **22.613** kN F_{min} = **9.788** kN

Design shear — $F = \max(abs(F_{max}),abs(F_{min}))$ = **22.613** kN

Total load on beam — W_{tot} = **22.613** kN

Glulam section details

Breadth of glulam section — b = **320** mm

Depth of glulam section — h = **1500** mm

Number of glulam sections in member — N = **1**

Overall breadth of glulam member — $b_b = N \times b$ = **320** mm

Glulam strength class - EN 14080:2013 - Table 5 — **GL32h**

Member details

Load duration - cl.2.3.1.2

Service class of timber - cl.2.3.1.3

Length of span

Length of bearing

Section properties

Cross sectional area of member — $A = N \times b \times h$ = **480000** mm^2

Section modulus — $W_y = N \times b \times h^2 / 6$ = **120000000** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **25600000** mm^3

Second moment of area — $I_y = N \times b \times h^3 / 12$ = **90000000000** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **4096000000** mm^4

Radius of gyration — $r_y = \sqrt{(I_y / A)}$ = **433.0** mm

$r_z = \sqrt{(I_z / A)}$ = **92.4** mm

γ = **0.287**

Torsional moment of inertia — $I_{tor} = \gamma \times \max(h, N \times b) \times \min(h, N \times b)^3$ = **14107084800** mm^4

Load configuration factor - exp.6.4 — $k_{c.90}$ = **1.750**

System strength factor - cl.6.6 — k_{sys} = **1.000**

Effective length - Table 6.1 — $L_{ef} = 0.8 \times L_{s1}$ = **12000** mm

Critical bending stress - exp.6.31 — $\sigma_{m.crit} = \pi \times \sqrt{[E_{0.g.05} \times I_z \times G_{0.g.05} \times I_{tor}]} / (L_{ef} \times W_y)$ = **48.922** N/mm^2

Relative slenderness for bending - exp.6.30 — $\lambda_{rel.m} = \sqrt{[f_{m.g.k} / \sigma_{m.crit}]}$ = **0.809**

Lateral buckling factor - exp.6.34 — $k_{crit} = 1.56 - 0.75 \times \lambda_{rel.m}$ = **0.953**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress — $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times (L_b + \min(L_b, 30\text{ mm})))$ = **0.544** N/mm^2

Design compressive strength — $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.g.k} / \gamma_M$ = **2.450** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.222**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress — $\sigma_{m.d} = M / W_y$ = **2.025** N/mm^2

Design bending strength — $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.g.k} / \gamma_M$ = **17.085** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **0.119**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress — $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.105** N/mm^2

Permissible shear stress — $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.g.k} / \gamma_M$ = **1.960** N/mm^2

$\tau_d / f_{v.d}$ = **0.054**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit — $\delta_{lim} = \min(14\text{ mm}, 0.003 \times L_{s1})$ = **14.000** mm

Instantaneous deflection due to permanent load — δ_{instG} = **6.382** mm

Final deflection due to permanent load — $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **10.211** mm

Instantaneous deflection due to variable load — δ_{instQ} = **2.822** mm

Factor for quasi-permanent variable action — ψ_2 = **0.3**

Final deflection due to variable load — $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **3.330** mm

Total final deflection — $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **13.542** mm

$\delta_{fin} / \delta_{lim}$ = **0.967**

PASS - Total final deflection is less than the deflection limit

GLULAM COLUMN WITH BIAXIAL BENDING AND AXIAL COMPRESSION

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.18

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 — γ_M = **1.250**

Member details

Load duration - cl.2.3.1.2 — Long-term

Service class - cl.2.3.1.3 — 1

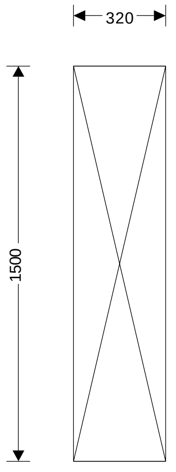
Glulam section details

Number of timber sections in member — N = **1**

Breadth of sections — b = **320** mm

Depth of sections — h = **1500** mm

Glulam strength class - EN 14080:2013 - Table 6 — **GL32c**



320x1500 glulam section

Cross-sectional area, A, 480000 mm²

Section modulus, W_y, 1.2×10⁸ mm³

Section modulus, W_z, 25600000 mm³

Second moment of area, I_y, 90000000000 mm⁴

Second moment of area, I_z, 4096000000 mm⁴

Radius of gyration, i_y, 433 mm

Radius of gyration, i_z, 92.4 mm

Glulam strength class GL32c

Characteristic bending strength, $f_{m.g.k}$, 32 N/mm²

Characteristic shear strength, $f_{v.g.k}$, 3.5 N/mm²

Characteristic compression strength parallel to grain, $f_{c.0.g.k}$, 24.5 N/mm²

Characteristic compression strength perpendicular to grain, $f_{c.90.g.k}$, 2.5 N/mm²

Characteristic tension strength parallel to grain, $f_{t.0.g.k}$, 19.5 N/mm²

Mean modulus of elasticity, $E_{0.g.mean}$, 13500 N/mm²

Fifth percentile modulus of elasticity, $E_{0.g.05}$, 11200 N/mm²

Shear modulus of elasticity, $G_{g.mean}$, 650 N/mm²

Characteristic density, $r_{g.k}$, 400 kg/m³

Span details

Unbraced length - Major axis — L_y = **6000** mm

Effective length - Major axis — $L_{e,y} = 0.9 \times L_y$ = **5400** mm

Unbraced length - Minor axis — L_z = **2000** mm

Effective length - Minor axis — $L_{e,z} = 0.9 \times L_z$ = **1800** mm

Bearing length — L_b = **100** mm

Analysis results

Design bending moment - Major axis — $M_{y,d}$ = **243** kNm

Design bending moment - Minor axis — $M_{z,d}$ = **243** kNm

Design shear force - Major axis — $F_{y,d}$ = **22.6** kN

Design axial compression force — P_d = **701.8** kN

Modification factors

Duration of load and moisture content - Table 3.1 — k_{mod} = **0.7**

Deformation factor - Table 3.2 — k_{def} = **0.6**

Depth factor for bending - Minor axis - exp.3.2 — $k_{h,m,z} = \min((600 \text{ mm} / b)^{0.1}, 1.1)$ = **1.065**

Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.67**

Check compression parallel to the grain - cl.6.1.4

Design axial compression — P_d = **701.8** kN

Design compressive stress — $\sigma_{c,0,d} = P_d / A$ = **1.462** N/mm²

Design compressive strength — $f_{c,0,d} = k_{mod} \times f_{c.0.g.k} / \gamma_M$ = **13.720** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.107**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check shear force - Section 6.1.7

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **22.6** kN

Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.105** N/mm²

Design shear strength — $f_{v,y,d} = k_{mod} \times f_{v.g.k} / \gamma_M$ = **1.960** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.054**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment in major axis — $M_{y,d}$ = **243** kNm

Design bending stress in major axis — $\sigma_{m,y,d} = M_{y,d} / W_y$ = **2.025** N/mm²

Design bending strength in major axis — $f_{m,y,d} = k_{mod} \times f_{m.g.k} / \gamma_M$ = **17.920** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.113**

Design bending moment in minor axis — $M_{z,d}$ = **243** kNm

Design bending stress in minor axis — $\sigma_{m,z,d} = M_{z,d} / W_z$ = **9.492** N/mm²

Design bending strength in minor axis — $f_{m,z,d} = k_{h,m,z} \times k_{mod} \times f_{m.g.k} / \gamma_M$ = **19.083** N/mm²

$\sigma_{m,z,d} / f_{m,z,d}$ = **0.497**

Combined bending checks - eq.6.11 & eq.6.12 — $\sigma_{m,y,d} / f_{m,y,d} + k_m \times \sigma_{m,z,d} / f_{m,z,d}$ = **0.461**

$k_m \times \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d}$ = **0.577**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 — $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} + k_m \times \sigma_{m,z,d} / f_{m,z,d}$ = **0.473**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d}$ = **0.588**

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending — $L_{e,y} = 0.9 \times L_y$ = **5400** mm

Slenderness ratio — $\lambda_y = L_{e,y} / i_y$ = **12.471**

Relative slenderness ratio - exp. 6.21 — $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.g.k} / E_{0.g.05})}$ = **0.186**

Effective length for z-axis bending — $L_{e,z} = 0.9 \times L_z$ = **1800** mm

Slenderness ratio — $\lambda_z = L_{e,z} / i_z$ = **19.486**

Relative slenderness ratio - exp. 6.22 — $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.g.k} / E_{0.g.05})}$ = **0.29**

Both $\lambda_{rel,y}$ <= 0.3 and $\lambda_{rel,z}$ <= 0.3, column stability check not required

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Fifth percentile shear modulus parallel to the grain — $G_{0.g.05} = E_{0.g.05} / 16$ = **700** N/mm²

Torsion factor — β = **0.287**

Torsional moment of inertia — $I_{tor} = \beta \times \min(h, b)^3 \times \max(h, b)$ = **14107084800** mm⁴

Effective length - Table 6.1 — $L_{ef} = 0.9 \times 2000 \text{ mm} + 2 \times h$ = **4800** mm

Critical bending stress - exp.6.31 — $\sigma_{m,crit} = \pi \times \sqrt{(E_{0.g.05} \times I_z \times G_{0.g.05} \times I_{tor})} / (L_{ef} \times W_y)$ = **116.087** N/mm²

Relative slenderness for bending - exp.6.30 — $\lambda_{rel,m} = \sqrt{(f_{m.g.k} / \sigma_{m,crit})}$ = **0.525**

Lateral buckling factor - exp.6.34 — k_{crit} = **1.000**

Beam stability check - exp.6.35 — $(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / f_{c,0,d}$ = **0.119**

PASS - Beam stability is acceptable

Pile Analysis

PILE ANALYSIS

In accordance with EN 1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

Pile details

Installation method — Drilled

Shape — 350 mm diameter

Length — L = **3800** mm

Material details

Material — Concrete

Concrete strength class — C40/50

Part. factor, concrete (EN1992-1-1 cl. 2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (EN1992-1-1 cl. 3.1.6(1)) — α_{cc} = **0.85**

Characteristic compression cylinder strength — f_{ck} = **40** N/mm^2

Design comp. strength (EN1992-1-1 cl. 3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm^2

Mean value of cyl. strength (EN1992-1-1 Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **48.0** N/mm^2

Secant mod. of elasticity (EN1992-1-1 Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **35.2** kN/mm^2

Modulus of elasticity — $E = E_{cm}$ = **35.2** kN/mm^2

Geometric properties

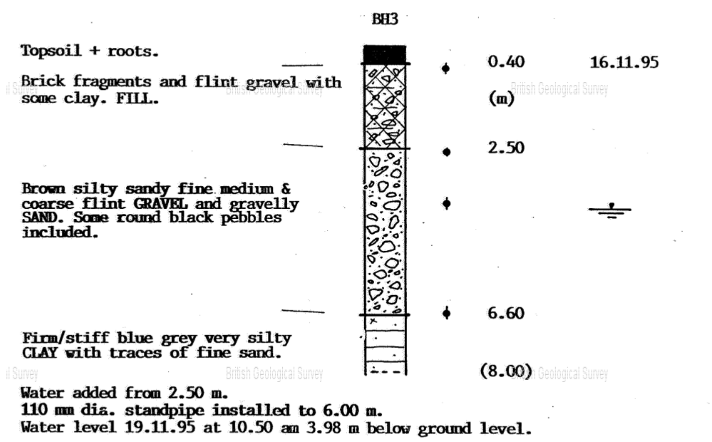
Pile section depth — h = **350** mm

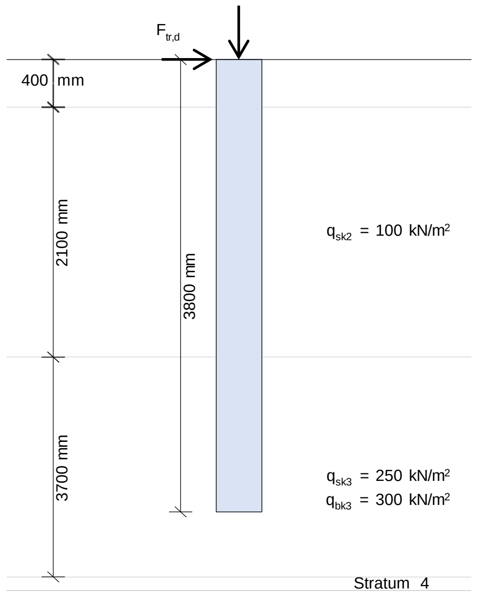
Bearing area — $A_{bearing} = \pi \times h^2 / 4$ = **0.096** m^2

Pile perimeter — $Perim_{pile} = \pi \times h$ = **1.1** m

Moment of inertia — $I = \pi \times h^4 / 64$ = **73662** cm^4

F_d





q_{ski} = Characteristic value, shaft resistance, q_{bki} = Characteristic value, base

Stratum details

Stratum	Geomaterial	Thickness, $t_{stratai}$ (mm)	Characteristic value, base, q_{bki} (kN/m^2)	Characteristic value, shaft, q_{ski} (kN/m^2)
1	Cohesive	400	-	100
2	Cohesive	2100	-	162.5
3	Cohesionless	3700	-	250
4	Cohesive	3000	350	200

Action details

Characteristic perm. unfav. action, compression — $G_{c,k,unfav}$ = **158** kN

Characteristic perm. fav. action, compression — $G_{c,k,fav}$ = **0** kN

Characteristic variable unfav. action, compression — $Q_{c,k}$ = **62.4** kN

Characteristic perm. unfav. action, tension — $G_{t,k,unfav}$ = **0** kN

Characteristic perm. fav. action, tension — $G_{t,k,fav}$ = **0** kN

Characteristic variable unfav. action, tension — $Q_{t,k}$ = **0** kN

Characteristic unfavourable perm. lateral action — $G_{tr,k,unfav}$ = **0** kN

Characteristic favourable permanent lateral action — $G_{tr,k,fav}$ = **0** kN

Characteristic variable lateral action — $Q_{tr,k}$ = **22.6** kN

Geotechnical partial and model factors:

Design approach 1:

Model factor on axial resistance — γ_{model} = **1.40**

Permanent unfavourable, A1 (Table A.3) — $\gamma_{G,unfav,A1}$ = **1.35**

Permanent favourable, A1 (1) — $\gamma_{G,fav,A1}$ = **1.00**

Variable unfavourable, A1 (Table A.3) — $\gamma_{Q,A1}$ = **1.50**

Permanent unfavourable, A2 (Table A.3) — $\gamma_{G,unfav,A2}$ = **1.00**

Permanent favourable, A2 (Table A.3) — $\gamma_{G,fav,A2}$ = **1.00**

Variable unfavourable, A2 (Table A.3) — $\gamma_{Q,A2}$ = **1.30**

Characteristic axial resistance

Characteristic axial base resistance — $R_{bk} = A_{bearing} \times q_{bk}$ = **57.7** kN

Characteristic axial shaft resistance per stratum

Stratum 1 — $R_{sk1} = q_{sk1} \times Perim_{pile} \times t_{strata1}$ =**44** kN

Stratum 2 — $R_{sk2} = q_{sk2} \times Perim_{pile} \times t_{strata2}$ =**375.2** kN

Stratum 3 — $R_{sk3} = q_{sk3} \times Perim_{pile} \times (L - D_{strata3})$ =**412.3** kN

Characteristic total axial shaft resistance — $R_{sk} = R_{sk1} + R_{sk2} + R_{sk3}$ = **831.5** kN

Axial compressive resistance

Load combination 1: A1 + M1 + R1

Design compression action — $F_{c,d,C1} = \gamma_{G,unfav,A1} \times G_{c,k,unfav} - \gamma_{G,fav,A1} \times G_{c,k,fav} + \gamma_{Q,A1} \times Q_{c,k}$

$F_{c,d,C1}$ = **306.9** kN

Partial resistance factor, bearing (Table A.7) — $\gamma_{b,R1}$ = **1.25**

Partial resistance factor, shaft (Table A.7) — $\gamma_{s,R1}$ = **1.00**

Design compressive resistance — $R_{c,d,C1} = (R_{bk} / \gamma_{b,R1} + R_{sk} / \gamma_{s,R1}) / \gamma_{model}$ = **626.9** kN

$F_{c,d,C1} / R_{c,d,C1}$ = **0.49**

PASS - Design compressive resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design compression action — $F_{c,d,C2} = \gamma_{G,unfav,A2} \times G_{c,k,unfav} - \gamma_{G,fav,A2} \times G_{c,k,fav} + \gamma_{Q,A2} \times Q_{c,k}$

$F_{c,d,C2}$ = **239.1** kN

Partial resistance factor, bearing (Table A.7) — $\gamma_{b,R4}$ = **1.60**

Partial resistance factor, shaft (Table A.7) — $\gamma_{s,R4}$ = **1.30**

Design compressive resistance — $R_{c,d,C2} = (R_{bk} / \gamma_{b,R4} + R_{sk} / \gamma_{s,R4}) / \gamma_{model}$ = **482.7** kN

$F_{c,d,C2} / R_{c,d,C2}$ = **0.495**

PASS - Design compressive resistance exceeds design load

Lateral analysis properties

Pile head fixity — Free

Eccentricity of applied action — e_{actual} = **0** mm

Lateral action duration — Long-term

Lateral stratum details

Overburden pressure, — $p_{ozSi} = \sum_{i=1}^{n} \gamma_i \times t_{stratai}$

Stratum	Characteristic cohesion, $c_{i,k}$ (kN/m^2)	Characteristic friction angle, $\phi_{i,k}$ (degrees)	Characteristic unit weight of soil, $\gamma_{i,k}$ (kN/m^3)	Characteristic overburden pressure, $p_{ozSi,k}$ (kN/m^2)
1	0	15	16	6.4
2	0	35	20.5	49.45
3	0	33	21.5	129
4	100	25	20	189

Load combination 1: A1 + M1 + R1

Partial factors:

Angle of shearing resistance (Table A.4) $\gamma_{\phi',M1}$ = **1.00**

Effective cohesion (Table A.4) $\gamma_{c',M1}$ = **1.00**

Undrained shear strength (Table A.4) $\gamma_{cu,M1}$ = **1.00**

Weight density (Table A.4) $\gamma_{\gamma,M1}$ = **1.00**

Lateral resistance factor $\gamma_{tr,R1}$ = **1.00**

Stratum	Design cohesion, $c_{i,d}$ (kN/m^2)	Design friction angle, $\phi_{i,d}$ (degrees)	Design unit weight of soil, $\gamma_{i,d}$ (kN/m^3)	Design overburden pressure, $p_{ozSi,d}$ (kN/m^2)
1	0	15	16	6.4
2	0	35	20.5	49.45
3	0	33	21.5	129
4	100	25	20	189

Resisting soil is divided into 10 segments to determine lateral resistance

From iteration, assume depth of point of rotation X = **3220** mm

Distance from lateral action to ground e = e_{actual} = **0** mm

Sum of moments about point of load application near zero

$\Sigma M_{tr} = M_{trS1} + M_{trS2} + M_{trS3} + M_{trS4} + M_{trS5} + M_{trS6} + M_{trS7} + M_{trS8} + M_{trS9t} + M_{trS9b} + M_{trS10}$ = **0** kNm

Sum of moments about point of rotation

$\Sigma M_X = M_{XS1} + M_{XS2} + M_{XS3} + M_{XS4} + M_{XS5} + M_{XS6} + M_{XS7} + M_{XS8} + M_{XS9t} + M_{XS9b} + M_{XS10}$ = **609.1** kNm

Calculated soil lateral resist. (Tomlinson Eqn 7.52) $R_{tr,calc} = \Sigma M_X / (e + X)$ = **189.1** kN

Design lateral action $F_{tr,d,C1} = \gamma_{G,unfav,A1} \times G_{tr,k,unfav} - \gamma_{G,fav,A1} \times G_{tr,k,fav} + \gamma_{Q,A1} \times Q_{tr,k}$

$F_{tr,d,C1}$ = **33.9** kN

Design lateral resistance $R_{tr,d,C1} = R_{tr,calc} / \gamma_{tr,R1}$ = **189.1** kN

$F_{tr,d,C1} / R_{tr,d,C1}$ = **0.179**

PASS - Design lateral resistance exceeds lateral load

Load combination 2: A2 + M2 + R4

Partial factors:

Angle of shearing resistance (Table A.4) $\gamma_{\phi',M2}$ = **1.25**

Effective cohesion (Table A.4) $\gamma_{c',M2}$ = **1.25**

Undrained shear strength (Table A.4) $\gamma_{cu,M2}$ = **1.40**

Weight density (Table A.4) $\gamma_{\gamma,M2}$ = **1.00**

Lateral resistance factor $\gamma_{tr,R4}$ = **1.00**

Stratum	Design cohesion, $c_{i,d}$ (kN/m^2)	Design friction angle, $\phi_{i,d}$ (degrees)	Design unit weight of soil, $\gamma_{i,d}$ (kN/m^3)	Design overburden pressure, $p_{ozSi,d}$ (kN/m^2)
1	0	12.1	16	6.4
2	0	29.3	20.5	49.45
3	0	27.5	21.5	129
4	80	20.5	20	189

Resisting soil is divided into 10 segments to determine lateral resistance

From iteration, assume depth of point of rotation X = **3211** mm

Distance from lateral action to ground e = e_{actual} = **0** mm

Sum of moments about point of load application near zero

$\Sigma M_{tr} = M_{trS1} + M_{trS2} + M_{trS3} + M_{trS4} + M_{trS5} + M_{trS6} + M_{trS7} + M_{trS8} + M_{trS9t} + M_{trS9b} + M_{trS10}$ = **1** kNm

Sum of moments about point of rotation

$\Sigma M_X = M_{XS1} + M_{XS2} + M_{XS3} + M_{XS4} + M_{XS5} + M_{XS6} + M_{XS7} + M_{XS8} + M_{XS9t} + M_{XS9b} + M_{XS10}$ = **368** kNm

Calculated soil lateral resist. (Tomlinson Eqn 7.52) $R_{tr,calc} = \Sigma M_X / (e + X)$ = **114.6** kN

Design lateral action $F_{tr,d,C2} = \gamma_{G,unfav,A2} \times G_{tr,k,unfav} - \gamma_{G,fav,A2} \times G_{tr,k,fav} + \gamma_{Q,A2} \times Q_{tr,k}$

$F_{tr,d,C2}$ = **29.4** kN

Design lateral resistance $R_{tr,d,C2} = R_{tr,calc} / \gamma_{tr,R4}$ = **114.6** kN

$F_{tr,d,C2} / R_{tr,d,C2}$ = **0.256**

PASS - Design lateral resistance exceeds lateral load

Lateral deflection analysis (Characteristic values)

Resisting soil is divided into 10 segments to determine lateral resistance

From iteration, assume depth of point of rotation X = **3220** mm

Distance from lateral action to ground e = e_{actual} = **0** mm

Sum of moments about point of load application near zero

$\Sigma M_{tr} = M_{trS1} + M_{trS2} + M_{trS3} + M_{trS4} + M_{trS5} + M_{trS6} + M_{trS7} + M_{trS8} + M_{trS9t} + M_{trS9b} + M_{trS10}$ = **0** kNm

Sum of moments about point of rotation

$\Sigma M_X = M_{XS1} + M_{XS2} + M_{XS3} + M_{XS4} + M_{XS5} + M_{XS6} + M_{XS7} + M_{XS8} + M_{XS9t} + M_{XS9b} + M_{XS10}$ = **609.1** kNm

Calculated soil lateral resist. (Tomlinson Eqn 7.52) $R_{tr,calc} = \Sigma M_X / (e + X)$ = **189.1** kN

Lateral deflection

Characteristic lateral action $F_{tr,k} = G_{tr,k,unfav} - G_{tr,k,fav} + Q_{tr,k}$ = **22.6** kN

Virtual point of fixity, from iteration $V_{zf} = R_{tr,calc} - P_{LatS1} - P_{LatS2} - P_{LatS3} - P_{LatS4} - P_{LatS5} - R \times P_{LatS6}$ = **0** kN

$z_f = (5 + R) \times L / 10$ = **2104** mm

Actual lateral deflection at top of pile $\delta_{Lat} = (F_{tr,k} \times (e + z_f)^3) / (3 \times E \times I)$ = **2.71** mm

Allowable lateral deflection $\delta_{LatAllow}$ = **25** mm

$\delta_{Lat} / \delta_{LatAllow}$ = **0.108**

PASS - Allowable lateral deflection exceeds actual lateral deflection

Pile Cap Design

PILE ANALYSIS

In accordance with EN 1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

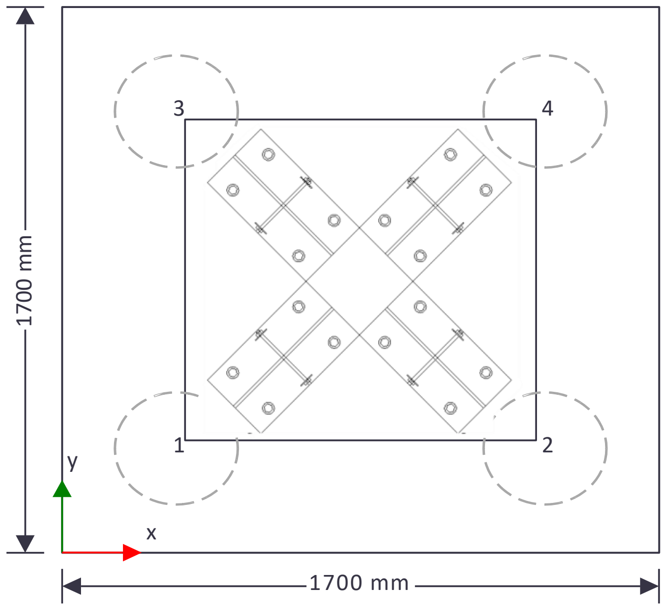
PILE CAP ANALYSIS & DESIGN

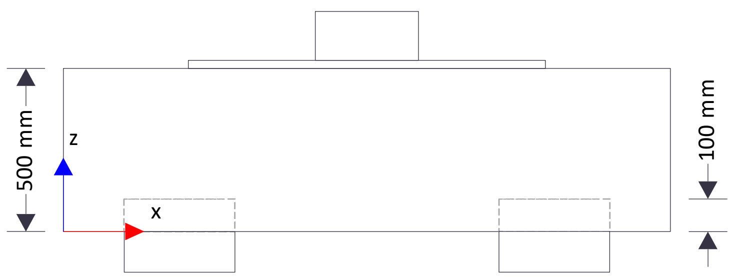
In accordance with EN1997-1-1:2004 & EN1992-1-1:2004 incorporating Corrigenda February 2009 and Corrigenda January 2008, and the UK National Annex

Tedds calculation version 2.0.11

Design summary - FAIL (1.283)

Pile analysis summary





Pile cap details

Pile cap width, along x axis	L_x = **1700** mm
Pile cap width, along y axis	L_y = **1700** mm
Total cap depth	D_{cap} = **500** mm
Density of concrete	γ_{conc} = **25** kN/m^3
Height of soil above pile cap	h_{soil} = **400** mm
Density of soil above pile cap	γ_{soil} = **15** kN/m^3
Pile cap area	$A_{cap} = L_x \times L_y$ = **28900.0** cm^2

Column details

Column width, along x axis	$l_{x,col}$ = **289** mm
Column width, along y axis	$l_{y,col}$ = **509** mm
Column location, x axis	x_c = **850** mm
Column location, y axis	y_c = **850** mm

Pile details

Pile material	Concrete
Concrete pile section	Circular
Pile diameter	h_{pile} = **350** mm
Combination C1 (A1 + M1 +R1)	
Allowable axial compression load	$P_{pC,allow1}$ = **483** kN
Allowable axial tension load	$P_{pT,allow1}$ = **0** kN
Allowable lateral load	$V_{p,allow1}$ = **115** kN
Combination C2 (A2 + M1 +R4)	
Allowable axial compression load	$P_{pC,allow2}$ = **483** kN
Allowable axial tension load	$P_{pT,allow2}$ = **0** kN
Allowable lateral load	$V_{p,allow2}$ = **115** kN
Number of piles	N_p = **4.000**
Pile embedment	d_{embed} = **100** mm
Pile spacing	s_p = **1050** mm
Edge distance	e = **150** mm
Edge distance from pile centre	E = **325** mm

Surcharge loads

Surcharge permanent load	$p_{G,sur}$ = **5.0** kN/m^2
Surcharge imposed loads	$p_{Q,sur}$ = **10.0** kN/m^2

Column axial loads

Axial permanent load	P_G = **634** kN
Total area axial permanent load	$P_{G,area} = \gamma_{conc} \times A_{cap} \times D_{cap} + p_{G,sur} \times (A_{cap} - l_{x,col} \times l_{y,col}) + h_{soil} \times \gamma_{soil} \times (A_{cap} - l_{x,col} \times l_{y,col})$ = **66.3** kN
Axial imposed load	P_Q = **249.7** kN
Total area axial imposed load	$P_{Q,area} = p_{Q,sur} \times (A_{cap} - l_{x,col} \times l_{y,col})$ = **27.4** kN

Column shear loads

Shear imposed load, along x axis	V_{xQ} = **22.6** kN
Shear imposed load, along y axis	V_{yQ} = **22.6** kN

Pile group centroid

Centroid location, x direction	$x_{pg,c} = (x_{p1} + x_{p2} + x_{p3} + x_{p4}) / N_p$ = **850** mm
Centroid location, y direction	$y_{pg,c} = (y_{p1} + y_{p2} + y_{p3} + y_{p4}) / N_p$ = **850** mm

Pile distance from centroid

Pile 1 centroid distance, x direction	$x_{p1,c} = x_{p1} - x_{pg,c}$ = **-525** mm
Pile 1 centroid distance, y direction	$y_{p1,c} = y_{p1} - y_{pg,c}$ = **-525** mm
Pile 2 centroid distance, x direction	$x_{p2,c} = x_{p2} - x_{pg,c}$ = **525** mm
Pile 2 centroid distance, y direction	$y_{p2,c} = y_{p2} - y_{pg,c}$ = **-525** mm
Pile 3 centroid distance, x direction	$x_{p3,c} = x_{p3} - x_{pg,c}$ = **-525** mm
Pile 3 centroid distance, y direction	$y_{p3,c} = y_{p3} - y_{pg,c}$ = **525** mm
Pile 4 centroid distance, x direction	$x_{p4,c} = x_{p4} - x_{pg,c}$ = **525** mm
Pile 4 centroid distance, y direction	$y_{p4,c} = y_{p4} - y_{pg,c}$ = **525** mm

Moment of inertia of pile group

Moment of inertia about x-x axis	$I_{xx} = y_{p1,c}^2 + y_{p2,c}^2 + y_{p3,c}^2 + y_{p4,c}^2$ = **1102500** mm^2
Moment of inertia about y-y axis	$I_{yy} = x_{p1,c}^2 + x_{p2,c}^2 + x_{p3,c}^2 + x_{p4,c}^2$ = **1102500** mm^2

Loading eccentricity

Eccentricity of column load, x direction	$e_{x,c} = x_c - x_{pg,c}$ = **0** mm
Eccentricity of column load, y direction	$e_{y,c} = y_c - y_{pg,c}$ = **0** mm

Permanent load pile forces

Permanent axial load on pile 1	$P_{p1,G} = (P_G + P_{G,area}) / N_p$ = **175** kN
Permanent axial load on pile 2	$P_{p2,G} = (P_G + P_{G,area}) / N_p$ = **175** kN
Permanent axial load on pile 3	$P_{p3,G} = (P_G + P_{G,area}) / N_p$ = **175** kN
Permanent axial load on pile 4	$P_{p4,G} = (P_G + P_{G,area}) / N_p$ = **175** kN

Imposed load pile forces

Imposed load moment about x-x axis	$M_{xQ,Des} = (-1) \times V_{yQ} \times D_{cap}$ = **-11.3** kNm
Imposed load moment about y-y axis	$M_{yQ,Des} = V_{xQ} \times D_{cap}$ = **11.3** kNm
Imposed shear load on each pile, x direction	$V_{p,x,Q} = V_{xQ} / N_p$ = **6** kN
Imposed shear load on each pile, y direction	$V_{p,y,Q} = V_{yQ} / N_p$ = **6** kN

Imposed axial load on pile 1 — $P_{p1,Q} = (P_Q + P_{Q,area}) / N_p + M_{yQ,Des} \times x_{p1,c} / I_{yy} + M_{xQ,Des} \times (-1) \times y_{p1,c} / I_{xx}$ = **59** kN

Imposed axial load on pile 2 — $P_{p2,Q} = (P_Q + P_{Q,area}) / N_p + M_{yQ,Des} \times x_{p2,c} / I_{yy} + M_{xQ,Des} \times (-1) \times y_{p2,c} / I_{xx}$ = **69** kN

Imposed axial load on pile 3 — $P_{p3,Q} = (P_Q + P_{Q,area}) / N_p + M_{yQ,Des} \times x_{p3,c} / I_{yy} + M_{xQ,Des} \times (-1) \times y_{p3,c} / I_{xx}$ = **69** kN

Imposed axial load on pile 4 — $P_{p4,Q} = (P_Q + P_{Q,area}) / N_p + M_{yQ,Des} \times x_{p4,c} / I_{yy} + M_{xQ,Des} \times (-1) \times y_{p4,c} / I_{xx}$ = **80** kN

Combination details

Imposed combination factor — ψ_0 = **0.70**

Wind combination factor — ψ_W = **0.50**

Snow combination factor — ψ_S = **0.50**

DA1 6.10 load combinations (ULS)

1.35G (0.489)

1.35G + 1.5Q (0.738)

1.00G (0.362)

1.00G + 1.3Q (0.578)

Combination 2 results: 1.35G + 1.5Q

Pile shear load, x direction — $V_{p,x} = 1.35 \times V_{p,x,G} + 1.5 \times V_{p,x,Q}$ = **8.5** kN

Pile shear load, y direction — $V_{p,y} = 1.35 \times V_{p,y,G} + 1.5 \times V_{p,y,Q}$ = **8.5** kN

Pile shear load, resultant — $V_{p,R} = \sqrt{(V_{p,x}^2 + V_{p,y}^2)}$ = **12** kN

$\max(V_{p,R} / V_{p,allow1}, 0)$ = **0.104**

PASS - Pile allowable shear load exceeds pile shear force

Pile 4 axial load — $P_{p4} = 1.35 \times P_{p4,G} + 1.5 \times P_{p4,Q}$ = **356.4** kN

$\max(P_{p4} / P_{pC,allow1}, 0)$ = **0.738**

PASS - Pile allowable compression load exceeds axial force

Pile cap design

Reinforcement details

Characteristic yield strength of reinforcement — f_{yk} = **550** N/mm²

Partial factor for reinforcing steel - Table 2.1N — γ_S = **1.15**

Design yield strength of reinforcement — f_{yd} = **478** N/mm²

Nominal cover to reinforcement

Nominal cover to top reinforcement — $c_{nom,top}$ = **75** mm

Nominal cover to bottom reinforcement — $c_{nom,bot}$ = **75** mm

Concrete details – Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class — C45/55

Aggregate type — Limestone

Aggregate adjustment factor - cl.3.1.3(2) — AAF = **0.9**

Characteristic compressive cylinder strength — f_{ck} = **45** N/mm²

Max. char. compressive cylinder strength in shear — $f_{ck,lim}$ = **50** N/mm²

Mean value of compressive cylinder strength — $f_{cm} = f_{ck} + 8$ N/mm² = **53** N/mm²

Mean value of axial tensile strength — $f_{ctm} = 0.3$ N/mm² $\times (f_{ck} / 1$ N/mm²$)^{2/3}$ = **3.8** N/mm²

Secant modulus of elasticity of concrete — $E_{cm} = 22$ kN/mm² $\times [f_{cm}/10$ N/mm²$]^{0.3} \times$ AAF = **32655** N/mm²

Ultimate strain - Table 3.1 — ε_{cu2} = **0.0035**

Shortening strain - Table 3.1 — ε_{cu3} = **0.0035**

Effective compression zone height factor — λ = **0.80**

Effective strength factor — η = **1.00**

Coefficient k_1 — k_1 = **0.40**

Coefficient k_2 — $k_2 = 1.00 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

Coefficient k_3 — k_3 = **0.40**

Coefficient k_4 — $k_4 = 1.00 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

Partial factor for concrete -Table 2.1N — γ_C = **1.50**

Compressive strength coefficient - cl.3.1.6(1) — α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15 — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **25.5** N/mm²

Compressive strength coefficient - cl.3.1.6(1) — α_{ccw} = **1.00**

Design compressive concrete strength - exp.3.15 — $f_{cwd} = \alpha_{ccw} \times \min(f_{ck}, f_{ck,lim}) / \gamma_C$ = **30.0** N/mm²

Maximum aggregate size — h_{agg} = **20** mm

Strength reduction factor, exp. 6.6N — $\nu = 0.6 \times (1 - (f_{ck} / 250$ N/mm²$))$ = **0.49**

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.12**

STR 6.10 load combinations (ULS_D)

1.35G (0.892)

1.35G + 1.5Q (1.283)

1.00G + 1.5W (0.661)

1.35G + 1.5Q + ψ_S×1.5S + ψ_W×1.5W (1.283)

Combination 10 results: 1.35G + 1.5Q + ψ_S×1.5S + ψ_W×1.5W

Moment design, about y-y axis, positive moment

Critical bending plane at face of column

Ultimate bendind moment — M_{Ed} = **297.6** kNm

Depth to tension reinforcement — $d = D_{cap} - d_{embed} - c_{nom,bot} - \phi_{x.bot} / 2$ = **315** mm

Redistribution ratio — δ = **1.000**

$K = M_{Ed} / (L_y \times d^2 \times f_{ck})$ = **0.039**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 \times d \times (1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}), 0.95 \times d)$ = **299** mm

Area of tension reinforcement required — $A_{s,req} = M_{Ed} / (f_{yd} \times z)$ = **2079** mm²

Tension reinforcement provided — (18) 20 mm ϕ bot bars (180 mm c/c)

Area of tension reinforcement provided — $A_{s,prov}$ = **5655** mm²

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d$ = **961** mm²

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times L_y \times D_{cap}$ = **34000** mm²

PASS - Area of reinforcement provided exceeds area required

Shear design, along x axis, right side – Section 6.2

Maximum design shear force at face of column — $V_{Ed.max}$ = **640.7** kN

Depth to reinforcement — $d_v = d_{x.bot}$ = **315** mm

Maximum design shear resistance, exp. 6.5 — $V_{Rd.max} = 0.5 \times L_y \times d_v \times \nu \times f_{cwd}$ = **3952.0** kN

$V_{Ed.max} / V_{Rd.max}$ = **0.16**

PASS - Design shear force at support is less than the maximum design shear resistance

Nearest valid pile to the column face — P4

Critical shear plane distance from column face — $a_{v.R} = \max(x_{PESP.p4} + e_{x,tolerance} - x_{R,col\_face}, 0 \text{ mm})$ = **350** mm

Design shear force at critical shear plane — $V_{Ed} = \text{abs}(P_{p2,ULS\_D10} \times a_{v.Rp2} / (2 \times d_v) + P_{p4,ULS\_D10} \times a_{v.Rp4} / (2 \times d_v) - (1.35 \times (\gamma_{conc} \times D_{cap} + \gamma_{soil} \times h_{soil} + p_{G,sur}) + 1.50 \times p_{Q,sur}) \times \max((L_x - x_{R,col\_face} - a_{v.R}) \times L_y, 0 \text{ mm}^2))$ = **359.4** kN

$k = \min(1 + (200 \text{ mm} / d_v)^{0.5}, 2)$ = **1.797**

Longitudinal reinforcement radio — $\rho_l = \min(A_{sx.bot.prov} / (L_y \times d_{x.bot}), 0.02)$ = **0.01056**

$v_{min} = 0.035$ N$^{0.5}$/mm $\times k^{3/2} \times f_{ck}^{1/2}$ = **0.57** N/mm²

Design shear resistance – exp. 6.2 a & b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d_v$

$V_{Rd.c}$ = **418.2** kN

$V_{Ed} / V_{Rd.c}$ = **0.86**

PASS - Design shear resistance exceeds design shear force

Column punching shear design – Section 6.4

Effective depth to reinforcement — $d_{eff} = (d_{x.bot} + d_{y.bot}) / 2$ = **305** mm

Maximum punching shear resistance - cl. 6.4.5(3) — $v_{Rd.max} = 0.5 \times \nu \times f_{cwd}$ = **7.38** N/mm²

$k = \min(1 + (200 \text{ mm} / d_{eff})^{0.5}, 2)$ = **1.81**

Longitudinal reinforcement ratio - cl. 6.4.4(1) — $\rho_{lx} = \min(A_{sx.bot.prov} / (L_y \times d_{x.bot}), 0.02)$ = **0.01056**

$\rho_{ly} = \min(A_{sy.bot.prov} / (L_x \times d_{y.bot}), 0.02)$ = **0.01128**

$\rho_l = \min((\rho_{lx} \times \rho_{ly})^{0.5}, 0.02)$ = **0.01091**

$v_{min} = 0.035$ N$^{0.5}$/mm $\times k^{3/2} \times f_{ck}^{1/2}$ = **0.57** N/mm²

Punching shear at column face

Loaded perimeter length — $l_{x,perim}$ = **289** mm

Loaded perimeter width — $l_{y,perim}$ = **509** mm

Column classification — Interior

Punching shear stress factor - fig. 6.21N — β_0 = **1.15**

Punching shear perimeter — $u_0 = 2 \times l_{x,perim} + 2 \times l_{y,perim}$ = **1596** mm

Area inside punching shear perimeter — $A_0 = l_{x,perim} \times l_{y,perim}$ = **1471** cm²

Maximum punching shear force — $V_{Ed.max} = \text{abs}(\text{Sum}(P_{p1,ULS\_D10}, P_{p2,ULS\_D10}, P_{p3,ULS\_D10}, P_{p4,ULS\_D10}) - 1.35 \times (\gamma_{conc} \times D_{cap} + p_{G,sur} + \gamma_{soil} \times h_{soil}) \times (A_{cap} - A_0) - 1.5 \times p_{Q,sur} \times (A_{cap} - A_0))$ = **1232.9** kN

Maximum punching shear stress – exp. 6.38 $v_{Ed.max} = \max(\beta_0 \times V_{Ed.max} / (u_0 \times d_{eff}), 0\ MPa) = \mathbf{2.91}\ N/mm^2$

$v_{Ed.max} / v_{Rd.max} = \mathbf{0.39}$

PASS - Design punching shear capacity exceeds max. punching shear load

Pile punching shear design, pile 4

Effective depth to reinforcement $d_{eff} = (d_{x.bot} + d_{y.bot}) / 2 = \mathbf{305}\ mm$

Maximum punching shear resistance - cl. 6.4.5(3) $v_{Rd.max} = 0.5 \times v \times f_{cwd} = \mathbf{7.38}\ N/mm^2$

$k = \min(1 + (200\ mm / d_{eff})^{0.5}, 2) = \mathbf{1.81}$

Longitudinal reinforcement ratio - cl. 6.4.4(1) $\rho_{lx} = \min(A_{sx.bot.prov} / (L_y \times d_{x.bot}), 0.02) = \mathbf{0.01056}$

$\rho_{ly} = \min(A_{sy.bot.prov} / (L_x \times d_{y.bot}), 0.02) = \mathbf{0.01128}$

$\rho_l = \min((\rho_{lx} \times \rho_{ly})^{0.5}, 0.02) = \mathbf{0.01091}$

$v_{min} = 0.035\ N^{0.5}/mm \times k^{3/2} \times f_{ck}^{1/2} = \mathbf{0.57}\ N/mm^2$

Punching shear at pile face

Column classification Corner

Loaded perimeter length $l_{x,perim} = \mathbf{425}\ mm$

Loaded perimeter width $l_{y,perim} = \mathbf{425}\ mm$

Punching shear perimeter $u_0 = l_{x,perim} + l_{y,perim} - d_p + \pi \times d_p / 4 = \mathbf{775}\ mm$

Maximum punching shear force $V_{Ed.max} = abs(P_{p4,ULS\_D10}) = \mathbf{356.4}\ kN$

Maximum punching shear stress – exp. 6.38 $v_{Ed.max} = \max(V_{Ed.max} / (u_0 \times d_{eff}), 0\ N/mm^2) = \mathbf{1.51}\ N/mm^2$

$v_{Ed.max} / v_{Rd.max} = \mathbf{0.20}$

PASS - Design punching shear capacity exceeds max. punching shear load

Pile punching shear design, pile 4 - Control perimeter

Control perimeter distance $a = 2 \times d_{eff} = \mathbf{610}\ mm$

Pile classification Corner

Rectangular perimeter length $l_{x,perim} = \mathbf{1035}\ mm$

Rectangular perimeter width $l_{y,perim} = \mathbf{1035}\ mm$

Punching shear perimeter $u_{2d} = l_{x,perim} + l_{y,perim} - 2 \times (d_p / 2 + a) + \pi \times (d_p / 2 + a) / 2 = \mathbf{1733}\ mm$

Design punching shear force $V_{Ed.2d} = abs(P_{p4,ULS\_D10}) = \mathbf{356.4}\ kN$

Design punching shear stress – exp. 6.38 $v_{Ed.2d} = \max(V_{Ed.2d} / (u_{2d} \times d_{eff}), 0\ N/mm^2) = \mathbf{0.67}\ N/mm^2$

Design punching shear resistance - exp. 6.50

$v_{Rd.c.2d} = \min(\max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}), v_{Rd.max})$

$v_{Rd.c.2d} = \mathbf{0.80}\ N/mm^2$

$v_{Ed.2d} / v_{Rd.c.2d} = \mathbf{0.85}$

PASS - Design punching shear capacity exceeds punching shear load

Corner pile shear design, pile 4

Design shear force $V_{Ed} = abs(P_{p4,ULS\_D10}) = \mathbf{356.4}\ kN$

Depth to reinforcement $d_{eff} = (d_{x.bot} + d_{y.bot}) / 2 = \mathbf{305}\ mm$

Width of design section $b = (\min(y_{p4}, L_y - y_{p4}) + \min(x_{p4}, L_x - x_{p4}) + (d_p / 2 + d_{eff} - e_{tolerance}) / \sin(45\ deg)) / \sin(45\ deg) = \mathbf{1729}\ mm$

$C_{Rd.c} = 0.18 / \gamma_C = \mathbf{0.12}$

$k = \min(1 + (200\ mm / d_{eff})^{0.5}, 2) = \mathbf{1.81}$

Longitudinal reinforcement ratio $\rho_{lx} = \min(A_{sx.bot.prov} / (L_y \times d_{x.bot}), 0.02) = \mathbf{0.01056}$

$\rho_{ly} = \min(A_{sy.bot.prov} / (L_x \times d_{y.bot}), 0.02) = \mathbf{0.01128}$

$\rho_l = \min((\rho_{lx} \times \rho_{ly})^{0.5}, 0.02) = \mathbf{0.01091}$

$v_{min} = 0.035\ N^{0.5}/mm \times k^{3/2} \times f_{ck}^{1/2} = \mathbf{0.57}\ N/mm^2$

Design shear resistance – exp. 6.2 a & b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times b \times d_{eff}$

Steel Bolt Design

Basic single shear loads for one grade 4.6 bolt in a two member timber connection

Timber grade	Minimum member thickness (mm)	Basic single shear load for selected grade 4.6 bolt diameters in a two member* timber connection (kN)							
		Direction of loading							
		Parallel to the grain				Perpendicular to the grain			
		M8	M12	M16	M20	M8	M12	M16	M20
C16	47	1.22	1.80	2.30	2.73	1.13	1.56	1.91	2.19
	72	1.46	2.68	3.52	4.19	1.39	2.39	2.93	3.36
	97	1.46	3.13	4.63	5.64	1.39	2.79	3.94	4.52
C24	47	1.33	2.04	2.59	3.09	1.23	1.76	2.16	2.47
	72	1.55	2.93	3.97	4.73	1.47	2.64	3.30	3.79
	97	1.55	3.42	5.05	6.37	1.47	3.07	4.43	5.11
D40	47	1.83	3.08	3.92	4.67	1.83	3.08	3.92	4.67
	72	1.91	4.02	5.98	7.16	1.91	4.02	5.98	7.16
	97	1.91	4.21	6.93	9.32	1.91	4.21	6.93	9.32
D50	47	2.12	3.78	4.81	5.73	2.12	3.78	4.81	5.73
	72	2.12	4.66	6.92	8.78	2.12	4.66	6.92	8.78
	97	2.12	4.66	8.09	10.82	2.12	4.66	8.09	10.82

According to BS 5268: Part 2: 2002: Appendix G:

Allowable shear force of 1 M20 bolt = 9.32 kN for D40 timber (assumed to be similar strength to GL32C)

The shear force on the connection = 46.2 kN (Secondary Beam, Maximum Shear)

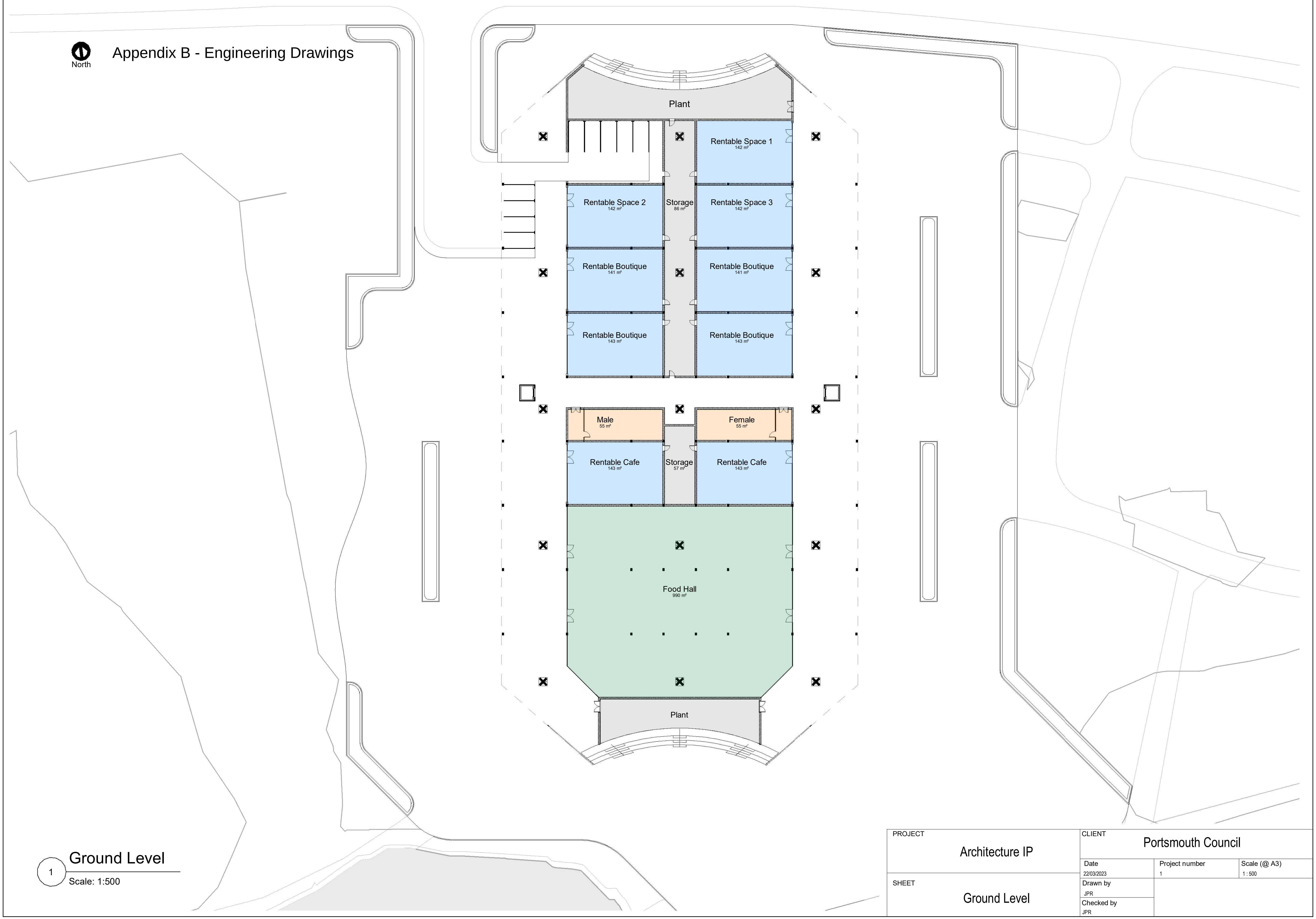
Therefore, 6 bolts = Allowable Shear,55.92 kN > Design shear, 46.2 kN

PASS – Design shear loading on connection

Spacing:

4d horizontal = 148 mm

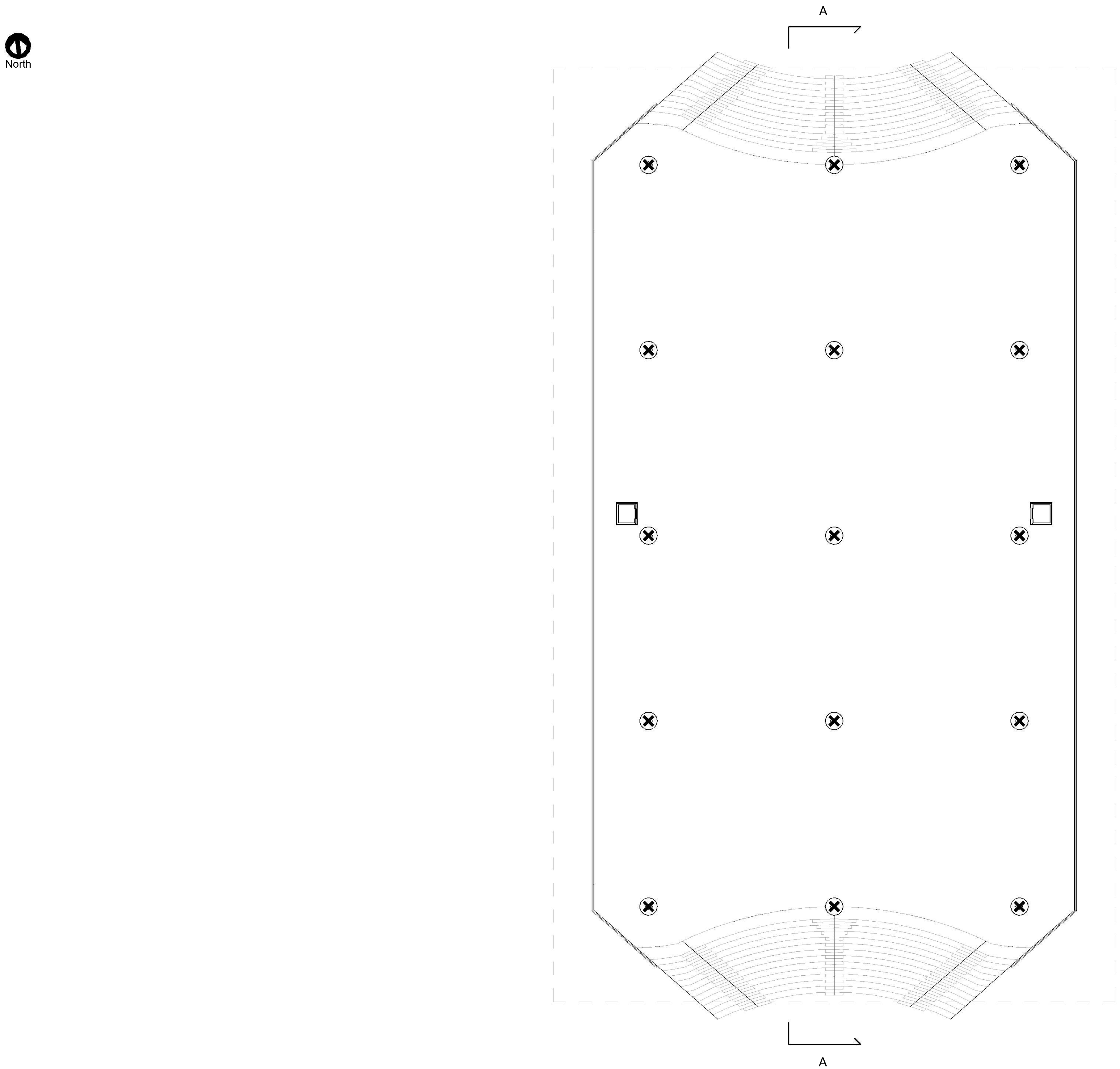
Appendix B - Engineering Drawings



1 Ground Level

Scale: 1:500

PROJECT Architecture IP	CLIENT Portsmouth Council		
	Date 22/03/2023	Project number 1	Scale (@ A3) 1 : 500
SHEET Ground Level	Drawn by JPR		
	Checked by JPR		

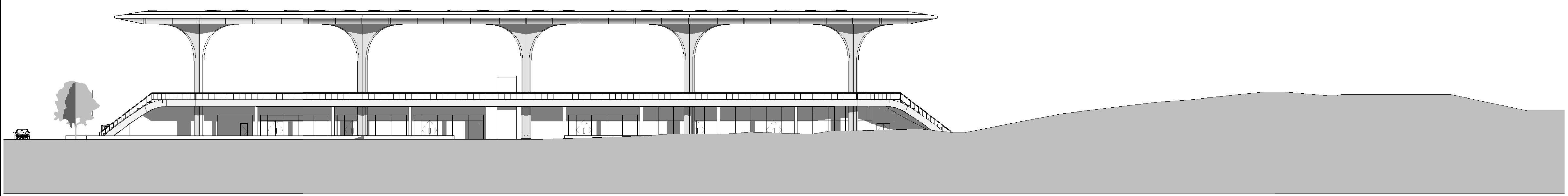


Level 1

2

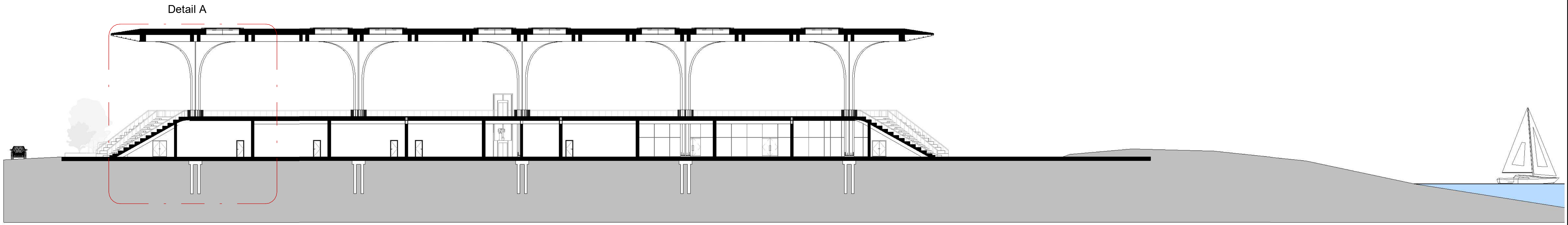
Scale: 1:500

PROJECT: Architecture IP	CLIENT: Portsmouth Council		
	Date: 22/03/2023	Project number: 1	Scale (@ A3): 1 : 500
SHEET: Level 1	Drawn by: JPR		
	Checked by: JPR		



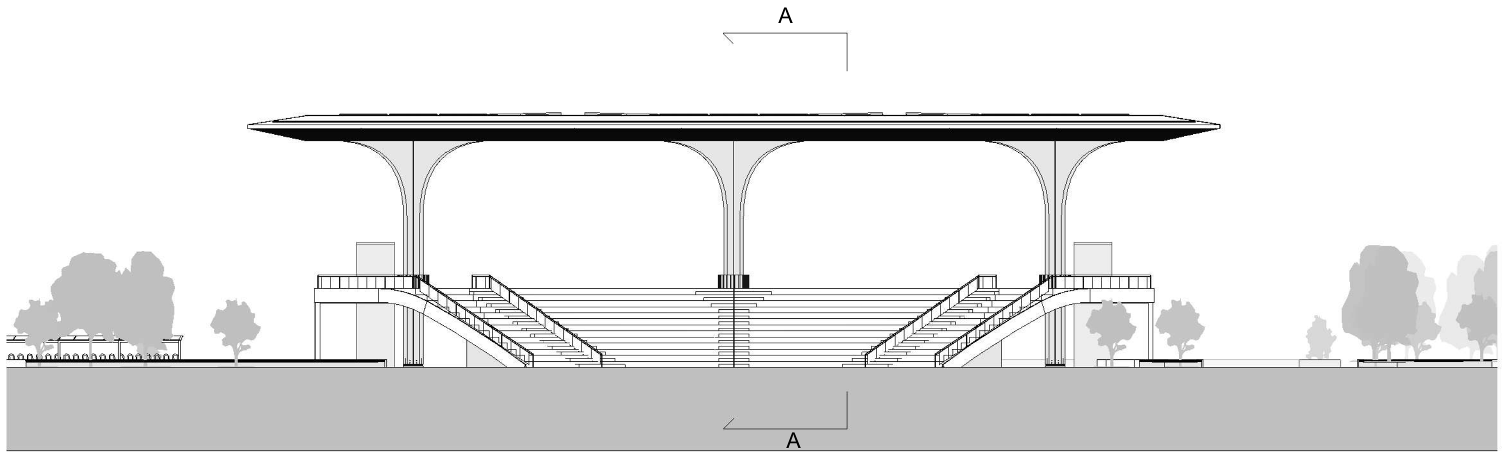
3 **West Elevation**

Scale: 1:500



4 **Section A**

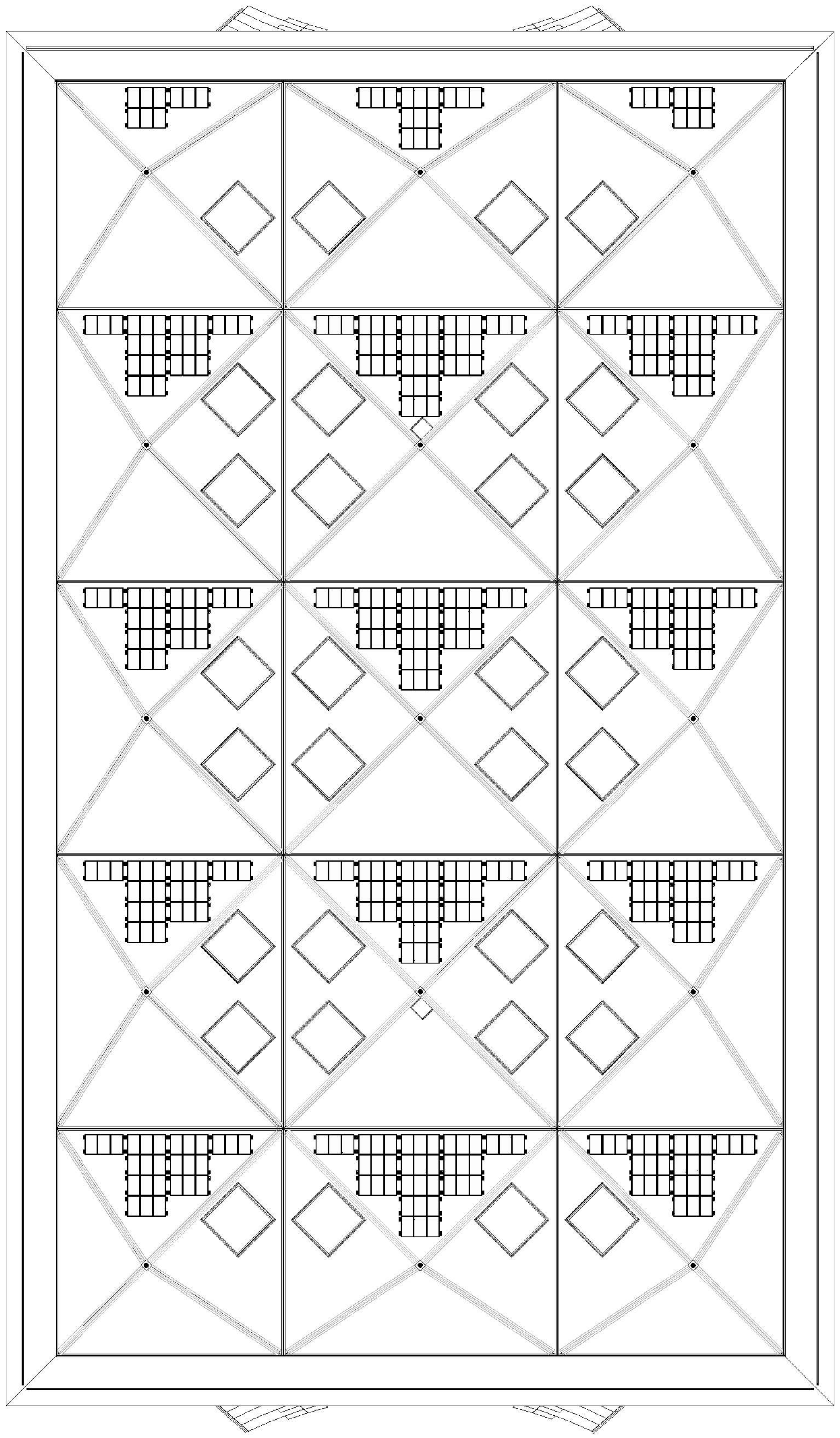
Scale: 1:500



5 **North Elevation**

Scale: 1:500

PROJECT Architecture IP	CLIENT Portsmouth Council		
	Date 22/03/2023	Project number 1	Scale (@ A3) 1 : 500
SHEET Elevations	Drawn by JPR		
	Checked by JPR		

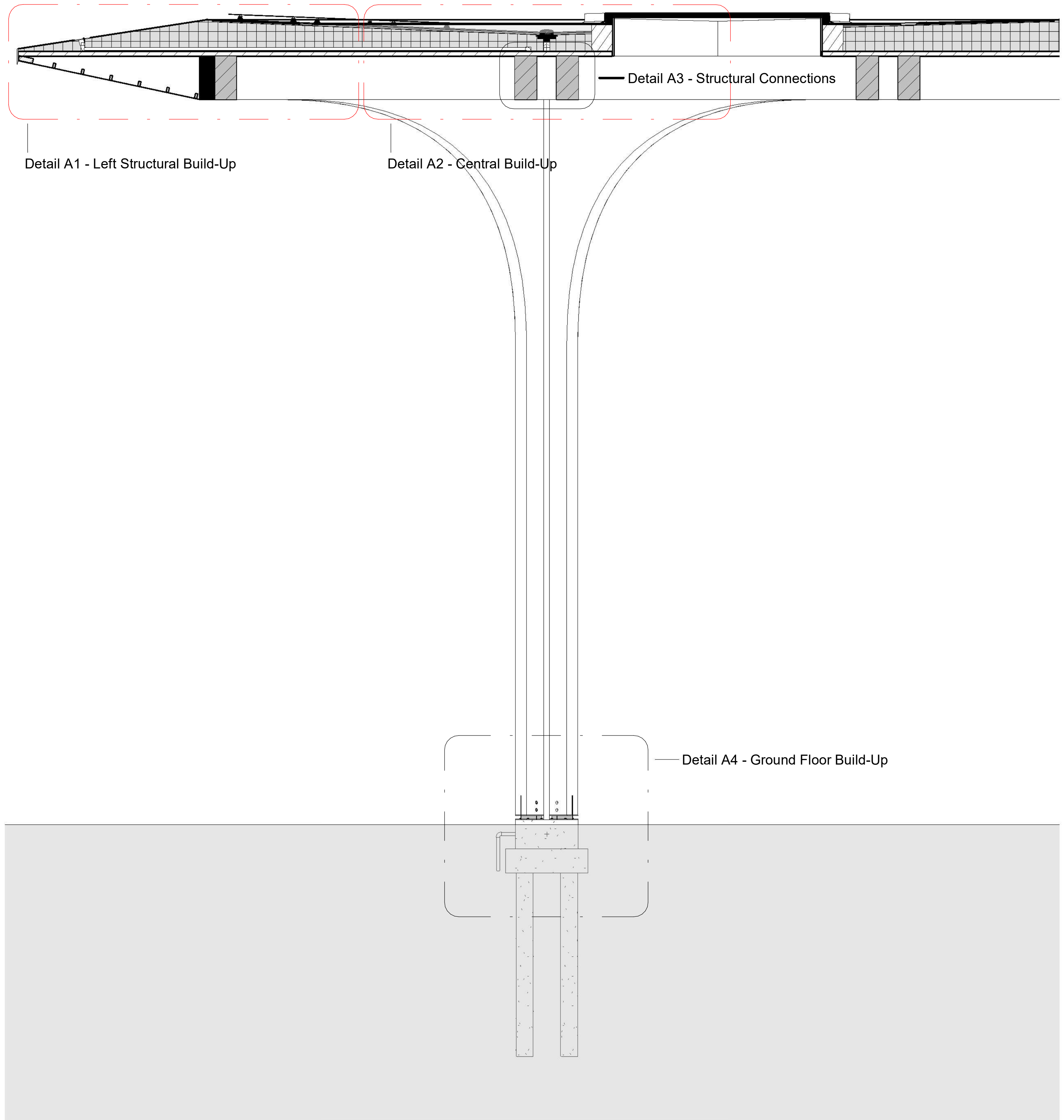


6

Roof Plan

Scale: 1:500

PROJECT	Architecture IP
SHEET	Roof Plan
CLIENT	Portsmouth Council
Date	22/03/2023
Project number	1
Scale (@ A3)	1 : 500
Drawn by	JPR
Checked by	JPR

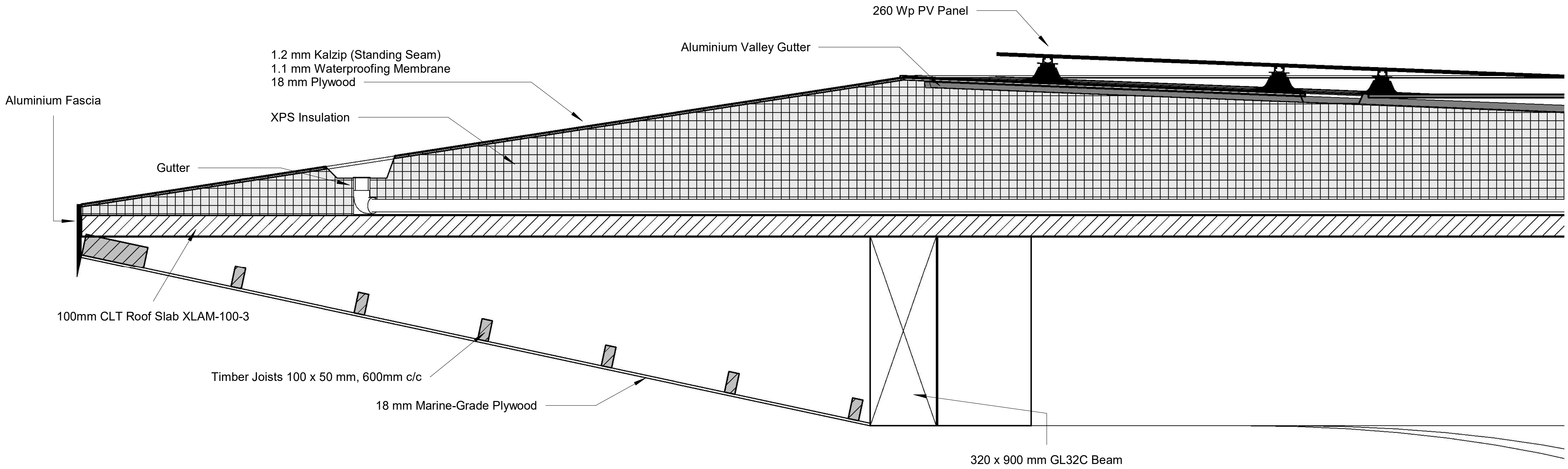


8

Detail A

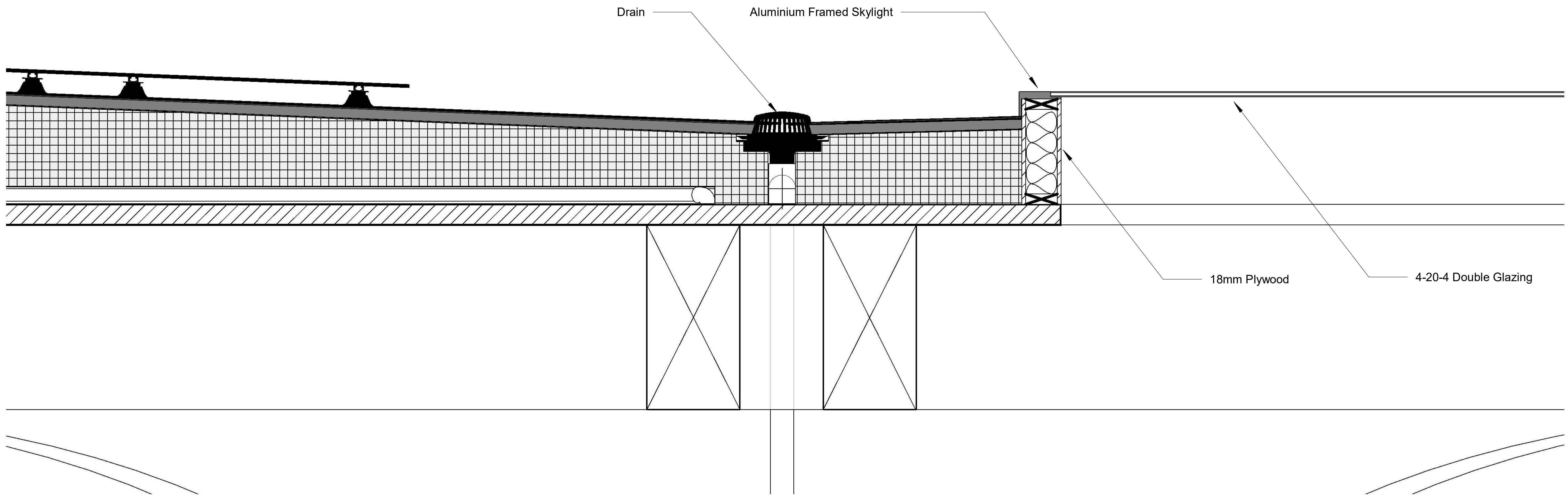
Scale: 1:100

PROJECT	CLIENT		
Architecture IP	Portsmouth Council		
	Date 22/03/2023	Project number 1	Scale (@ A3) 1 : 100
SHEET	Drawn by JPR		
Detail A	Checked by JPR		



9 **Detail A1 - Left Structural Build-Up**

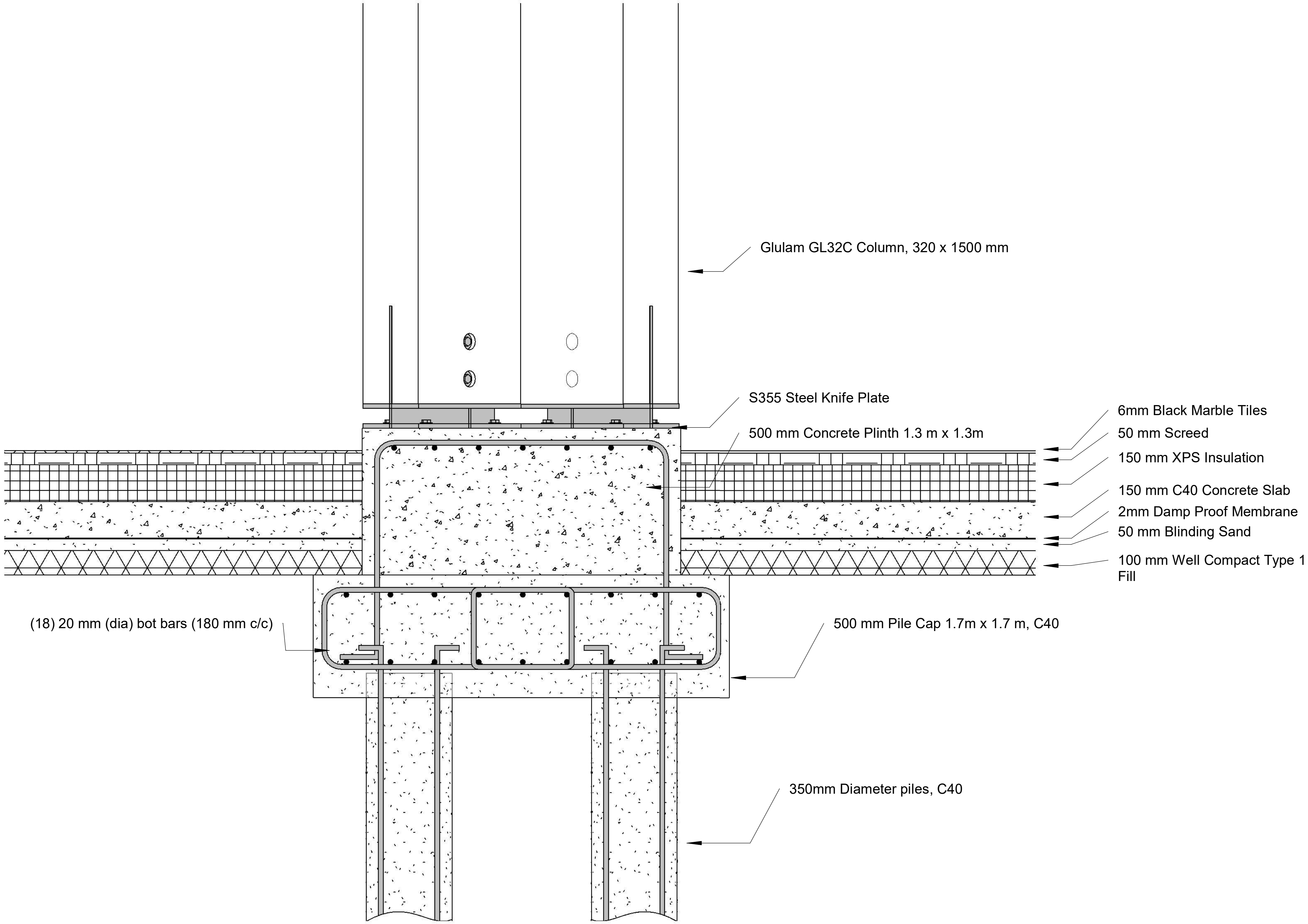
Scale: 1:20



10 **Detail A2 - Central Build-Up**

Scale: 1:20

PROJECT: Architecture IP	CLIENT: Portsmouth Council		
	Date: 22/03/2023	Project number: 1	Scale (@ A3): 1 : 20
SHEET: Architectural Detail	Drawn by: JPR		
	Checked by: JPR		

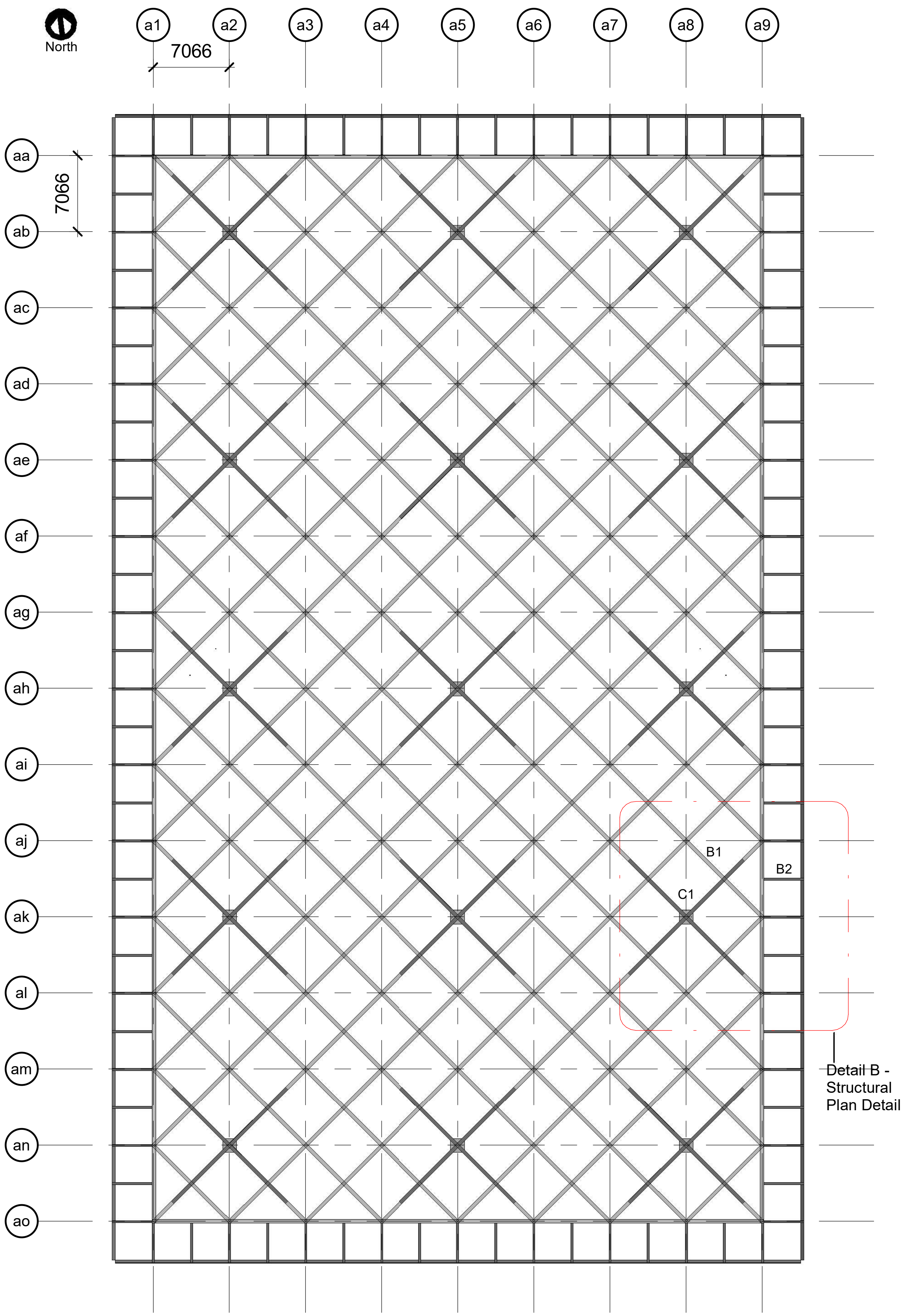


11

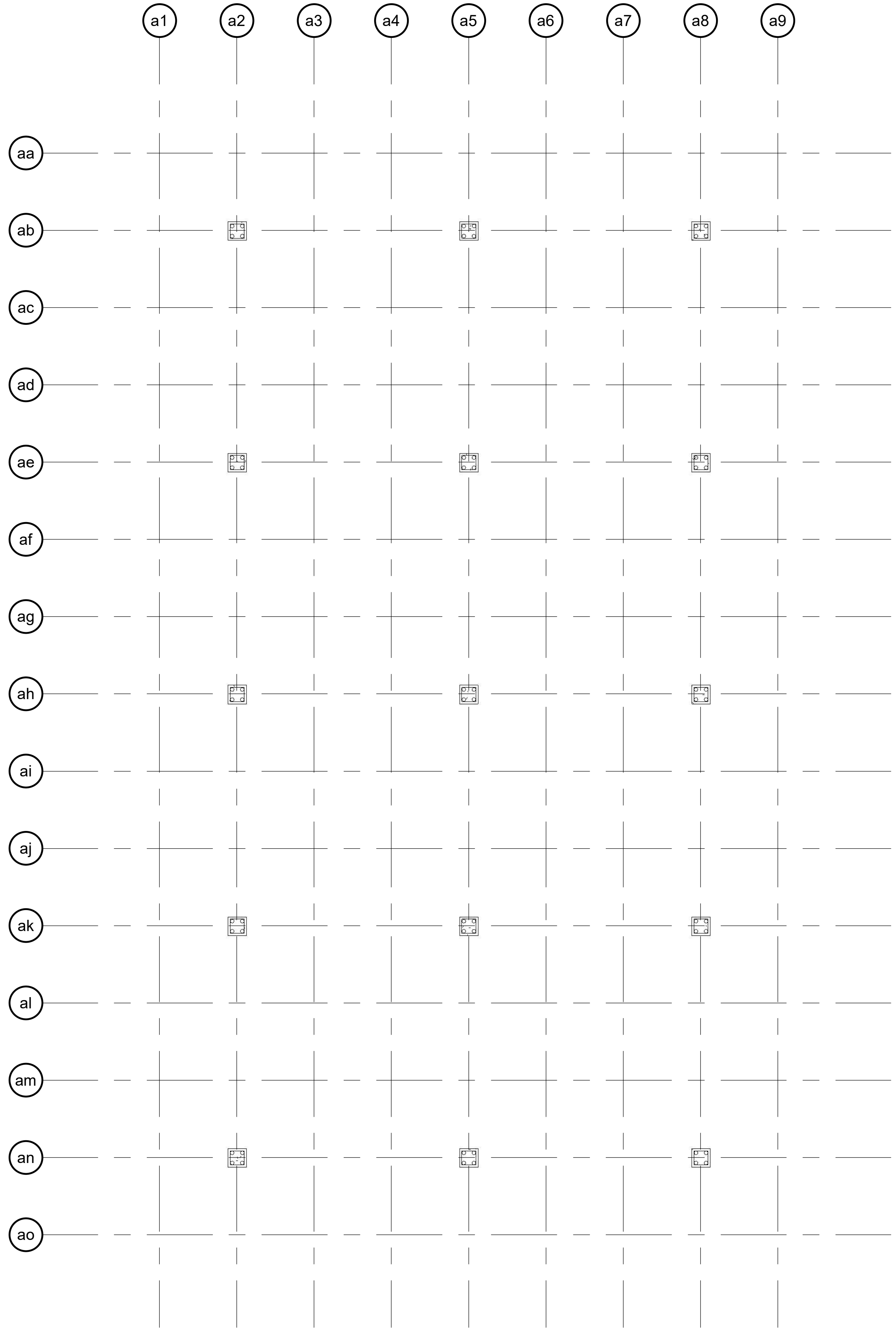
Detail A4 - Ground Floor Build-Up

Scale: 1:20

PROJECT Architecture IP	CLIENT Portsmouth Council		
	Date 22/03/2023	Project number 1	Scale (@ A3) 1 : 20
SHEET Structural Detail	Drawn by JPR		
	Checked by JPR		



12 Structural Plan
Scale: 1:500



13 Foundation Plan
Scale: 1:500

Measurements

All measurements are in mm unless stated otherwise.

Beams
B1 - GL32C, 900 x 320
B2 - GL32C, 900 x 200 cut to fall

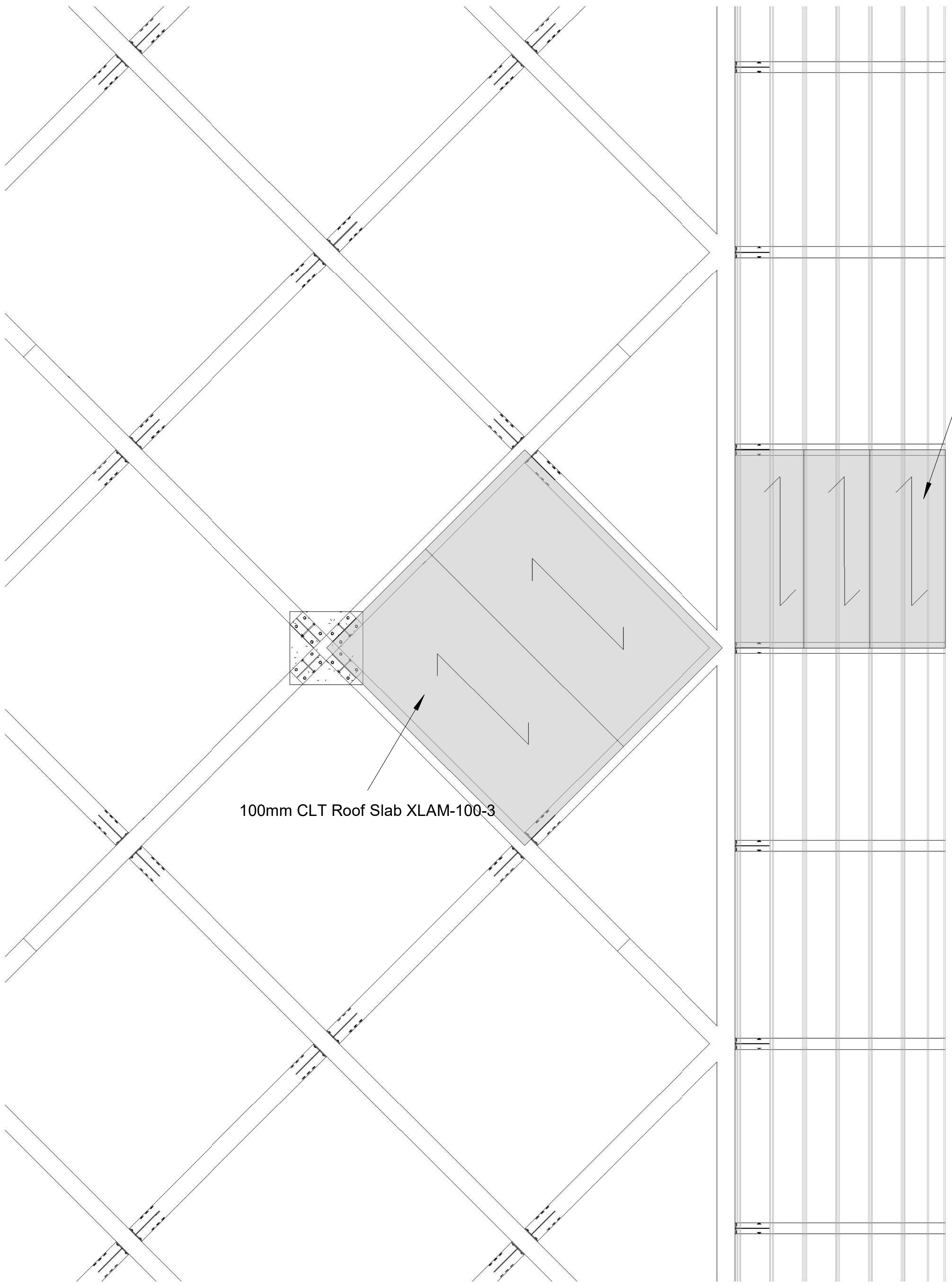
Columns
C1 - GL32C, 320 x 1500 with tapered end

Plinth
Typical Plinth - C40, 1300 x 1300 x 500
(18) 20 mm f bot bars (180 mm c/c)

Pile Cap
Typical Pile Cap - C40, 1700 x 1700 x 500
(18) 20 mm f bot bars (180 mm c/c)

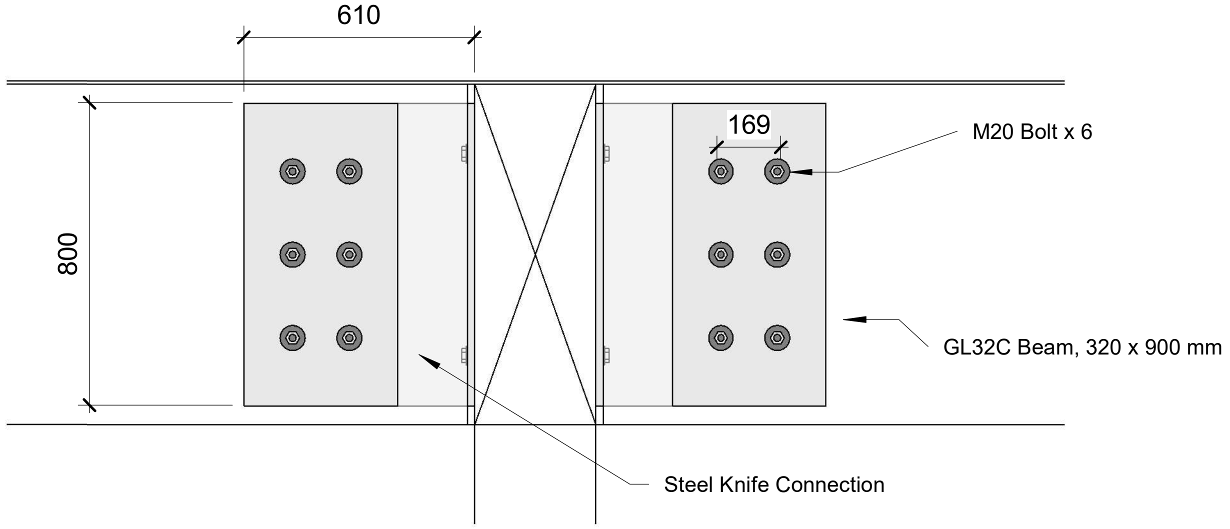
Piles
Typical Pile - C40, 350 x 3800
(2) 20 mm f bot bars

PROJECT	CLIENT		
Architecture IP	Portsmouth Council		
	Date: 22/03/2023	Project number: 1	Scale (@ A3): 1 : 500
SHEET: Structural Plans	Drawn by: JPR		
	Checked by: JPR		



14 Detail B - Structural Plan Detail

Scale: 1:100



15 Detail A3 - Structural Connections

Scale: 1:20

PROJECT	CLIENT		
Architecture IP	Portsmouth Council		
	Date 22/03/2023	Project number 1	Scale (@ A3) As indicated
SHEET Structural Detail 2	Drawn by JPR		
	Checked by JPR		

PORTSMOUTH OPEN MARKET PROPOSAL

Architectural Engineering Project

Jake Robson – 18/05/2023

INTRODUCTION

The Pyramids Centre Portsmouth

The Pyramids Centre Portsmouth

Problems:

- Unattractive
- Near end of design-life
- Introspective design
- Doesn't attract enough visitors

INTRODUCTION

The Pyramids Centre Portsmouth

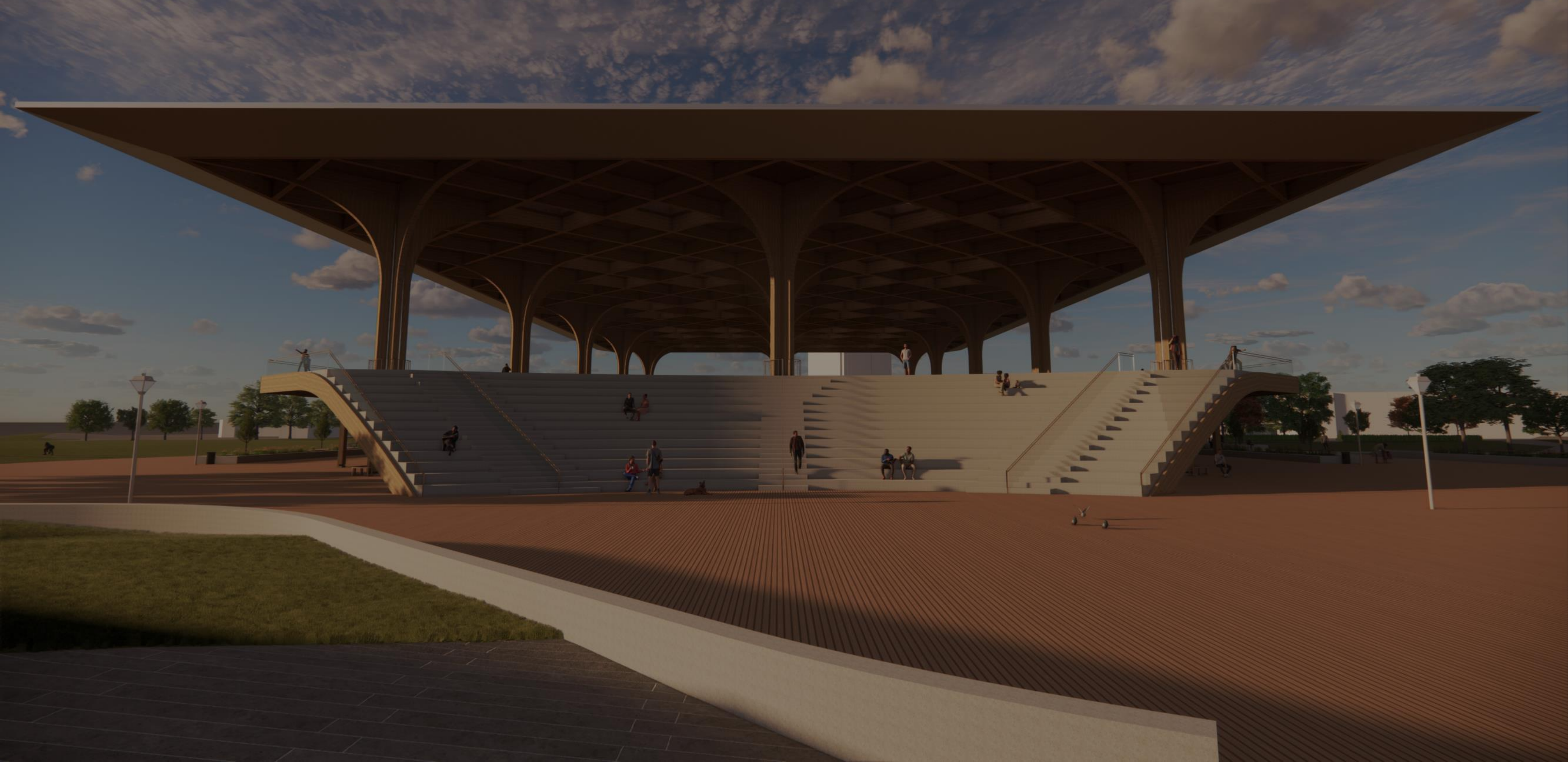
Google Image of Castle Feld

Problems:

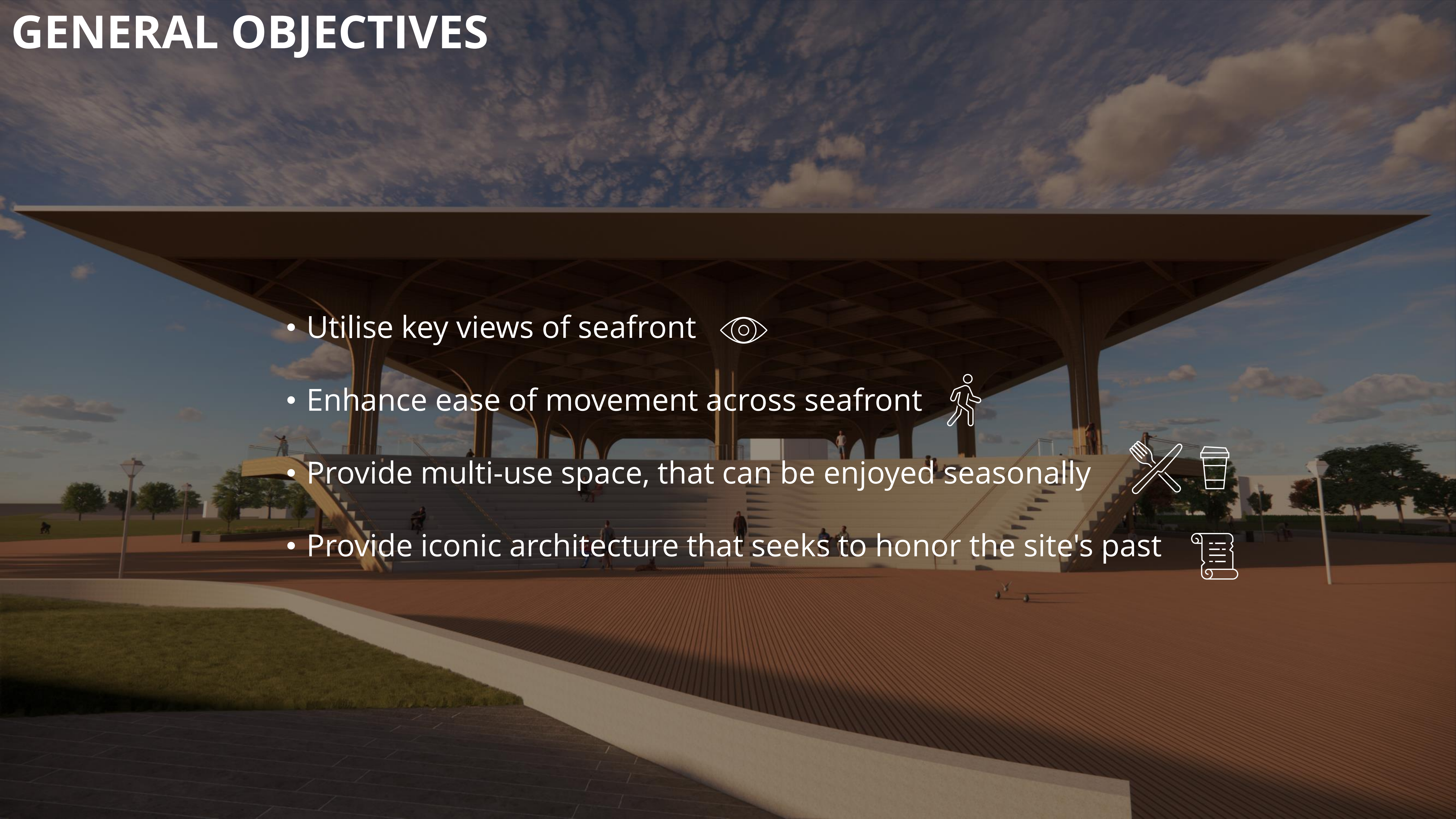
- Key features block horizontal movement
- Lack of opportunities for socializing
- Poor linkage and legibility (In need of a coastal central hub)

PROJECT BRIEF

Replace the outdated ‘Pyramid’s Centre’ with an iconic structure that accommodate a mix of uses



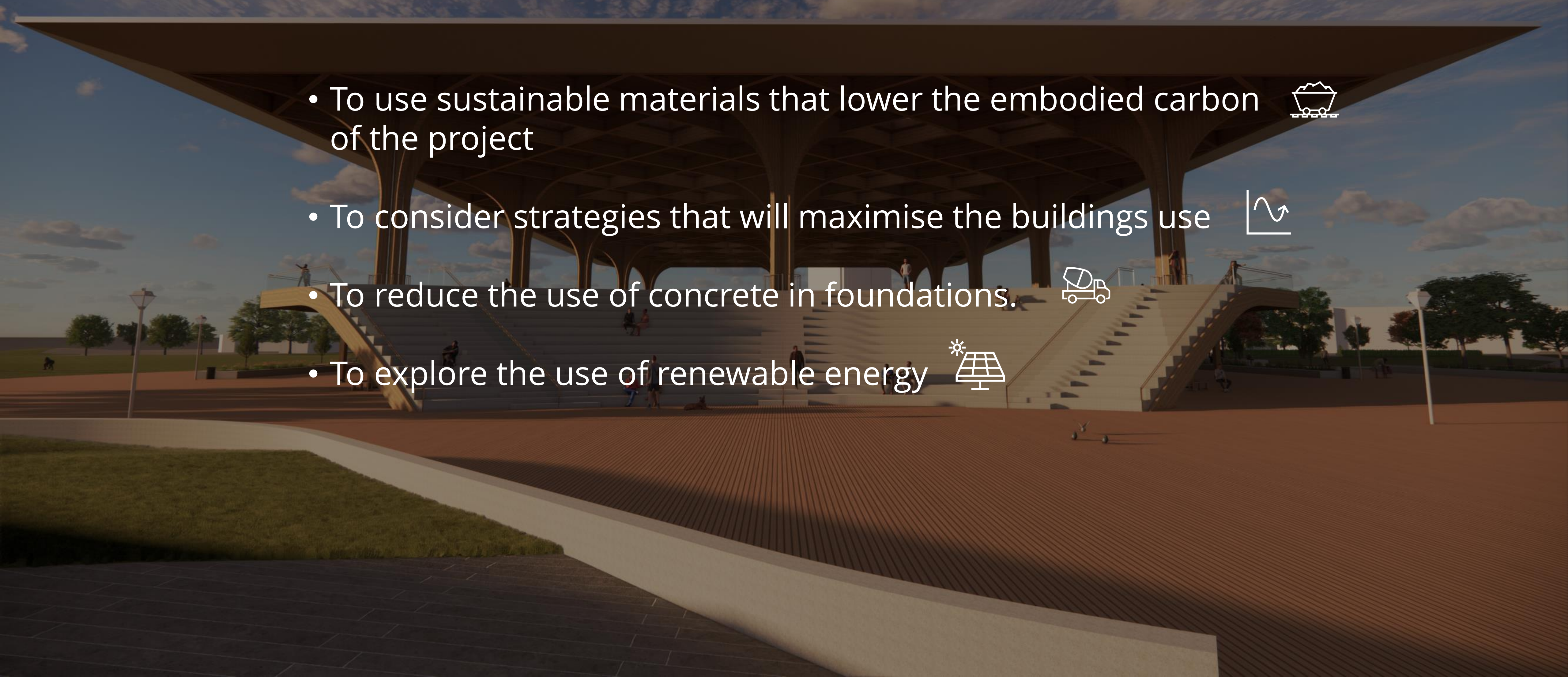
GENERAL OBJECTIVES



- Utilise key views of seafront
- Enhance ease of movement across seafront
- Provide multi-use space, that can be enjoyed seasonally
- Provide iconic architecture that seeks to honor the site's past

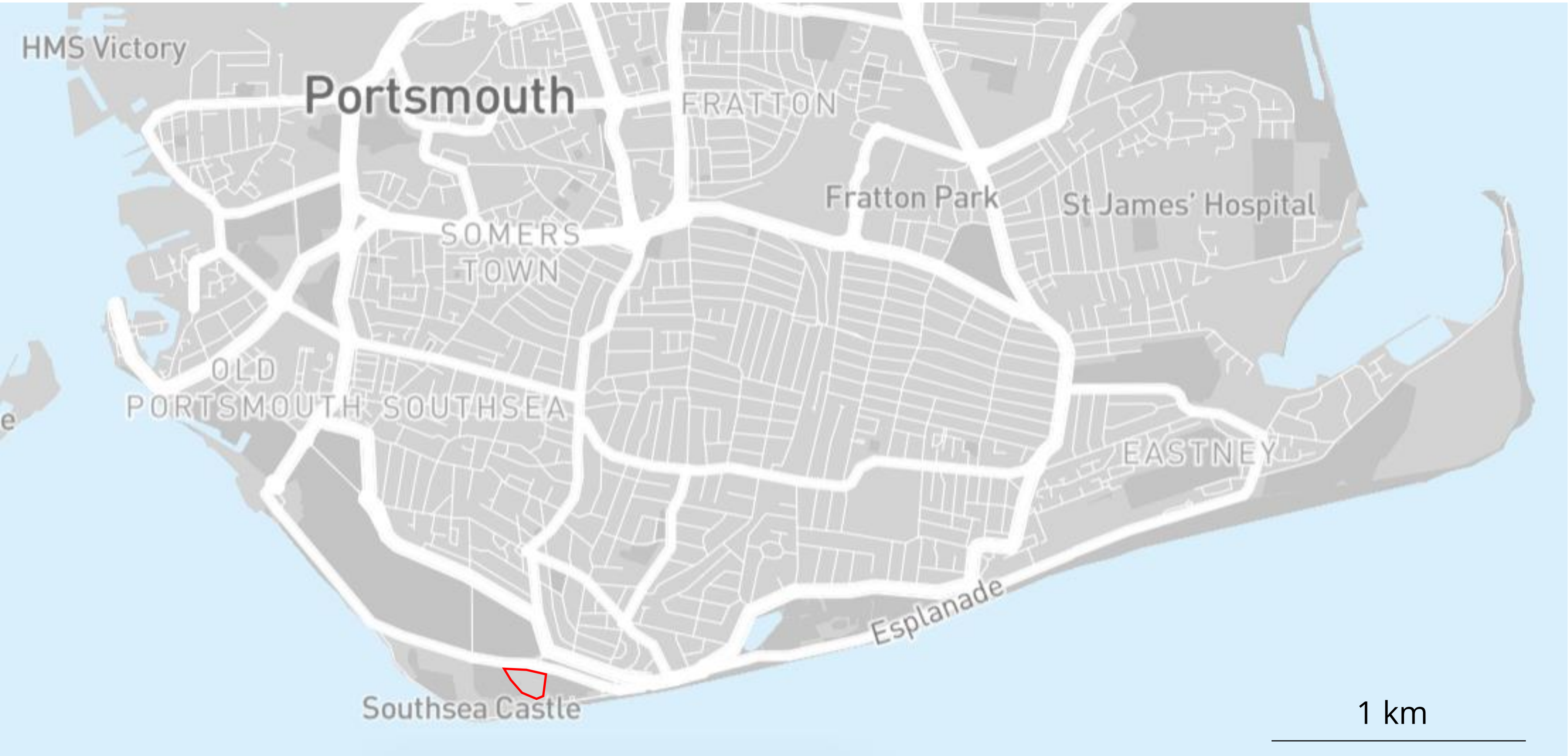
SUSTAINABILITY OBJECTIVES

- To use sustainable materials that lower the embodied carbon of the project
- To consider strategies that will maximise the buildings use
- To reduce the use of concrete in foundations.
- To explore the use of renewable energy



THE SITE

- Site Boundary



Map of the seafront of Portsmouth

Map of UK

SITE ANALYSIS

- Site Boundary

- Blocked Movement

SITE ANALYSIS

- Site Boundary
- Road Access
- Parking:
  1 – 132 Spaces
  2 – 141 Spaces
- Bus Route 1
- Bus Route 2

SITE ANALYSIS

- Site Boundary

- Sun Path

- Bore Hole Data:
0.4 m Topsoil
2.5 m Gravely Sand
6.6m Stiff Blue Clay

1 – The Castle

2 – Castle Field

3 – Rock Garden

SITE VIEWS

- Site Boundary

View 1

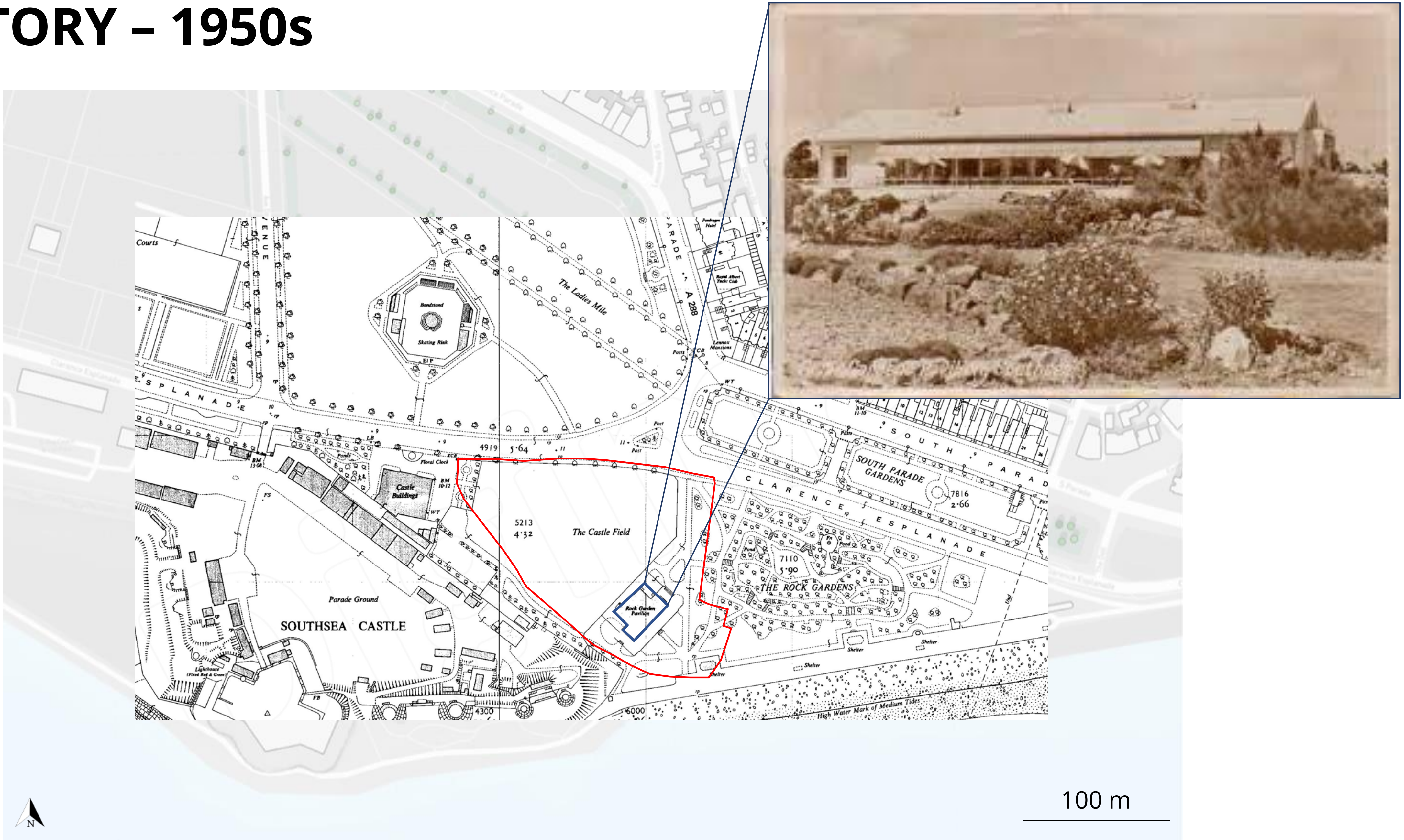
View 2

View 3

SITE HISTORY – 1950s

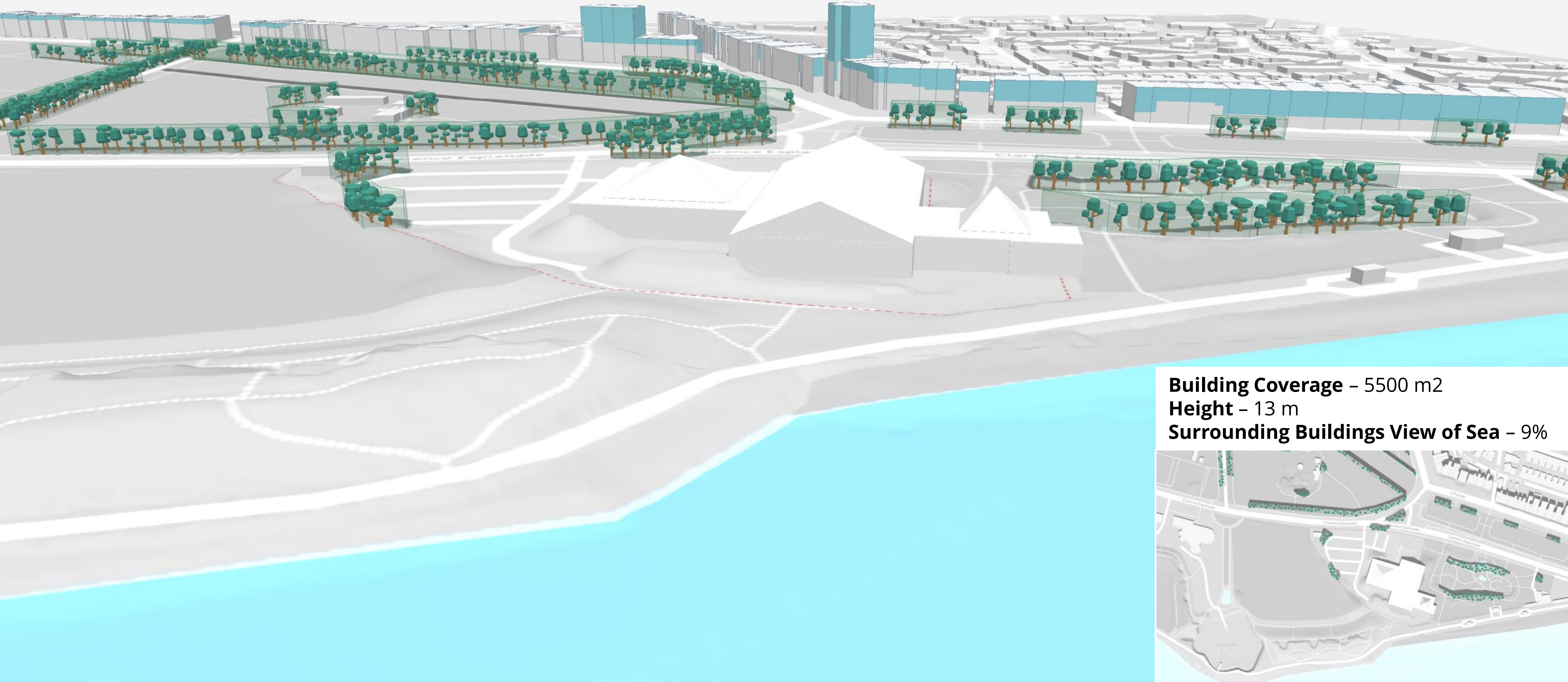
- Site Boundary

- Rock Garden Pavilion



VIEW ANALYSIS

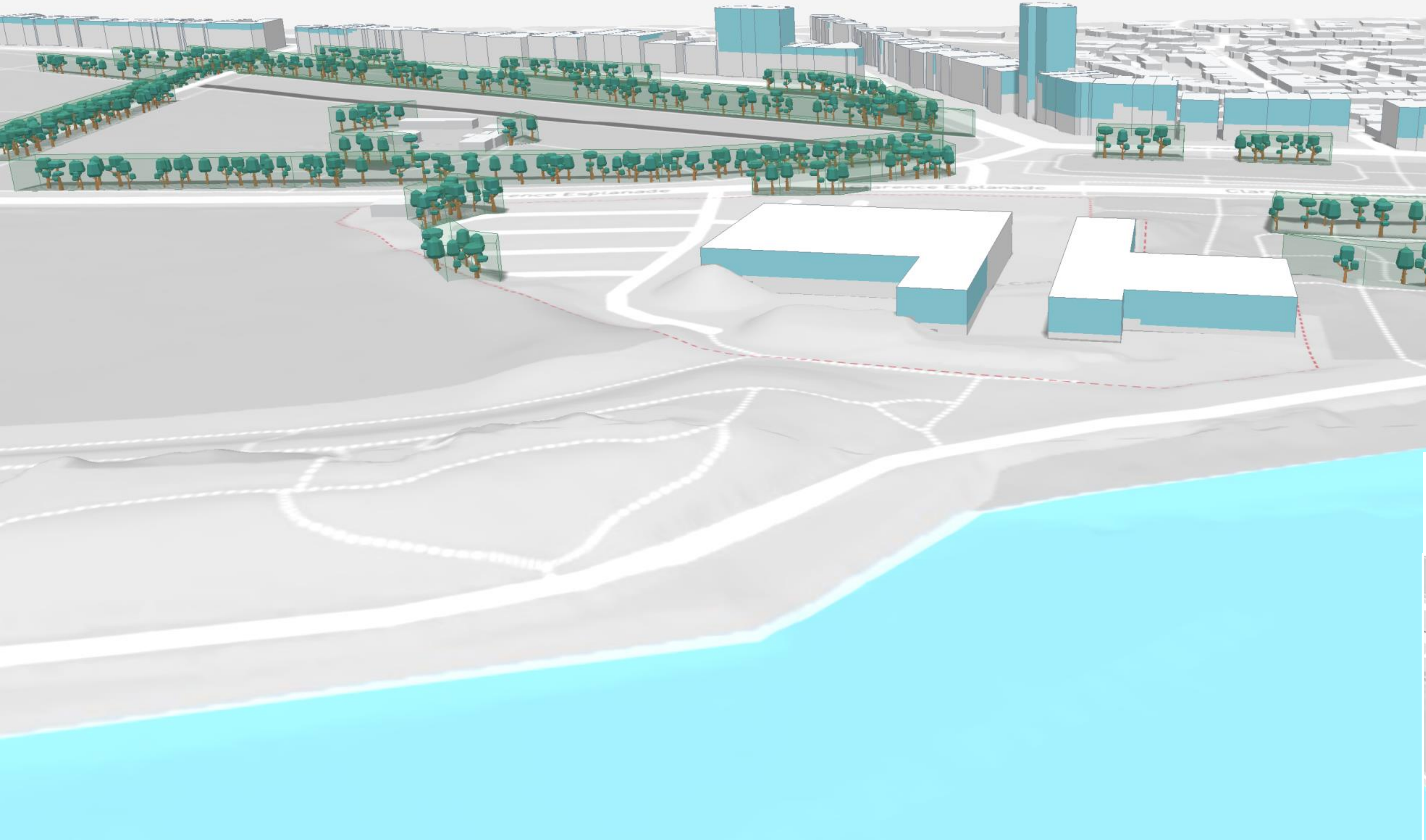
Analysing the % of surrounding façades that can see the sea



Building Coverage – 5500 m2
Height – 13 m
Surrounding Buildings View of Sea – 9%

VIEW ANALYSIS

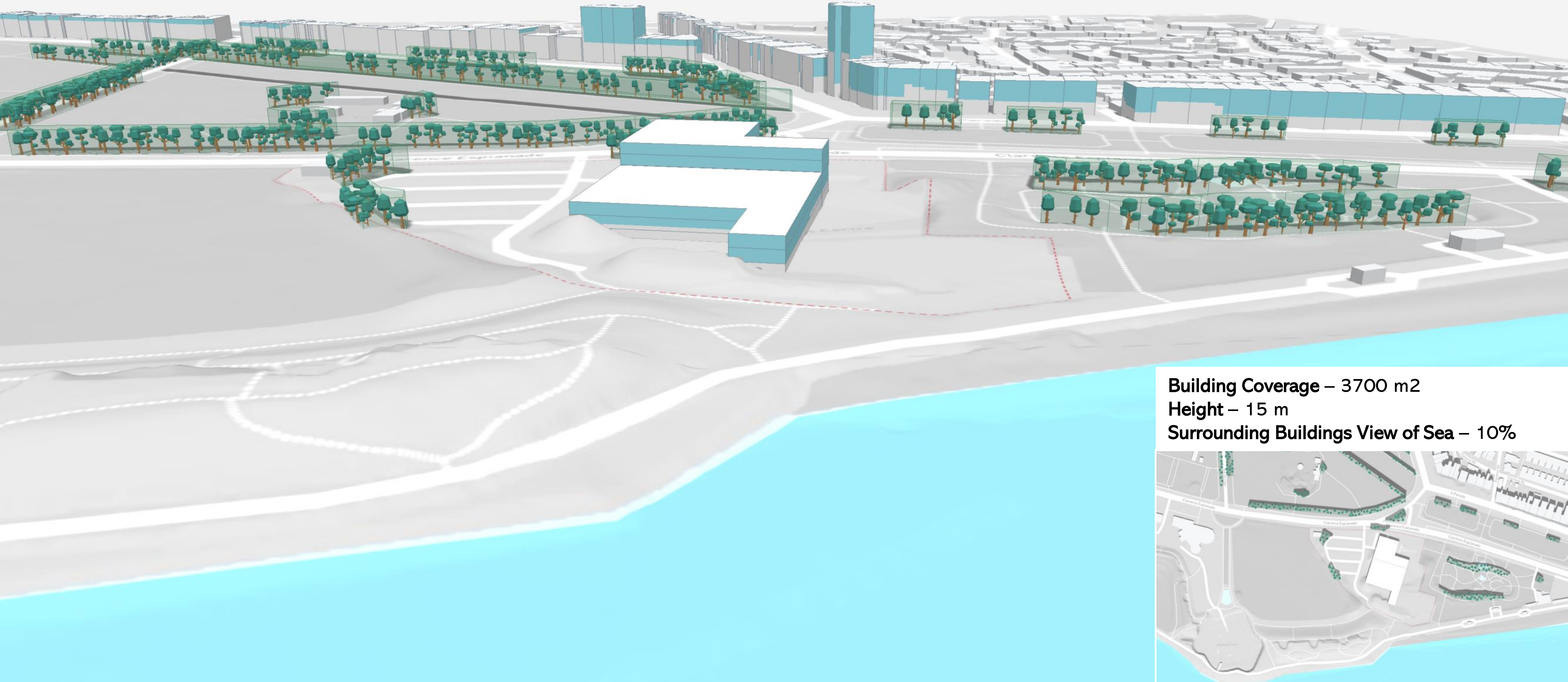
View of sea from surrounding buildings – Pyramids Concrete Base



Building Coverage – 3700 m2
Height – 9 m
Surrounding Buildings View of Sea – 10%

VIEW ANALYSIS

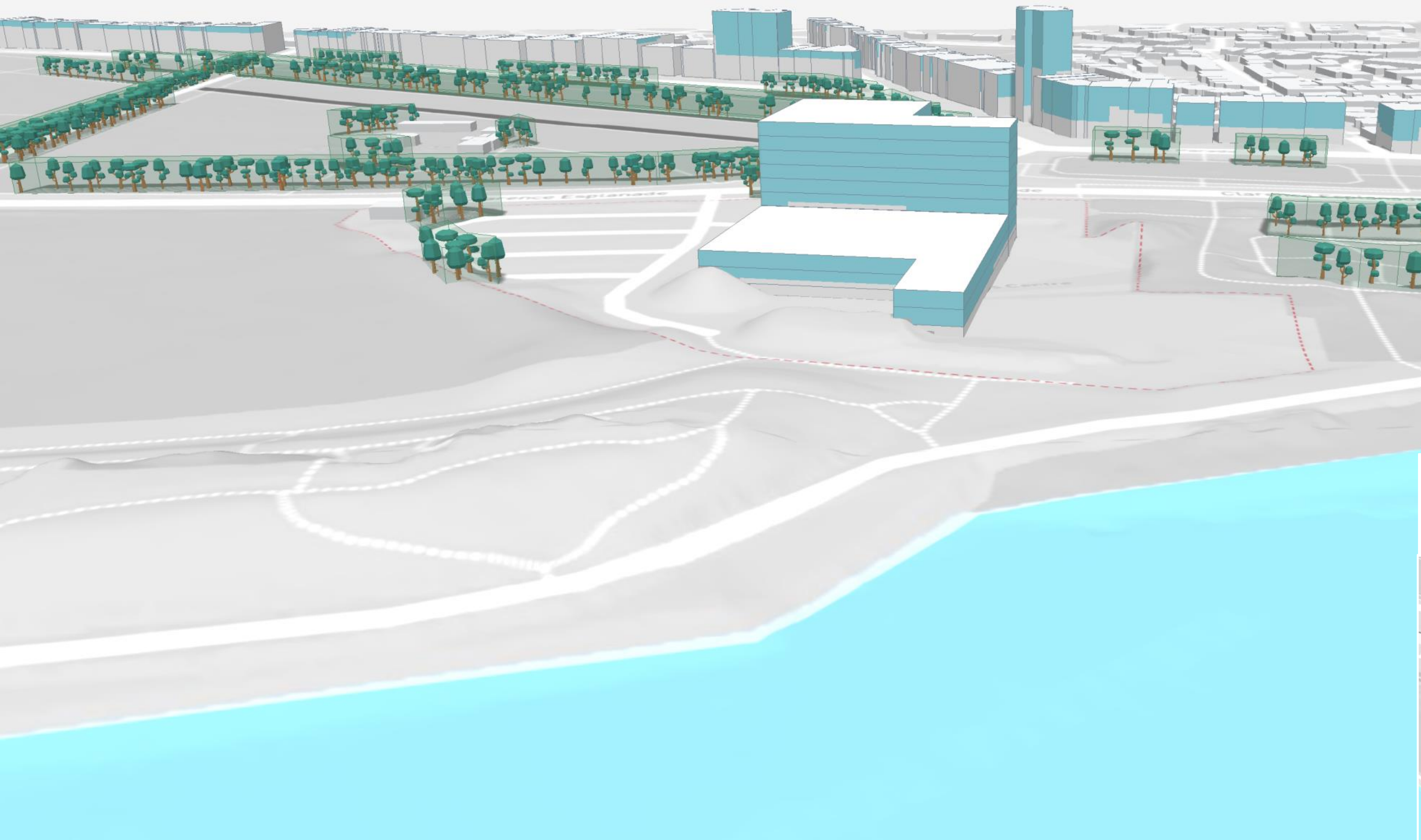
View of sea from surrounding buildings – North Development 5 Storeys



Building Coverage – 3700 m2
Height – 15 m
Surrounding Buildings View of Sea – 10%

VIEW ANALYSIS

View of sea from surrounding buildings – North Development 9 Storeys

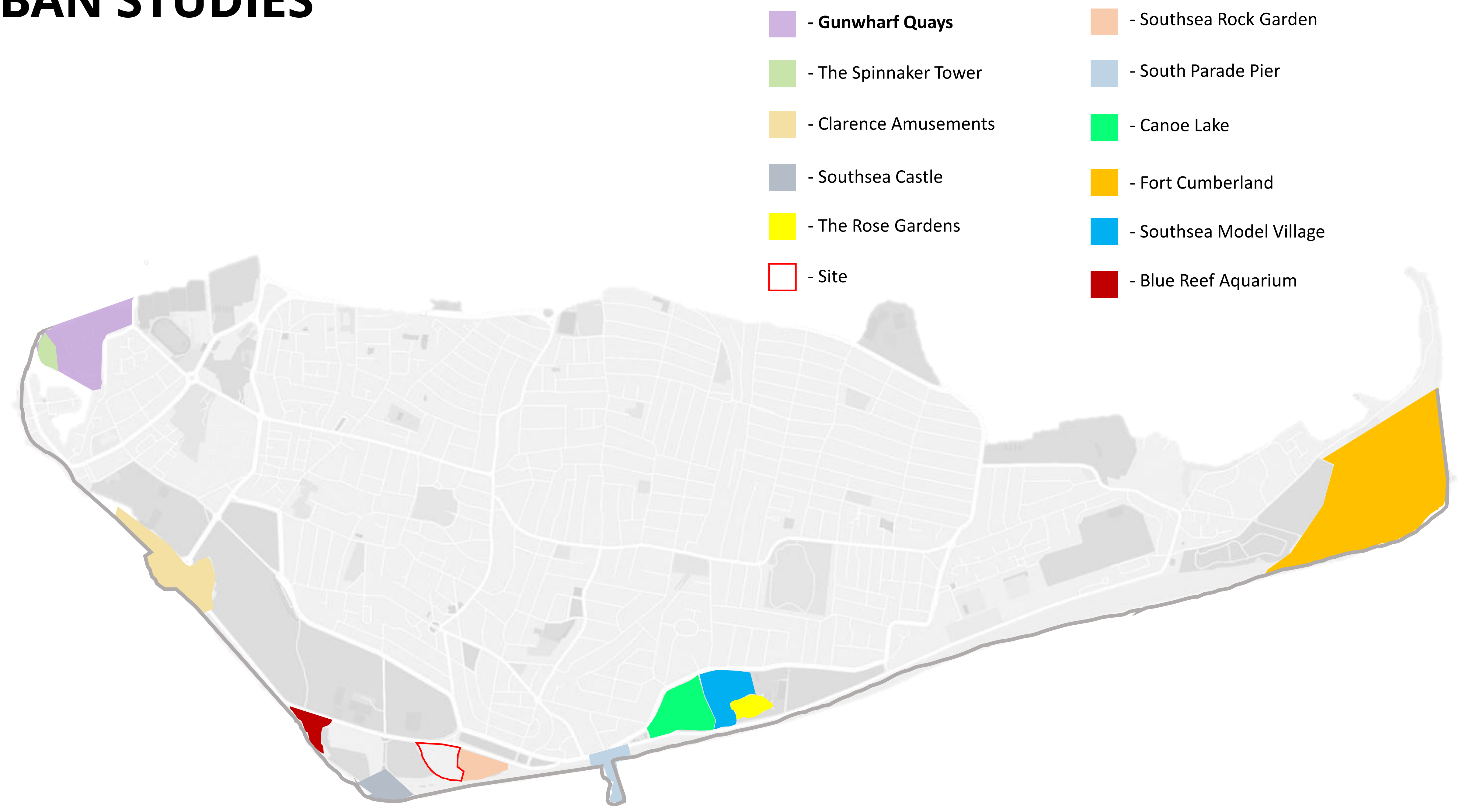


Building Coverage – 3700 m2
Height – 27 m
Surrounding Buildings View of Sea – 10%

VIEW ANALYSIS

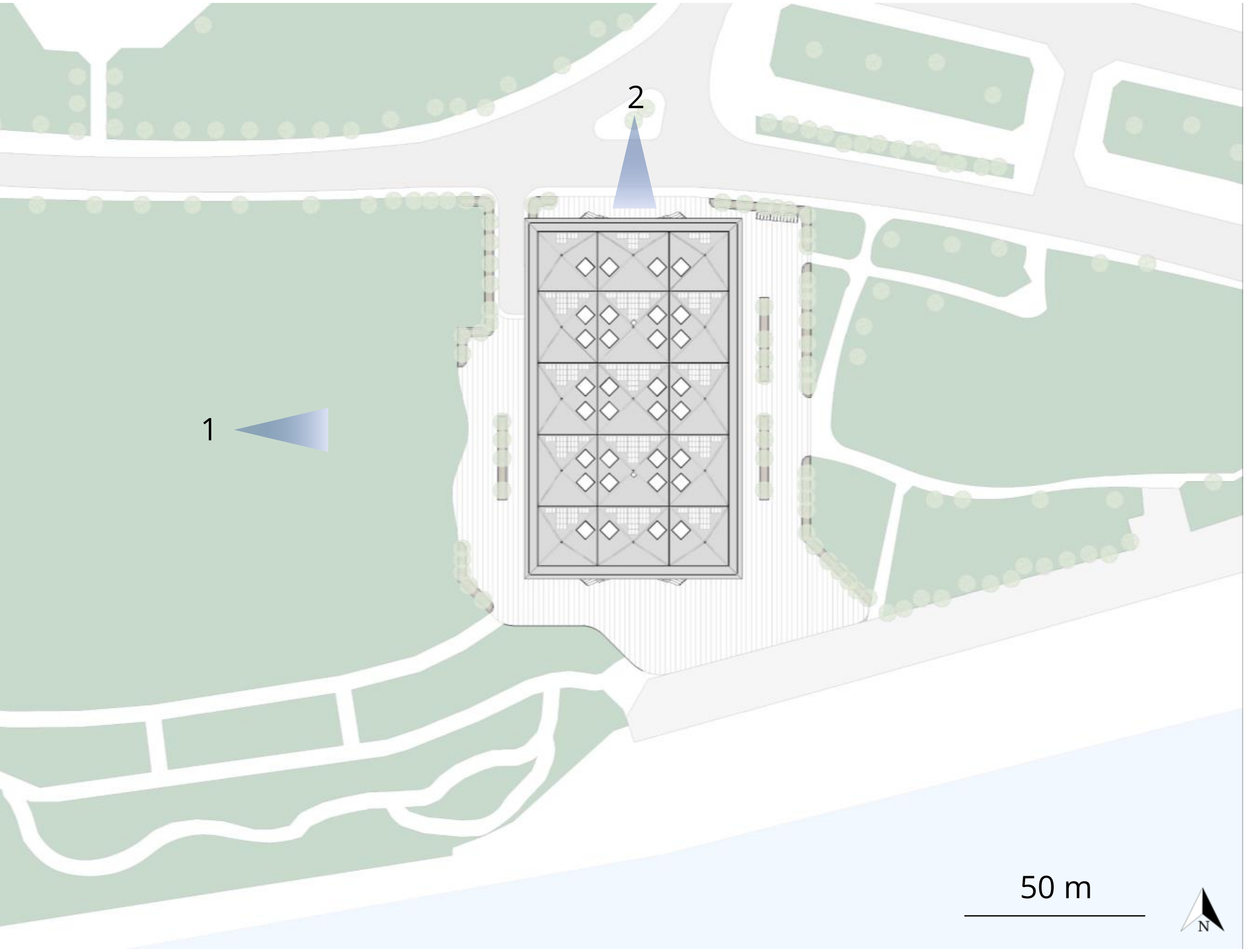
- Site Boundary
- Area not affecting surrounding building's view of sea
- Area affecting surrounding building's view of sea
- Area limited to 9m height to not affect surrounding building's view of sea

URBAN STUDIES



DRONE FOOTAGE

FINAL CONCEPT

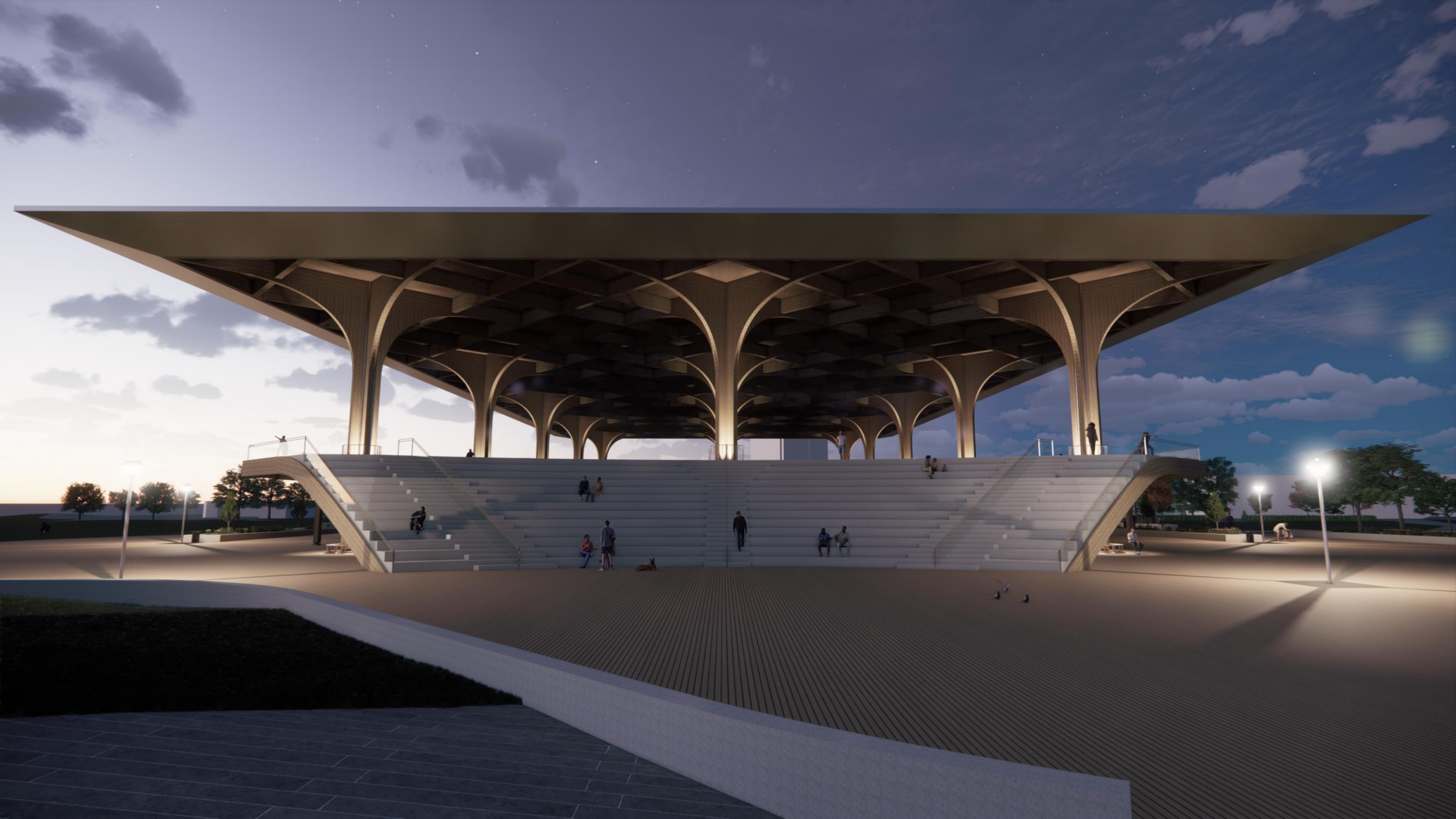


Site Plan

View 1

View 2

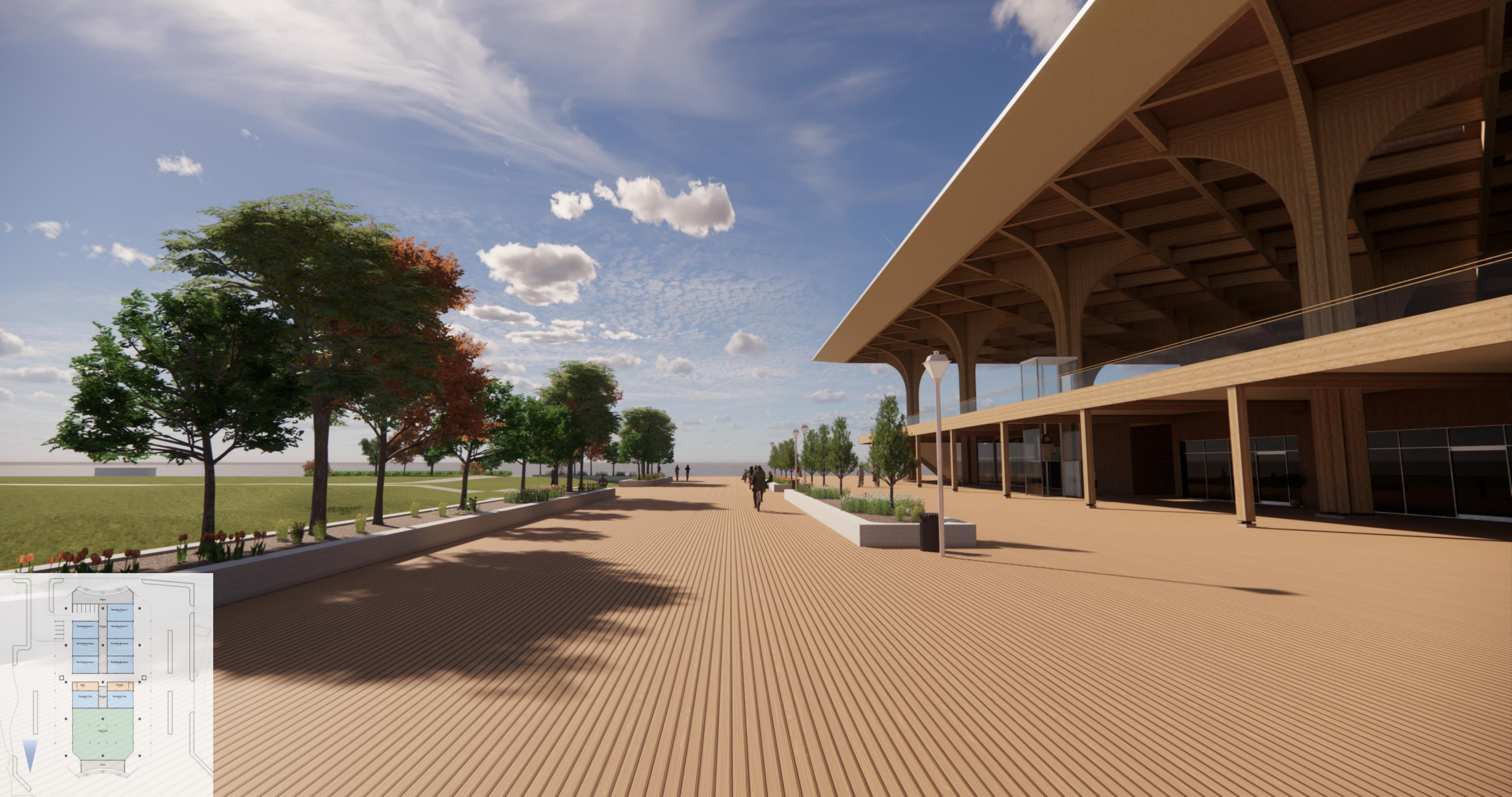
FINAL CONCEPT



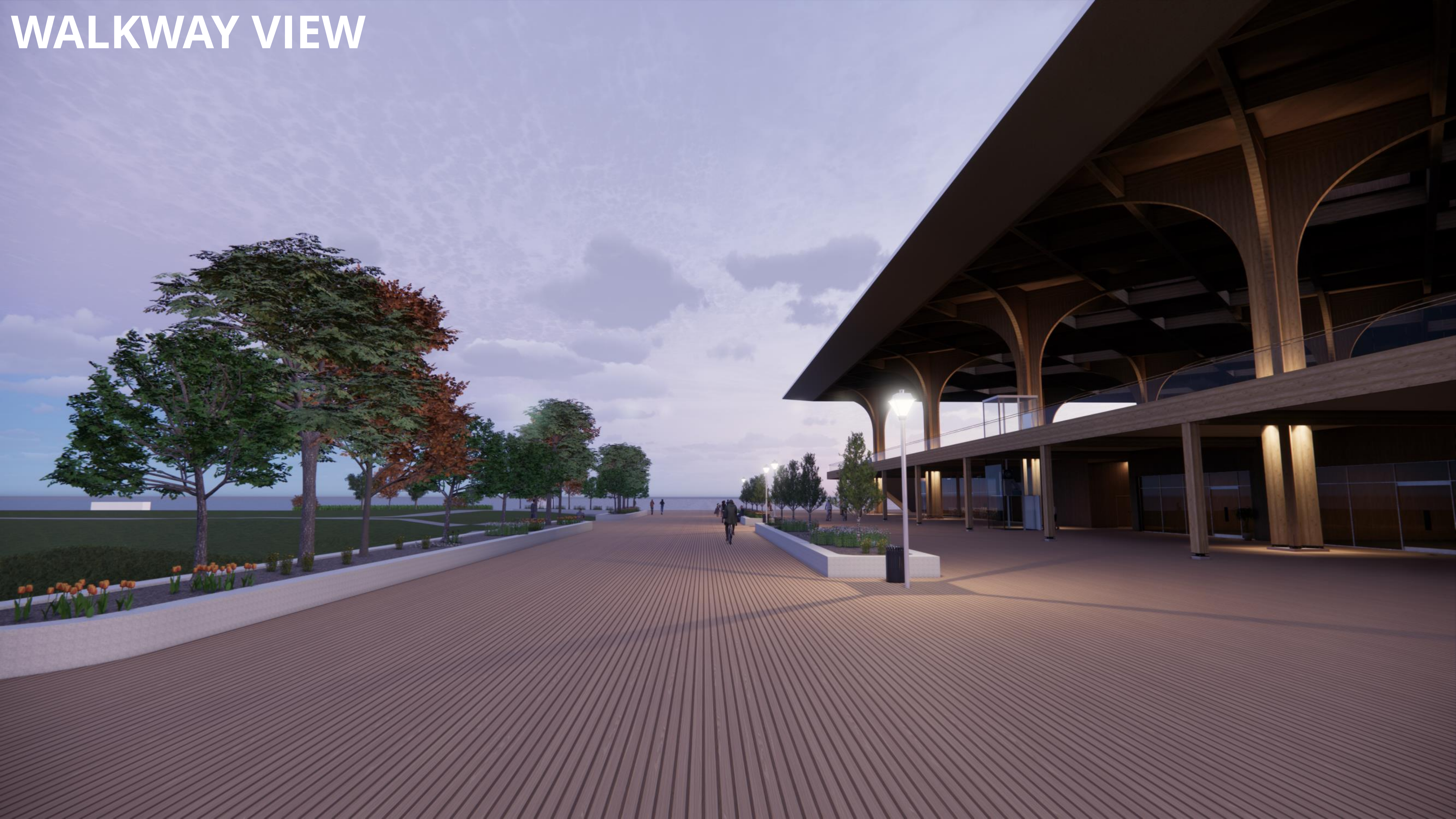
Rentable Space 1
Rentable Space 2
Storage
Rentable Space 3
Rentable Boutique
Rentable Boutique
Rentable Boutique
Rentable Boutique
Male
Female
Rentable Cafe
Storage
Rentable Cafe
Food Hall
Plant

WEST VIEW

WALKWAY VIEW



WALKWAY VIEW

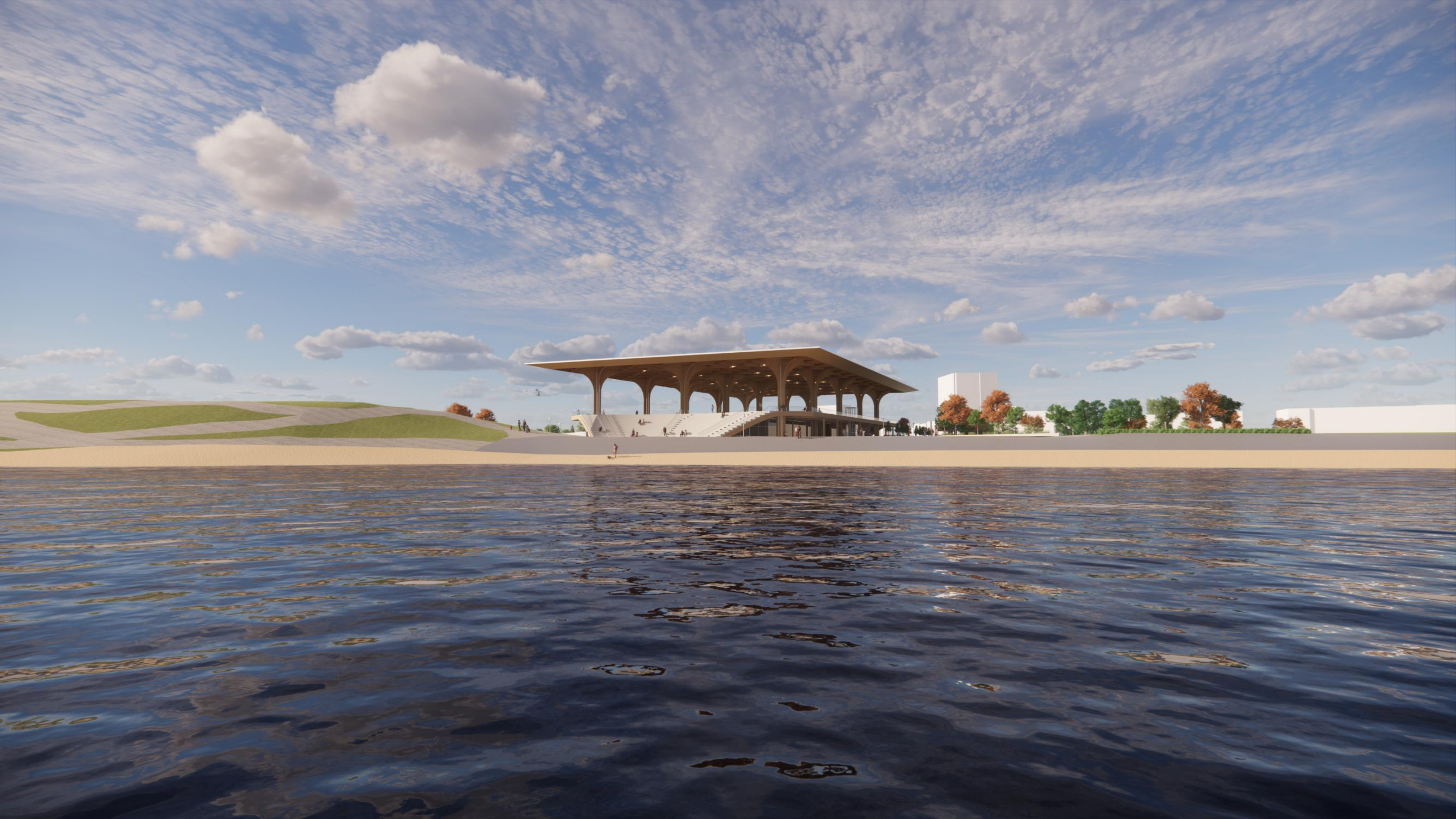


FOOD HALL

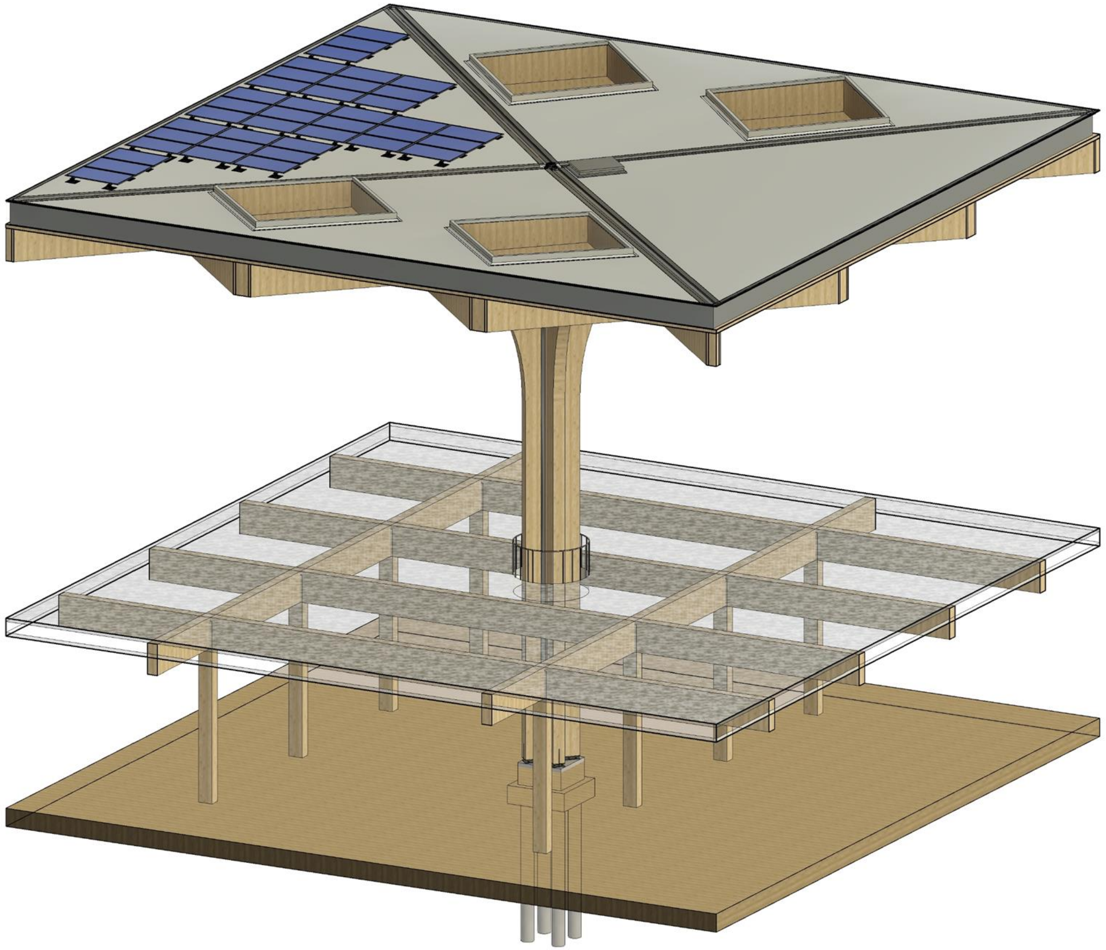
View of Food Hall

TOILETS AND GLASS ELEVATOR

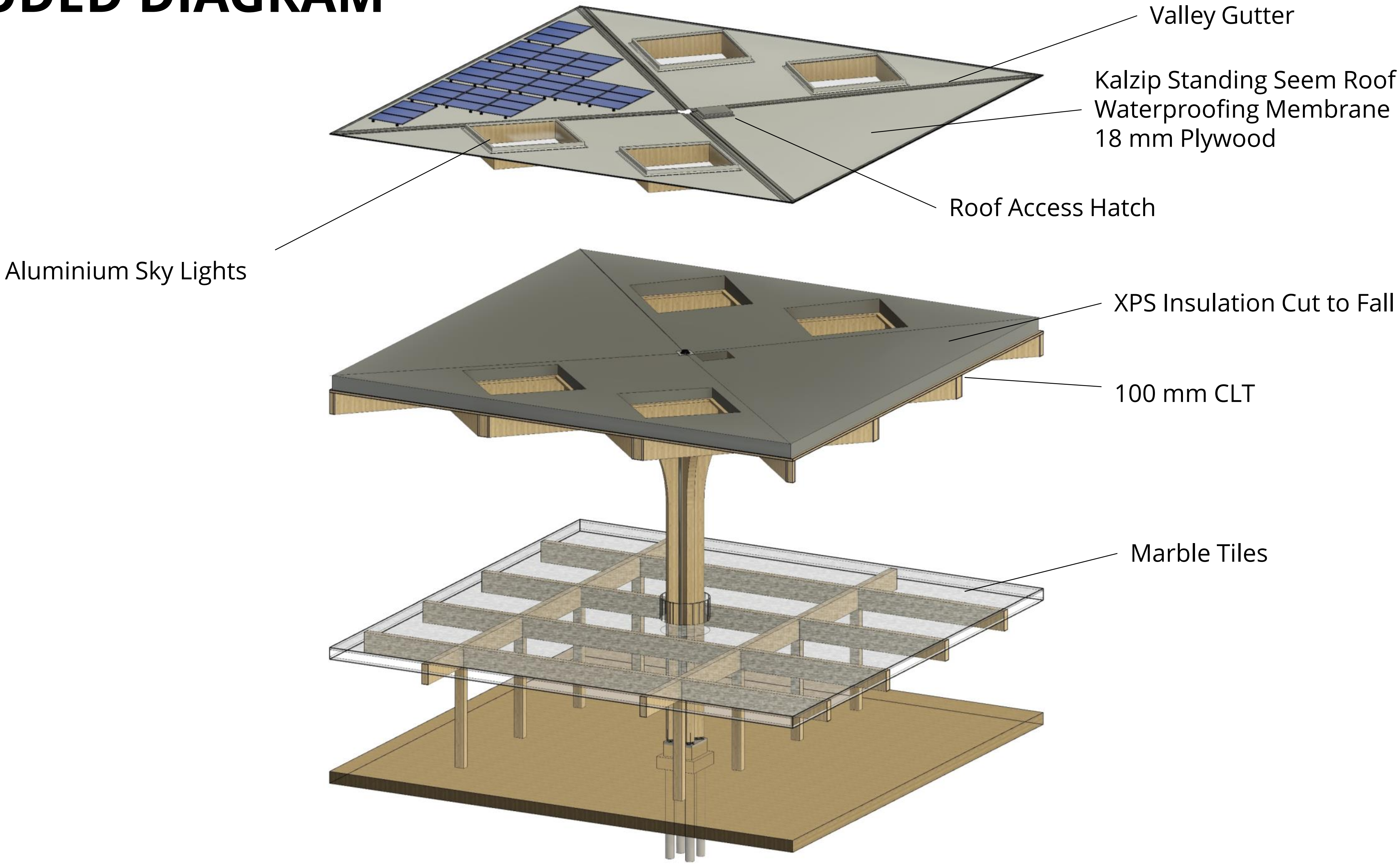
UNLOADING BAY



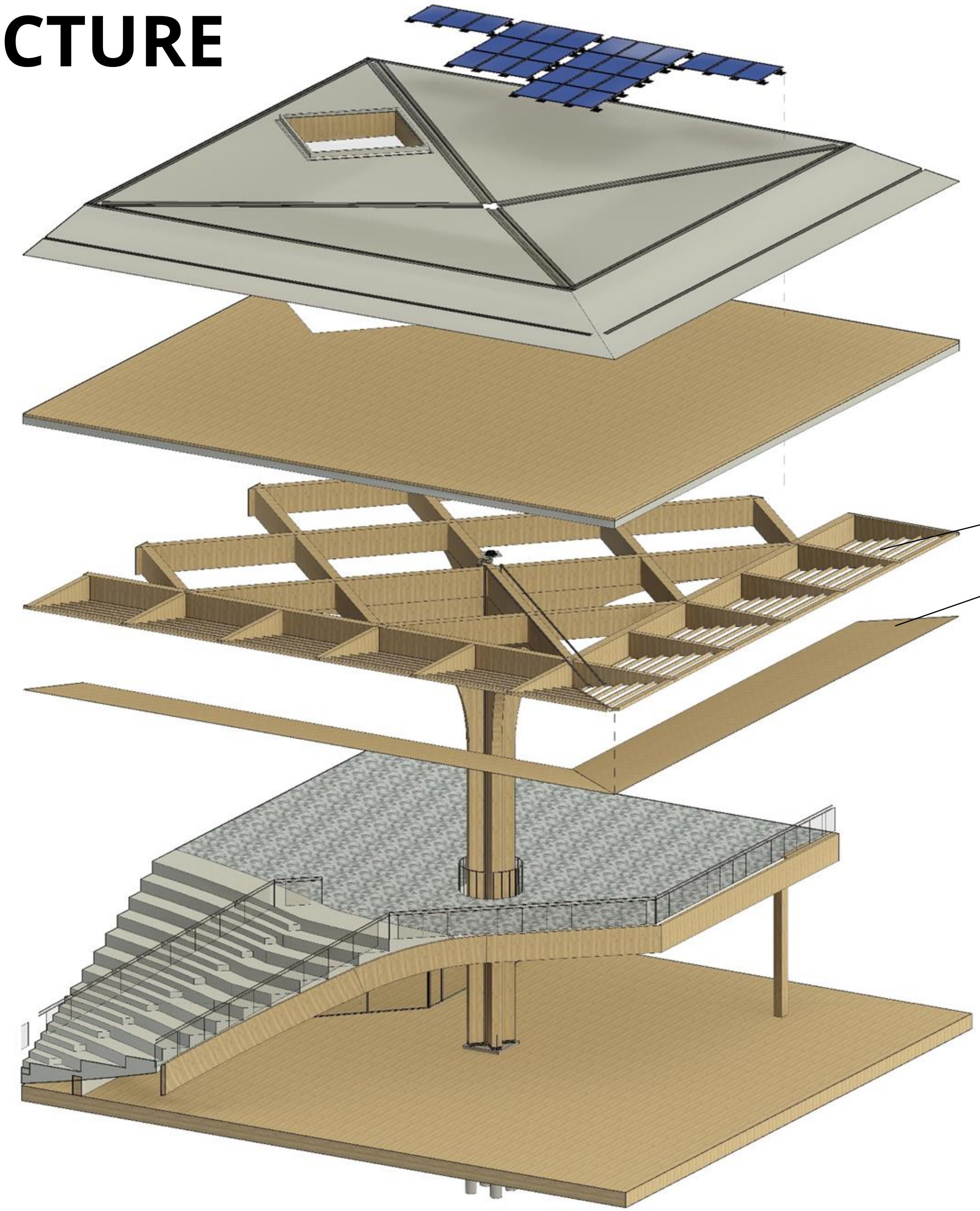
‘TREE COLUMN’ 3D SECTION



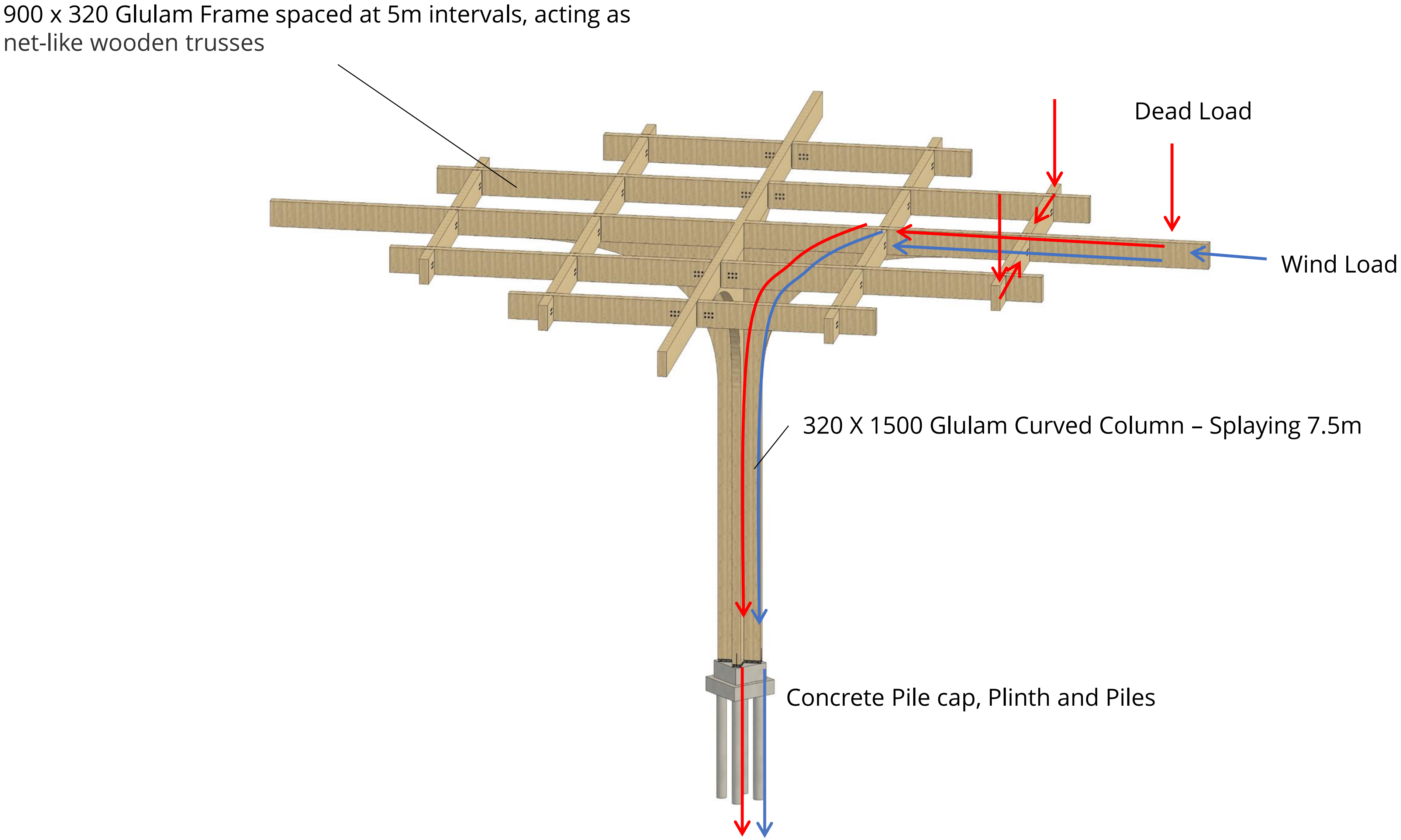
EXPLODED DIAGRAM



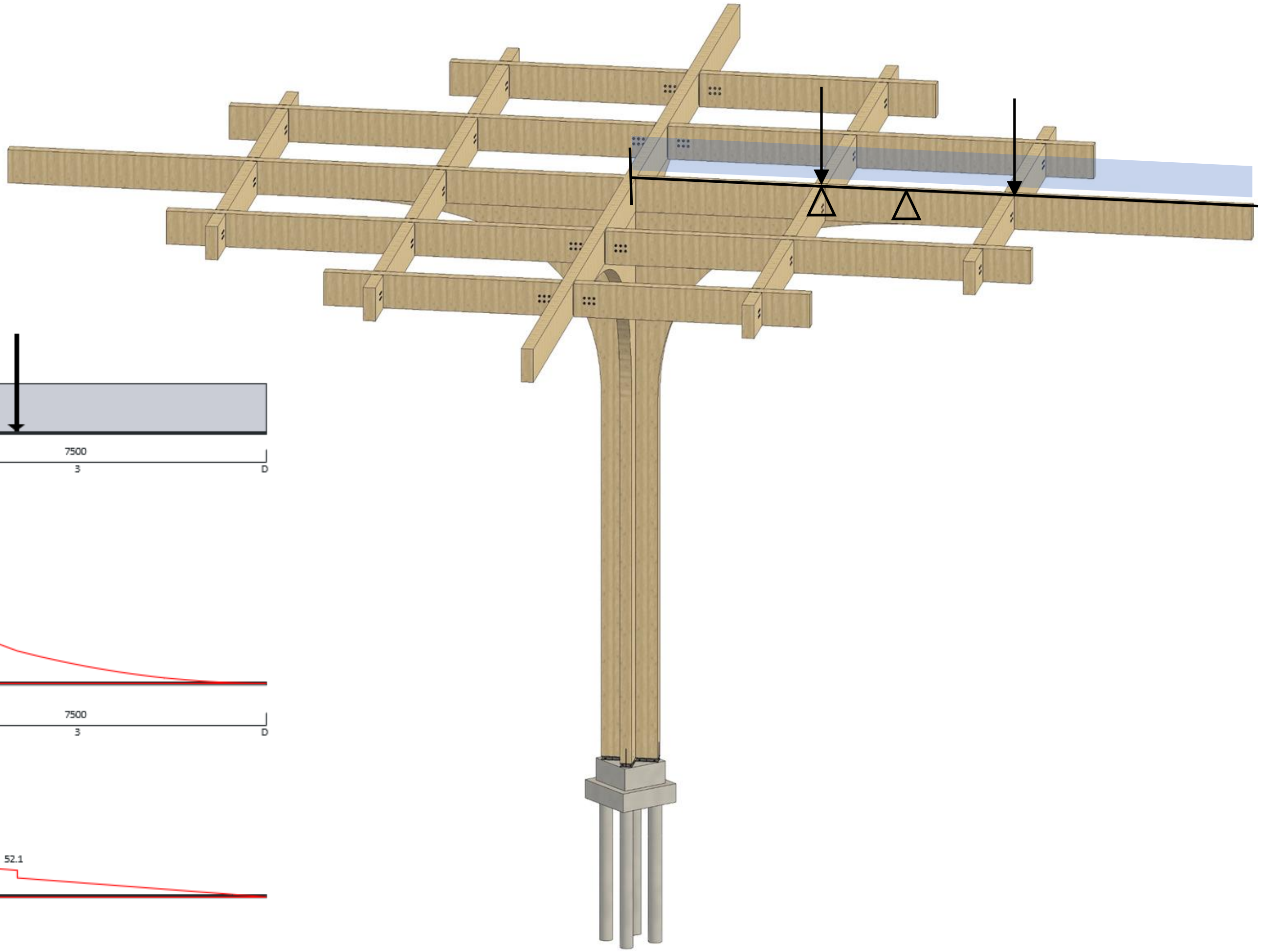
SECONDARY STRUCTURE



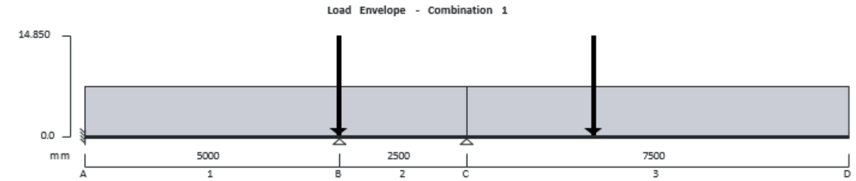
PRIMARY STRUCTURE

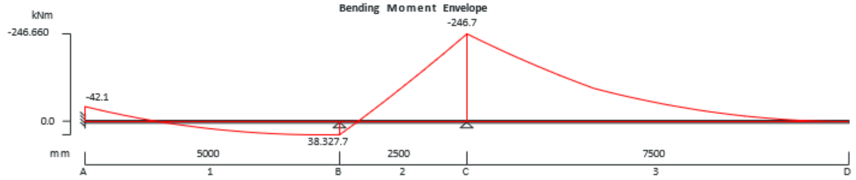


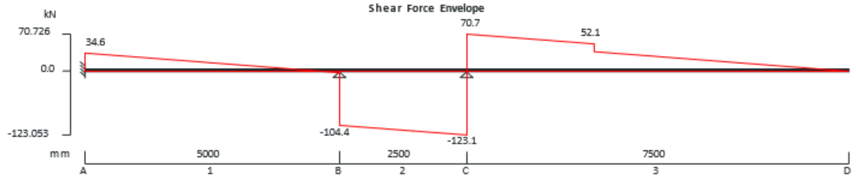
PRIMARY STRUCTURE



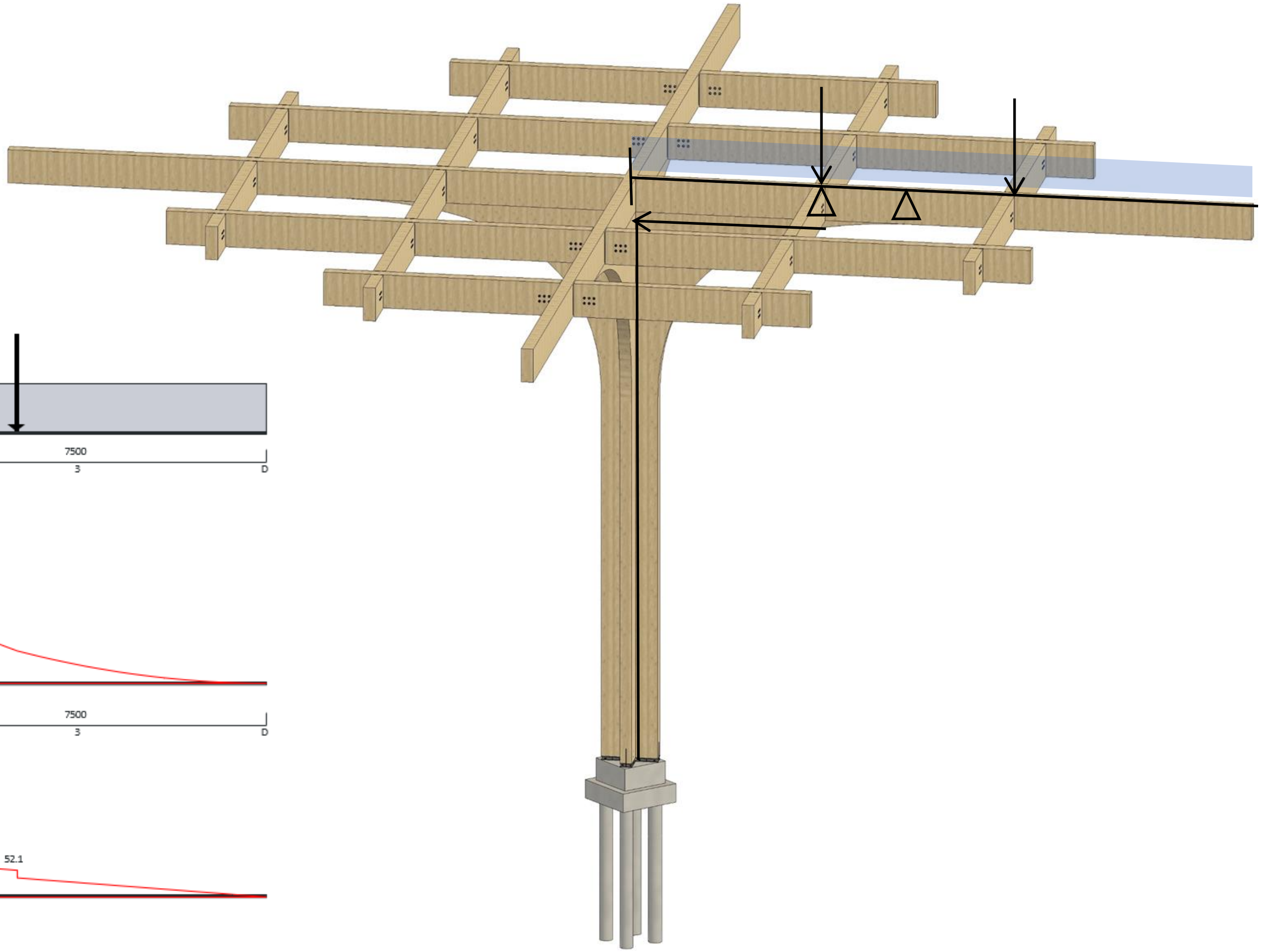
GL32C Beam Design



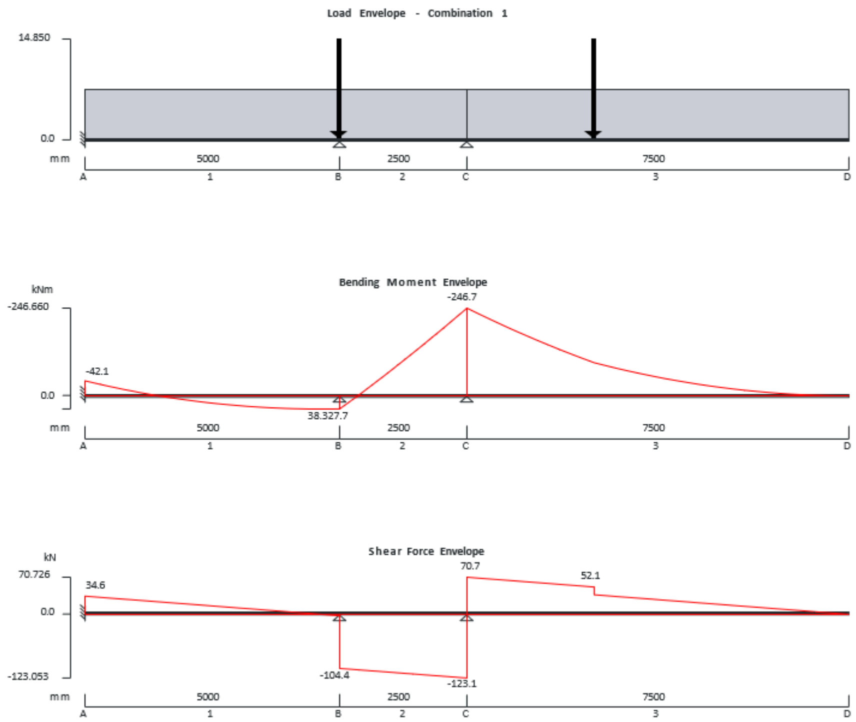




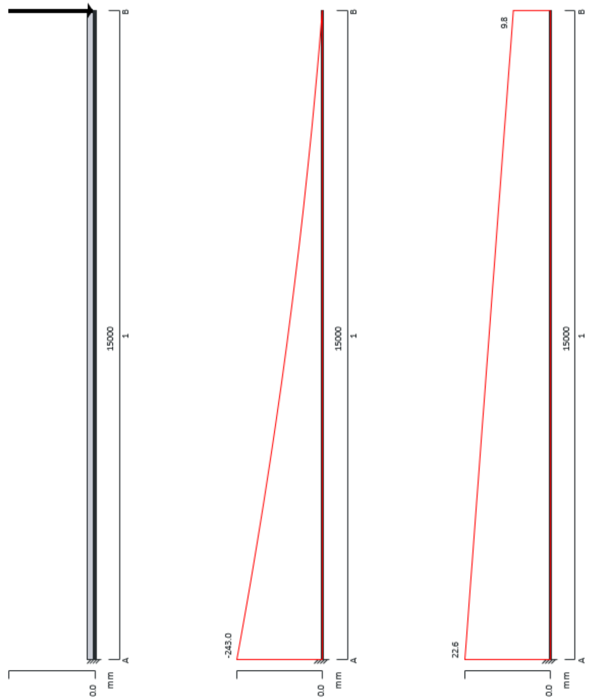
PRIMARY STRUCTURE



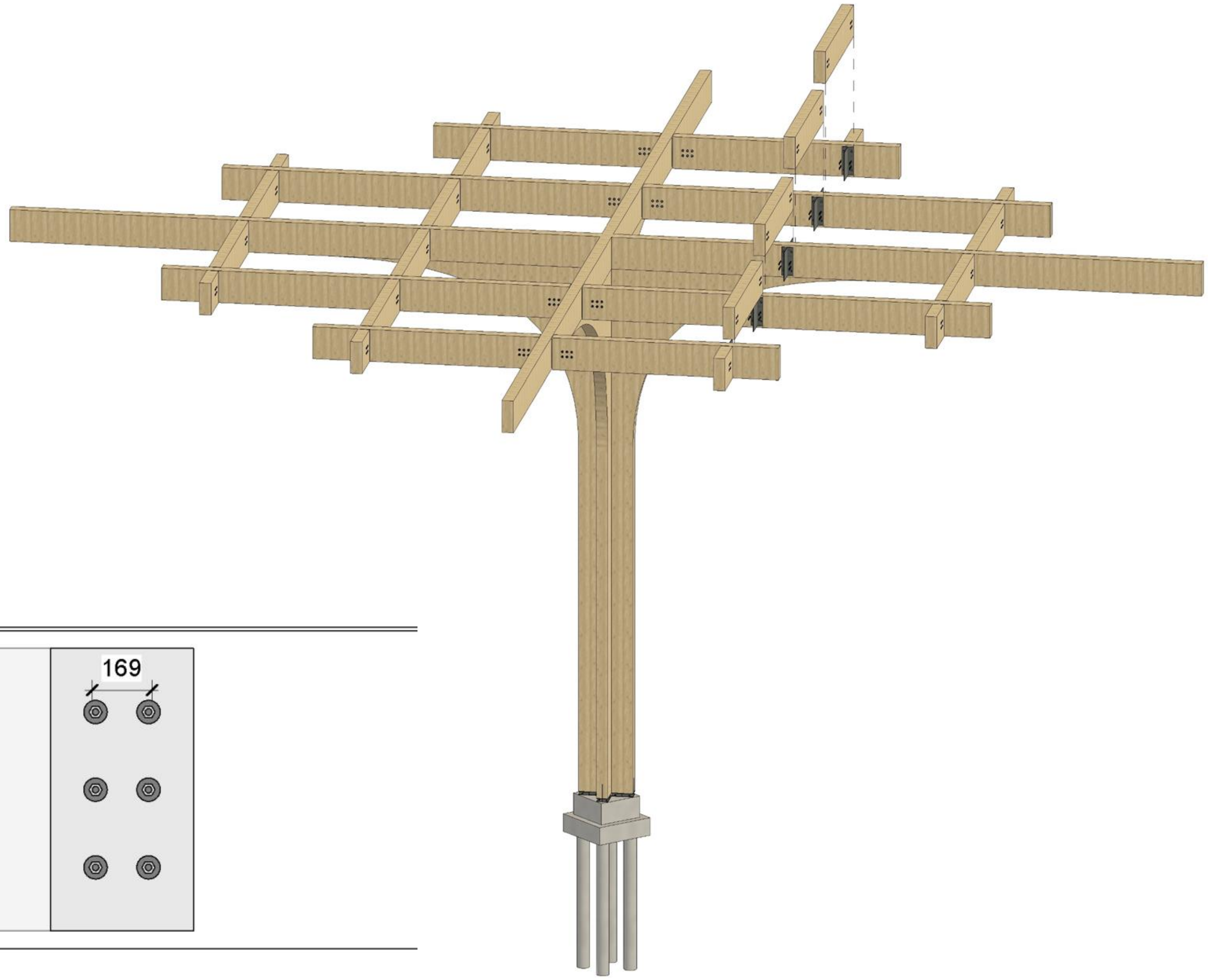
GL32C Beam Design



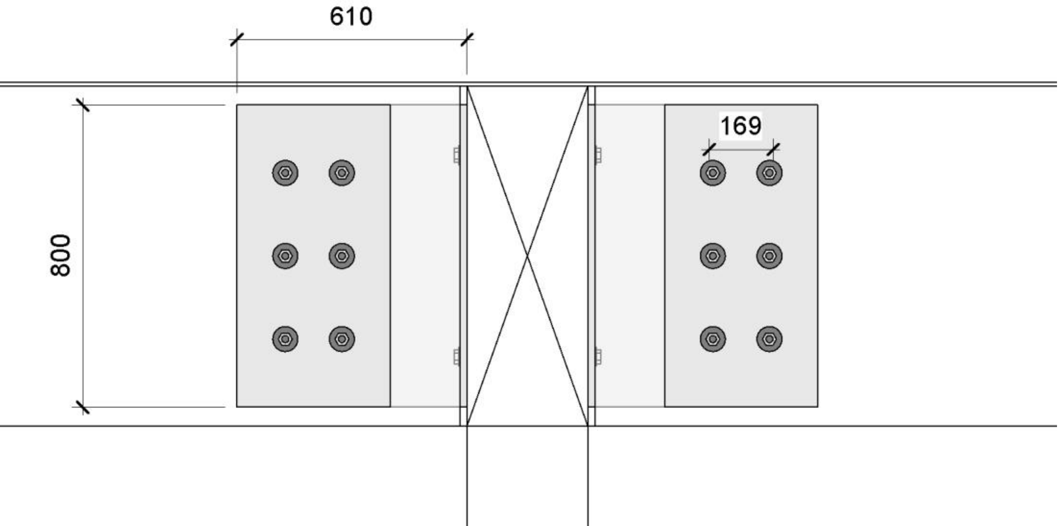
GL32C Column Design



PRIMARY STRUCTURE CONNECTIONS



Steel Knife Plate Connection



EMBODIED CARBON CALCULATION

Project Life Cycle Embodied Carbon:

	A	B	C	D	Biogenic Carbon
Total [tCO_2e]	1253	80	2709	-894	-2597

Total Embodied Carbon [tCO_2e]	3148
Total Sequestered Carbon [tCO_2e]	2597
Net Embodied Carbon	551

Scheme performance against targets:

SCORS: 137 kgCO2e/m2 Scheme meets target

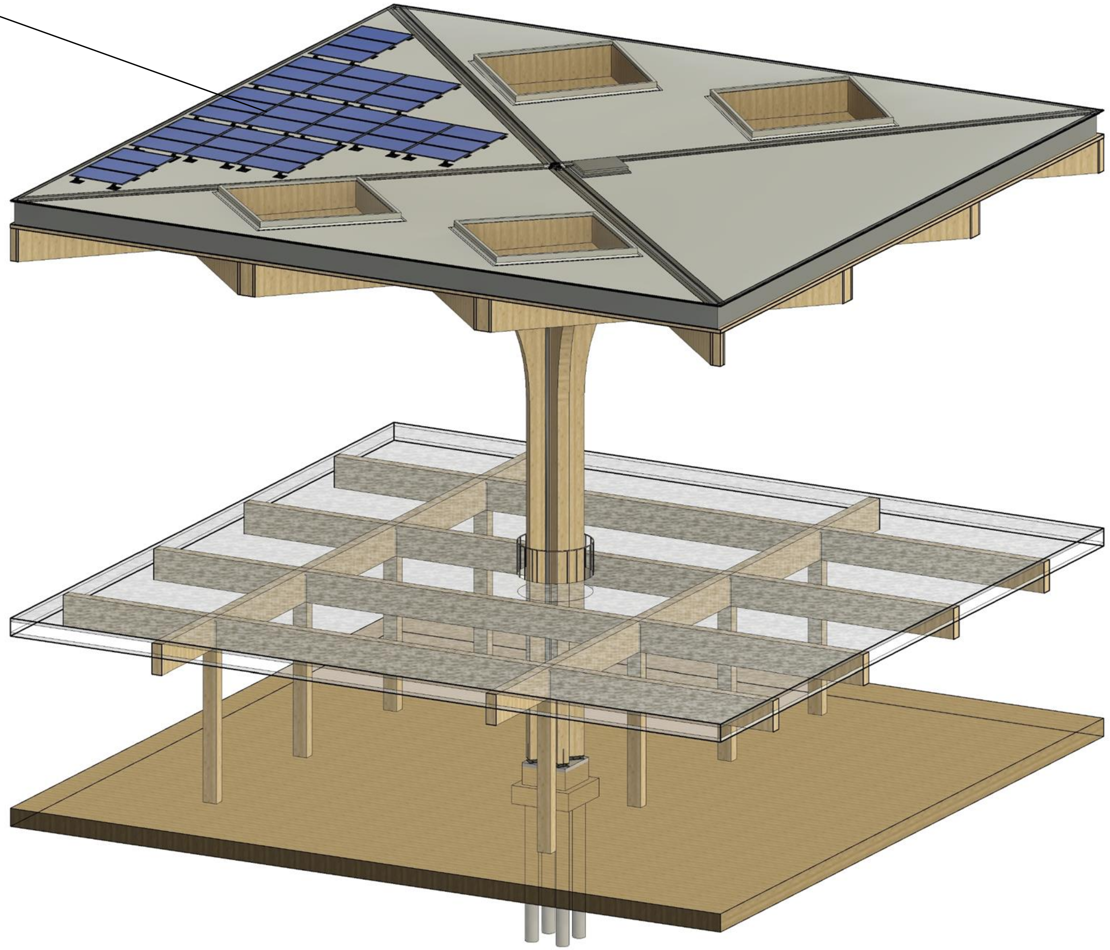
RIBA: 144 kgCO2e/m2 Scheme meets target

ENERGY

408 Panels at 300 Wp

Generates 104,040 kWh per year

Saves £22,000 per year



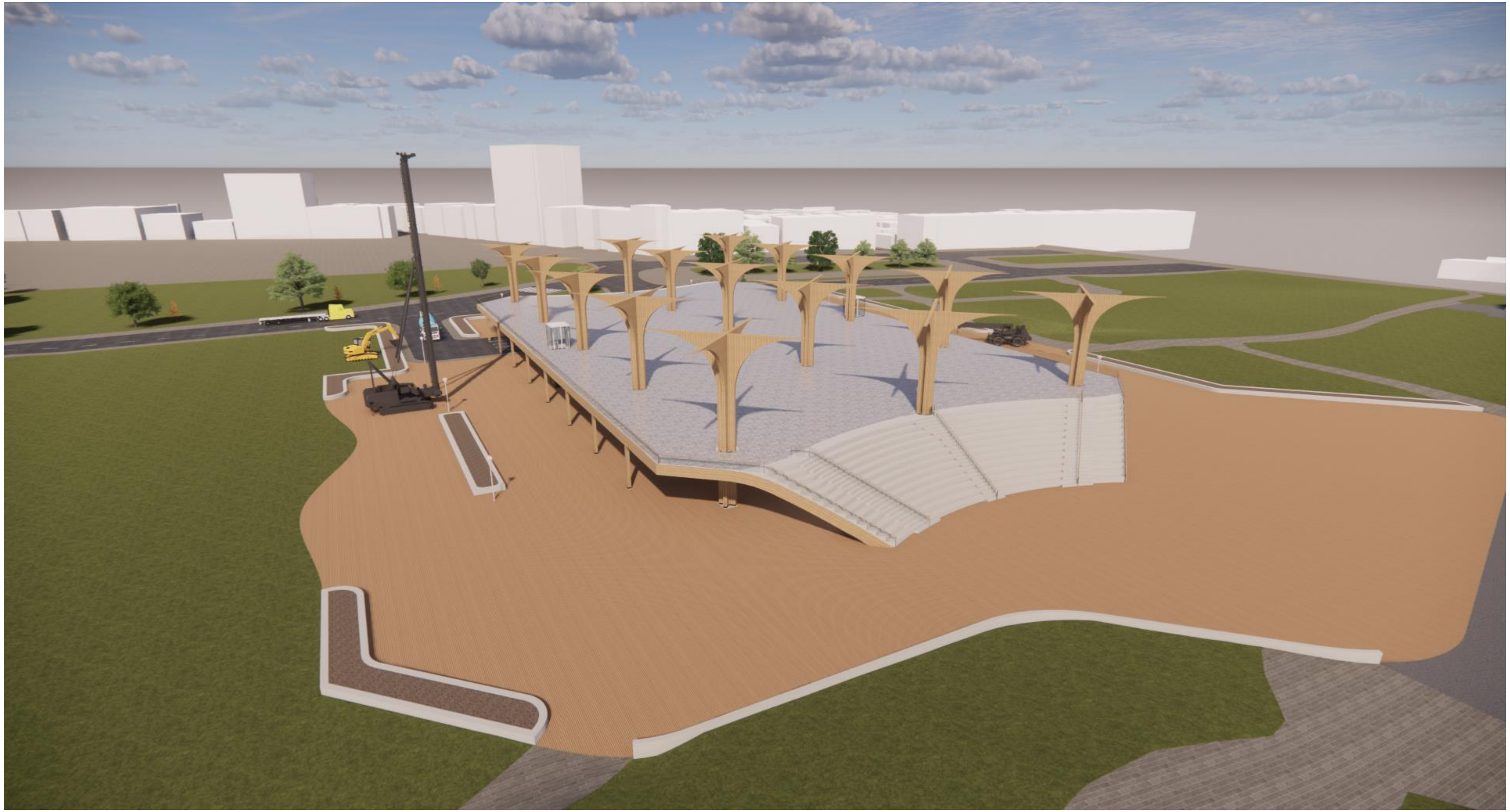
CONSTRUCTION PROCESS

Foundation

CONSTRUCTION PROCESS

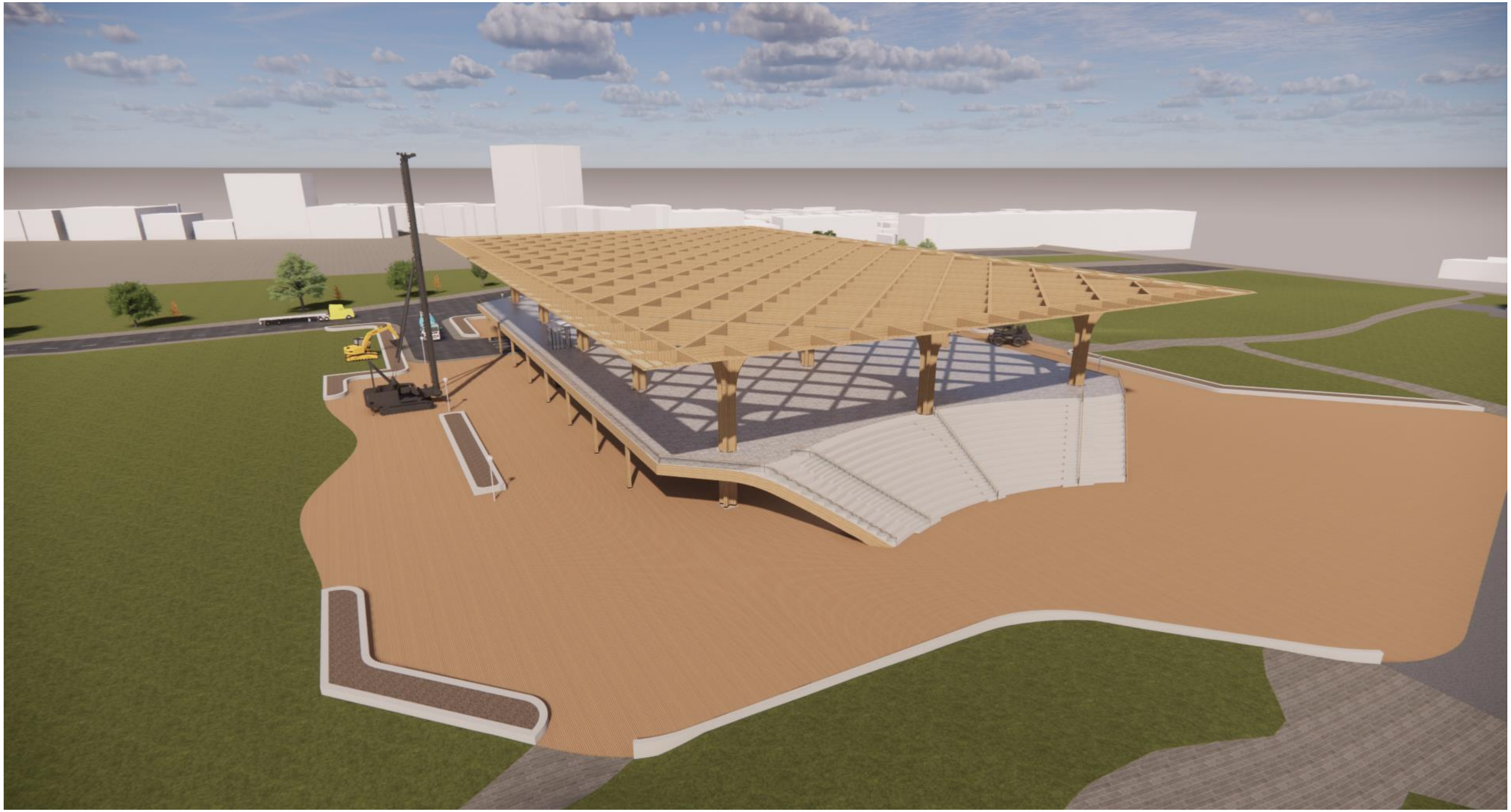
Deck Structure, Canopy Columns, Concrete Base

CONSTRUCTION PROCESS



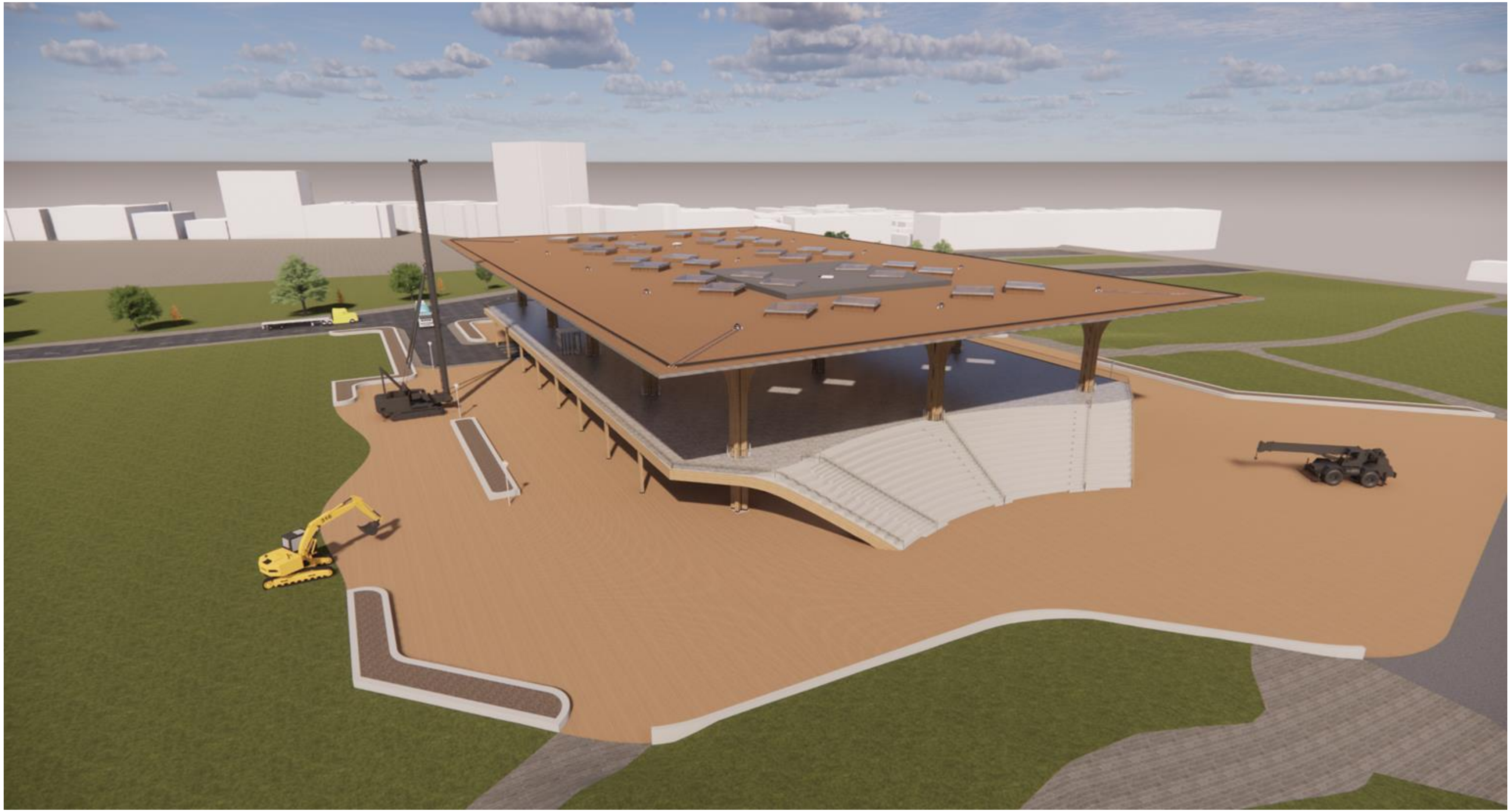
Stairs, CLT Deck Slab, Landscaping

CONSTRUCTION PROCESS



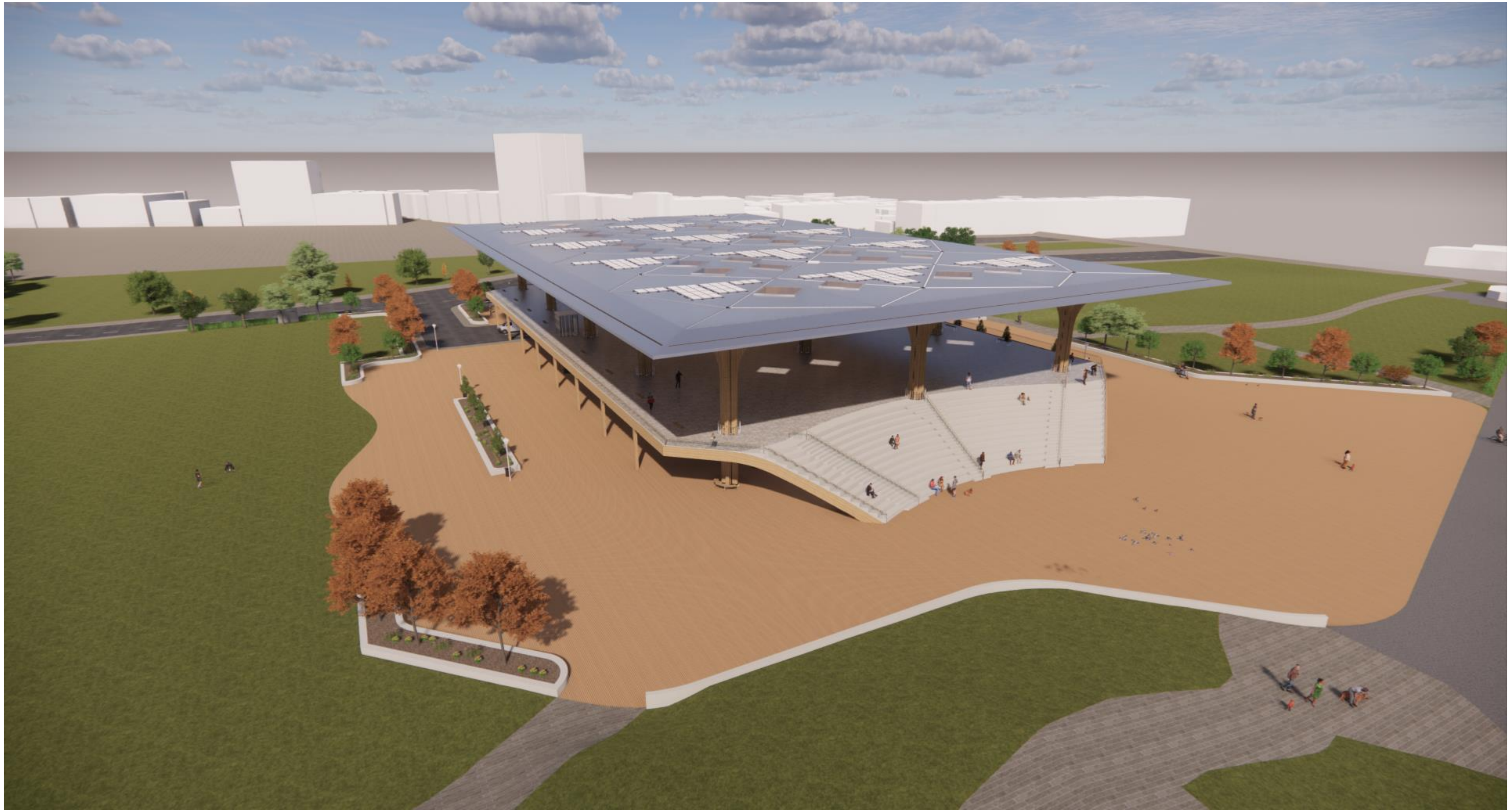
Glulam Frame for Pavilion

CONSTRUCTION PROCESS



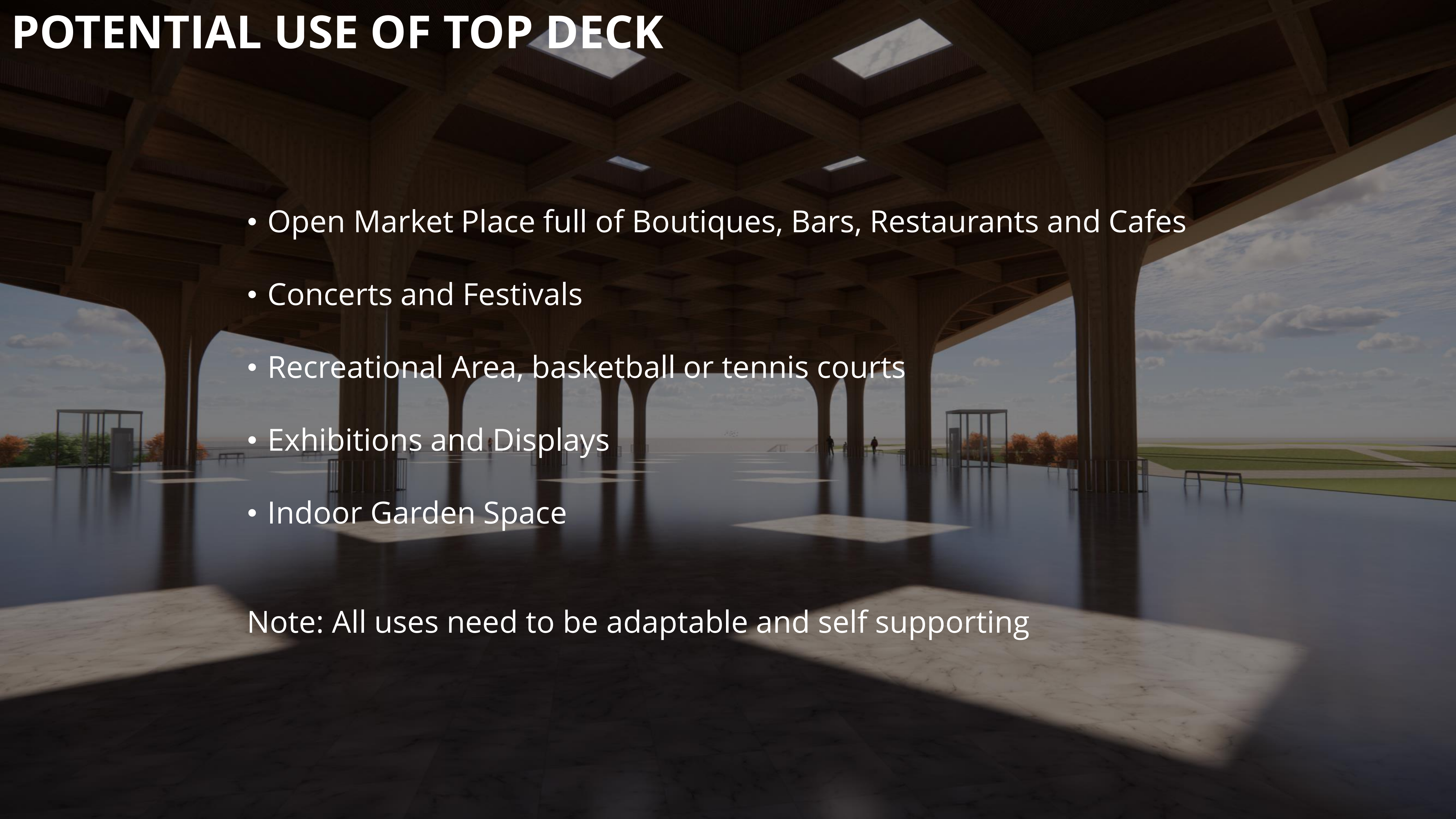
CLT Roof Slab and Insulation Cut to Fall

CONSTRUCTION PROCESS



Roof Finishes and Planting of Trees

POTENTIAL USE OF TOP DECK



- Open Market Place full of Boutiques, Bars, Restaurants and Cafes
- Concerts and Festivals
- Recreational Area, basketball or tennis courts
- Exhibitions and Displays
- Indoor Garden Space

Note: All uses need to be adaptable and self supporting

THANKS FOR LISTENING

Appendix D Physical Model – Scale 1:500