DESIGN OF THE SOUTHFIELD RUGBY CLUB NEW GRANDSTAND

Structural Concrete 2011

Entry No. 11-001
24/6/2011

CONCEPTUAL DESIGN REPORT

The following report presents the initial structural design concept that was commissioned for the replacement of the Southfield Rugby Club Grandstand. This report includes an appraisal of two options and a recommendation is made for the choice of design. For the recommended design, the foundation scheme, material specifications, construction procedure, and robustness are also discussed. Verification for structural viability, drawings, and plans for sustainability of the structure have been provided in the appendices.

AN APPRAISAL OF TWO DESIGN OPTIONS

For the replacement of the Southfield Rugby Club Grandstand, we have developed two designs for consideration, both of which will enhance the experience of watching a match of the recently promoted team, while keeping with the basic requirements set out by the client.

DESIGN SOLUTION 1

The first design solution shall have four frames; one on each end and two spaced 12 meters apart in the centre. Each frame consists of a roof beam, two columns, and a support beam for the seating. The centerpiece of this design concept is the roof beam, which will be curved to represent the shape of a rugby ball. Figures 1 to 4 show the overall layout of the structure as well as details of the load paths.

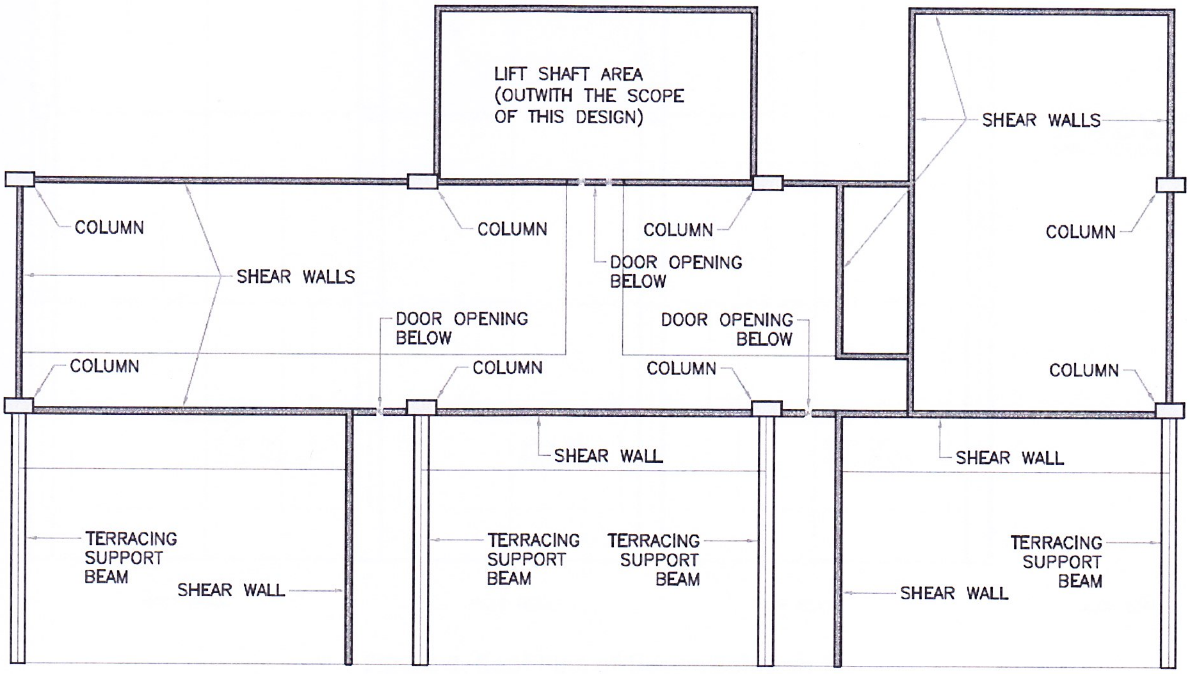


Figure 1. Ground Floor Plan of Design Solution 1.

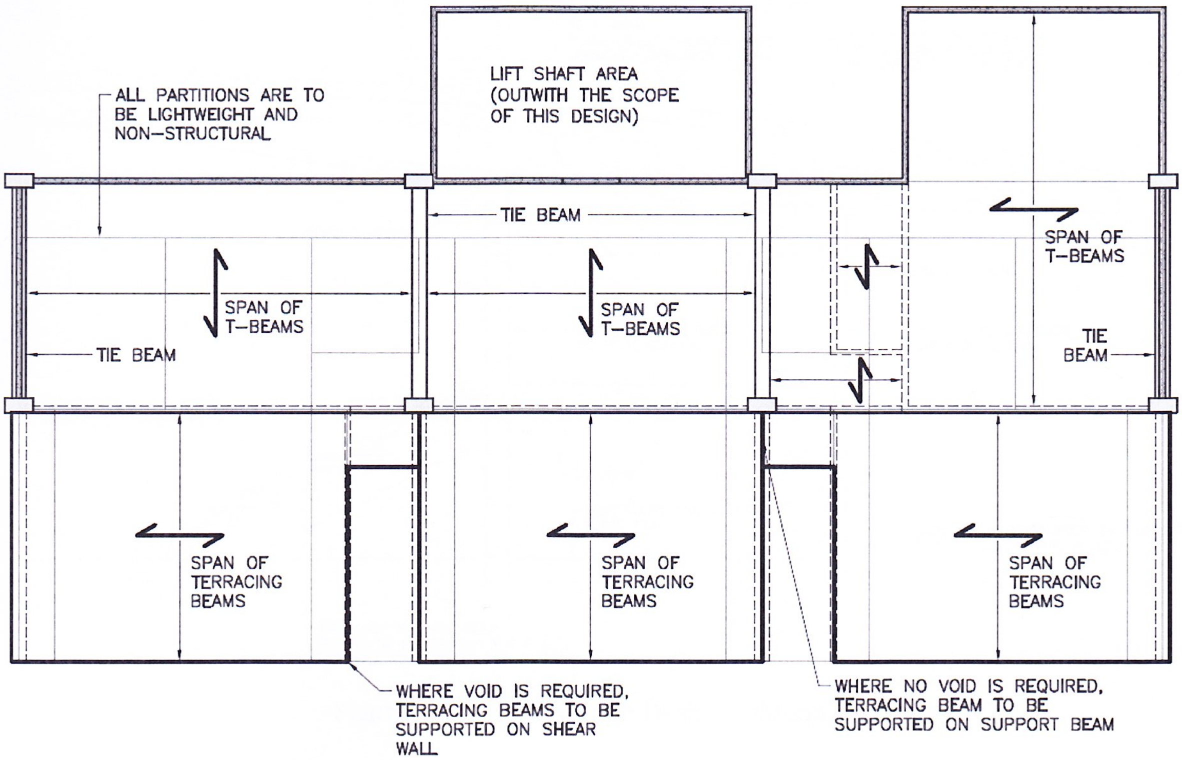


Figure 2. First Floor Plan of Design Solution 2.

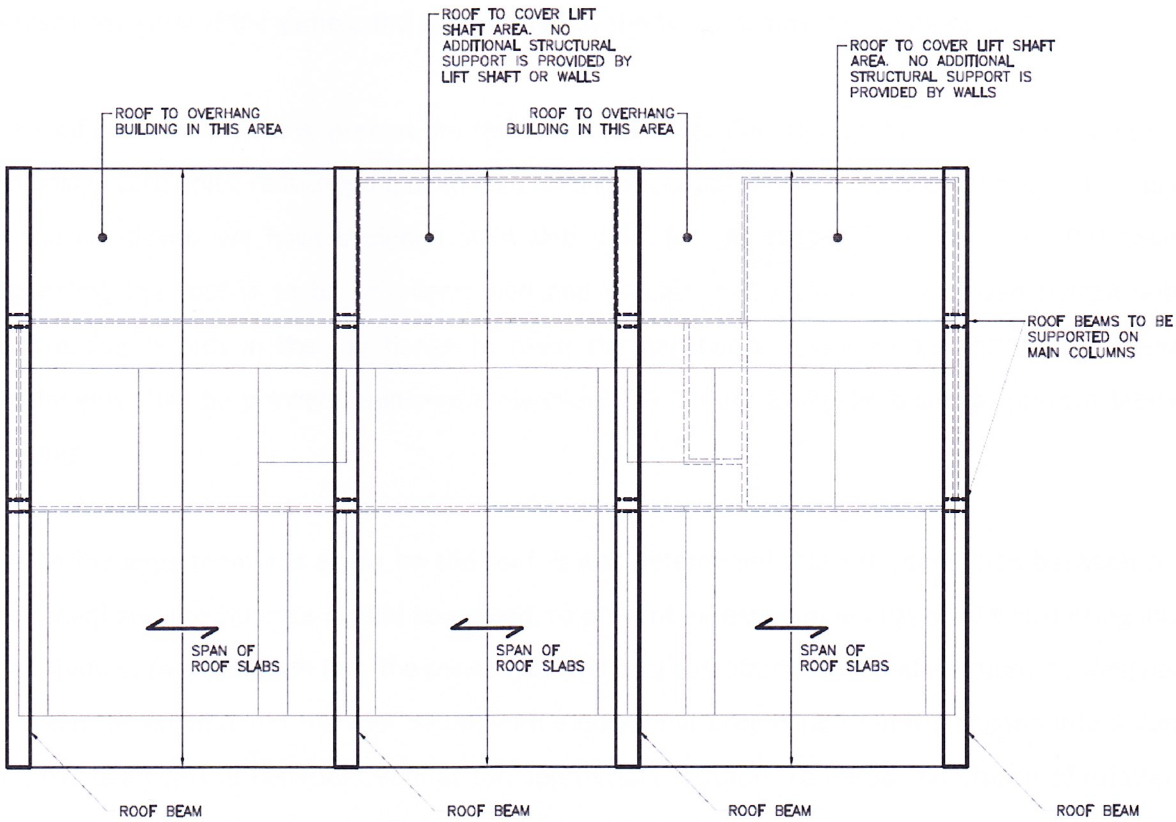


Figure 3. Roof Plan of Design Solution 1.

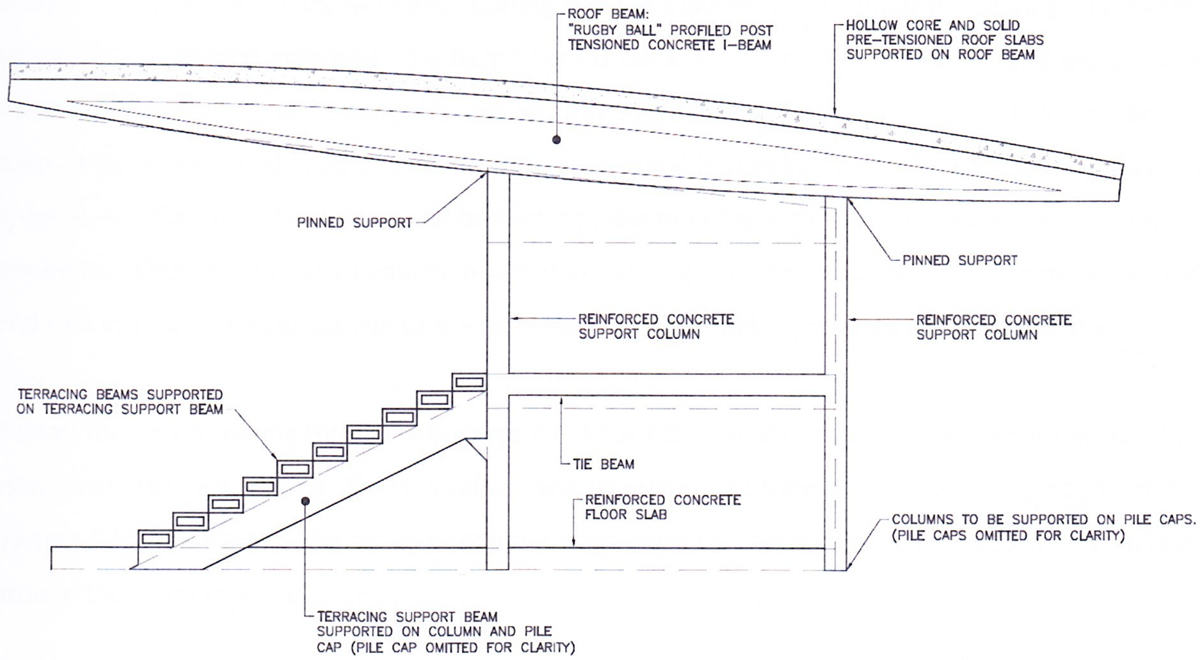


Figure 4. Section through Design Solution 1.

In this concept, the floors of the internal structure are to be supported solely by shear walls, while the frames carry the weight of the roof and terraced seating. The roof is sloped, allowing for an unrestricted view of the games, and is supported by the two columns in each frame.

The roof itself will consist of precast pre-tensioned slab units. On the front half of the roof, the slabs are hollow core units, reducing the dead load on the structure. However, to prevent overturning due to the cantilever, we have designed solid slab units for the back half of the roof. The beam supporting the roof is to be post-tensioned and precast in the shape of a curved I-beam with prestressing strands in the top flange to resist the negative moment caused by the cantilever. Diaphragms shall be provided approximately every 2.5 meters along the beam to prevent lateral buckling.

Due to the large moments acting on the roof, it was determined that the connection between the roof beam and the columns should be pinned, to prevent excessive moments from transferring into the columns. To accomplish this, the connection will be a combination of an elastomeric bearing pad between the column and the roof beam, with a dowel bar projecting from the column into a duct that has been cast in the roof beam at the appropriate location. To reduce resistance of rotation, loose materials such as sand will be placed in the duct before grouting. Design of additional reinforcement around the connection will be required as well.

In addition to the roof slabs, the tiered seating and the support beams under the seating will also be precast pre-tensioned members. The tiered seating consists of terraced box beams and the support beam is a solid beam of rectangular section, framing into the first column. The top face of the support beam will be shaped in order to provide a suitable bearing for each of the box beams. In order to achieve this, shear studs will be cast into the member which will protrude from the top of the beam. Once the precast beam is installed in place on site, the bearing sections can be formed and cast in-situ, with connectivity to the main support beam being provided via the shear studs.

Behind the tiered seating there will be large windows between the frames, allowing for unrestricted views from the box seating. These windows will be supported from a beam spanning between the frames, which will also serve as an additional diaphragm for the frames. Cladding will be provided around the framing for aesthetics.

The first floor structure shall consist of reinforced concrete beams, in the form of T-beams and L-beams, where the flange of the beams will act as the first floor slab. The first floor beams are to be supported on reinforced concrete shear walls, which will act as the inner leaf of the external wall and also act as partition walls within the building where appropriate. The reinforced concrete shear walls will be installed between each frame, in line with the columns, acting as a diaphragm in order to provide stability and shear resistance in the lateral direction. All wind loads acting on the sides of the structure will therefore be transmitted to these shear walls. As well as being provided below the first floor slabs, the shear walls will also be provided above the first floor slabs in order to act as a diaphragm. The shear walls above the first floor slabs will not be subject to any imposed vertical loading or any vertical dead loading from other members.

It is intended to then clad the external walls with precast concrete cladding as per the specification. There will be an insulated cavity between the cladding and the shear walls in order to prevent cold bridging. The precast concrete cladding units will be hung from the main columns and will transmit any wind loads onto the columns. If it is deemed necessary during detailed design, additional columns could be installed to reduce the span of the precast concrete cladding. This would have the added benefit of reducing the wind loading on the main frame columns. If additional columns are necessary, the columns would need to be strategically positioned such that additional shear walls can be provided behind them to provide resistance to the horizontal forces.

Finally, for the connection of the cladding to the columns, channel and bolt fixings will be installed. The channels are to be cast vertically into the columns. T-headed bolts will then be inserted into the channel and turned 90^0. These bolts are to protrude through the precast cladding panels with a nut and washer used to secure the cladding to the column.

DESIGN SOLUTION 2

The second design solution shall have a total of five frames, with one on each end, one in the centre, and two strategically placed within the remaining space on each side. Each frame consists of a roof beam, two columns, the seating support beam, and a horizontal beam spanning the two columns and supporting the first floor. The unique design element in this solution is the arched roof, as can be seen in Figure 5 below. Figures 6 to 9 show the overall layout of the structure along with the load paths.

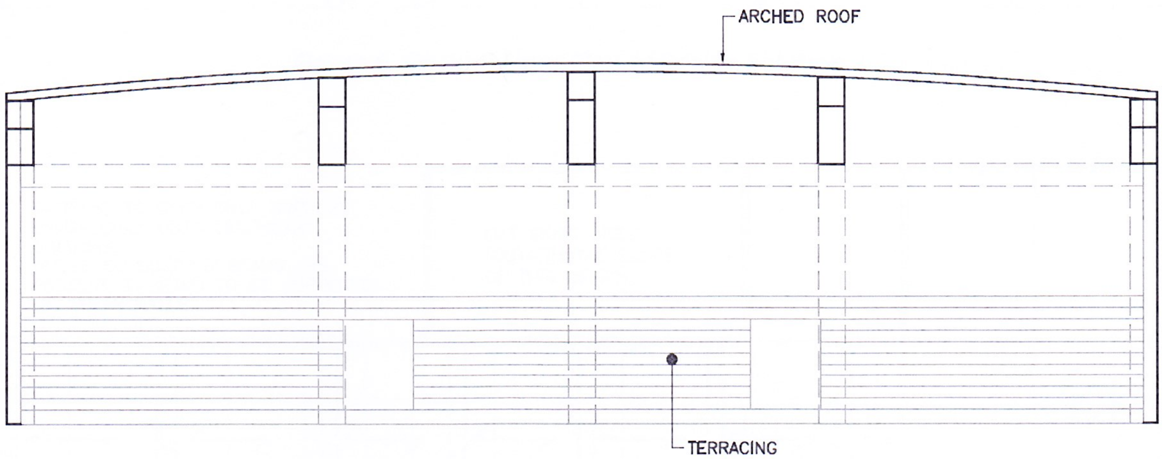


Figure 5. Front Elevation of Design Solution 2.

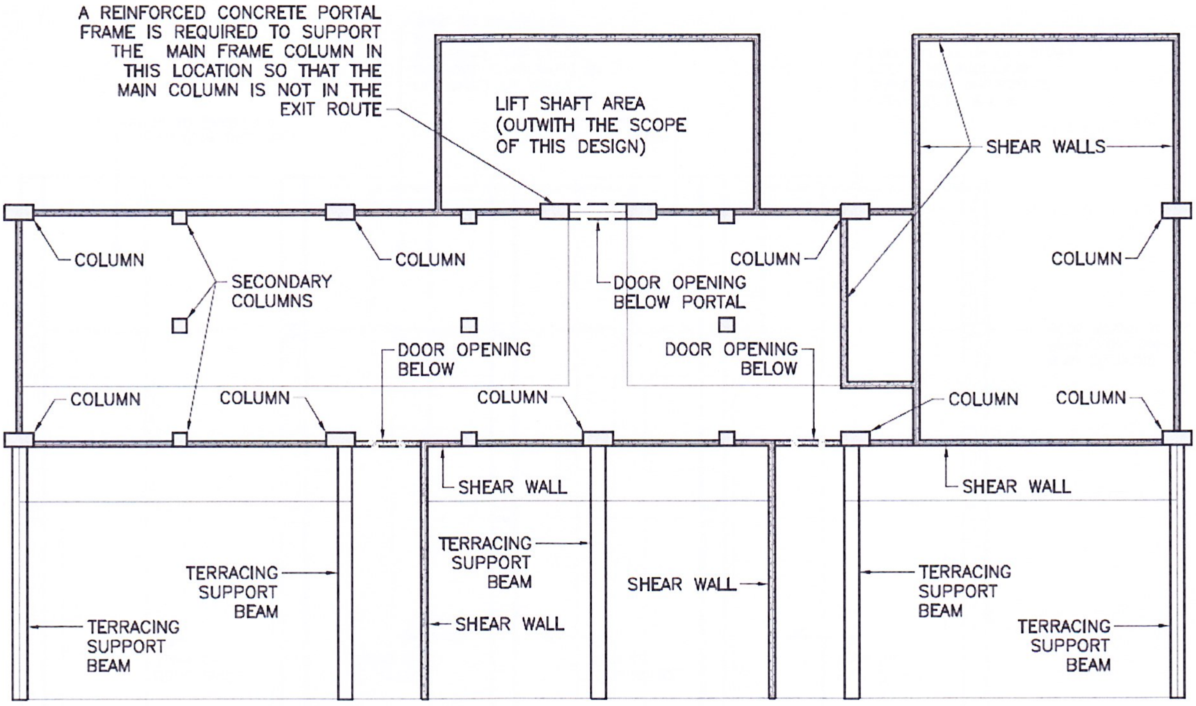


Figure 6. Ground Floor Plan of Design Solution 2.

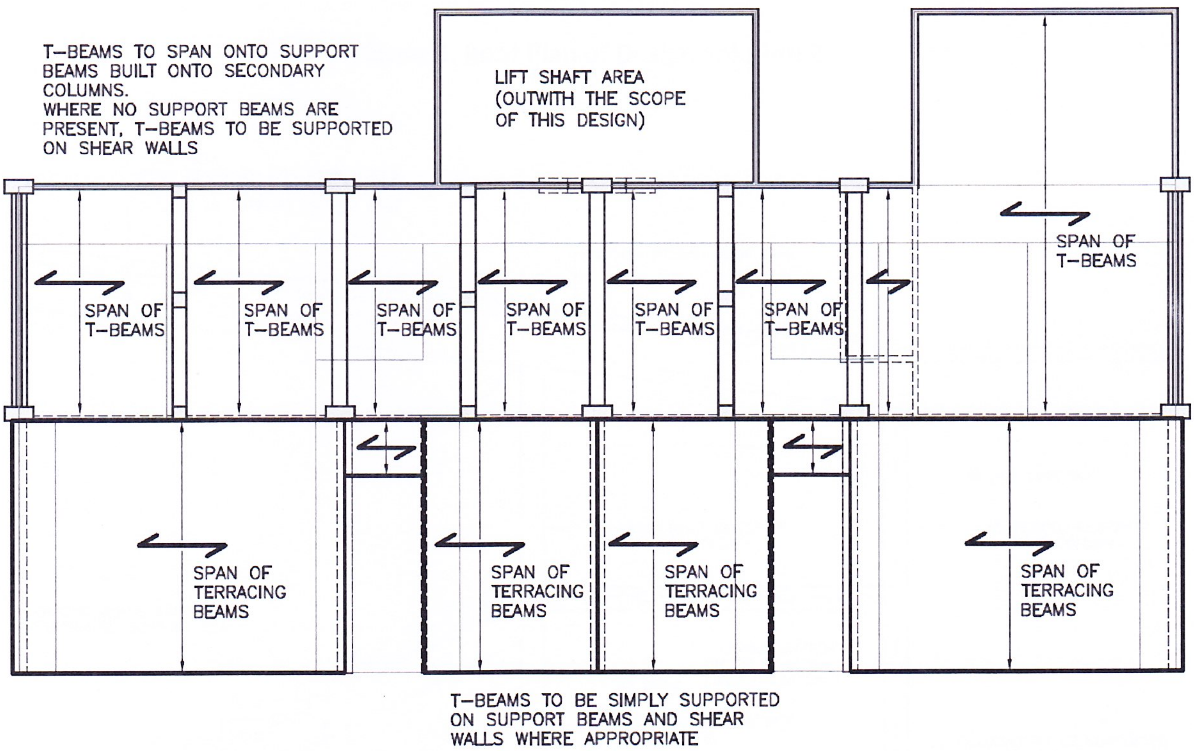


Figure 7. First Floor Plan of Design Solution 2.

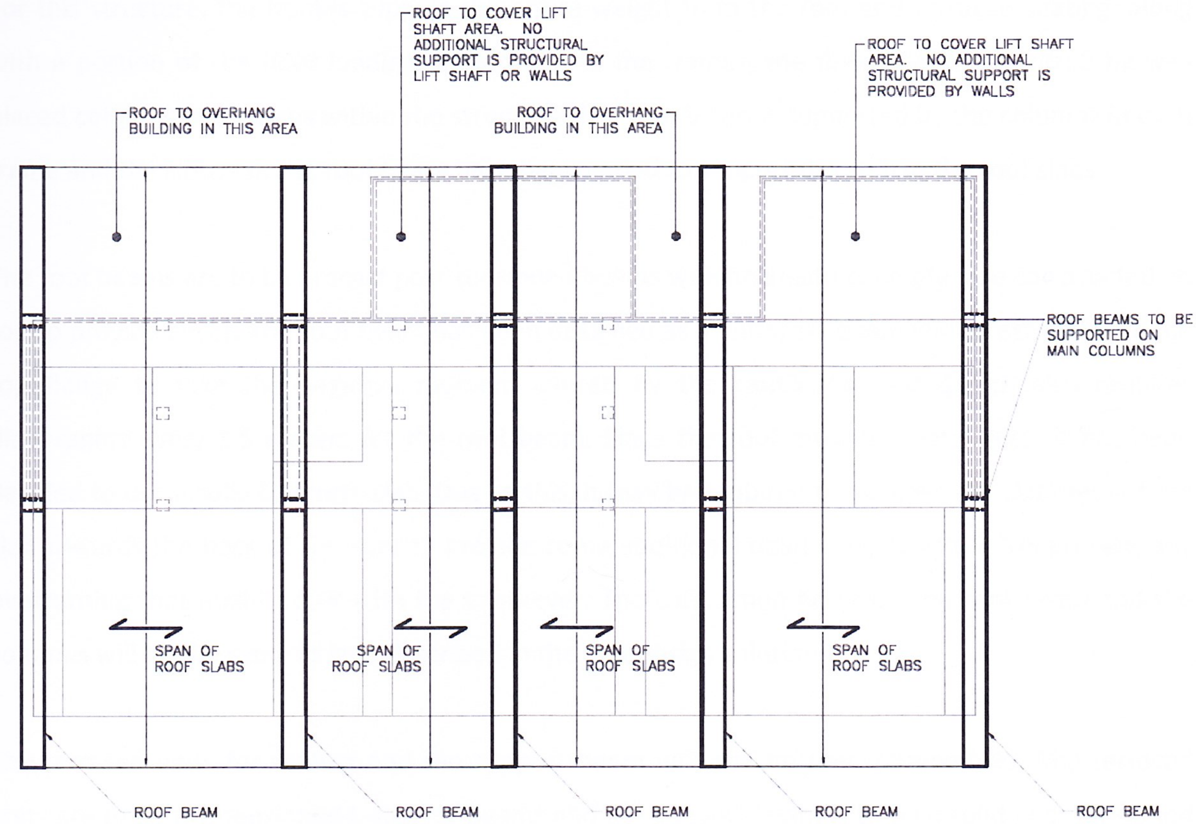


Figure 8. Roof Plan of Design Solution 2.

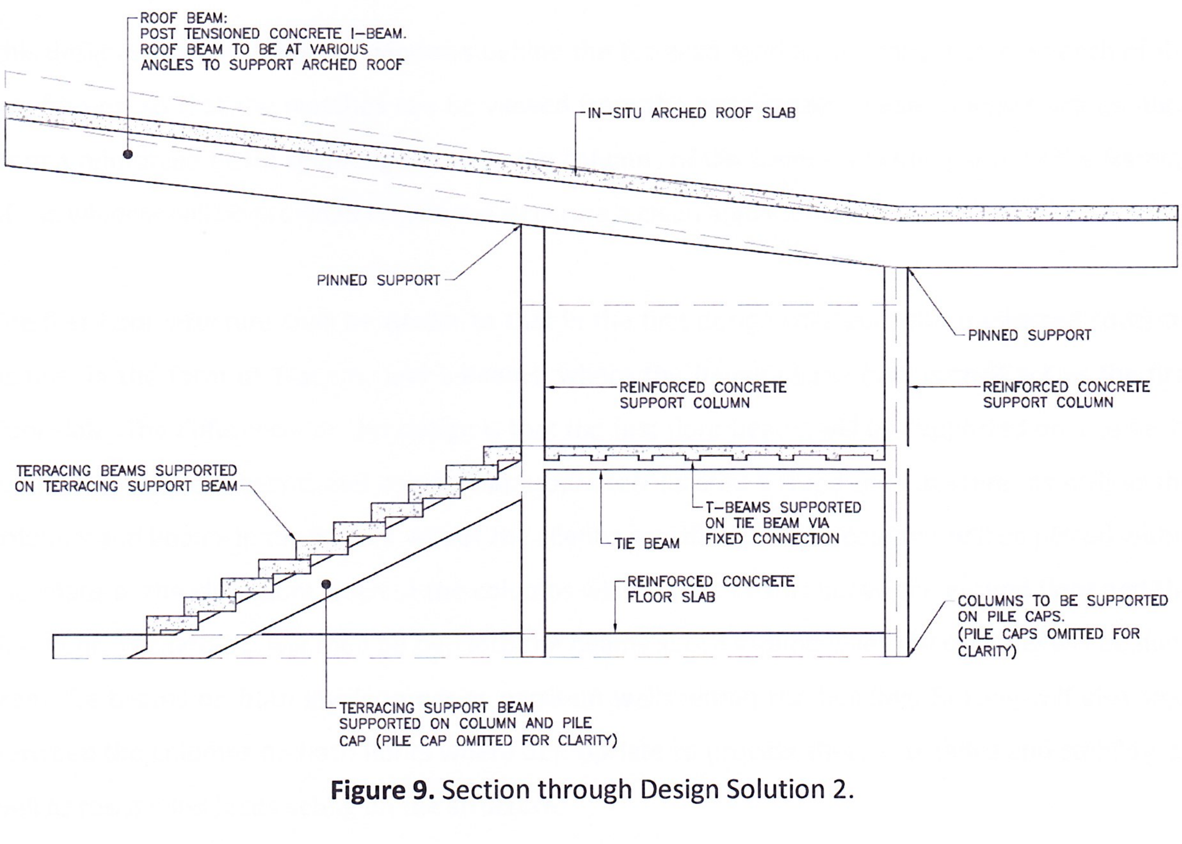


Figure 9. Section through Design Solution 2.

For this structure, the frames support the entire weight from the roof and terraced seating, along with a portion of the floor loading. In addition to the frames, the floor is also supported by well placed columns and beams within the structure. The roof beam is supported by the columns in each frame and the slopes of the roof beam have been varied to support the arch in the roof slabs.

The roof beams are to be precast post-tensioned beams with an in-situ concrete slab constructed on top to provide cover. The roof beam has been designed as a uniform I-beam with prestressing in the top flange to take the negative moment caused by the cantilever. This design also requires diaphragms every 2.5 meters for the roof beam. Since the roof must be cast in-situ, it has been decided to use a solid concrete slab. Due to this, it may be required to increase the thickness of the slab towards the back of the roof to provide some additional dead weight, which will prevent any overturning that might occur with the cantilever. The connection between the roof beam and the columns will be the same as that described in the first design solution.

The terraced units for seating and the support beam will also be precast members. The terraced units are to be pre-tensioned L-shaped beams and the support beam is to be a solid post-tensioned beam of rectangular section. This beam will frame into the first column and support the terracing in a similar fashion to that described in the first design solution.

This design will also have large windows behind the terraced seating, spanning between each of the five frames, so that the matches can be viewed from the box seating. These windows will be hung from a reinforced beam spanning between the columns of the frames. Cladding around the framing of the window will be provided to give the structure a clean appearance.

The first floor structure shall be similar to that in the first design solution, with reinforced concrete beams, in the form of T-beams and L-beams, where the flange of the beams shall act as the first floor slab. The difference for this design is that the first floor beams will be supported on a series of reinforced concrete beams and columns appropriately located around the structure, as well as the columns and beams in the frames. As per the client's specifications, no columns will be placed within the space of the club room. Each of the columns will support beams above the ground floor and the first floor. The flooring will then be tied into the beams above ground level and cladding will be hung from the beams on both levels to act as partition walls within the building. Bracing will also span between the columns on both floors where appropriate to provide shear resistance and stability, as well as resist wind loads acting on the structure.

The external walls will have precast concrete cladding as per the specification with an insulated cavity, preventing cold bridging. As in the first solution, the precast concrete cladding units will be hung from the main columns and will transmit any wind loads onto the columns. The number of columns required around the structure to support the cladding and resist the wind load will be determined in final design. The connections for the cladding are to be the same as that described in the first design solution.

ADDITIONAL DESIGN NOTES

For both designs, sustainable construction has been incorporated as much as possible, without significantly affecting the cost of the design. In the concrete, the use of silica fume or granulated blast furnace slag (ggbs) is recommended. Both are industrial byproducts that can be recycled as a cement replacement. These byproducts have beneficial effects on the concrete as well, which will be discussed further under the topic of material properties. The use of recycled concrete aggregate (RCA) was also considered, however with the high performance that will be demanded of the concrete, it was determined that its use will not be feasible for this project. However, use of RCA may be considered in the pavement designs around the grandstand.

Each design has as many precast units as possible. Since they can be made in advance, the construction phase will be much faster and simpler. A second benefit of precast members is better quality control. Production in a factory provides superior mix designs, better equipment, more accurate dimensions, and better curing since it is a controlled environment. The waste from production can often be recycled as well, in contrast with concrete cast in the field, which creates more waste. The higher quality will help to ensure long lasting use of the structure.

To extend the life span and sustainability of the structure it is also recommended to apply a silane-based water repellent or a similar product to all surfaces. This repellent is typically used for impregnating or priming reinforced concrete after the concrete has hardened, providing a dramatic reduction in the absorption of chlorides and water. It also provides resistance to alkalis and protection against frost. It is environmentally compatible and allows for good adhesion of paints as well.

EVALUATION OF THE DESIGN SOLUTIONS

There are several advantages and disadvantages for each solution that has been developed. Two main points that have been considered in selecting the most appropriate and beneficial concept are the cost and ease of construction. In comparing the costs of the two designs, it was determined that the first design solution is more economical and buildable for a number of reasons.

First, with only four frames rather than five, there are fewer concrete members to design and manufacture in Design Solution 1. This also carries through to the foundations, where only four large foundations for the support of the frames will need to be constructed. It should be noted that due to the wider spacing between the frames, the capacity and size of the members and foundation must be greater than those in the second design solution; however, this disadvantage is outweighed by the fact that there is one less frame to build and support.

One advantage that the second design solution has over the first is that the concrete members in the design are of standard shapes and sizes in contrast with the first design solution which requires the production of a large curved prestressed I-beam. The manufacturing of precast elements in the second solution will be much simpler. Nonetheless, the second solution also requires more in-situ construction with difficult formwork and temporary supports required for the arched roof, making the construction phase longer and more complicated. It has been determined that advantage of the ease of construction with more precast members in Design Solution 1 prevails over the ease of manufacture for the precast members in Design Solution 2.

A third advantage for first solution is that the design is more robust than that for the second solution. The support of the flooring is completely separate from the framing, allowing for disproportionate collapse in the unfortunate occurrence of a local failure. Additional details of this aspect will be discussed later.

Overall, it is also felt that the first solution provides a more aesthetically pleasing design, thereby encouraging more fans to attend the matches and make use of the grandstand. Based on the advantages listed here, it is recommended that the first design concept be selected. From this point on, discussions will be based on that solution.

FOUNDATION SCHEME

The foundation scheme to be adopted comprises of tubular steel piles bearing onto the chalk layer. An insitu concrete "plug" will then be poured into the pile, with continuity reinforcement within the concrete in the pile and protruding into the pile caps. A pile group with a suitable insitu cast reinforced concrete pile cap will be provided at each support location to support the main structural frame. Piles installed at an angle, known as raked piles, will also be provided in order to help withstand horizontal loads. The ground floor slab will be supported on a carpet of piles installed in a suitable grid. During construction this pile carpet will be used to support the heavier machinery needed to install raked piles and any larger piles. Piles will also be provided to support the shear walls which support the first floor structure. The shear walls will be supported on insitu concrete ground beams supported on insitu concrete pile caps on these piles. The pile groups, pile caps and ground beams will be required to take the axial load of the shear walls and first floor, as well as the moments due to the fixed connection to the shear walls.

This system has been adopted for many reasons. The made ground is not a suitable foundation material. Voids, the remains of old foundations, and contamination may be present. If contamination is present, then the made ground could be expensive and difficult to excavate and dispose of. Strip foundations on the sand layer were considered to support the shear walls, however there are a number of reasons why it would likely prove difficult to found on this layer. The high water table is one of the primary reasons, as sheet piling and dewatering may be required to excavate and provide a suitable formation. The problem of disposing of the made ground also arises. Although the clay layer may provide a suitable bearing stratum for large pad foundations on which to support the structure, the aforementioned problems of dewatering of the formation and disposal of the made ground and sand are even more relevant. Combined with the fact that this solution would involve deep excavations requiring temporary supports, founding onto the clay would clearly be uneconomical. There are also a number of health and safety concerns with deep foundations. The use of piled foundations bearing onto the chalk layer is therefore the most viable option.

The use of tubular steel piles has been selected over other piling methods due to the fact that the supports are subjected to large horizontal forces. As well as being suitable for large vertical forces, such as those experienced in this structure, steel tubular piles are able to accommodate significant horizontal loads. These piles are also suitable for being driven through any obstructions which may exist in the made ground in the form of old foundations.

The use of insitu concrete pile caps has been adopted for the main frame supports due to the fact that the pile caps should provide continuation into the piles to resist horizontal forces and applied moments. This continuation will be provided by lapping the continuity reinforcement, which is encased in the concrete plug within the pile, onto the reinforcement in the pile cap. The reinforcement and the top of the steel pile can then be encased in the insitu concrete. Large pile caps supported on many piles including raked piles will be provided to support the main frame, below each column and at the foot of the terracing area. Smaller pile caps over a smaller pile group, which may possibly include the use of raked piles, will be provided to support the ground beams which support the shear walls and floors. Ground beams and piles will also be provided perpendicular to the main pile caps to tie the structure together and help distribute horizontal forces. Insitu concrete ground beams have been selected in order to provide continuity with the pile cap reinforcement, as well as providing continuity with the shear walls above. A kicker will be cast on the top of the ground beam, with starter bars protruding, and a construction joint provided at the top of the kicker. The reinforcement for the shear walls can then be tied to the starter bars and the shuttering of the shear walls can also be clamped to the kicker. This will assist in the construction of the shear walls as well as ensuring a fixed connection between the shear walls and the ground beams.

MATERIAL PROPERTIES

Considering the required capacity of the structural members, it was decided to use C50/60 concrete. The properties of this concrete can be seen in Table 1. Due to the structures' close proximity to the sea, it is recommended to include a cementitious material such as silica fume or ground granulated blastfurnace slag (ggbs) in the mix design. Both ingredients reduce the permeability of the concrete, thereby providing greater resistance to the penetration of chloride ions from the salt water. A superplasticizer should be used in the concrete mix as well in order to achieve a low water/cement ratio, which will reduce the permeability of the concrete and therefore increase its durability.

Table 1. Concrete Properties from EN 1992-1-1 Table 3.1

C50/60			
f_{ck} (MPa)	50	f_{cm} (MPa)	58
$f_{ck,cube}$ (MPa)	60	f_{ctm} (MPa)	4.1
E_{cm} (GPa)	37		

For the prestressing steel in the post-tensioned roof beam and pre-tensioned terracing support beam Y1820S7G strands with a 15.2mm diameter are to be used. For the terraced box beams, Y1860S7 strands with a 16mm diameter have been chosen. And for the roof slabs, it was decided to use Y1860S7 strands with a 12.5mm diameter. The properties of these can be seen in Table 2. A 50mm duct has been assumed for the post-tensioned roof beam, with 4 strands in each duct.

Table 2. Properties of Prestressing Strands from BS 5896:1980 Table 6 and EN 10138-3

Y1820S7G Strands					
ϕ_p (mm)	15.2	f_{pk} (MPa)	1820	Ultimate Prestress (kN)	300
A_p (mm^2)	165	$f_{p0.1k}$ (MPa)	1560	Initial Prestress (kN)	219
Y1860S7 Strands					
ϕ_p (mm)	12.5	f_{pk} (MPa)	1860	Ultimate Prestress (kN)	173
A_p (mm^2)	93	$f_{p0.1k}$ (MPa)	1600	Initial Prestress (kN)	126
Y1860S7 Strands					
ϕ_p (mm)	16	f_{pk} (MPa)	1860	Ultimate Prestress (kN)	279
A_p (mm^2)	150	$f_{p0.1k}$ (MPa)	1600	Initial Prestress (kN)	204

For the reinforcing steel, a strength class of B500B was chosen. Table 3 presents the properties for this steel class that have been used in design.

Table 3. Properties of Reinforcement from BS 4449:2005 Table 4

Grade: B500B			
Yield Strength	f_{yk}	500	MPa
Modulus of Elasticity	E_s	200	GPa
Elongation at max force	e_{uk}	5.0	%

METHOD STATEMENT FOR SAFE CONSTRUCTION

- The existing facility is to be demolished in a safe manner, the site cleared, and the ground is to be made level and free from obstructions or excavations prior to any construction works taking place.
- Full PPE must be worn at all times including high visibility clothing, hard hats, protective footwear and any additional PPE required for specific tasks, such as respirator masks and gloves.

- Any excavation, including the lift pit, shall be supported or battered to a safe slope, with barriers erected to prevent pedestrians from accessing the area.
- Temporary scaffolding shall be provided at all areas where working at height is required. The scaffolding should be designed adequately and the installation carried out by a professionally approved scaffolder. All scaffolding installed should then be checked by the approved scaffolder and clear signage and fencing put in place to ensure that no person may access any scaffolding before it is complete. Similarly, signage must be installed to show which areas of scaffolding have been approved as being complete.
- Once the site is cleared, the piling carpet should be installed prior to any heavy machinery or stored materials arriving on site. A separate piling carpet shall be installed initially, and a concrete slab cast as soon as possible, in order to take the loading of a large crane which will be required later in the construction.
- The piling carpet shall provide a safe working area for machinery. A banksman shall be present to direct each machine operator as to where safe working areas are. The banksman will also ensure that a safe distance is kept between the plant machinery and people on the site.
- Once the carpet is in place and the piles are installed, pile testing of a number of piles will be carried out in order to ensure that the safe working load for the piles has been achieved.
- The continuation reinforcement should be installed into the top of each of the piles and insitu concrete poured to provide a plug in the top of the pile. The insitu pile caps should then be excavated, the reinforcement tied, and the concrete poured.
- The in-situ ground beams, columns, and tie beams can then be constructed once the pile caps have cured to sufficient capacity. Temporary bracing and support structure should be provided to the columns and tie beams during pouring, curing, and until the strength of the frame is sufficient to withstand any applied loading.
- The ground floor shear walls will be constructed once the ground beams have cured to sufficient capacity. The order of construction of the shear walls may be amended in order to allow plant to access to building.
- The lift pit shall be excavated in a safe manner as directed above, and the lift shaft base and walls constructed.
- The ground floor slabs shall be constructed in bays, on a "hit and miss" basis, in order to facilitate differential movement of the individual slabs. Once the initial pour (the "hit") has cured sufficiently, the "miss" areas of slab can be poured, and suitable movement joints

provided between. Any waterproofing and membranes shall be included in the ground floor construction as appropriate.

- Once the columns have cured sufficiently, as well as the pile caps, the precast pre-tensioned support beams for the tiered seating can then be installed. The support beams shall be lifted into position by a crane and each will be supported by one pile cap and also a connection to a column. The bearing sections can then be constructed in situ.
- Once the shear walls, as well as the fixed connection to the ground beams, the foundations themselves, and the ground floor slab, have cured sufficiently, the in-situ first floor T-beams can be constructed. The formwork for the T-beams should be adequately designed to withstand all loads that will be applied during construction, with props to support the formwork on the ground floor slab installed. This formwork must only be removed once the first floor has cured sufficiently to withstand any applied loading itself.
- While the first floor slab is curing, the precast concrete wall cladding shall be supported on the columns. Insulation and waterproofing shall be applied between the cladding and the shear walls as specified in detailed design.
- Once the bearing sections to support the terracing have achieved the required structural capacity, the precast pre-tensioned terracing beams can be installed. These shall be lifted in by crane and supported by the bearing sections.
- The cladding and any glazed panels can then be installed behind the terracing and in front of the boxes. The seating can then be installed into the terracing beams.
- The precast post-tensioned roof beams can then be installed. These shall be lifted into place by crane and supported by the main columns.
- The precast pre-tensioned hollow core and solid roof slabs shall then be installed. These will be lifted into place by crane and supported on the roof slabs. **The order of installation of the roof slabs is important.** The roof slabs should be installed in a particular order such that the cantilever is not over-loaded before there is suitable dead load on the stabilizing span. Similarly too much loading onto the middle span may produce an applied sagging moment for which the roof beam is not designed. Therefore further calculations should be provided and a clear schedule of erection should be prepared and followed to ensure that the slabs are erected in the correct order. *An alternative that may be proposed by the contractor may be to provide additional temporary supports to the roof beams in order to ensure stability and to reduce any sagging moments applied to the beam.*

- After these stages the building shall be wind and watertight, ready for any plumbing, electrical, plant and furniture installation to take place, as well as any soft furnishings or finishing.
- The order of construction of the insitu pile caps, ground beams, shear walls, columns, tie beams and first floor slabs may be programmed in order to speed up construction. For example, the ground beams at one side of the building may be cured sufficiently to take loading while the ground beams at the opposite side of the building are just being poured. In this case the construction of the shear walls could commence at one end of the building simultaneously with the construction of the ground beams at the other. This example shows how the construction of all of the in-situ members can be programmed efficiently in order to reduce the time of construction of the building.

STATEMENT REGARDING ROBUSTNESS IN THE DESIGN

By definition, a sufficiently robust design is one that can withstand arbitrary damage, such as the removal of any member in the design. For this structure, the removal of some members, such as the roof beam would cause damage, however the structure is designed so that this damage will be kept to a minimum with only local failures. By designing higher capacity members or adding additional members throughout the structure, it may be possible to make the design more robust, however it was determined that the cost of this would be significant, and based on a risk assessment the current design is sufficient enough.

The current design is such that each of the three sections could stand on its own, thereby avoiding disproportionate collapse. Simple spans have been assumed between each of the frames. Therefore, in the unfortunate event of a local failure, only one section of the structure will be affected. The structure is also designed such that the floors of the building will be supported only by the shear walls in the structure and the roof and terracing will be supported entirely by the four frames. Therefore, if a failure occurs with the floors, it will not affect the roof, terracing, and frames. Similarly, if a failure occurs with the terracing, it will not affect the flooring and shear walls of the main structure. While it is unavoidable that a failure in the roof would affect the terracing and/or the floor slabs underneath, the design is such that this failure would be kept localized in one section of the structure.

In addition to creating a robust design that will prevent the total collapse of the structure, precautions have also been taken in the design to ensure that even local failures will not occur. Efforts have been made to protect the structure from adverse environmental conditions, with the recommended use of silica fume or ggbs in the cement, as well as application of a sealant on all surfaces of the concrete as previously mentioned. There is also a plan for the maintenance and sustainability of the structure in Appendix 3. And accessibility to all main structural elements and their connections for inspection will also be ensured in the final design so that a thorough inspection and maintenance plan can be put in place to monitor and preserve the condition of the structure.

LETTER REGARDING EXTENSION OF THE ROOF

To the Southfield Rugby Club Owner:

This letter is in response to your request for the implications of extending the roof by one meter. We have completed a brief investigation, and have determined that extending the roof would be possible. However, it would require extensive additional design, and would increase the cost significantly. The following modifications would be required:

- Additional dead load on the back of the roof, to help prevent uplift due to the extension on the front. This can be achieved by designing a roof slab with a greater depth or extending the length of the roof in the back as well.
- Reanalysis and redesign of the curved roof beam due to increased length and loading.
- Reanalysis and redesign of the columns and foundations due to increased loading on the structure.

We have concluded that extending the roof by one meter is possible; however it will require the modification of several of main elements in the design. These changes will most likely raise the cost of the Grandstand significantly.

Respectfully yours,

11-001 Consultants

APPENDIX 1: VERIFICATION OF STRUCTURAL VIABILITY

The following pages provide calculations, verifying the viability of our chosen design. For each member type, the worst case was designed for. These calculations confirm that the design is possible; however, since this is only a design concept it should be noted that additional calculations will be required for the final design.

Due to the many load combinations which need to be calculated in a building of this type, it was decided that the use of computer modelling software was the most appropriate and efficient method for determining the maximum loading on each member. CSC S-Frame software was chosen due to the ability to model various combinations of member loading in order to calculate the worst case design. The methodology used was to model the main frame in two dimensions. Member sizes were assumed initially and were re-modelled through an iterative process in order to provide the correct stiffness properties for each member. Each of the members was set to have a dead load of zero, so that the dead load could be applied separately. This served two purposes; the first was to allow the dead loading to be factored by the appropriate factors for beneficial and non-beneficial loading, and the second was to allow the curved roof beam loading to be applied accurately. In order to determine the most onerous load combination, different load cases were applied to each span of each member. One of these load cases assumed that the dead load was favourable, therefore it was factored down in accordance with BS EN 1990:2002 Table A1.2(A) Note 1. The unfavourable load case assumed fully factored variable and permanent loading in accordance with BS EN 1990:2002 Table A1.2(A) Note 2. The design loads for each case were calculated in accordance with BS EN 1990:2002 Equations (6.10), (6.10a) and (6.10b). Every feasible combination of each span of each member subject to either unfavourable or favourable loading was then modelled. In conjunction with the favourable load applied to the roof beams, uplift was applied to the individual spans, whereas downward wind loading was applied to the spans subject to unfavourable loading. This method was conservative and ensured that the roof will be stable with regards to equilibrium, and also structurally able to withstand an unlikely event (i.e. full downwards wind and variable load applied to certain spans, and full uplift and reduced permanent load applied to other spans). Under no load combination is the roof subject to a positive (sagging) moment; therefore no reinforcement to take positive moments is required.

This process made it clear that for each member the critical load case was different, i.e. the worst case for the roof beam was not necessarily the worst case for the columns. The computer data was then collated and the members were individually designed using their respective worst case loading. An approximate hand calculation was then completed of one load combination on the roof beam in order to verify the data in terms of axial loads on the columns and bending moment forces on the roof beam. To facilitate simpler calculations, the roof beam was assumed to be straight rather than curved. Although the computer model is far more accurate due to more accurate loading we feel that this hand calculation demonstrates that the computer model is correct and that the correct methodology has been applied in its construction. Due to the limitations of space within the report it has been deemed appropriate to provide the worst case bending moments on the roof from the model.

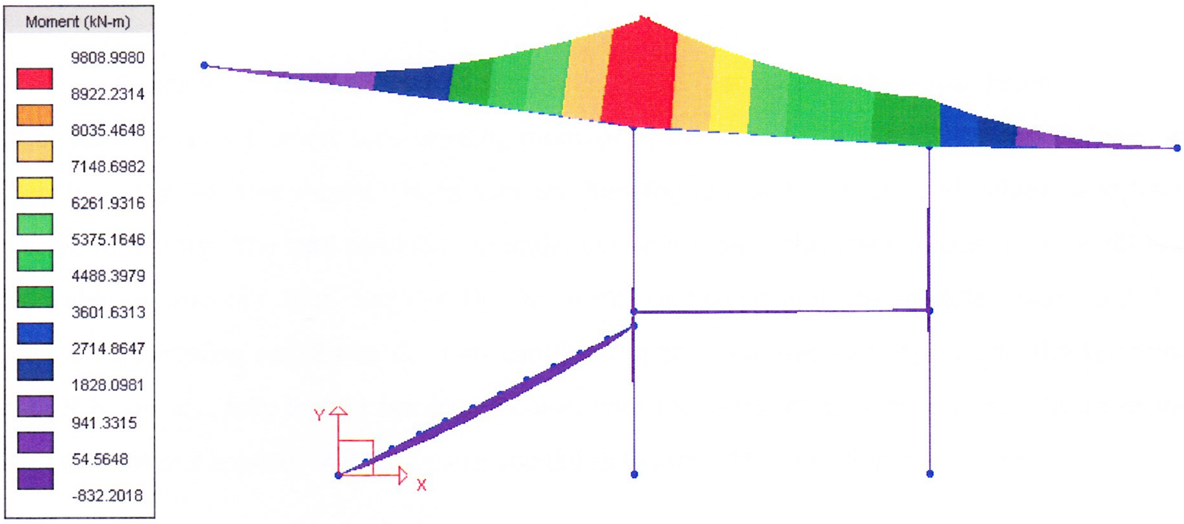


Figure A1.1. Worst case bending moment at the front support of the roof beam.

Figure A1.1 shows the worst case Bending Moment at the support to the front of the building as taken from the S-Frame model. The worst case bending moment within the terracing support beam is also shown. Note that the bending moment diagram and values have been inverted for clarity. The load combination under consideration in this diagram consisted of each span of the roof beam subject to fully factored unfavourable variable (imposed) load and permanent (dead) load, applied to one of the central frames. The load case applied to the terracing support beam was fully loaded terracing beams, with the worst case loading as per the terracing beam calculations applied. Wind was also applied to the right of the building in this case.

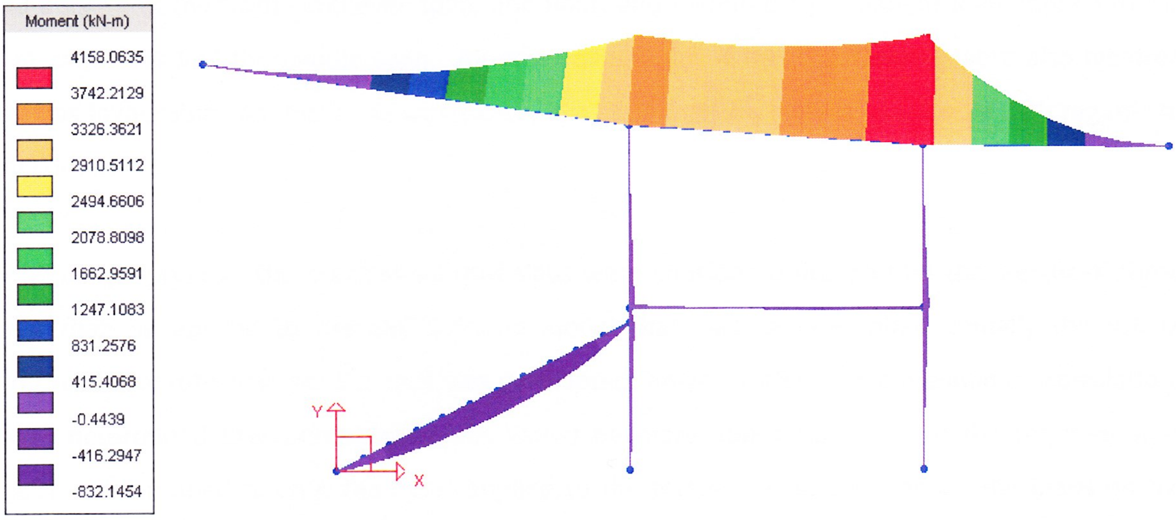


Figure A1.2. Worst case bending moment at the rear support of the roof beam.

Figure A1.2 shows the worst case bending moment at the support to the rear of the building, as taken from the S-Frame model. Note that the bending moment diagram and values have been inverted for clarity. The load combination under consideration in this diagram consists of uplift and favourable permanent load applied to the front cantilever and the middle span, and full unfavourable loading applied to the rear cantilever span. The load case applied to the terracing support beam was fully loaded terracing beams, with the worst case loading as per the terracing beam calculations applied. Wind was also applied to the left of the building in this case.

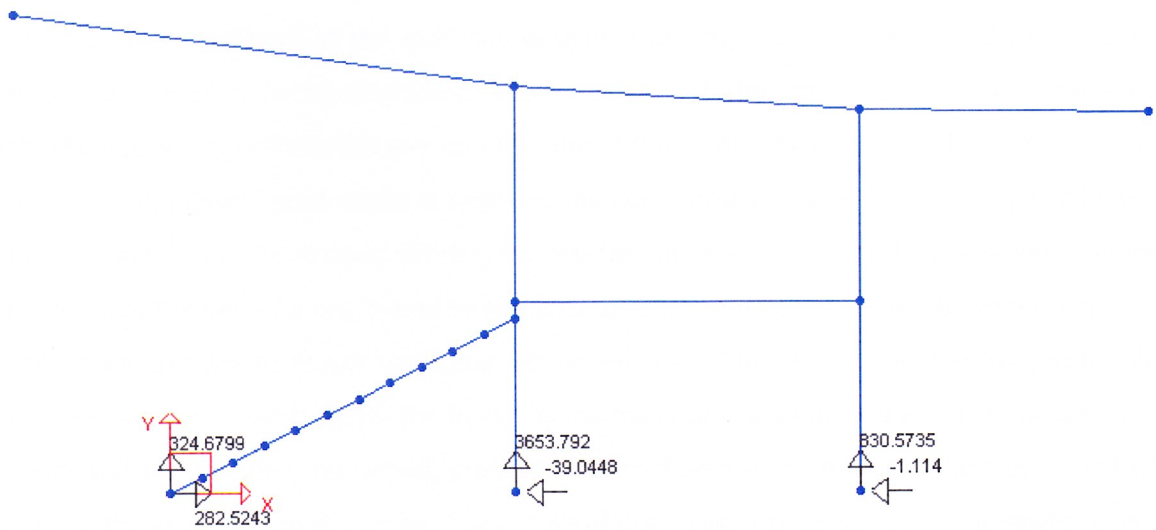


Figure A1.3. Worst case for column reactions.

The structure was also checked in each combination that the forces at the base were all downwards, i.e. there is an upwards reaction at the base. Figure A1.3 shows the worst case, in terms of the smallest downward force in either of the columns. In this case the reaction is at the rear column. The load combination under consideration in Figure A1.3 includes uplift and favourable permanent load applied to the front cantilever span, and uplift and favourable permanent load applied to the front cantilever and the middle span. All other dead loads within the building were also factored down as favourable. As this is the worst case, it is clear that the building is suitable with regards to equilibrium.

In the design process, the prestressed roof slabs were considered first so that the weight of these could then be applied to the CSC S-Frame model that was just described. Initially the use of reinforced concrete slabs for the roof was considered, however after some preliminary calculations it was determined that prestressed slabs would be more appropriate due to the required span length and the need to limit the loads applied to the frames. To further reduce the loads on the structure, it was also decided to use hollow-core slabs on the front half of the roof. These were designed assuming a small I-Beam with the flange width equal to the spacing of the hollow cores. The final result was a pre-tensioned hollow core slab with a 360 mm depth and 170 mm spacing between the cores. To prevent overturning of the structure due to the cantilever roof, a pre-tensioned solid slab was designed for the back of the roof to add some dead load. Some additional prestressing strands had to be added to support the extra weight; however, it is designed to the same depth as the hollow core depths, so the aesthetics of the structure are not affected. Details of these designs can be seen in the following pages.

Following the completion of the roof slab designs, the design of the prestressed roof beam was completed. This beam is the centrepiece of our design, with the shape representing that of a rugby ball. The depth ranges from 500 mm to 1770 mm at the centre. As the results from the load model show, the roof beam must resist a negative moment throughout its length. This is due to the cantilever in the roof. To account for this, the prestressing is in the top of the beam, with the flange on the top of the beam having twice the depth as the bottom flange. Due to the curved shape of the beam, it was decided to design it as a post-tensioned beam. This allows more flexibility in the shape of the prestressing strands within the beam, as the ducts can easily be cast in curved positions. Four strands will be threaded, tensioned, grouted, and anchored in each duct, requiring a total of 12 ducts for the 48 strands in the design. The details of this design can be seen in the following pages.

In addition to the roof, the tiered seating and support beam have also been designed with prestressing. The tiered seating has been designed as prestressed for similar reasons to those of the prestressed roof slabs. The result is a series of 400mm x 800mm pre-tensioned box beams stacked on top of the 750mm x 500mm pre-tensioned solid support beam. The support beam has been designed for flexure; however due to it's positioning in the frame, it should also be checked for axial compression in the final design.

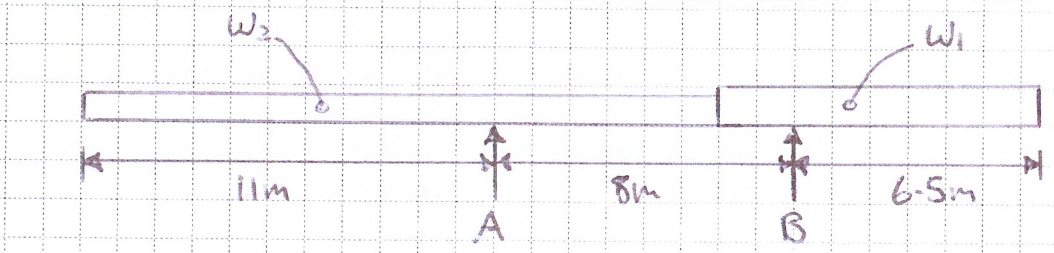
For all of the prestressed concrete designs, strength requirements as well as any limits set by the Eurocodes have been met in relation to durability and fire resistance, beam sizing, prestressing force and eccentricity, concrete stress limits, ultimate moment resistance, and deflection limits. For these checks, some initial assumptions were made on the prestress losses. In the final design, detailed calculations of prestress losses should be made and checks on final stresses should be performed. For several of the members, additional sections should be checked in the final design as well, to determine if and where debonding of the strands is required. Also, future consideration should be given to shear reinforcement design, end block design, and any additional un-tensioned reinforcement required. Detailed calculations for the diaphragms in the roof and full design of the connections will be required as well.

The main columns have been designed using the worst case loads taken from the S-Frame Model. The worst case loading has been assumed as being the highest axial vertical load to occur to a column in conjunction with the bending moment applied to the column during the same load case. The columns have all been designed on this basis. The columns are to be 500mm deep x 1000mm wide and of reinforced concrete. There is to be a tie beam between the columns which will be at first floor level. This beam has not been designed during conceptual design and therefore must be considered during detailed design. Due to axial forces it may required for the tie beam to be prestressed. Due to the difficulty of connecting a prestressed beam to insitu columns other bracing arrangements may be considered at a later date, which are beyond the scope of this report.

The first floor will be supported on in situ concrete shear walls. In the design of the shear walls, we have provided the required capacity to resist the vertical shear loads from the floor. In addition to these loads, wind loads will act horizontally, however it has been determined that a more detailed design of these walls will be done in the full design. We believe that the shear walls will have the capacity to take the wind loads, however if this turns out not to be the case, other options may be considered such as a combination of columns and horizontal beams to support the floor loads and wind loads. The shear walls have been designed in order to accommodate the worst case floor load, which occurs beneath the office, due to the 9m span above the club room. The shear walls have been designed as 200mm thick reinforced concrete, which are to be fully fixed to the ground beams.

The in-situ floors, which shall span onto the shear walls, have been designed for the worst case, which occurs in the office and nearby boxes due to the 9m span above the club room. The floors have been designed as T-Beams, with the top flange acting as the first floor slab. The T-beams have been designed as being 500mm deep overall, with a 300mm wide web. The flange depth is to be 250mm and the spacing of the webs is to be 1000mm. The T-beam has been designed with regards to bending and shear and assumes that the beams are to be simply supported on the shear walls. During detailed design the top flange will be required to be designed with regard to spanning between the webs, and also shear at the web-flange interface.

CHECK ROOF BEAM BENDING MOMENTS



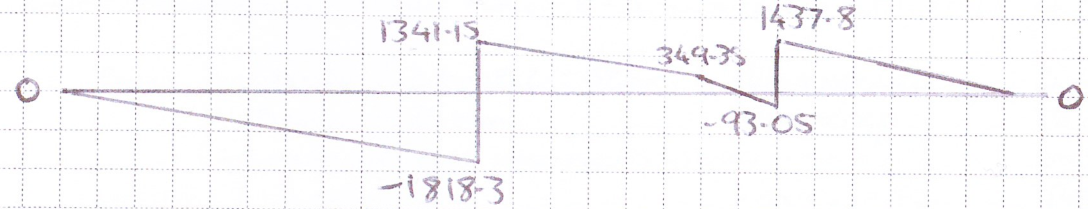
Determine w_1 : Slab loading $= 14.54 \times 13 = 189$ kN/m
Beam loading $= (1.77 \times 0.2) + (0.8 \times 0.5) + (0.8 \times 0.25)$
$\times 25 \times 1.35$
$= 32.2$ kN/m
$\therefore w_1 = 221.2$ kN/m

Determine w_2 : Slab loading $= 1.7411 \times \frac{1000}{170} \times 13 = 133.14$
Beam loading $= 32.2$ kN/m
$\therefore w_2 = 165.3$ kN/m

Take moments about A: $\Sigma CWM = \Sigma ACWM$
$\Rightarrow [(8.5 \times 221.2) \times 10.25] + [(6 \times 165.3) \cdot 3] = [(11 \times 165.3) \times 5.5)] + 8B$

$\Rightarrow B = 1530.85$
$\therefore A = (8.5 \times 221.2) + (6 \times 165.3) + (11 \times 165.3) - B$
$= 3159.45$ kN

Shear force diagram: (kN)



Bending moment = Area under Shear force diagram

B.M.@A $= 1818.3 \times 11/2 = 10000.7$ kNm (-ve moment)

B.M.@B $= 1437.8 \times 6.5/2 = 4672.9$ kNm (-ve moment)

These bending moments are slightly higher than those determined using S-Frame, however the full height of the beam has been assumed in this calculation, which is a larger load than applied to the cantilever spans in S-Frame. This has been taken as sufficient evidence to verify the computer (S-Frame) Model.

DURABILITY REQUIREMENTS

THE FOLLOWING CALCULATIONS COVER ALL MEMBERS IN THE DESIGN

DESIGN WORKING LIFE = 50 YEARS (CATEGORY 4) — BS EN 1992-1-1 TABLE NA.2.1

EXTERNAL MEMBERS: EXPOSURE CLASS = XS1 (BY THE SEA) — TABLE 4.1
STRUCTURAL CLASS = S3 (-1 FOR > C40/50) — TABLE 4.3N

TO SATISFY BOND AND ENVIRONMENTAL REQUIREMENTS:
$C_{min} = \max[C_{min,b};\ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add};\ 10mm]$ — CL. 4.4.1.2(2)

$C_{min,b} = 2.5 \times \phi_p = 2.5 \times 16mm = 40mm$ PRE-TENSIONED — CL. 4.4.1.2(3)
$= \phi_{DUCTS} = 50\ mm$ POST-TENSIONED
$= \phi_{BAR} = 25\ mm$ REINFORCED — N.A. RECOMMENDS:
(ASSUMED MAXIMUM DIMENSIONS FOR EACH TYPE) — BS 8500-1
$C_{min,dur} = 30mm$ APPLIES TO ALL — TABLE A.4
BS EN 1992-1-1
$\Delta C_{dur,\gamma} = 0$ $\Delta C_{dur,st} = 0$ $\Delta C_{dur,add} = 0$ — TABLE NA.1

$C_{min} = \max[40;\ 30;\ 10] = 40mm$ PRETENSIONED
$= \max[50;\ 30;\ 10] = 50mm$ POST-TENSIONED
$= \max[25;\ 30;\ 10] = 30\ mm$ REINFORCED

INTERNAL MEMBERS: EXPOSURE CLASS = XC1 — TABLE 4.1
STRUCTURAL CLASS = S3 (-1 FOR > C30/37) — TABLE 4.3N

TO SATISFY BOND AND ENVIRONMENTAL REQUIREMENTS:
$C_{min} = \max[C_{min,b};\ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add};\ 10mm]$ — CL. 4.4.1.2(2)

$C_{min,b} = 2.5 \times \phi_p = 2.5 \times 15.2 = 38mm$ PRE-TENSIONED — CL. 4.4.1.2(3)
$= \phi_{BAR} = 25mm$ REINFORCED — NA RECOMMENDS:
(ASSUMED MAXIMUM DIMENSIONS FOR EACH TYPE) — BS 8500-1
$C_{min,dur} = 15\ mm$ APPLIES TO ALL — TABLE A.4
BS EN 1992-1-1
$\Delta C_{dur,\gamma} = 0$ $\Delta C_{dur,st} = 0$ $\Delta C_{dur,add} = 0$ — TABLE NA.1

$C_{min} = \max[38;\ 15;\ 10] = 38mm$ PRE-TENSIONED
$= \max[25;\ 15;\ 10] = 25mm$ REINFORCED

FOR THE SELECTION OF VALUES FOR $C_{min,dur}$ IN BS-8500-1 WE HAVE DETERMINED THAT THE USE OF TABLE A.4 RATHER THAN THE RECOMMENDED TABLE A.5 IS APPROPRIATE, GIVEN THAT THE DESIGN LIFE OF OUR STRUCTURE IS ONLY 50 YEARS.

TO SATISFY REQUIREMENTS FOR FIRE RESISTANCE: REI = 90min.

$C_{min,fire} = a - \phi_s/2$

BS EN 1992-1-2 TABLE 5.8

PRE-TENSIONED ROOF SLABS: $h_{min} = 100mm$ (<360mm OK!)

$a = 30mm \quad \phi_s = 12.5mm$

$C_{min,fire} = 30 - 12.5/2 = 23.75mm < C_{min} = 40mm$

TABLE 5.5

PRE- & POST-TENSIONED BEAMS: $b_{min} = 200mm$

$a = 45mm \quad \phi_s = 15.2mm$

$C_{min,fire} = 45 - 15.2/2 = 37.4mm < C_{min} = 40mm$ & 38mm

(CHOSE MOST CONSERVATIVE VALUES OF THE P/S BEAMS)

- FIRE RESISTANCE FOR REINFORCED MEMBERS WILL BE CHECKED UNDER EACH INDIVIDUAL DESIGN.
- FOR THE PRESTRESSED MEMBERS, THE BOND AND ENVIRONMENTAL REQUIREMENTS WILL CONTROL OVER FIRE RESISTANCE.

REQUIRED NOMINAL COVER

BS EN 1992-1-1 CL.4.4.1.1(2)P

$C_{nom} = C_{min} + \Delta C_{dev}$

- FABRICATION OF ALL MEMBERS WILL BE SUBJECTED TO A QUALITY ASSURANCE SYSTEM. IT MUST ALSO BE ASSURED THAT ACCURATE MEASUREMENT DEVICES ARE USED IN THE MANUFACTURING OF PRECAST ELEMENTS.
  THEREFORE, IN ACCORDANCE WITH BS EN 1992-1-1 CL. 4.4.1.3(3), THE FOLLOWING REDUCED VALUES OF ΔC_{dev} WILL BE ASSUMED:

IN-SITU CONCRETE: $\Delta C_{dev} = 5mm$

PRECAST ELEMENTS: $\Delta C_{dev} = 0mm$

$C_{nom} = 40mm + 0mm = 40mm$ PRE-TENSIONED EXTERNAL MEMBERS

$C_{nom} = 50mm + 0mm = 50mm$ POST-TENSIONED EXTERNAL MEMBERS

$C_{nom} = 30mm + 5mm = 35mm$ REINFORCED EXTERNAL MEMBERS

$C_{nom} = 38mm + 0mm = 38mm$ PRE-TENSIONED INTERNAL MEMBERS

$C_{nom} = 25mm + 5mm = 30mm$ REINFORCED INTERNAL MEMBERS

PRESTRESSED CONCRETE DESIGN

PROPERTIES FOR ALL PRESTRESSED MEMBERS:

PRESTRESS LOSS: $\alpha = 1-0.10 = 0.90$ $\beta = 1-0.25 = 0.75$ BS EN 1992-1-1

C50/60 CONCRETE: $f_{ck} = 50$ MPa $f_{cm} = 58$ MPa TABLE 3.1

$f_{ck,cube} = 60$ MPa $f_{ctm} = 4.1$ MPa

$E_{cm} = 37$ GPa

CEM 52.5R CEMENT: $s = 0.20$ CL. 3.1.2(6)

TRANSFER TIME: $t = 7$ DAYS

STRESS LIMITS AT TRANSFER:

$\beta_{cc} = e^{s\left(1-\sqrt{\frac{28}{t}}\right)} = e^{0.2\left(1-\sqrt{\frac{28}{7}}\right)} = 0.8187$ EQ (3.2)

$f_{cm}(t) = \beta_{cc} \cdot f_{cm} = 0.8187 \cdot 58 = 47.4864$ MPa EQ (3.1)

$f_{ck}(t) = f_{cm}(t) - 8 = 47.4864 - 8 = 39.4864$ MPa CL. 3.1.2(5)

$E_{cm}(t) = (f_{cm}(t)/f_{cm})^{0.3} \cdot E_{cm} = 34.8$ GPa EQ (3.5)

$f'_{max} = 0.6 f_{ck}(t) = \underline{23.6918 \text{ MPa}}$ CL. 5.10.2.2(5)

$f'_{min} = \underline{-1.0 \text{ MPa}}$

STRESS LIMITS AT SERVICE:

$f_{max} = 0.6 f_{ck} = \underline{30 \text{ MPa}}$ CL. 7.2(2)

$f_{min} = \underline{0 \text{ MPa}}$

THE FIRST STEP WAS TO DESIGN THE SLAB UNITS FOR THE ROOF SO LOADING ON THE FRAME COULD BE DETERMINED.

HOLLOW-CORE ROOF SLAB DESIGN - PRE-TENSIONED

ASSUMED 170 mm WIDTH FOR DESIGN (SPACING OF CORES)

PROPERTIES: $h = 360$ mm $b = 170$ mm $h_w = 196$ mm

$t_{ft} = 50$ mm $t_{fb} = 115$ mm $t_w = 50$ mm

$L = 14.0$ m

$A = 37800 \text{ mm}^2$ $I = 5.868 \times 10^8 \text{ mm}^4$

$y_b = 159.881$ mm $y_t = 200.119$ mm

$Z_b = 3.6703 \times 10^6 \text{ mm}^3$ $Z_t = 2.9323 \times 10^6 \text{ mm}^3$

DESIGN LOADS: AT TRANSFER DUE TO SELF-WEIGHT

$w_i = (A/10^6) \times 25 \text{ kN/m}^3 = 0.945$ kN/m $M_i = \frac{w_i L^2}{8} = \underline{23.1525 \text{ kNm}}$

AT SERVICE DUE TO SELF-WEIGHT + SERVICE

$w_{imp} = 1.0 \text{ kN/m}^2 \times 0.17 \text{ m} = 0.17$ kN/m $w_{WIND} = 1.5 \times 0.17 = 0.255$ kN/m

$w_d = w_i + w_{imp} + w_{WIND} = 1.370$ kN/m $M_d = \frac{w_d L^2}{8} = \underline{33.565 \text{ kNm}}$

CHECK SIZING OF THE SLAB:

$$Z_{tREQ} = \frac{\alpha M_d - \beta M_i}{\alpha f_{max} - \beta f'_{min}} = \frac{0.9 \times 33.565 \times 10^6 - 0.75 \times 23.153 \times 10^6}{0.9 \times 30 - 0.75 \times -1.0} = 462850 \text{ mm}^3$$

$$Z_{bREQ} = \frac{\alpha M_d - \beta M_i}{\beta f'_{max} - \alpha f_{min}} = \frac{0.9 \times 33.565 \times 10^6 - 0.75 \times 23.153 \times 10^6}{0.75 \times 23.6918 - 0.9(0)} = 722840 \text{ mm}^3$$

$Z_t = 2932300 \text{ mm}^3 > Z_{tREQ} = 462850 \text{ mm}^3$ OK!

$Z_b = 3670300 \text{ mm}^3 > Z_{bREQ} = 722840 \text{ mm}^3$ OK!

DETERMINE PRESTRESS FORCE AND ECCENTRICITY:

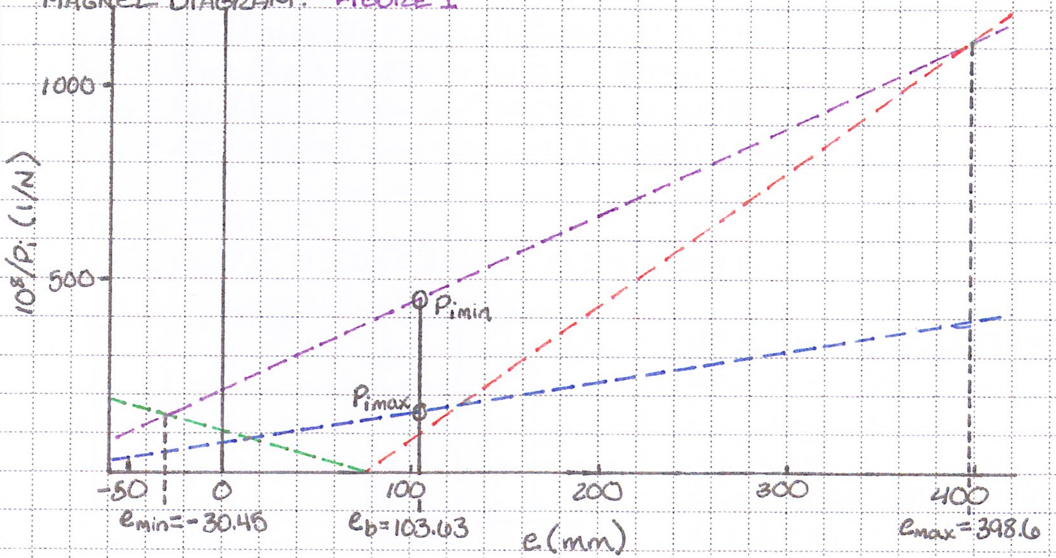
$$\frac{1}{P_i} \leq \frac{\alpha(Z_t/A - e)}{Z_t \cdot f'_{min} - M_i} = 10^{-8}(-267.6636 + 3.4503e) \quad \text{(a) ---}$$

$$\frac{1}{P_i} \geq \frac{\alpha(Z_b/A + e)}{Z_b \cdot f'_{max} + M_i} = 10^{-8}(79.3654 + 0.8174e) \quad \text{(b) ---}$$

$$\frac{1}{P_i} \leq \frac{\beta(Z_t/A - e)}{Z_t \cdot f_{max} - M_d} = 10^{-8}(106.9414 - 1.3786e) \quad \text{(c) ---}$$

$$\frac{1}{P_i} \geq \frac{\beta(Z_b/A + e)}{Z_b \cdot f_{min} + M_d} = 10^{-8}(216.9624 + 2.2345e) \quad \text{(d) ---}$$

MAGNEL DIAGRAM: FIGURE 1



SELECTION OF STRANDS: Y1860S7 STRANDS – $\phi_p = 12.5$mm $A_p = 93.0 \text{ mm}^2$

$f_{pk} = 1860$ MPa $f_{p0.1k} = 1600$ MPa $P_{0STR} = 134$ kN $P_{iSTR} = 126$ kN

$P_{uSTR} = 173$ kN

$e_b = Y_b - c_{nom} - \phi_p/2 = 159.881 - 50 - 12.5/2 = 103.631$ mm CONSERVATIVELY USED $c_{nom} = 50$mm

$e_{min} = -30.45 < e_b = 103.631 < e_{max} = 398.60$ OK!

$P_{imax} = 10^8/(79.3654 + 0.8174 \times 103.631) = 609490$ N

$P_{imin} = 10^8/(216.9624 + 2.2345 \times 103.631) = 222980$ N

$P_{REQ} = P_{imin}/0.75 = 297,272.2$ N $= 297.27$ kN BS EN 1992-1-1 Cl. 5.10.3(2)

SELECTION OF STRANDS CONTINUED: $N_{STR} = 2$

$P_{pk} = P_{uSTR} \cdot N_{STR} = 173 \cdot 2 = 346\text{ kN} > P_{REQ} = 297.27\text{ kN}$ OK!

$P_i = P_{iSTR} \cdot N_{STR} = 126 \cdot 2 = 252\text{ kN} > P_{i,min} = 222.95\text{ kN}$ OK!

$< P_{imax} = 609.49\text{ kN}$ OK!

CHECK CONCRETE STRESSES

AT TRANSFER STAGE:

$$f'_{t,BM} = \frac{\alpha P_i}{A} - \frac{\alpha P_i e_b}{Z_t} + \frac{M_i}{Z_t} = 5.8803\text{ MPa} > f'_{min} = -1.0\text{ MPa} \quad \text{OK!}$$

$$f'_{b,BM} = \frac{\alpha P_i}{A} + \frac{\alpha P_i e_b}{Z_b} - \frac{M_i}{Z_b} = 6.0956\text{ MPa} < f'_{max} = 23.69\text{ MPa} \quad \text{OK!}$$

AT SERVICE STAGE:

$$f_{t,BM} = \frac{\beta P_i}{A} - \frac{\beta P_i e_b}{Z_t} + \frac{M_d}{Z_t} = 9.7671\text{ MPa} < f_{max} = 30.0\text{ MPa} \quad \text{OK!}$$

$$f_{b,BM} = \frac{\beta P_i}{A} + \frac{\beta P_i e_b}{Z_b} - \frac{M_d}{Z_b} = 1.1914\text{ MPa} > f_{min} = 0\text{ MPa} \quad \text{OK!}$$

CHECK ULTIMATE MOMENT CAPACITY:

$E_s = 200000\text{ MPa}$ $\gamma_s = 1.15$ $\gamma_{p,fav} = 0.90$ $\varepsilon_c = 0.0035$ $\varepsilon_{pd} = 0.0200$

$f_{pd} = \sigma_p = f_{p0.1k}/\gamma_s = 1600/1.15 = 1391.3\text{ MPa}$

$f_{pi} = P_{iSTR}/A_p = 128000/93.0 = 1354.8\text{ MPa}$

$\varepsilon_{po} = f_{pi}/E_s = 1354.8/200000 = 0.0068$

$\varepsilon_{pu} = \Delta\varepsilon_p + \gamma_{p,fav} \cdot \varepsilon_{po} = \Delta\varepsilon_p + 0.9(0.0068) = \Delta\varepsilon_p + 0.0061$

$\varepsilon_c = 0.0035$

$\eta \alpha_{cc} f_{ck}/\gamma_c = 1.0 \times 0.85 \times f_{ck}/1.5 = 0.567 f_{ck}$

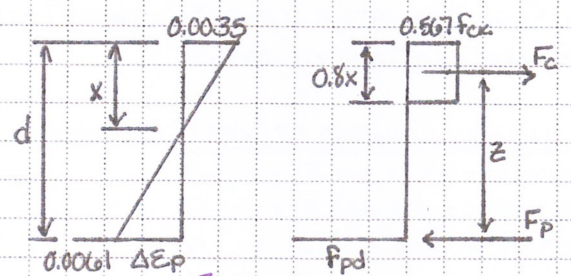


FIGURE 2

$d = h - (y_b - e_b) = 303.75\text{mm}$

$z = d - 0.4x = d - y$

$$\frac{\varepsilon_c}{x} = \frac{\Delta\varepsilon_p}{d-x}$$

$\Delta\varepsilon_p = 0.0035(303.75 - x)/x$

$A_p = 2\text{ STRANDS} \times 93\text{ mm}^2 = 186\text{ mm}^2$

@ $x = 78\text{ mm}$, $F_p = F_c$

$\Delta\varepsilon_p = 0.0035(303.75 - 78)/78 = 0.0101$

$F_p = \min[(\Delta\varepsilon_p + \gamma_{p,fav} \cdot \varepsilon_{po}) \cdot E_s, f_{pd}] \cdot A_p = \min(0.0162 \times 200000, 1391.3) \cdot A_p = 258780\text{ N}$

$F_c = [t_{fb} \cdot b + t_w(0.8x - t_{fb})] \cdot 0.567 f_{ck} = 9120.0 \times 0.567 \times 50 = 258550\text{ N}$

$\varepsilon_{pu} = \Delta\varepsilon_p + 0.0061 = 0.0162 < \varepsilon_{pd} = 0.0200$ OK!

ULTIMATE MOMENT OF RESISTANCE

$A_c = t_{fb} \cdot b + t_w(0.8x - t_{fb}) = 9120\text{ mm}^2$ $y = 0.5A_c/b = 26.82\text{ mm}$

$M_{Rd} = F_p(d - y) = 258780(303.75 - 26.82) = \underline{71.66\text{ kNm}}$

MAXIMUM DESIGN BENDING MOMENT AT ULS — BS EN 1990

$\gamma_G = 1.35$ $\gamma_Q = 1.50$ $\xi = 0.925$ — TABLE NA.A1.2(B)

$\psi_{0ROOF} = 0.7$ $\psi_{0WIND} = 0.5$ — TABLE NA.A1.1

EQ 6.10b CONTROLS – $W_{Ed} = \xi \gamma_G W_i + \gamma_Q W_{WIND} + \gamma_Q \psi_{0ROOF} W_{WIND} = 1.7411$ kN/m — EQ(6.10b)

$M_{Ed} = W_{Ed} L^2/8 = 1.7411 (14^2)/8 = \underline{42.66 \text{ kNm}}$

$M_{Ed}/M_{Rd} = 42.66/71.66 = 0.5952 < 1.0$ OK!

CHECK DEFLECTIONS ($< L/250 = 0.0560$ m) — BS EN 1992-1-1 CL. 7.4.1

AT TRANSFER: $P_{TR} = P_i + W_i L = 265.23$ kN

$y_{TMAX} = P_{TR} \cdot e_b \cdot L^2/8 E_{cm}(t) \cdot I = 0.0329\text{m} < 0.056\text{m}$ OK!

AT APPLICATION OF FINISHES: $P_F = \alpha P_i + W_i L = 240.03$ kN

$y_{FMAX} = P_F \cdot e_b \cdot L^2/8 E_{cm} \cdot I = 0.0281\text{m} < 0.056\text{m}$ OK!

AT SERVICE: $\psi_2 = 0$ — BS EN 1990 TABLE NA.A1.1

$P_S = \beta P_i + W_i L + \psi_2 (W_{IMP} + W_{WIND}) = 202.23$ kN

$\varphi(\infty, t_0) = 1.3$ — BS EN 1992-1-1 FIG. 3.1

$E_{c,eff} = E_{cm}/(1+1.3) = 16.09$ GPa

$y_{SMAX} = P_S \cdot e_b \cdot L^2/8 E_{ceff} \cdot I = 0.0544\text{m} < 0.056\text{m}$ OK!

SOLID ROOF SLAB DESIGN – PRE-TENSIONED

ASSUMED 1000mm WIDTH FOR DESIGN, $L = 14.0$m

FOLLOWED SAME PROCESS AS HOLLOW-CORE SLAB DESIGN

PROPERTIES: $h = 360$mm $A = 360000 \text{ mm}^2$ $y_b = y_t = 180$mm

$I = 3.888 \times 10^9 \text{ mm}^4$ $Z_b = Z_t = 21.6 \times 10^6 \text{ mm}^3$

DESIGN LOADS: $W_i = 9$ kN/m $M_i = 220.5$ kNm

$W_d = 11.5$ kN/m $M_d = 281.75$ kNm

$W_{Ed} = 14.54$ kN/m $M_{Ed} = 356.20$ kNm

CHECK DEPTH: $Z_{tREQ} = 3.1784 \times 10^6 \text{ mm}^3 < 21.6 \times 10^6 \text{ mm}^3$ OK!

$Z_{bREQ} = 4.9637 \times 10^6 \text{ mm}^3 < 21.6 \times 10^6 \text{ mm}^3$ OK!

PRESTRESSING: Y1860S7 STRANDS – $\phi_p = 12.5$mm; $e_b = 50$ mm; $N_{STR} = 28$

$e_{min} = -7.8\text{mm} < e_b = 50\text{mm} < e_{max} = 362.5\text{mm}$ OK!

$P_{pk} = 173 \text{ kN} \cdot 28 = 4844 \text{ kN} > P_{REQ} = 4653.5$ kN OK!

$P_{min} = 3416.2 \text{ kN} < P_i = 3528 \text{ kN} < P_{max} = 7396.4$ kN OK!

CONCRETE STRESS: $f'_{tBM} = 11.6783 \text{ MPa} > f'_{min} = -1.0$ MPa OK!

$f'_{bBM} = 5.6917 \text{ MPa} < f'_{max} = 23.69$ MPa OK!

$f_{tBM} = 14.2690 \text{ MPa} < f_{max} = 30.0$ MPa OK!

$f_{bBM} = 0.4310 \text{ MPa} > f_{min} = 0$ MPa OK!

ULTIMATE MOMENT: $F_p = F_c = 3623$ kN @ $x = 160$ mm $\varepsilon_{pu} = 0.0076 < \varepsilon_{pd} = 0.02$ OK!

$M_{Ed}/M_{Rd} = 356.20 \text{ kNm}/601.41 \text{ kNm} = 0.5923 < 1.0$ OK!

DEFLECTIONS: $y_{TMAX} = 0.0330\text{m} < 0.056\text{m}$ OK! $y_{FMAX} = 0.0281\text{m} < 0.056\text{m}$ OK!

$\varphi(\infty, t_0) = 1.3$; $E_{Ceff} = 16.09$ GPa $y_{SMAX} = 0.0543\text{m} < 0.056\text{m}$ OK!

The Concrete Centre

Project Details:	Date:	Entry Reference:
Structural Concrete 2011 Southfield Grandstand	24/6/11	11-001
	Page: 8	Of: 33

TERRACED SEATING BOX BEAM DESIGN - PRE-TENSIONED

FOLLOWED SAME PROCEDURE AS HOLLOW-CORE SLAB DESIGN

PROPERTIES: $h = 400mm$ $b = 800mm$ $h_w = 200mm$

$t_{ft} = t_{fb} = 100mm$ $t_w = 100mm$

$A = 200000 mm^2$ $I = 3.8667 \times 10^9 mm^4$

$y_b = y_t = 200mm$ $Z_b = Z_t = 1.933 \times 10^7 mm^3$

$L = 14.0m$ $c_{nom} = 40mm$

DESIGN LOADS: $w_i = 5.0 kN/m$ $M_i = 122.5 kNm$

$w_d = 9.72 kN/m$ $M_d = 238.14 kNm$

$w_{Ed} = 12.78 kN/m$ $M_{Ed} = 313.20 kNm$

CHECK SIZING: $Z_{tREQ} = 4.4126 \times 10^6 mm^3 < Z_t = 1.933 \times 10^7 mm^3$ OK!

$Z_{bREQ} = 6.8913 \times 10^6 mm^3 < Z_b = 1.933 \times 10^7 mm^3$ OK!

PRESTRESSING: Y1860S7 STRANDS - $\phi_p = 16.0mm$; $P_{uSTR} = 279 kN$; $P_{iSTR} = 204 kN$

$e_b = y_b - c_{nom} - \phi_p/2 = 152 mm$; $N_{STR} = 9$

$e_{min} = -17.29mm < e_b = 152mm < e_{max} = 287.18mm$ OK!

$P_{pk} = 279 \times 9 = 2511 kN > P_{REQ} = 1702.5 kN$ OK!

$P_{imin} = 1276.9 kN < P_i = 204 \times 9 = 1836 kN < P_{imax} = 2694 kN$ OK!

CONCRETE STRESSES: $f'_{tBM} = 1.6069 MPa > f'_{min} = -1.0 MPa$ OK!

$f'_{bBM} = 14.9171 MPa < f'_{max} = 23.6918 MPa$ OK!

$f_{tBM} = 8.3765 MPa < f_{max} = 30.0 MPa$ OK!

$f_{bBM} = 5.3935 MPa > f_{min} = 0 MPa$ OK!

ULTIMATE MOMENT: $F_p = F_c = 1878 kN$ @ $x = 103.5mm$

$\varepsilon_{pu} = \Delta\varepsilon_p + \gamma_{p,fav}\varepsilon_{po} = 0.0145 < \varepsilon_{pd} = 0.0200$ OK!

$M_{Rd} = F_p(d - 0.4x) = 583.39 kNm$

$M_{Ed}/M_{Rd} = 0.5369 < 1.0$ OK!

DEFLECTIONS: $L/250 = 0.0560m$ $L/175 = 0.0800$

$y_{Tmax} = 0.0527m$; $y_{Fmax} = 0.0448m < 0.0560m$ OK!

$y_{Smax} = 0.0790m < 0.0800m$ OK!

SEATING SUPPORT BEAM DESIGN: - PRE-TENSIONED

FOLLOWED SAME PROCEDURE AS HOLLOWED-CORE SLAB DESIGN

PROPERTIES: $h = 750mm$ $b = 500mm$ $L = 9.0m$
$A = 375000mm^2$ $I = 1.7578 \times 10^{10} mm^4$
$y_b = y_t = 375mm$ $z_b = z_t = 4.6875 \times 10^7 mm^3$
$c_{nom} = 40mm$

DESIGN LOADS: AT TRANSFER $M_i = 94.9$ kNm LOADS DETERMINED IN
AT SERVICE $M_d = 808$ kNm CSC S-FRAME MODEL
FACTORED $M_{Ed} = 865$ kNm

CHECK SIZING: $z_{tREQ} = 2.3628 \times 10^7 mm^3 < z_t = 4.6875 \times 10^7 mm^3$ OK!
$z_{bREQ} = 3.6901 \times 10^7 mm^3 < z_b = 4.6875 \times 10^7 mm^3$ OK!

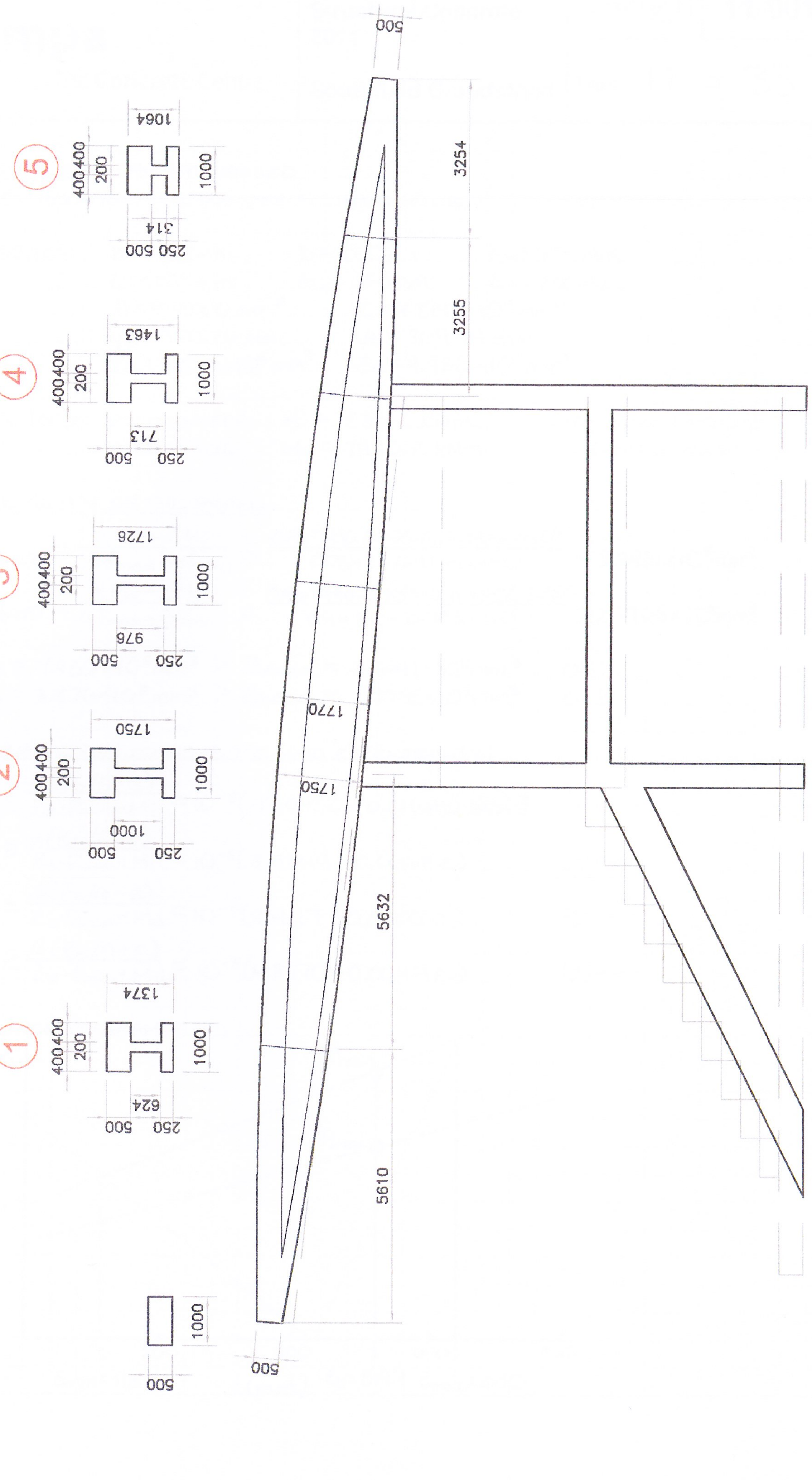
PRESTRESSING: Y1820S7G STRANDS – $\phi_p = 15.2mm$; $P_{uSTR} = 300kN$; $P_{iSTR} = 219kN$
$e_b = 150 mm$; $N_{STR} = 20$
$e_{min} = 18.5mm < e_b = 150mm < e_{max} = 167.8mm$ OK!
$P_{pk} = 300 \cdot 20 = 6000kN > P_{REQ} = 5221 kN$ OK!
$P_{imin} = 3915.8 kN < P_i = 4380 (= 219 \times 20) < P_{imax} = 4870.6 kN$ OK!

CONCRETE STRESSES: $f'_{tBM} = -0.0774 MPa > f'_{min} = -1.0 MPa$ OK!
$f'_{bBM} = 21.1014 MPa < f'_{max} = 23.6918 MPa$ OK!
$f_{tBM} = 15.4777 MPa < f_{max} = 30.0 MPa$ OK!
$f_{bBM} = 2.0423 MPa > f_{min} = 0 MPa$ OK!

ULTIMATE MOMENT: $F_p = F_c = 4477 kN$ @ $x = 395mm$
$\varepsilon_{pu} = \Delta\varepsilon_p + \gamma_p f_{pay} : \varepsilon_{po} = 0.0072 < \varepsilon_{pd} = 0.0200$ OK!
$M_{Rd} = F_p(d - 0.4x) = 1642.9$ kNm
$M_{Ed} / M_{Rd} = 0.5263 < 1.0$ OK!

DEFLECTIONS: $L/250 = 0.036m$
$y_{TMAX} = 0.0111m$; $y_{FMAX} = 0.0109m$; $y_{SMAX} = 0.0234m < 0.036m$ OK!

Entry Ref: 11-00
Page 10 of 33



ROOF BEAM DESIGN – POST-TENSIONED

SECTION 2 – LOCATION BEARING THE MAXIMUM MOMENT

PROPERTIES: $h = 1750$ mm, $b = 1000$ mm, $h_w = 1000$ mm
$t_{ft} = 500$ mm, $t_{fb} = 250$ mm, $t_w = 200$ mm
$A = 950000\ mm^2$, $I = 3.7995 \times 10^{11}\ mm^4$
$y_b = 980.26$ mm, $y_t = 769.74$ mm
$Z_b = 3.876 \times 10^8\ mm^3$, $Z_t = 4.936 \times 10^8\ mm^3$

DESIGN LOADS: AT TRANSFER $M_i = 1535.2$ kNm
AT SERVICE $M_d = 7990.2$ kNm

LOADS DETERMINED IN CSC S-FRAME MODEL

CHECK SIZING OF THE BEAM:

$$Z_{tREQ} = \frac{\alpha M_d - \beta M_i}{\beta f'_{max} - \alpha f_{min}} = \frac{0.9 \times 7990.2 \times 10^6 - 0.75 \times 1535.2 \times 10^6}{0.75 \times 23.6918 - 0.9 \times 0} = 3.3991 \times 10^8\ mm^3$$

$$Z_{bREQ} = \frac{\alpha M_d - \beta M_i}{\alpha f_{max} - \beta f'_{min}} = \frac{0.9 \times 7990.2 \times 10^6 - 0.75 \times 1535.2 \times 10^6}{0.9 \times 30 - 0.75 \times -1.0} = 2.1765 \times 10^8\ mm^3$$

$Z_t = 4.936 \times 10^8\ mm^3 > Z_{tREQ} = 3.3991 \times 10^8\ mm^3$ OK!
$Z_b = 3.876 \times 10^8\ mm^3 > Z_{bREQ} = 2.1765 \times 10^8\ mm^3$ OK!

DETERMINE PRESTRESS FORCE AND ECCENTRICITY:

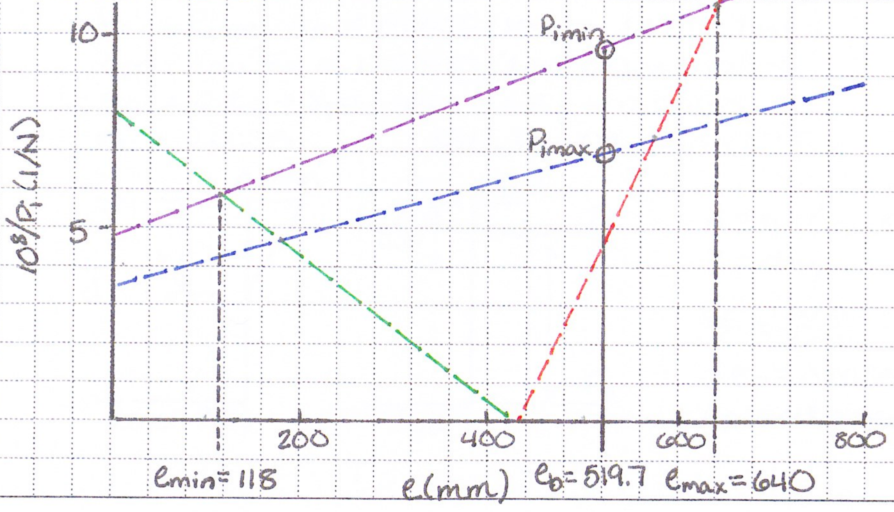
$$\frac{1}{P_i} \le \frac{\alpha(Z_b/A - e)}{Z_b \cdot f'_{min} - M_i} = 10^{-8}(-19.0932 + 0.0468e) \quad \text{(a)}$$

$$\frac{1}{P_i} \ge \frac{\alpha(Z_t/A + e)}{Z_t \cdot f'_{max} + M_i} = 10^{-8}(3.5349 + 0.0068e) \quad \text{(b)}$$

$$\frac{1}{P_i} \le \frac{\beta(Z_b/A - e)}{Z_b \cdot f_{max} - M_d} = 10^{-8}(8.4117 - 0.0206e) \quad \text{(c)}$$

$$\frac{1}{P_i} \ge \frac{\beta(Z_t/A + e)}{Z_t \cdot f_{min} + M_d} = 10^{-8}(4.8770 + 0.0094e) \quad \text{(d)}$$

MAGNEL DIAGRAM



SELECTION OF STRANDS: Y1820S7G STRANDS – $\phi_p = 15.2$ mm $A_p = 165\,mm^2$

$f_{pk} = 1820$ MPa $\quad f_{p0.1k} = 1560$ MPa $\quad P_{USTR} = 300$ kN $\quad P_{iSTR} = 219$ kN

$e_b = y_t - c_{nom} - \phi_p/2 = 769.74 - 60 - 15.2/2 = 702.14\,mm > e_{max} = 640\,mm$

USE $e_b = y_t - t_{ft}/2 = 769.74 - 250 = 519.74\,mm$

$e_{min} = 118\,mm < e_b = 519.74\,mm < e_{max} = 640\,mm$ OK!

$P_{imax} = 10^8/(4.8770 + 0.0094 \times 519.74) = 10251000\,N$

$P_{imin} = 10^8/(3.5349 + 0.0068 \times 519.74) = 14143000\,N$

BS EN 1992-1-1 CL. 5.10.3(2)

$P_{REQ} = P_{imin}/0.75 = 13667000\,N = 13667\,kN$

USE 48 STRANDS ($= N_{STR}$)

$P_{pk} = P_{USTR} \cdot N_{STR} = 300 \cdot 48 = 14400\,kN > P_{REQ} = 13667\,kN$ OK!

$P_i = P_{iSTR} \cdot N_{STR} = 219 \cdot 48 = 10512\,kN > P_{imin} = 10251\,kN$ OK!

$< P_{imax} = 14143\,kN$ OK!

CHECK CONCRETE STRESSES

AT TRANSFER STAGE:

$$f'_{t,BM} = \frac{\alpha P_i}{A} + \frac{\alpha P_i e_b}{Z_t} - \frac{M_i}{Z_t} = 16.8102\,MPa < f'_{max} = 23.6918\,MPa \quad \text{OK!}$$

$$f'_{b,BM} = \frac{\alpha P_i}{A} - \frac{\alpha P_i e_b}{Z_b} + \frac{M_i}{Z_b} = 1.2334\,MPa > f'_{min} = -1.0\,MPa \quad \text{OK!}$$

AT SERVICE STAGE:

$$f_{t,BM} = \frac{\beta P_i}{A} + \frac{\beta P_i e_b}{Z_t} - \frac{M_d}{Z_t} = 0.4129\,MPa > f_{min} = 0\,MPa \quad \text{OK!}$$

$$f_{b,BM} = \frac{\beta P_i}{A} - \frac{\beta P_i e_b}{Z_b} + \frac{M_d}{Z_b} = 18.3418\,MPa < f_{max} = 30.0\,MPa \quad \text{OK!}$$

CHECK ULTIMATE MOMENT CAPACITY:

$E_s = 200000$ MPa $\quad \gamma_s = 1.15 \quad \gamma_{pfav} = 0.90 \quad \varepsilon_c = 0.0035 \quad \varepsilon_{pd} = 0.020$

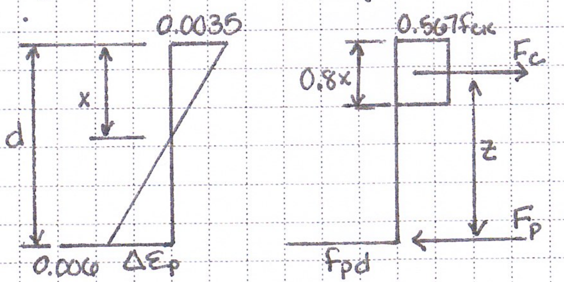
$f_{pd} = \sigma_p = f_{p0.1k}/\gamma_s = 1560/1.15 = 1356.5$ MPa

$f_{pi} = P_{iSTR}/A_p = 219000/165 = 1327.27$ MPa

$\varepsilon_{p0} = f_{pi}/E_s = 1327.27/200000 = 0.0066$

$\varepsilon_{pu} = \Delta\varepsilon_p + \gamma_{pfav} \cdot \varepsilon_{p0} = \Delta\varepsilon_p + 0.0060$

$\varepsilon_c = 0.0035 \qquad \eta\alpha_{cc} f_{ck}/\gamma_c = 0.567 f_{ck}$



$d = h - (y_t - e_b) = 1500\,mm$

$z = d - y$

$$\frac{\varepsilon_c}{x} = \frac{\Delta\varepsilon_p}{d - x}$$

$\Delta\varepsilon_p = 0.0035(1500 - x)/x$

$A_p = 48$ STRANDS $\times 165\,mm^2 = 7920\,mm^2$

ULTIMATE MOMENT CAPACITY CONTINUED

@ $x = 1125$ mm $F_p = F_c$

$\Delta\varepsilon_p = 0.0035(1500-1125)/1125 = 0.0011$

$F_p = \min[(\Delta\varepsilon_p + \gamma_{pfav}\cdot\varepsilon_{po})\cdot E_s, f_{pd}]\cdot A_p = \min[1420, 1356.5]\cdot 7920 = 10744$ kN

$F_c = [t_{fb}\cdot b + t_w(0.8x - t_{fb})]\cdot(0.567 f_{ck}) = 380000\cdot(0.567\times 50) = 10773$ kN

$\varepsilon_{pu} = \Delta\varepsilon_p + 0.0060 = 0.0011 + 0.0060 = 0.0071 < \varepsilon_{pd} = 0.0200$ OK!

ULTIMATE MOMENT OF RESISTANCE:

$A_c = 380000\ mm^2$ $\quad y = 0.5 A_c/b = 190$ mm

$M_{Rd} = F_p(d-y) = 10744(1500-190) = \underline{14074\ kNm}$

MAXIMUM DESIGN BENDING MOMENT AT ULS:

$M_{Ed} = \underline{9808.89\ kNm}$

$M_{Ed}/M_{Rd} = 0.6969 < 1.0$ OK!

BS EN 1992-1-1
CL. 7.4.1(4)

<u>CHECK DEFLECTIONS:</u> $(< L/250 = 0.0440m,\ L = 11m)$

AT TRANSFER
$w_i = M_i\cdot 8/L^2 = 1535.2\cdot 8/11^2 = 101.5$ kN/m
$P_{TR} = P_i + w_i L = 11628$ kN
$y_{TMAX} = 5P_{TR}\cdot e_b\cdot L^2/48 E_{cm}(t)\cdot I = 0.0068m < 0.044m$ OK!

AT APPLICATION OF FINISHES
$w_d = M_d\cdot 8/L^2 = 7990.2\cdot 8/11^2 = 528.3$ kN/m
$P_F = \alpha P_i + w_d L = 15272$ kN
$y_{FMAX} = 5P_F\cdot e_b\cdot L^2/48 E_{cm}\cdot I = 0.0071m < 0.044m$ OK!

AT SERVICE
$\psi_2 = 0$
$P_S = \beta P_i + w_d L + \psi_2(w_{IMP} + w_{WIND}) = 13695$ kN — BS EN 1990 TABLE NA.A1.1
$\varphi(\infty, t_0) = 1.5$ — BS EN 1992-1-1 FIG. 3.1
$E_{ceff} = E_{cm}/(1+1.5) = 14.8$ GPa
$y_{SMAX} = 5P_S\cdot e_b\cdot L^2/48 E_{ceff}\cdot I = 0.0160m < 0.044m$ OK!

mpa The Concrete Centre	Project Details: **Structural Concrete 2011** **Southfield Grandstand**	Date: 24/6/11	Entry Reference: **11-001**
		Page: 14	Of: 33

SUMMARY OF RESULTS:

USING THE SAME PROCEDURE, SECTIONS 1, 3, 4 & 5 HAVE BEEN VERIFIED.

① SIZING: $Z_t = 3.248 \times 10^8 mm^3 > Z_{treq} = 8.457 \times 10^7 mm^3$ OK!
$Z_b = 2.637 \times 10^8 mm^3 > Z_{breq} = 5.415 \times 10^7 mm^3$ OK!
PRESTRESS: $e_{min} = -205 mm < e_b = 365.67 mm < e_{max} = 543 mm$ OK!
$P_{pk} = 14400 kN > P_{REQ} = 4752 kN$ OK!
$P_{min} = 3564 kN < P_i = 10512 kN < P_{max} = 10787 kN$ OK!
STRESSES: $f_t' = 20.36 MPa < f'_{max}$ $f_b' = -0.9396 MPa > f'_{min}$ OK!
$f_t = 11.82 MPa > f_{min}$ $f_b = 5.55 MPa < f_{max}$ OK!
MOMENT: $M_{Ed} / M_{Rd} = 0.2522 < 1.0$ OK!
DEFLECTION: $Y_{TMAX} = 0.0071 m$; $Y_{FMAX} = 0.0068 m$; $Y_{SMAX} = 0.0126 m < 0.044 m$ OK!

③ SIZING: $Z_t = 4.824 \times 10^8 mm^3 > Z_{treq} = 1.747 \times 10^8 mm^3$ OK!
$Z_b = 3.793 \times 10^8 mm^3 > Z_{breq} = 1.119 \times 10^8 mm^3$ OK!
PRESTRESS: $e_{min} = -129 mm < e_b = 509.7 mm < e_{max} = 875 mm$ OK!
$P_{pk} = 14400 kN > P_{REQ} = 8307 kN$ OK!
$P_{min} = 6230 kN < P_i = 10512 kN < P_{max} = 14171 kN$ OK!
STRESSES: $f_t' = 16.73 MPa < f'_{max}$ $f_b' = 1.4647 > f'_{min}$ OK!
$f_t = 6.79 MPa > f_{min}$ $f_b = 10.31 MPa < f_{max}$ OK!
MOMENT: $M_{Ed} / M_{Ed} = 0.4820 < 1.0$ OK!
DEFLECTION: $Y_{TMAX} = 0.0059 m$; $Y_{FMAX} = 0.0061 m$; $Y_{SMAX} = 0.0134 m < 0.044 m$ OK!

④ SIZING: $Z_t = 3.634 \times 10^8 mm^3 > Z_{treq} = 1.479 \times 10^8 mm^3$ OK!
$Z_b = 2.918 \times 10^8 mm^3 > Z_{breq} = 9.470 \times 10^7 mm^3$ OK!
PRESTRESS: $e_{min} = -109 mm < e_b = 401.6 mm < e_{max} = 569 mm$ OK!
$P_{pk} = 14400 kN > P_{REQ} = 7828 kN$ OK!
$P_{min} = 5871 kN < P_i = 10512 kN < P_{max} = 12885 kN$ OK!
STRESSES: $f_t' = 18.94 MPa < f'_{max}$ $f_b' = 0.2157 MPa > f'_{min}$ OK!
$f_t = 7.75 MPa > f_{min}$ $f_b = 10.19 MPa < f_{max}$ OK!
MOMENT: $M_{Ed} / M_{Rd} = 0.3889 < 1.0$ OK!
DEFLECTION: $Y_{TMAX} = 0.0068 m$; $Y_{FMAX} = 0.0070 m$; $Y_{SMAX} = 0.0161 m < 0.044 m$ OK!

⑤ SIZING: $Z_t = 1.993 \times 10^8 mm^3 > Z_{treq} = 3.696 \times 10^7 mm^3$ OK!
$Z_b = 1.723 \times 10^8 mm^3 > Z_{breq} = 2.366 \times 10^7 mm^3$ OK!
PRESTRESS: $e_{min} = -167 mm < e_b = 243.4 mm < e_{max} = 448 mm$ OK!
$P_{pk} = 14400 kN > P_{REQ} = 3232 kN$ OK!
$P_{min} = 2424 kN < P_i = 10512 kN < P_{max} = 12833 kN$ OK!
STRESSES: $f_t' = 22.24 MPa < f'_{max}$ $f_b' = -0.6194 > f'_{min}$ OK!
$f_t = 14.87 MPa > f_{min}$ $f_b = 3.72 MPa < f_{max}$ OK!
MOMENT: $M_{Ed} / M_{Ed} = 0.1599 < 1.0$ OK!
DEFLECTION: $Y_{TMAX} = 0.0095 m$; $Y_{FMAX} = 0.0085 m$; $Y_{SMAX} = 0.0180 m < 0.044 m$ OK!

DESIGN OF REINFORCED CONCRETE FLOORS

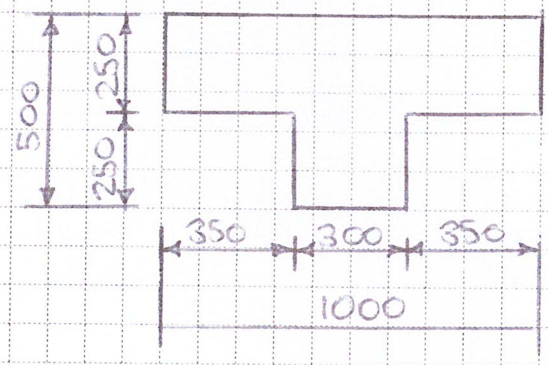
Adopt T-Section Beams.

Design for worst case span, i.e. 9m span above club room.

Design as simply supported.

Design Data:

Span of main beams	$L_b = 9m$
Span of slabs (centres of main beams)	$L_s = 1m$
Thickness of slabs	$h_s = 250mm$
Web width of beams	$b_w = 300\ mm$
Overall Depth	$h = 500\ mm$
Characteristic variable (imposed) load	$q_k = 5.0\ kN/m^2$
Nominal Density of reinforced concrete	$\gamma_{conc} = 25\ kN/m^3$
Fire Resistance	REI = 90 min



Section through T-Section Beam

Solution:

Effective Flange Width, b_{eff}

EN 1992-1-1

Figure 5.2 — Use middle span $\therefore l_0 = 0.7 L_b$

$l_0 = 0.7 \times 9000 = 6300mm$

Figure 5.3

Net half flange widths, b_1 and b_2

$$b_1 = b_2 = \frac{Ls - bw}{2} = \frac{1000 + 300}{2}$$

$$= 350\text{mm}$$

Eqs. (5.7a)/(5.7b)

Effective half flange widths b_{eff1} and b_{eff2}

$$b_{eff1} = b_{eff2} = \min(0.2b_1 + 0.1l_0 ; 0.2l_0 ; b_1)$$

$$= \min(0.2 \times 350 + 0.1 \times 9000 ; 0.2 \times 9000 ; 350)$$

$$= \min(970 ; 1800 ; 350)$$

$$= 350\text{mm}$$

Eq. (5.7)

Effective Flange width, b_{eff}

$$b_{eff} = \min(b_{eff1} + b_{eff2} + h_w ; Ls)$$

$$= \min(350 + 350 + 300 ; 1000)$$

$$= 1000\text{mm}$$

The effective flange width is 1000mm

Beam Size and Concrete Cover

EN1992-1-2
Clause 5.6.2(1)
Table 5.5

Minimum dimensions of reinforced concrete beam
Beam is simply supported
Assume $a = 55$ mm $\therefore$ $b_{min} = 150 < 300 \therefore$ OK

Clause 5.6.4(1)

Minimum height of concrete beams exposed to fire on all sides:
$h_{min} = b_{min} = 150$ mm
$h = 500$ mm > 150 mm $\therefore$ The beam height is OK

Clause 5.6.4(1)

Eq. (5.12)

Minimum cross sectional area of reinforced beams exposed to fire on all sides
Min cross sectional area = $A_{cmin} = 2b^2_{min}$
$A_{cmin} = 2 \times 150^2 = 45000$ mm^2
$A_c = b_w \times h = 300 \times 500 = 150\,000$ mm^2
$A_c > A_{c\,min}$ $\therefore$ Cross sectional area OK

Reinforcement Required for bending Resistance

Effective depth, d $d = h - a = 500 - 55 = 445$mm

Characteristic values of the UDL load

Cross sectional area of beam $A_{c,Tbeam} = 1 \times 0.25 + 0.3 \times 0.25$
$= 0.325 \, m^2$

Characteristic permanent load of beam $g_{k_1} = A_{c,Tbeam} \times \gamma_{conc}$
$= 0.325 \times 25 = 8.125 \, kN/m$

Characteristic imposed load $q_{k_1} = q_k \times L_s$
$= 5 \times 1 = 5.0 \, kN/m$

UK NA to EN 1990
Table NA.A1 2(B) $\gamma_G = 1.35$ $\gamma_Q = 1.5$

EN 1990
Eq (6.10)

$q_d = \gamma_G \times g_{k_1} + \gamma_Q \times q_{k_1}$
$= 1.35 \times 8.125 + 1.5 \times 5$
$= 18.469 \, kN/m$

Design bending moment at midspan

$M_{Ed} = \frac{wL^2}{8}$ where $w = q_d$, $L = L_b$
$= \frac{18.469 \times 9^2}{8}$
$= 186.999$
$= 187 \, kNm$

Design Shear at supports

$V_{Ed} = \frac{wL}{2}$
$= \frac{18.469 \times 9}{2}$
$= 83.11 \, kN$

Bending Reinforcement at mid-span

$K = \frac{M_{Ed}}{b_{eff} d^2 f_{ck}} = \frac{187 \times 10^6}{1000 \times 445^2 \times 50}$
$= 0.01889$

$\delta = \min(1 - 0\%; 1) = 1$

$K' = 0.6 \times 1 - 0.18 \times 1^2 - 0.21 = 0.21$

$K' > K \therefore$ No compression reinforcement

$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K}\right]$$

$$= \frac{d}{2}\left[1 + \sqrt{1 - 3.53 \times 0.01889}\right]$$

$$= 0.983d > 0.95d$$

The lever arm is taken as 0.95d

$$As_{req} = \frac{M_{Ed}}{f_{yd}z} = \frac{187 \times 10^6}{\left(\frac{500}{1.15}\right) \times 0.95 \times 445}$$

$$= 1017.4 \text{ mm}^2$$

Provide 25 mmØbars x 4, i.e. 4H25 (1963.5mm^2)

Minimum Reinforcement Spacing

EN1992-1-1

Clause 8.2(2)

$$S_{smin} = S_{sclear} + Ø_s$$
$$= \text{Max}[k_1 Ø_s;\ d_g + k_2;\ 20\text{mm}] + Ø_s$$
$$= \text{Max}[25;\ 25;\ 20\text{mm}] + 25$$
$$= 50\text{mm}$$

where $k_1 = 1$ and $k_2 = 5$mm

Maximum number of bars per row

$$N_{bar,max} = \frac{b_w - 2a_{sd}}{S_{smin}} + 1$$

$$= \frac{300 + 2 \times 55}{50} + 1 = 4.8$$

$4.8 > 4 \therefore$ Spacing OK

Design suitable reinforcement with respect to shear

EN1992-1-1
Eq(3.15)

Design compressive Strength

$$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c}$$

$$= \frac{1 \times 50}{1.5} = 33.33 \text{ MPa}$$

UK NA to EN 1992-1-1
Where $\alpha_{cc} = 1.0$ is used for shear checks
Design strength of shear reinforcement, f_{ywd}
$f_{ywd} = f_{ywk}/\gamma_s = 500/1.15 = 434.78$ MPa

EN 1992-1-1
Clause 6.2.3
Eq (6.9)

Design shear strength for members requiring vertical shear reinforcement

$$V_{Rdmax} = \frac{V_{Rdmax}}{b_w Z} = \frac{a_w v_1 f_{cd}}{\cot\theta + \tan\theta}$$

$$= \frac{1.0 \times 0.48 \times 33.33}{\cot 21.8^\circ + \tan 21.8^\circ} = 5.52 \text{ MPa}$$

where:

Clause 6.2.3(3) — a_w is a coefficient considering the stress state in the compression chord, $a_w = 1.0$

Clause 6.2.3(3) — v_1 is a strength reduction factor for cracked concrete in shear, calculated as:

$$v_1 = v = 0.6\left(1 - \frac{f_{ck}}{250}\right) = 0.48$$

Eq (6.7) — θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force, and $\theta = 21.80^\circ$ is taken from $\cot\theta = 2.5$

Design Shear Reinforcement at support

EN 1992-1-1 — Design shear force at outer support

$V_{Ed} = 83.11$ kN

Clause 6.2.3(1) — Lever arm in shear analysis $Z = 0.9d$ may be used

Design shear stress at the outer support, $v_{Ed} = \frac{V_{Ed}}{b_w Z} = \frac{83.11 \times 10^3}{300 \times 0.95 \times 445}$

$= 0.6553$ MPa

$V_{Rd\,Max} = 5.52$ MPa

$\therefore \cot\theta = 2.5$ or $\theta = 21.80^\circ$ is adopted and nominal links are provided

Try H8 2 leg links

EN1992-1-1

$$A_{sw} = 2 \times \pi \times 8^2/4 = 100.5 \text{ mm}^2$$

Eq (6.8)

Maximum spacing of shear reinforcement

$$S_{wmax1} = \frac{A_{sw}\, f_{ywd} \cot\theta}{v_{Ed}\, b_w}$$

$$= \frac{100.5 \times 434.78 \times 2.5}{0.6553 \times 300}$$

$$= 555.67 \text{ mm}$$

Eq (9.8N)

$$S_{wmax2} = \min(0.75d;\ 600\text{mm})$$

$$= \min(333.75;\ 600)$$

$$= 333.75 \text{ mm}$$

$$S_{wmax} = \min(S_{wmax1};\ S_{wmax2})$$

$$= 333.75 \text{mm}$$

Provide H8 shear reinforcement with 2 legs @ 300mm centres

Check the suitability of the beam with respect to deflection.

EN1992-1-1

Clause 7.4.2(2)

Reference Reinforcement Ratio, ρ_0

$$\rho_0 = \sqrt{f_{ck}}\ 10^{-3} = \sqrt{50} \times 10^{-3} = 7.07 \times 10^{-3}$$

Required Reinforcement Ratio ρ

$$\rho = A_{s\,req}/(b_w d) = 1017.4/(300 \times 445)$$

$$= 7.62 \times 10^{-3} > \rho_0 = 7.07 \times 10^{-3}$$

Required compression reinforcement ratio $\rho' = 0$

Eq (7.16b)

Limit span-depth ratio

$$\left(\frac{L}{d}\right)_{limit} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right]$$

$$= 1\left[11 + 1.5\sqrt{50} \times \frac{7.07 \times 10^{-3}}{7.62 \times 10^{-3}} + \frac{1}{12}\sqrt{50}\sqrt{\frac{0}{7.07 \times 10^{-3}}}\right]$$

$$= 20.04$$

UK NA to EN 1992-1-1

Table NA5

Where $K = 1.0$ for simply supported beams.

EN 1992-1-1

Eq (7.17)

$$F_1 = \frac{500 A_{s,prov}}{f_{yk} A_{s,req}} = \frac{500 \times 1964}{500 \times 1017.4}$$

$$= 1.930$$

Clause 7.4.2(2)

$$F_2 = \max\left[\left(1.1 - 0.1\frac{L_s}{b_w}\right); (0.8)\right]$$

$$= \max\left[1.1 - 0.1 \times \frac{1000}{300}; 0.8\right]$$

$$= \max\left[0.767, 0.8\right] = 0.8$$

Clause 7.4.2(2)

$$F_3 = \min\left(\frac{7}{L_b}; 1.0\right) = \min\left(\frac{7}{9}, 1.0\right) = \frac{7}{9}$$

$$= 0.778$$

Clause 7.4.2(2)

Final limit span-depth ratio:

$$\left(\frac{L}{d}\right)_{limit,fin} = \left(\frac{L}{d}\right)_{limit} F_1 F_2 F_3$$

$$= 20.04 \times 1.93 \times 0.8 \times 0.778$$

$$= 24.078$$

Actual span-depth ratio

$$\left(\frac{L}{d}\right)_{actual} = \frac{9000}{445} = 20.225 < 24.078$$

As $\left(\frac{L}{d}\right)_{actual} < \left(\frac{L}{d}\right)_{limit\_final}$

The designed T-beam is adequate with respect to deflection.

Check Maximum and Minimum Reinforcement Areas

EN 1992-1-1

Table 3.1

Eq (9.1N)

Minimum reinforcement area, $A_{s,min}$

$f_{ctm} = 4.1$ MPa

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_w d = 0.26 \times \frac{4.1}{500} \times 300 \times 445$$

$$= 685.28 \text{ mm}^2 > 0.0013 b_w d$$

$$0.0013 b_w d = 0.0013 \times 300 \times 445 = 173.55 \text{ mm}^2$$

EN 1992-1-1
Clause 9.2.1.1(3)

Maximum Reinforcement Area $A_{s,max}$

$A_{s,max} = 0.04 A_{cbeam}$
$= 0.04 \times 300 \times 500$
$= 6000 \text{ mm}^2$

$A_{s,min} = 685.28 < A_{s,prov} = 1964 < A_{s,max} = 6000$

$\therefore$ The Reinforcement is adequate

Check Assumed Effective Depth

Effective Depth d

$d = h - C_{nom} - \varnothing_{slink} - \varnothing_s/2$
$= 500 - 35 - 8 - \frac{25}{2} = 444.5 \text{mm}$

$444.5 \approx 445 \text{mm}$

Therefore all calculations are valid.

The present design of the T-beam is adequate with respect to bending, shear, and deflection.

The Concrete Centre	Project Details: Structural Concrete 2011 — Southfield Grandstand	Date: 24/6/11	Entry Reference: 11-001
		Page: 23	Of: 33

DESIGN OF REINFORCED CONCRETE COLUMNS

For the purpose of preliminary conceptual design, the worst case column loading has been assumed as the highest axial load to occur to a column within the frame, combined with the bending moment to which the column is subjected due to the same load combination.

NOTE: Should it be determined during detailed design that it is actually the highest moment due to another load combinat which is critical, additional design will be required. Measures to reduce the moment within the columns could also be considered, such as providing bracing, portalised bracing structures, or diaphragms between the columns. The design of these members is beyond the scope of this report.

DESIGN LOADING (from computer model)

$N_{Ed} = 4257.22$ kN
$M_{Ed} = 138.74$ kNm

DESIGN DATA:

Breadth of column	$b = 1000$ mm
Depth of column	$h = 500$ mm
Clear height of column	$L = 4.25$m
Standard Fire Resistance	REI = 90 min
Strength class of concrete	= C50/60
Characteristic strength of reinforcement	$f_{yk} = 500$ MPa

SOLUTION:

Minimum dimensions to satisfy fire requirements:

EN 1992-1-2
Clause 5.3.2(2)

$l_{0fi} \approx 0.7 \times 4.25 = 2.975\ m < 3.0m$
$e_{max} = 0.15h = 0.15 \times 500 = 75\ mm$
$e = M_{0Ed\,fi} / N_{0Ed\,fi} < e_{max}$

Clause 2.4.2

$E_{d.fi} = \eta_{fi}\, E_d$
As a simplification a recommended value of $\eta_{fi} = 0.7$ may be used

$e = M_{0Ed,fi}/N_{0Ed,fi} = M_{Ed}/N_{Ed}$
$= (138.74 \times 10^6)/(4257.22 \times 10^3)$
$= 31.65\,mm < e_{max}$

Clause 5.3.2(3) Note 1

$\mu_{fi} = N_{Ed,fi}/N_{Rd}$

The reduction factor η_{fi} may be used instead of μ_{fi} for the design load level as a safe simplification η_{fi} assumes that the column is fully loaded at normal temperature design

Table 5.2a

For R90, $b_{min}/a = 350/53$
or $b_{min}/a = 450/40$

Since $b = 1000mm > 350mm$ use $a \geq 53$
$h = 500mm > 350mm$ use $a \geq 53$

Column dimensions are adequate with respect to fire resistance.

EN 1992-1-1

Assume 8mm diameter shear links and 25mm diameter bars

$C_{min,fire} = a - \varnothing_s/2 - \varnothing_s\,link$
$= 53 - 25/2 - 8$
$= 32.5$

$C_{nom} = 35mm > C_{min,fire}$

$\therefore C_{nom} = 35mm$ is adequate with respect to fire.

Clause 5.8.3.2(3)

The effective length of compression members in braced frames is given by:

Equation (5.15)

$$L_0 = 0.5\,l \times \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) \times \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

where k_1 and k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively

$k = (\theta/M) \times (EI/L)$

PD 6687

The stiffness of the restraint beam can be taken as $2EI/L$

$\therefore (M/\theta) = (2EI/L)_{restraint\,beam}$

$k = (\theta/M) \times (EI/L)$
$= 1/(\Sigma 2EI/L)_{restraint\,beams} \times (EI/L)_{column}$

For a 500x1000 column

$$I_{column} = \frac{1000 \times 500^3}{12} = 1.04167 \times 10^{10}$$

$$(I/L)_{column} = (1.04167 \times 10^{10})/4250$$
$$= 2450980\ mm^3$$

For a 500x500 beam

$$I_{beam} = \frac{500 \times 500^3}{12} = 5208.33 \times 10^6$$

$$(\Sigma 2I/L)_{restraint\ beam} = (2 \times 5208.33 \times 10^6)/7750$$
$$= 1344086.2\ mm^3$$

EN1992-1-1 Clause 5.8.3.2(3)

Note: a minimum value of 0.1 is recommended for k

$$\therefore\ k_1 = \max\{[1/(\Sigma 2EI/L)_{restraint\ beams} \times (EI/L)_{column}];\ 0.1\}$$
$$= \max\{[(1/1344088.2) \times 2450980];\ 0.1$$
$$= 1.82$$

$$k_2 = \infty\ (\text{pinned})$$

EN1992-1-1 Equation (5.15)

$$l_0 = 0.5l \times \sqrt{\left(1 + \frac{1.82}{0.45 + 1.82}\right) \times \left(1 + \frac{\infty}{0.45 + \infty}\right)}$$

$$= 0.5L\sqrt{(1.80) \times (2)}$$

$$= 0.5L \times 1.898$$
$$= 0.949L$$
$$= 0.949 \times 4250$$
$$= 4033.25\ mm$$

NB: $\left(\frac{\infty}{0.45 + \infty}\right)$ is indeterminate, however as the worst case of $\left(\frac{k_2}{0.45 + k_2}\right)$ is tending towards 1, the value of 1 has been used, conservatively.

Clause 5.8.3.1(1)	Limiting Slenderness λ_{lim}
	$\lambda_{lim} = (20 \times A \times B \times C / \sqrt{n})$
	where:
	φ_{ef} is not known ∴ assume $A = 0.7$
	ω is not known ∴ assume $B = 1.1$
	$C = 1.7 - r_m$
	where r_m = moment ratio $= M_{01}/M_{02}$ and $\lvert M_{02}\rvert \geq \lvert M_{01}\rvert$
	∴ $M_{01} = 0$, $M_{02} = 138.74$ kNm
	∴ $r_m = 0$, and $C = 1.7$
	$n = N_{Ed}/(A_c f_{cd})$
Clause 3.1.6(1)P	$f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$
UK NA to EN 1992-1-1	α_{cc} may be taken conservatively as 0.85 for all phenomena
EN1992-1-1 Table 2.1N	For persistant and transient actions
	$\gamma_c = 1.5$ and $\gamma_s = 1.15$
	$f_{cd} = \alpha_{cc} f_{ck}/\gamma_c = (0.85 \times 50)/1.5 = 28.3$ MPa
	$n = N_{Ed}/(A_c f_{cd}) = 4257 \times 10^3/(500 \times 1000 \times 28.3)$
	$= 0.30086$
	$\lambda_{lim} = (20 \times A \times B \times C / \sqrt{n})$
	$= 20 \times 0.7 \times 1.1 \times 1.7/\sqrt{0.301}$
	$= 47.72$
	$\lambda_{actual} = l_0/i$ where $i = \sqrt{I/A}$
	$= h/\sqrt{12}$
	$= 500/\sqrt{12}$
	$= 144.33$ mm
	$\lambda_{actual} = 4033.25/144.33$
	$= 27.945$
Clause 5.8.3.1(1)	Since $\lambda_{actual} < \lambda_{lim}$ the column is not slender and 2nd order moments do not have to be considered.
Clause 5.8.8	Method based on nominal curvature.
Clause 5.8.8.2(1)	Design moment $M_{Ed} = M_{0Ed} + M_2$
	M_{0Ed} is the 1st order moment including the effect of imperfections.
	$M_2 = 0$ (i.e no 2nd order moment)
Clause 5.2(9)	For an isolated column in a braced system, e = accidental eccentricity

$e = l_0/400 = 4033.25/400 = 10.08\,mm$

Clause 6.1(4)

$\geq e_0 \geq h/30 = \frac{500}{30} = 16.67\,mm$

$\geq 20\,mm$

$e = 20\,mm$

$N_{Ed} = 4257.22\,kN$
$M_{Ed} = [(138.74) + (4257.22 \times 0.02)]$
$= 218.88\,kNm$

$\frac{M_{Ed}}{bh^2 f_{ck}} = \frac{218.88 \times 10^6}{1000 \times 500^2 \times 50} = 0.0175$

$\frac{N_{Ed}}{bh\,f_{ck}} = \frac{4257.22 \times 10^3}{1000 \times 500 \times 50} = 0.17$

$\frac{d_2}{h} = \frac{(35 + 8 + 25/2)}{500} = 0.111$

Use the design chart for $d^2/h = 0.15$
(Attached - can be sourced from:
www.concretecentre.com/pdf/ccip_concise_column_graphs_extract.pdf)

From design chart, $\frac{A_s f_{yk}}{bh f_{ck}} \approx 0$
which would make $A_s = 0$

Therefore assume: $\frac{A_s f_{yk}}{bh f_{ck}} = 0.1$

$\therefore A_s = \frac{0.1 \times 1000 \times 500 \times 50}{500}$
$= 5000\,mm^2$

Clause 9.5.2(2)

$A_{s,min} = \frac{0.1 N_{Ed}}{f_{yd}} = \frac{0.1 \times 4257.22 \times 10^3}{(500/1.15)}$
$= 979\,mm^2$

$\geq 0.002 A_c = 0.002 \times 500 \times 1000 = 1000$

$\therefore A_{s,min} = 1000\,mm^2$

	Project Details:	Date:	Entry Reference:
mpa The Concrete Centre	**Structural Concrete 2011**	24/6/11	**11-001**
	Southfield Grandstand	Page: 28	Of: 33

UK NA to EN 1992-1-1
Clause 9.5.2(3)

$A_{s,max} = 0.04\,A_c$
$= 0.04 \times 500 \times 1000$
$= 20000\,mm^2$

$A_{s,min} < A_{s,required} < A_{s,max}$

Clause 9.5.2(1)

Minimum diameter of longitudinal bars $= 12\,mm$

ADOPT 11 H25 BARS ($5400\,mm^2$) LONGITUDINALLY

EN 1992-1-1
Clause 9.5.3(1)

Link diameter $\geq 6\,mm \geq \varnothing_{longitudinal}/6$
$= 25/6 = 6.25\,mm$

$\therefore$ Use 8mm dia links

UK NA to EN 1992-1-1
Clause 9.5.3(3)

Maximum spacing of links $S_{cl,tmax}$
– Use the recommended value
$S_{cl,tmax} \leq 20 \times \min \varnothing_{longitudinal} = 500\,mm$
$\leq$ lesser dimension of column $= 500\,mm$
$\leq 400\,mm$

$\therefore$ Use 300mm spacing

ADOPT H8 LINKS @ 300 mm CENTRES

Project Details:	Date:	Entry Reference:
Structural Concrete 2011	24/6/11	**11-001**
Southfield Grandstand	Page: 29	Of: 33

DESIGN OF REINFORCED CONCRETE SHEAR WALLS

For the purpose of preliminary design, the shear walls have been designed to withstand the worst case vertical load, i.e. from the T-beams above the club room at ULS. During detailed design, the shear walls should be checked in order to ensure that they can withstand any shear loads acting horizontally due to wind or another action in the lateral direction, due to the fact that the shear walls are to act as a diaphragm between frames.
The bases of the shear walls are to be fully fixed to the ground beams. The top of the shear walls are to have a pinned connection to the T-beams, ensuring that there are no applied moments about the minor axis (other than accidental moments due to eccentricity).

NOTE: As above, during detailed design the shear walls should be checked in order to ensure that they can act as a suitable diaphragm. This is beyond the scope of this preliminary design

THIS DESIGN IS PRESENTED AS A SUMMARY AND SHOULD BE READ IN CONJUNCTION WITH THE "DESIGN OF REINFORCED CONCRETE COLUMNS" CALCULATIONS FOR DETAILS OF TABLES AND CLAUSES USED. DEVIATIONS FROM THE COLUMN PROCEDURE SHALL BE CLEARLY IDENTIFIED.

DESIGN LOADING:

N_{Ed} = 113.5 kN/m (from T-beam design shear load + wall above [$1m \times 0.2m \times 25 kN/m^3 \times 4.5m \times 1.35$])

DESIGN DATA:

Design for 1m run of wall $b = 1000mm$
Thickness of wall $h = 200mm$
Clear height of wall $L = 4.25m$
Standard Fire Resistance REI = 90 min
Strength class of concrete = C50/60
Characteristic strength of reinforcement $f_{yk} = 500 MPa$

SOLUTION:

Minimum dimensions to satisfy fire requirements:
EN 1992-1-2

$REI = 90$
$\mu_{fi} = \eta_{fi} = 0.70$

$b/a = 170/25$

(DEVIATION FROM COLUMN DESIGN PROCEDURE!)

Table 5.4 — Walls exposed both sides:

$C_{min,fire} = a - \varnothing_s/2 - \varnothing_{s\ transverse}$
$= 25 - 25/2 - 8$
$= 4.5mm$

$C_{nom} > C_{min,fire}$
$35 > 4.5$
$h = 200 > 170$
∴ Dimensions are adequate with respect to fire.

DETERMINE THE EFFECTIVE LENGTH:
(DEVIATION FROM COLUMN PROCEDURE)

EN 1991-1-1
Figure 5.7 — $l_0 = 0.7l$ (pinned - fixed)

∴ $l_0 = 0.7 \times 4250$
$= 2975mm$

Limiting Slenderness:

$A = 0.7$
$B = 1.1$
$C = 1.7 - r_m$ where $r_m = 1$
$= 0.7$
$n = N_{Ed}/(A_c f_{cd})$
$= 0.020$

$f_{cd} = 28.3MPa$

$\lambda_{lim} = 20 \times A \times B \times C / \sqrt{n} = 76.23$

	Project Details:	Date:	Entry Reference:
mpa The Concrete Centre	**Structural Concrete 2011** **Southfield Grandstand**	24/6/11	**11-001**
		Page: 31	Of: 33

$\lambda_{actual} = l_0 / i$

where: $i = h / \sqrt{12}$
$= 200 / \sqrt{12}$
$= 57.7 \text{ mm}$

$\lambda_{actual} = 2975 / 57.7$
$= 51.56$

$\lambda_{actual} < \lambda_{lim}$ ∴ NOT SLENDER
∴ No need to consider 2nd order moments

Method based on nominal curvature:

$M_{Ed} = M_{0Ed} + M_2$, where $M_2 = 0$
$e = l_0/400 = 2975\text{mm}/400$
$= 7.4\text{mm}$
$\geq e_0 \geq h/30 = 6.67\text{ mm}$
$\geq 20\text{mm}$

∴ $e = 20\text{mm}$

$N_{Ed} = 113.5 \text{ kN}$

$M_{Ed} = 113.5 \times 0.02$
$= 2.27 \text{ kNm}$

$$\frac{M_{Ed}}{b h^2 f_{ck}} = \frac{2.27 \times 10^6}{1000 \times 200^2 \times 50} = 0.00135$$

$$\frac{N_{Ed}}{b h \, f_{ck}} = \frac{113.5 \times 10^3}{1000 \times 200 \times 50} = 0.01135$$

$$\frac{d_2}{h} = \frac{(35 + 8 + 25/2)}{200} = 0.2775$$

∴ Use design chart for $d_2/h = 0.25$ (attached) (see page 33)

From design chart: $\dfrac{A_s f_{yk}}{b h f_{ck}} = 0$

which would make $A_s = 0$

∴ Assume $\dfrac{A_s f_{yk}}{b h f_{ck}} = 0.1$

$$A_s = \frac{0.1 \times 1000 \times 200 \times 50}{500}$$
$$= 2000\,mm^2$$

$$A_{s,min} = \frac{0.1 N_{Ed}}{f_{yd}} = \frac{0.1 \times 113.5 \times 10^3}{(500/1.15)} = 26.105\,mm^2$$

$$\geq 0.002 A_c = 0.002 \times 200 \times 1000 = 400\,mm^2$$

$$\therefore A_{s\,min} = 400\,mm^2$$

$$A_{s\,max} = 0.04 A_c$$
$$= 0.04 \times 200 \times 1000 = 8000$$

$$A_{s\,min} < A_{s\,required} < A_{s\,max}$$

Min $\varnothing$ = 12mm

ADOPT H25 BARS @ 200mm CENTRES ($2454\,mm^2/m$) LONGITUDINALLY ALONG LENGTH OF WALL (VERTICAL BARS AT LONGITUDINAL CENTRES)

Determine transverse reinforcement:

Link diameter $\geq 6 \geq \varnothing_{longitudinal}/6 = 6.25$

$\therefore$ Use 8mm$\varnothing$ transverse reinforcement

Max spacing, $S_{cl\,max}$ $\leq 20 \times \varnothing_{longitudinal} = 500mm$
$\leq$ lesser dimension of wall = 200mm
$\leq$ 400mm

} 200mm

$\therefore$ Use 200mm Spacing.

ADOPT H8 TRANSVERE REINFORCEMENT @ 200mm CENTRES (May be subject to change during detailed design in order to resist lateral loading)

mpa The Concrete Centre	Project Details: Structural Concrete 2011 / Southfield Grandstand	Date: 24/6/11	Entry Reference: 11-001
		Page: 33	Of: 33

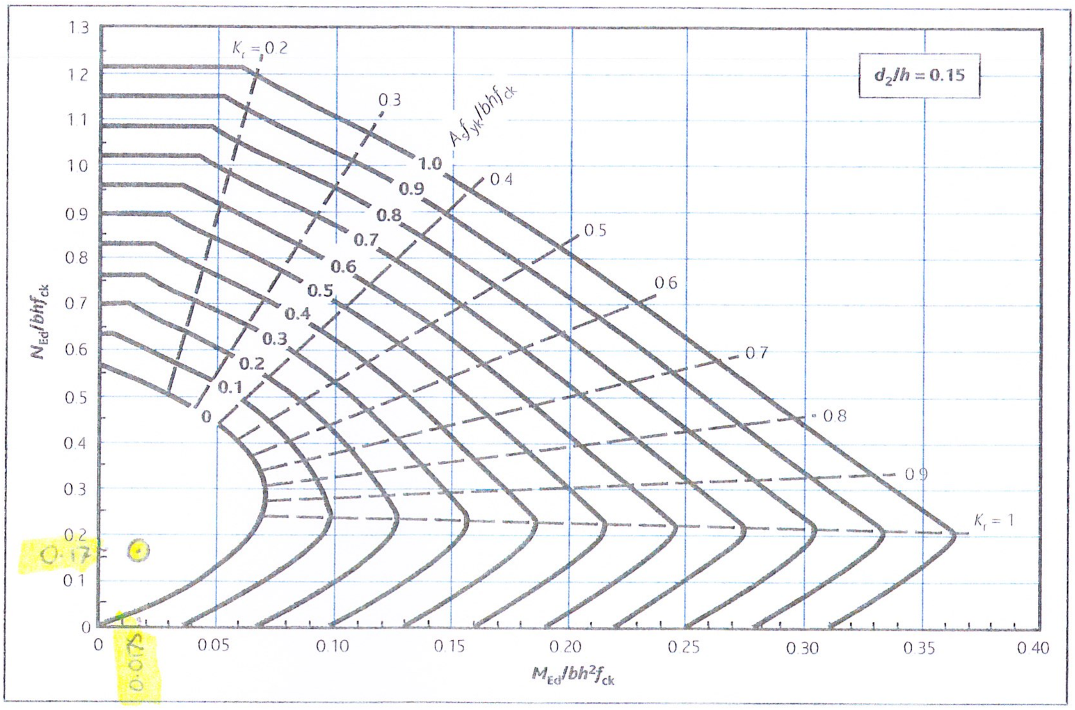


Figure 15.5c)
Rectangular columns $d_2/h = 0.15$

DESIGN CHART FOR COLUMN DESIGN

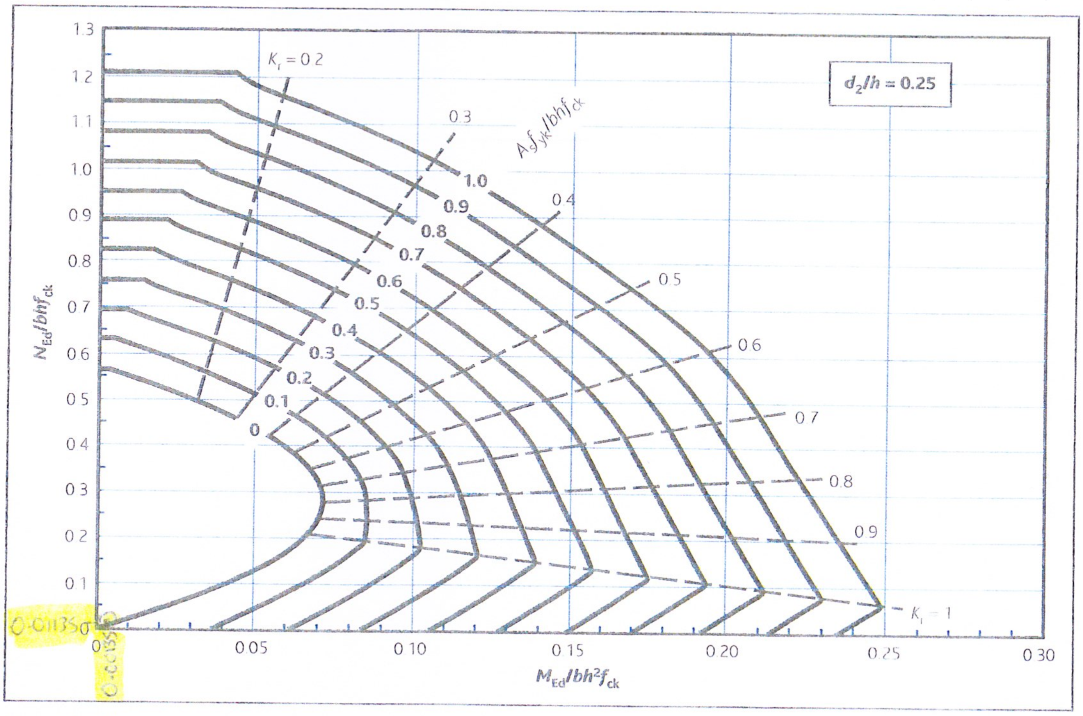
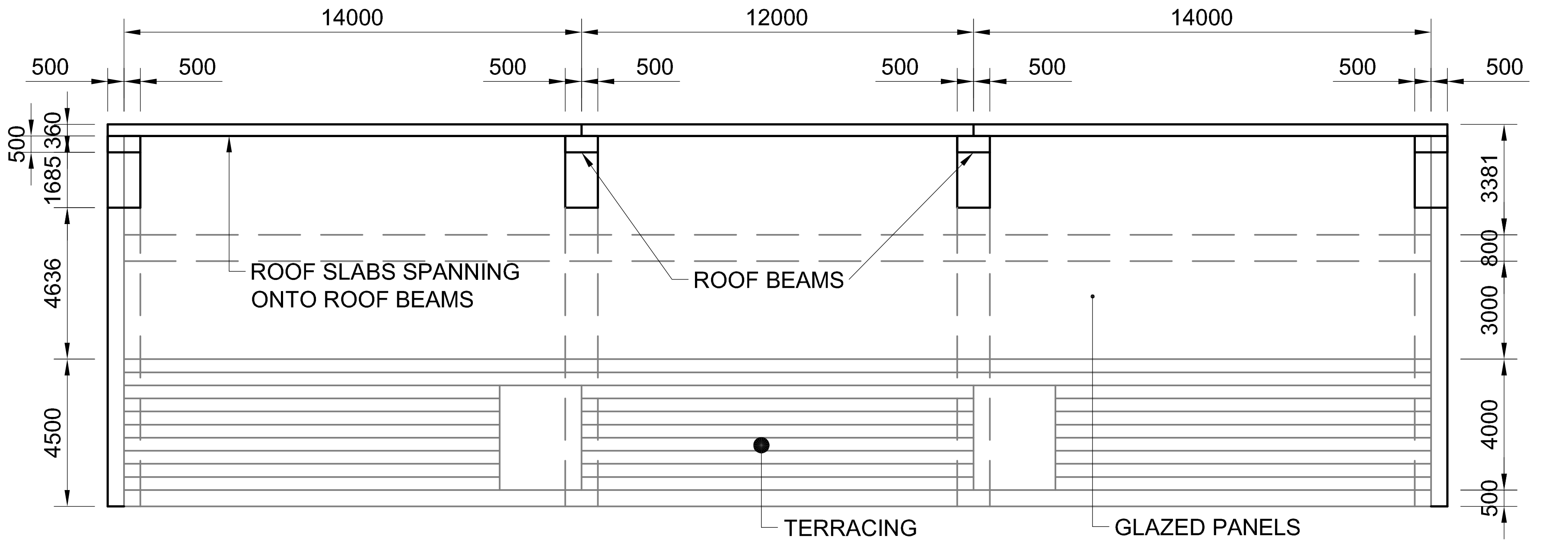


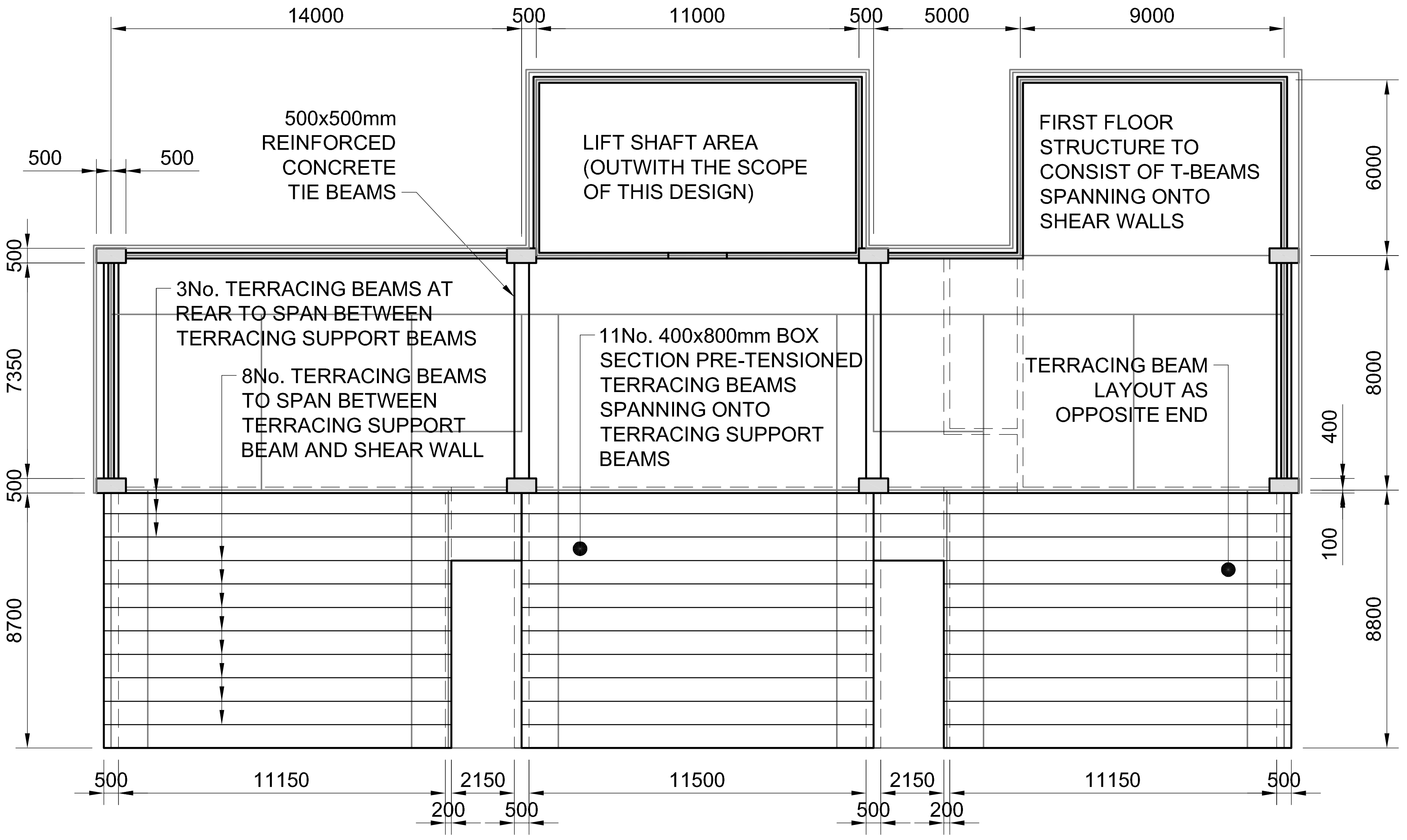
Figure 15.5e)
Rectangular columns $d_2/h = 0.25$

DESIGN CHART FOR SHEAR WALL DESIGN

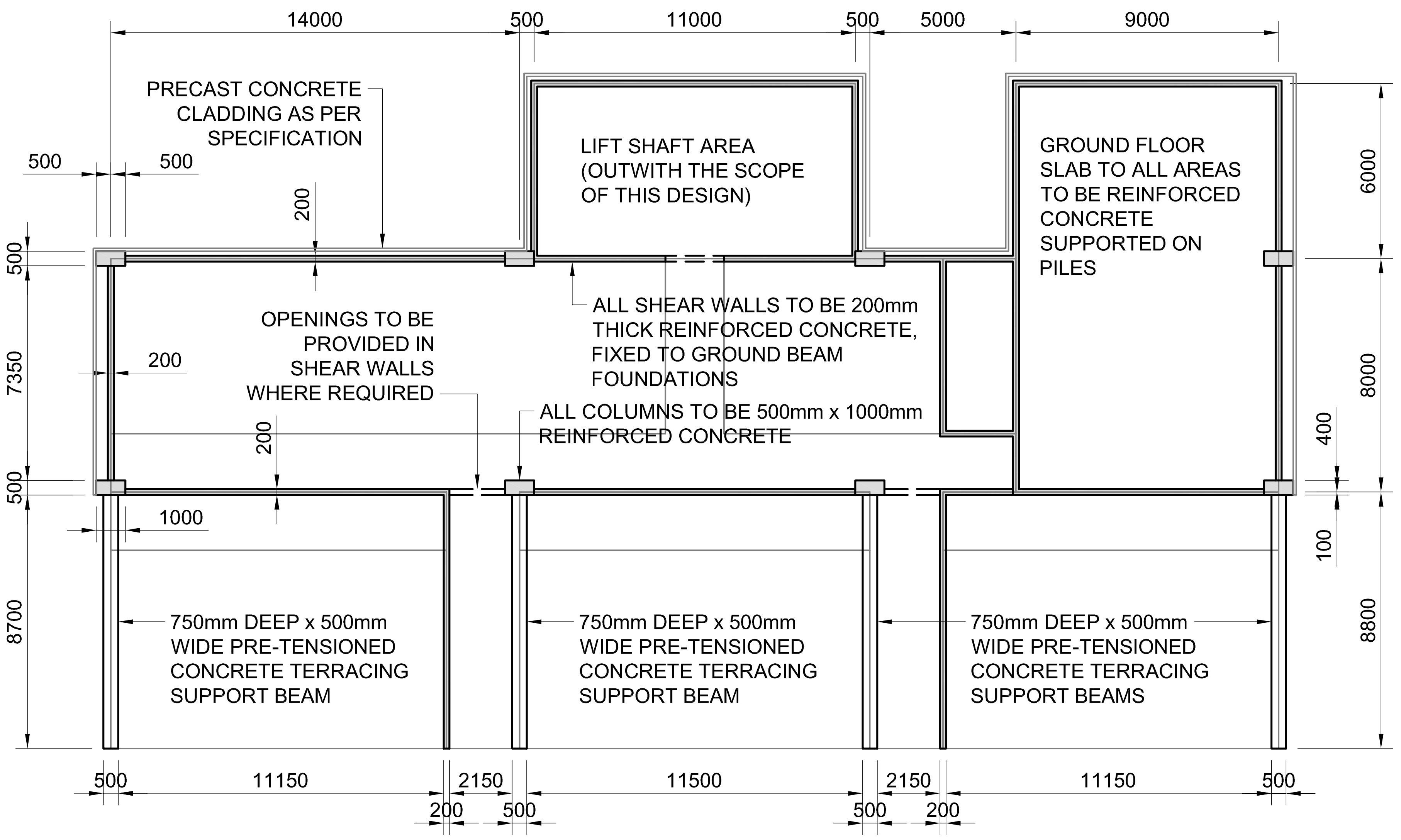
APPENDIX 2: DRAWINGS



ELEVATION 1:200

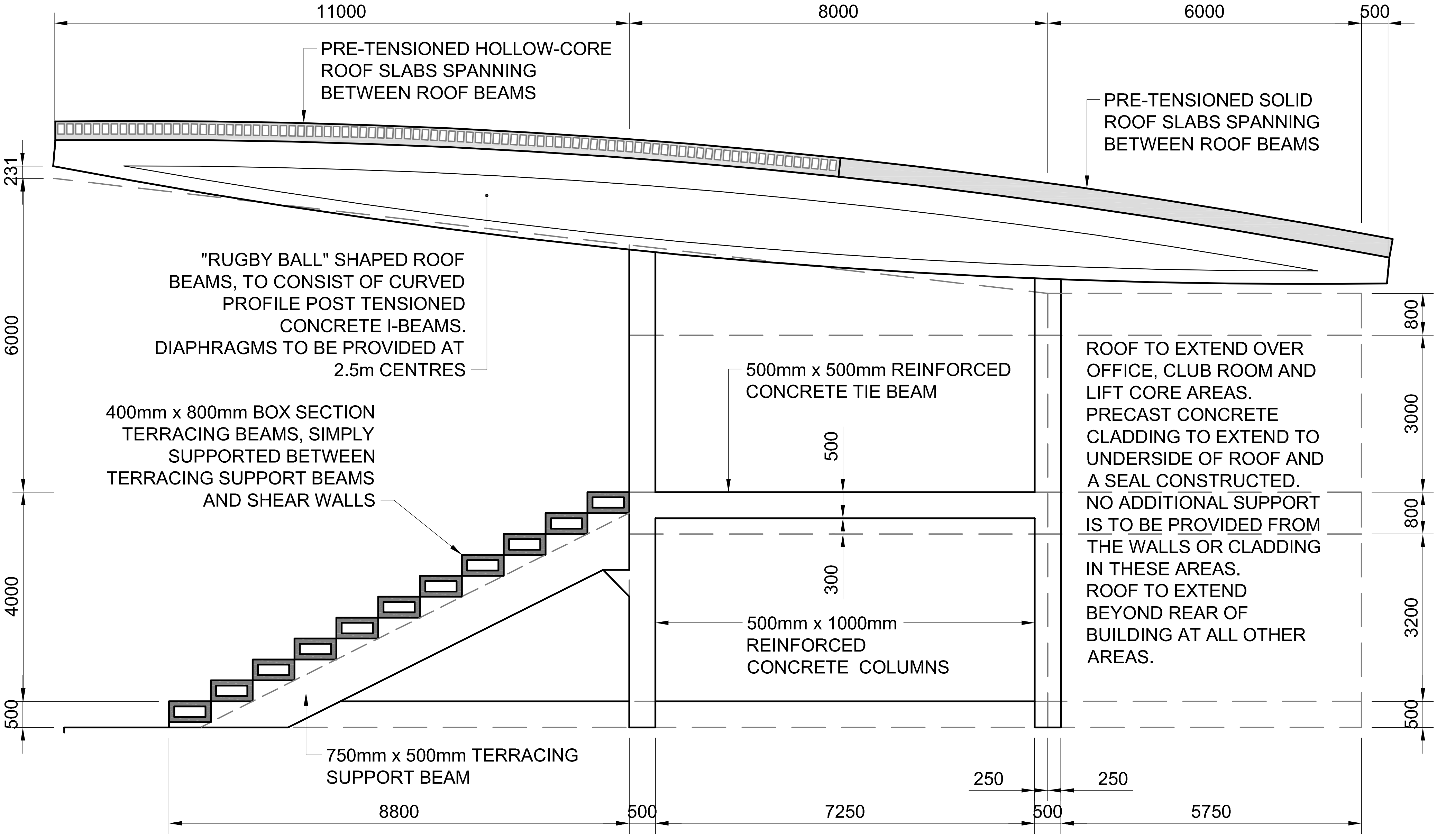


FIRST FLOOR PLAN 1:200

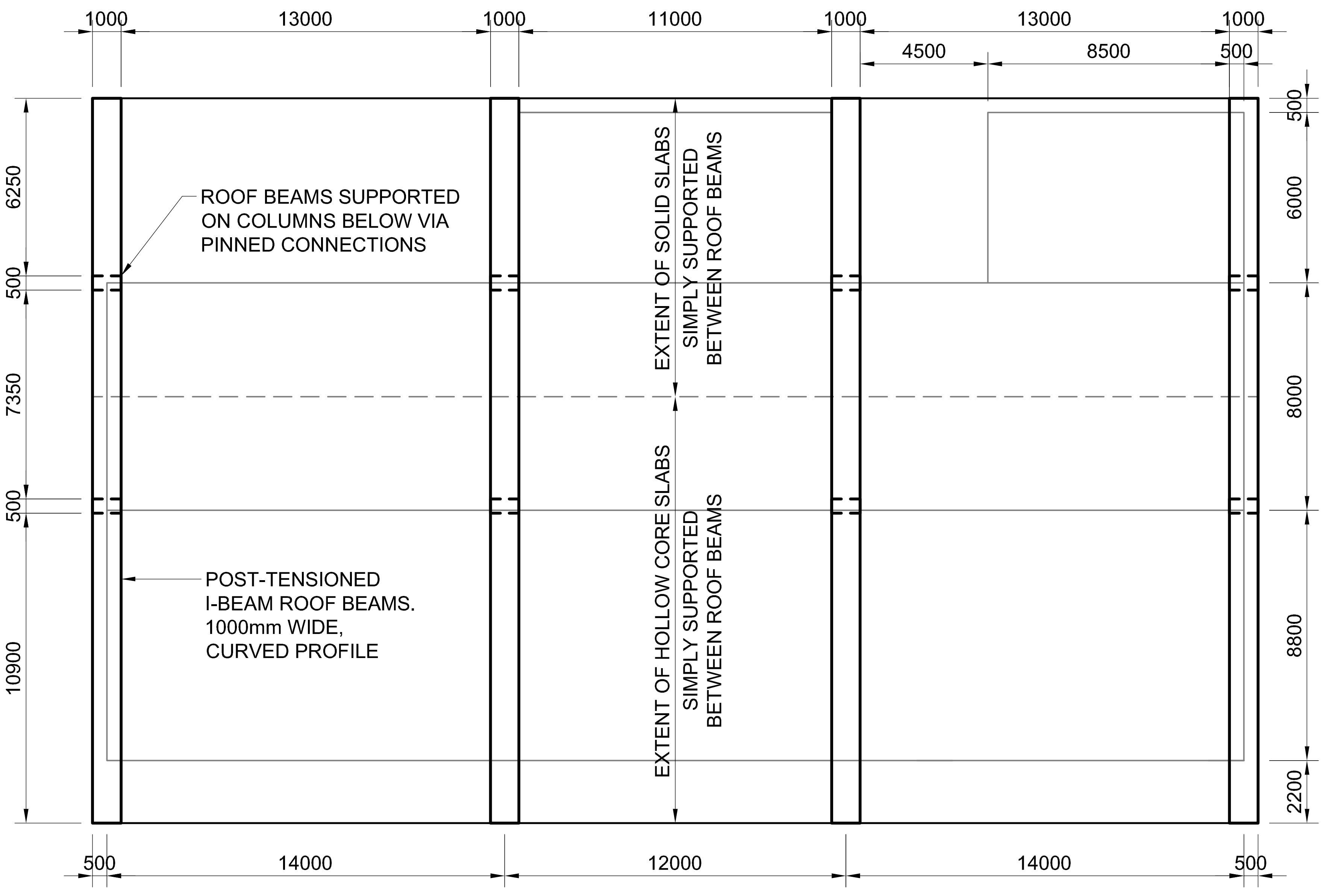


GROUND FLOOR PLAN 1:200

Entry No. 11-001
Appendix 2 Drawing 1

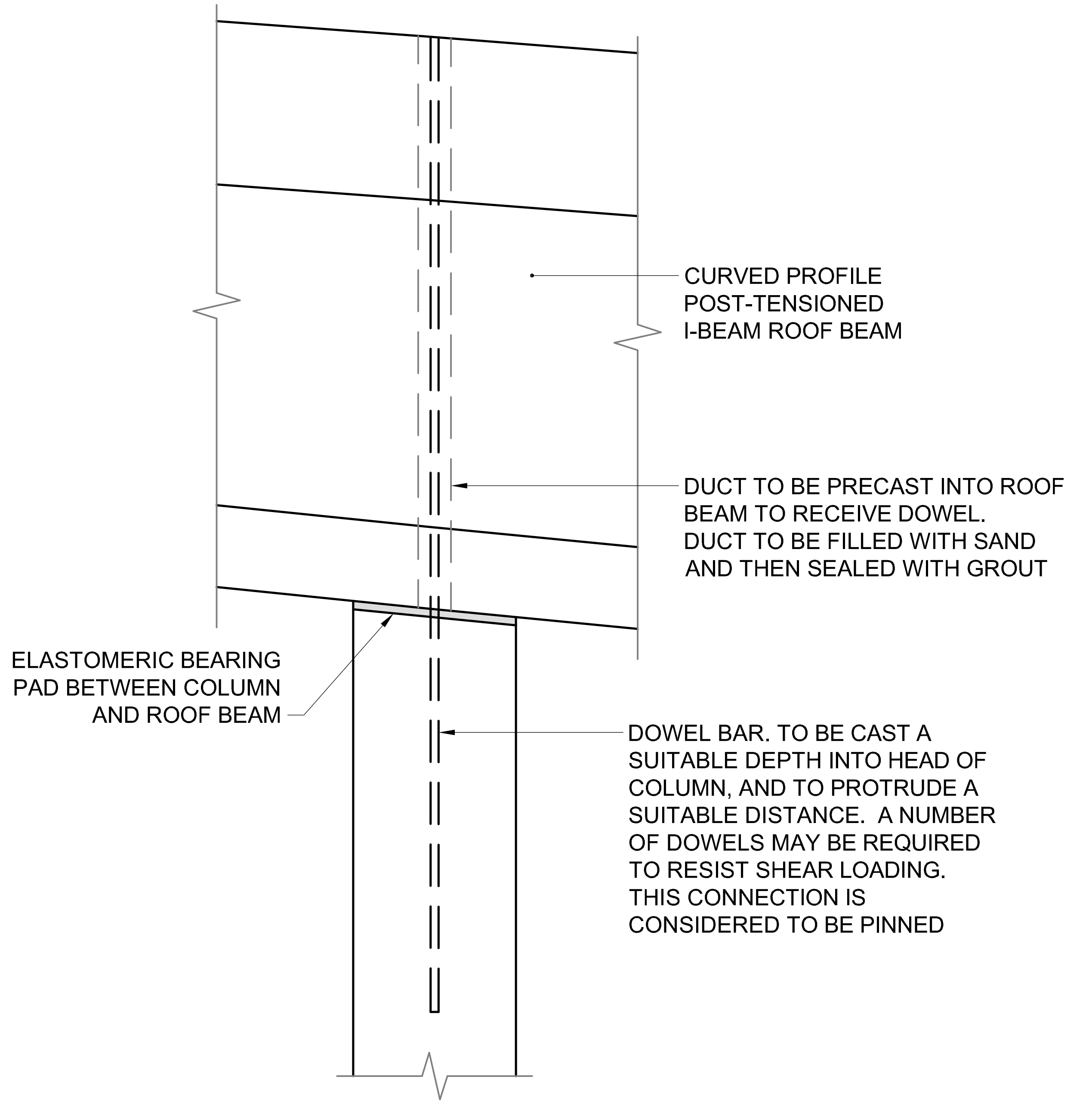


SECTION 1:100

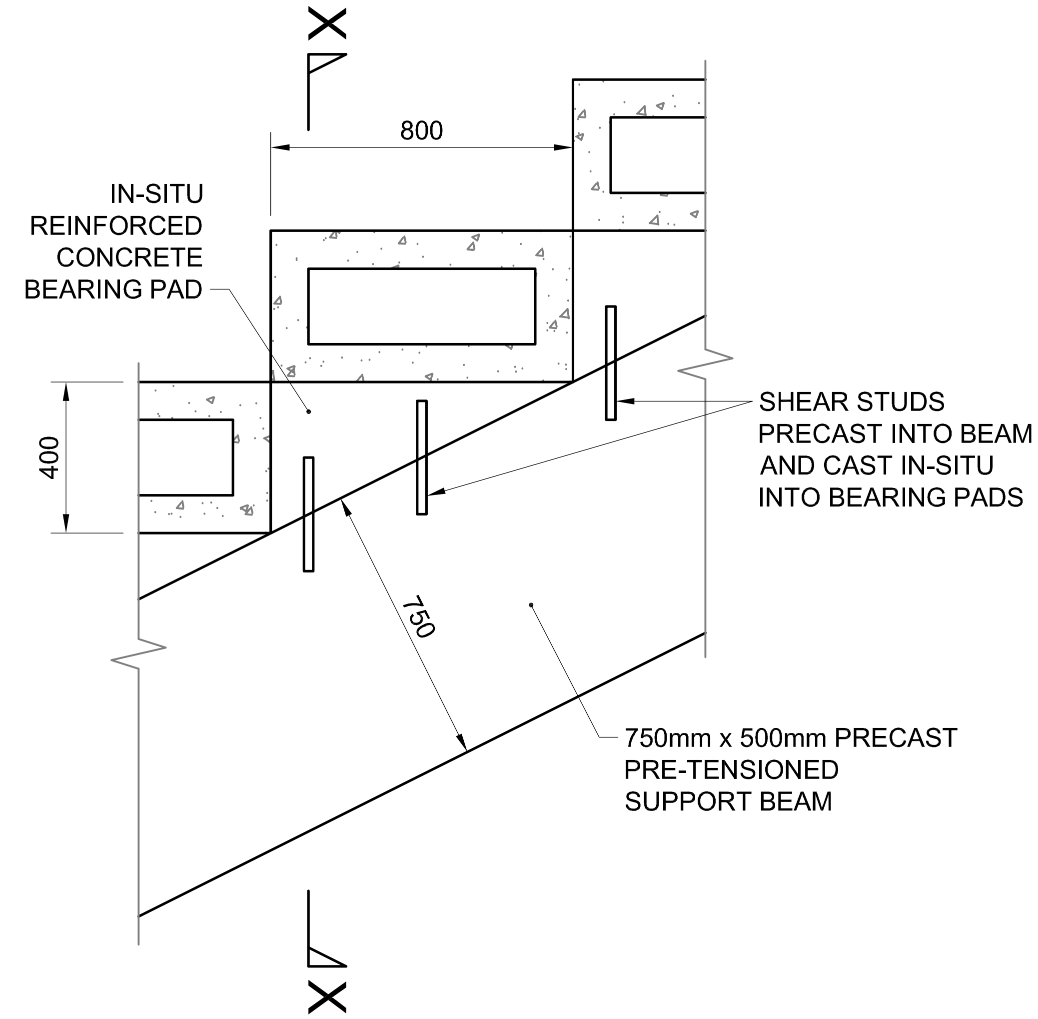


ROOF PLAN 1:200

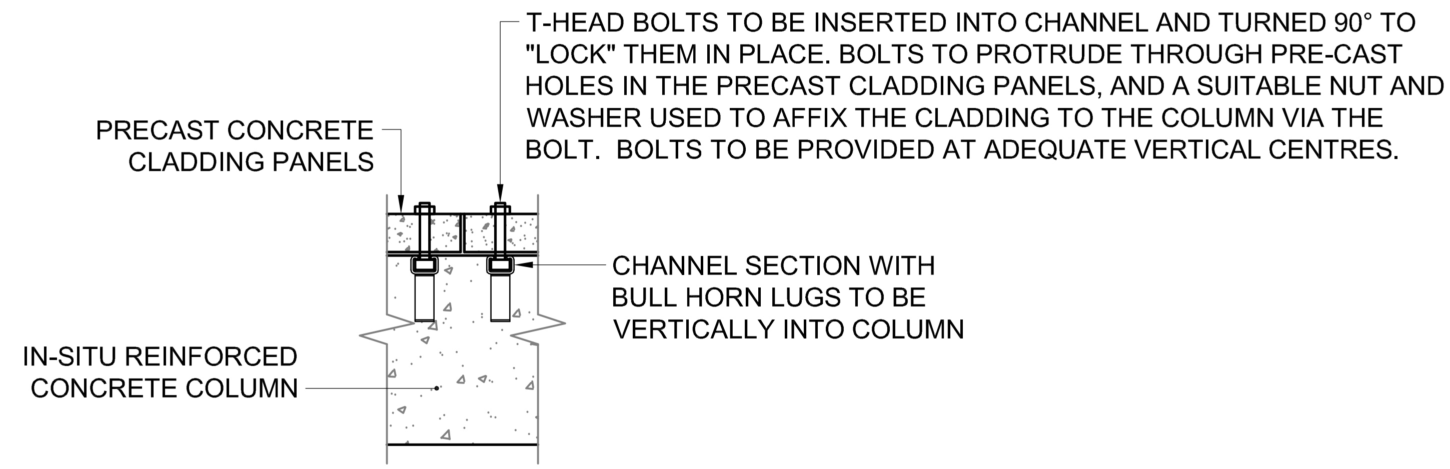
Entry No. 11-001
Appendix 2 Drawing 2



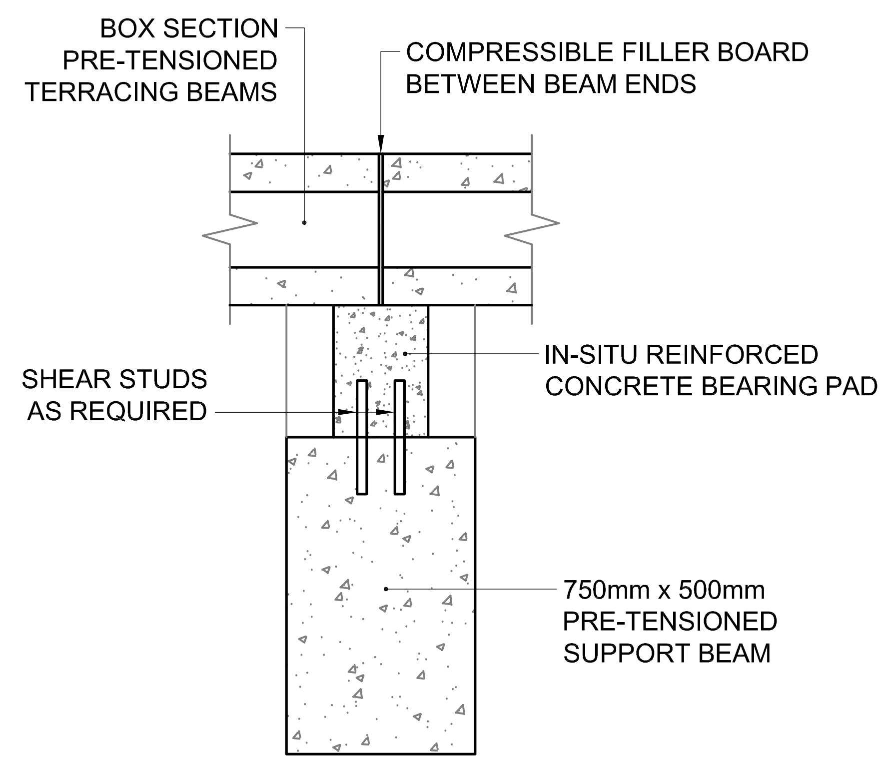
DETAIL a) JUNCTION BETWEEN ROOF BEAM AND SUPPORTING STRUCTURE 1:20



DETAIL b) CONNECTION BETWEEN THE SEATING SECTION AND THE SUPPORTING SECTION 1:20



DETAIL c) FIXING ARRANGEMENT FOR A PRECAST CONCRETE CLADDING PANEL 1:20



SECTION X-X 1:20

Entry No. 11-001
Appendix 2 Drawing 3

APPENDIX 3: SUSTAINABILITY PLAN

USER MANUAL

This user manual has been developed in order to ensure that the structure is maintained in a serviceable condition. This will ensure sustainable construction by identifying and addressing any problems before they become too extensive, thus reducing the amount of repairs required. This will also help control the life cycle costs of the structure. In order to achieve the maximum benefit from this user manual, the findings of all inspections carried out should be recorded and collated. It may be useful for an individual within the client's organization to be responsible for this information. It is recommended that a computerized database be established with all inspection information.

The recommended inspection scheme is as follows:

- Initial full inspection noting any defects from construction. To be carried out by Engineer.
  (Once)
- Full in-depth inspection and documentation of the following to be carried out by Engineer:
  (Every 2 years)
  Look for cracks, corrosion, spalling, water seepage, damage, etc.
    - Prestressed concrete elements
    - Reinforced concrete elements
    - Joints/connections
    - Any exposed connections to foundations
    - Any exposed foundations
- Interim inspections of high stress elements and problem areas. To be carried out by Engineer.
  (Every year)
- Inspections following catastrophic events or natural disasters. To be carried out by Engineer.
  (Determined by client)
- Inspections following any modifications to the structure. To be carried out by Engineer.
  (Determined by client)
- Detailed investigation of any defects, including non-destructive testing of the concrete, concrete sample testing, or further investigation. To be carried out by specialist inspector.
  (Determined by Engineer)

- Routine inspection. Report any defects or monitor known defects. Monitor and document crack widths and lengths in the concrete. May be carried out by inspector or assistant who may be a member of the on-site staff.
  (Every 6 months)
- Remove debris from the structure, preventing deterioration. To be carried out by on-site staff.
  (Monthly)
- On-site staff should report any defects immediately to client.
  (Daily)

Health and Safety should be considered at all times during any inspection, investigation or repair works. Access to working areas should be restricted and inaccessible to members of the public. Where possible, inspections should take place when the public do not have access to the building. Any non-urgent repair works or modifications should be scheduled during the rugby close season. Any urgent repair works or modifications should be carried out in accordance with the Health and Safety advice given above, and if possible, the building be closed to the general public during the works.