Possible solution to past CM examination question

Question 3 - April 2010

Footbridge

by Saprava Bhattacharya

Footbridge question

Client's requirements

1. A new footbridge is required to cross a major urban highway to provide access to a commercial centre: see Fig. Q3
2. The footbridge is to cross the highway at an angle of 30 degrees. At the east end of the bridge a ramp is required to descend to ground level. Provision is to be made for a future extension of the bridge further to the east.
3. No loading may be transferred from the footbridge to the commercial centre building and an expansion joint is required at this junction. Column supports to the footbridge are permitted only within the highway planting strips and the central carriageway divider. No columns are permitted under the east end of the bridge.
4. The maximum permitted gradient of the ramp is 1:12. Horizontal landings are required in the ramp at vertical intervals of not more than 3.5m, and the length of each landing must be not less than 2.0 m.
5. A 1.0m high parapet is required for both the footbridge and the ramp. The clear widths of the footbridge and ramp are to be 6.0m and 4.0m respectively.
6. A minimum clearance of 0.8m is required from the edge of carriageway to the face of any structure. The minimum required headroom under the footbridge is 5.1m above the carriageway level.
7. Temporary access to the highway carriageways is available each night between midnight and 5:00am.

Imposed loading

8. Footbridge loading 5.0kN/m^2

Site conditions

9. The site is located in the centre of a city. Basic wind speed is 46m/s based on a 3 second gust; the equivalent mean hourly wind speed is 23m/s.
10. Ground Conditions Ground level - 0.5m Made ground 0.5m – 30.0m Sandstone. Allowable bearing pressure 1000kN/m^2

Omit from consideration

11. Detailed structural design of the footings for the ramp.

SECTION 1 ***(50 marks)***

a. Prepare a design appraisal with appropriate sketches indicating two distinct and viable solutions for the proposed structure including the foundations. Indicate clearly the functional framing, load transfer and stability aspects of each scheme. Identify the solution you recommend, giving reasons for your choice. (40 marks)

b. After the design has been completed, the client advises that he wishes to create a garden under the west end of the footbridge and wishes to avoid any columns in this area (shown as the dotted line in Figure Q3). Write a letter to the client explaining how your design could be modified to accommodate this change. (10 marks)

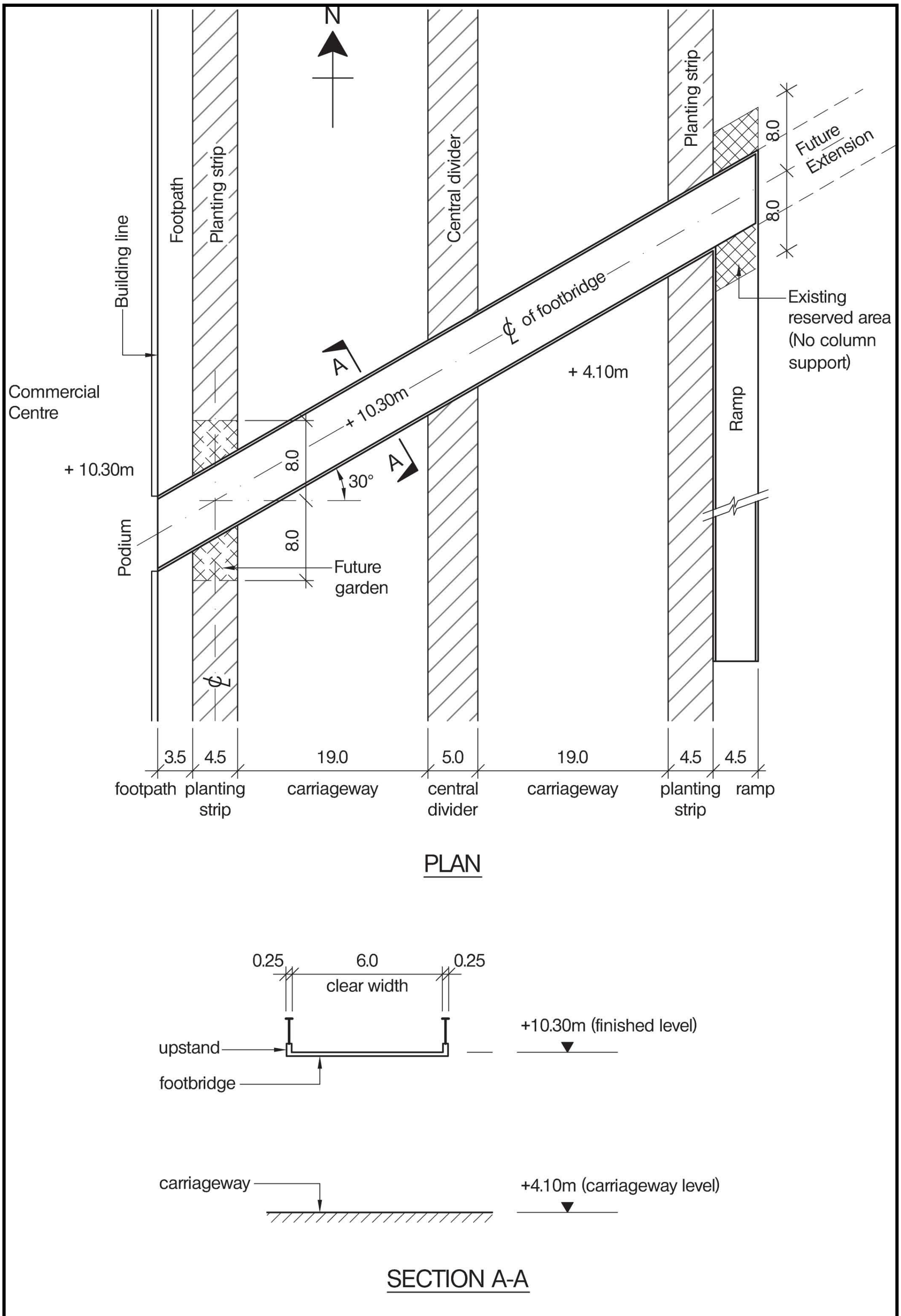
SECTION 2 ***(50 marks)***

For the solution recommended in Section 1(a):

c. Prepare sufficient design calculations to establish the form and size of all the principal structural elements including the bridge foundations and the ramp. (20 marks)

d. Prepare general arrangement plans, sections and elevations to show the dimensions, layout and disposition of the structural elements and critical details for estimating purposes. (20 marks)

e. Prepare a detailed method statement for the safe construction of the footbridge and its ramp and an outline construction programme. (10 marks)



NOTE: All dimensions are in metres

FIGURE Q3

Introduction:

Initial response to this question reminds any bridge engineer the footbridges over busy motorway or trunk road in urban areas of various part of UK. For example Ely Road footbridge in London over A406 as shown in Figure A & B (Footbridge at Hong Kong):

Figure A: Ely Road footbridge over A406 North Circular next to the shopping centres.

Figure B: Footbridge at Hong Kong

Understanding the question and visualisation of the site in three dimensions is the most important step to solve the problem. An imaginary view of the site is shown in the Figure C below:

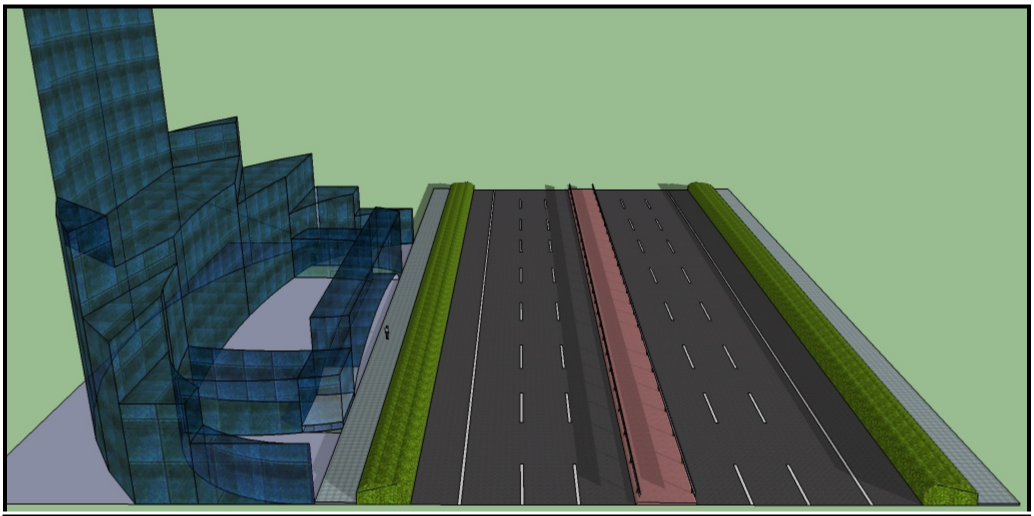


Figure C: An imaginary three dimensional view of the site as in question

Key observations from Client's requirement => *constraints*

- Major urban highway, temporary access to the highway carriageways is available each night between midnight and 5:00am :
  - *Least (as above) / no disruption acceptable*
  - *Prefabricated / Pre-cast form of deck only acceptable*
  - *Cast in situ deck only possible if permanent formwork is used*
- 30° Skewed Alignment:
  - *Detailing is affected throughout, especially at the connection with ramp*
  - *Effect of skew on the proposed form of superstructure's behaviour.*
- No load to be transmitted to the commercial building:
  - *Superstructure must be cantilevered / overhanged from either end supports*
  - *Expansion joint at west end shall be designed to accommodate the possible movement*
- Column supports to the footbridge are permitted only within the highway planting strips and the central carriageway divider with a minimum clearance of 0.8m is required from the edge of carriageway to the face of any structure:
  - *Limited space for substructure & foundation*
  - *Lighter superstructure is more appropriate to avoid skew effect on the substructure.*
- The clear widths of the footbridge and ramp are to be 6.0m and 4.0m respectively:
  - *Much wider than UK's standard width for foot bridges and approach ramps*
  - *Omission of central support using a longer superstructure will be extremely difficult.*
  - *Transport of prefabricated superstructure shall be planned since design stage.*
  - *Erection of the major components of the superstructure shall be considered in design.*
- The minimum required headroom under the footbridge is 5.1m above the carriageway level. The maximum permitted gradient of the ramp is 1:12. Horizontal landings are required in the ramp at vertical intervals of not more than 3.5m, and the length of each landing must be not less than 2.0 m.
  - *It is lesser than UK standard of 5.7m, but for a ramp in 1:12, less than 70m long with one landing would be required.*

- *Considering the width of each carriageway, spans are to be determined and their allowable deflection limit will have to be added on top of the permissible headroom.*

All the above points are better visualised by the following three dimensional figure D

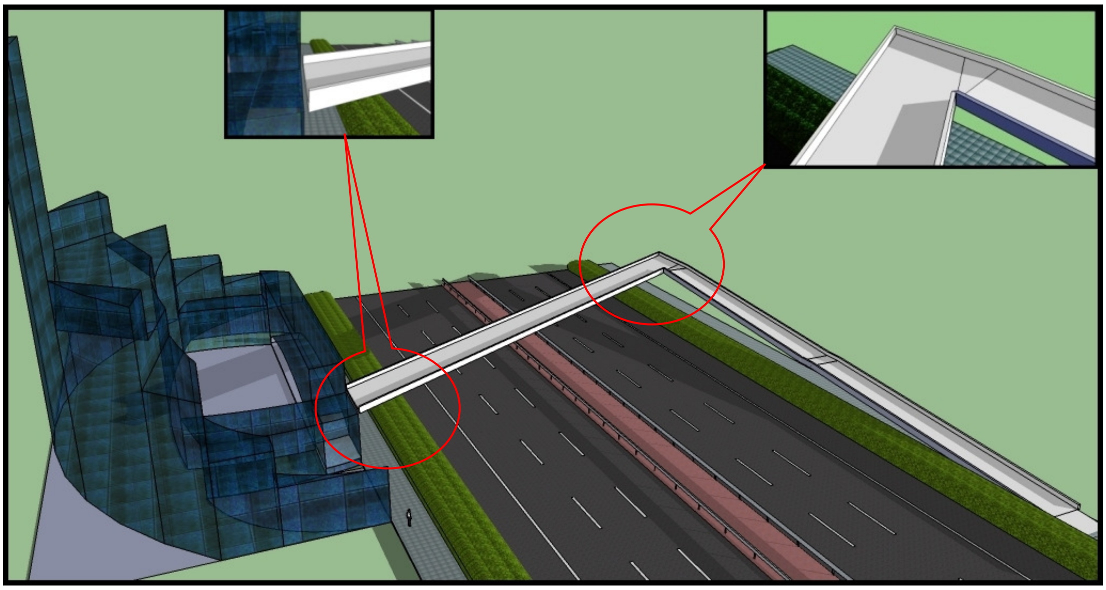


Figure D: A 3D representation of the client's requirements leading to the constraints

A complete three dimensional visualisation of what is described in the question and particularly the client's requirements are the most important to the person whoever will attempt it. Any solution is put forward has to be cross checked to all the above constraints.

<u>The most appropriate two distinct viable solutions possible for this problem:</u>

For the two distinct and viable solutions there are many examples for the structural forms in the common design offices. Cast in situ deck slab (using pre-cast formwork) on top of simple or continuous beams or even simple through truss, overhanged at either side to meet client's requirement should be the most appropriate solutions since there is no aesthetic requirement from client. However both the solutions shall be designed and detailed keeping all the above constraints in consideration.

Solution 1:

Cast in situ deck slab on top of continuous steel beams as shown in Figure E below:

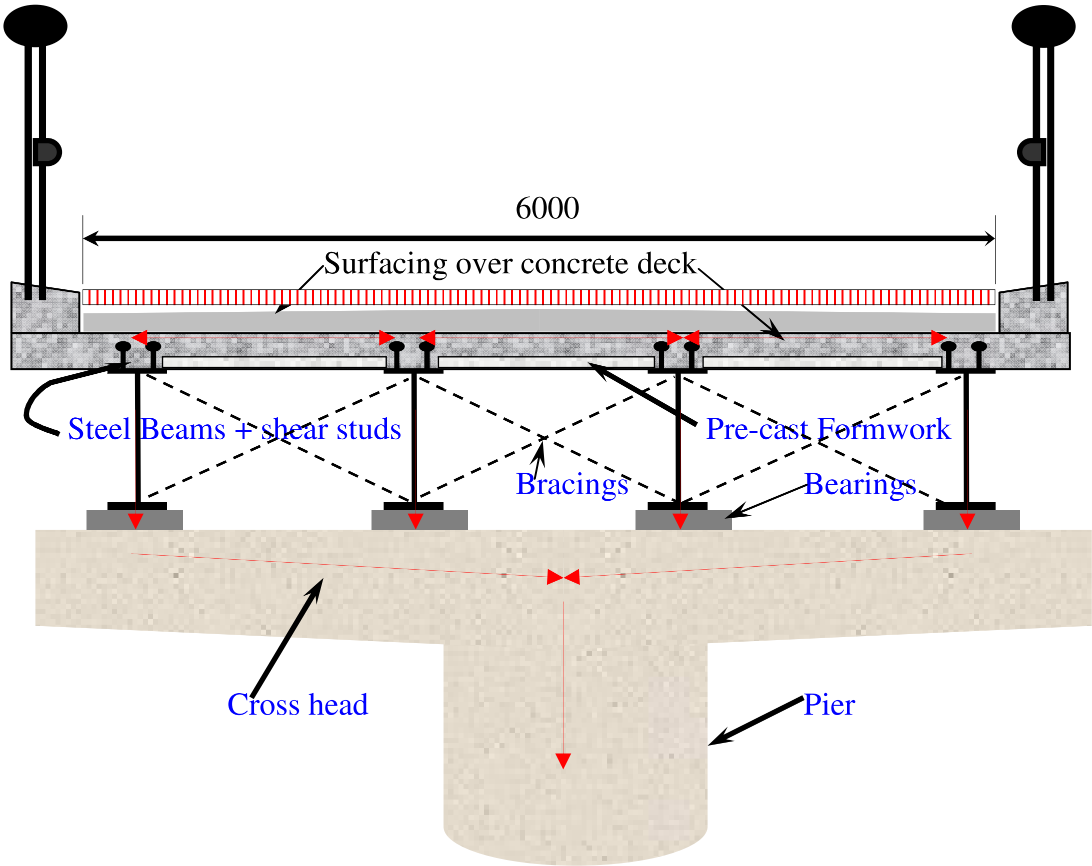


Figure E: Typical cross section of the solution 1 with load & load path

This is the simplest possible solution for the given problem. In the table below it is shown how it overcomes the above mentioned constrains

List of Constraints	**Ways to overcome them**
Least / no disruption Prefabricated / Pre-cast form of deck Cast in situ deck on permanent formwork	Erection of pair of pre fabricated girders and casting of slab on top of the pre-cast formwork will minimise disruption.
Detailing to accommodate 30 degree skew Superstructure's behaviour due to the skew. Superstructure to be cantilevered / overhanged No load to be transferred to the Commercial building.	Beams used underside of deck to accommodate the triangular portion to match the ramp slope. Skew induced behaviour is dealt with by pair of steel composite plate girders. Provision of expansion joint between the cantilever end and building will ensure no load is transferred to the building.

Limited space for substructure & foundation Lighter superstructure is more appropriate to avoid skew effect on the substructure.	Single circular pier with cross head supporting the light weight superstructure will be least affected by the skewed superstructure.
6m wide foot bridge and 4m wide ramp Omission of central support using a longer superstructure will be extremely difficult. Transport of prefabricated superstructure shall be planned since design stage. Erection of the major components of the superstructure shall be considered in design.	Two pairs of girders for footbridge and a pair of girders for ramp are enough for the spans. Use of central pier support and the introduction of splice at the point of contra-flexure at either side, will make it easier for transportation and erection of prefabricated steel girders leading to a very simple solution.

Solution 2:

Cast in situ deck slab through simply supported Warren Truss Girders as shown in Figure F below:

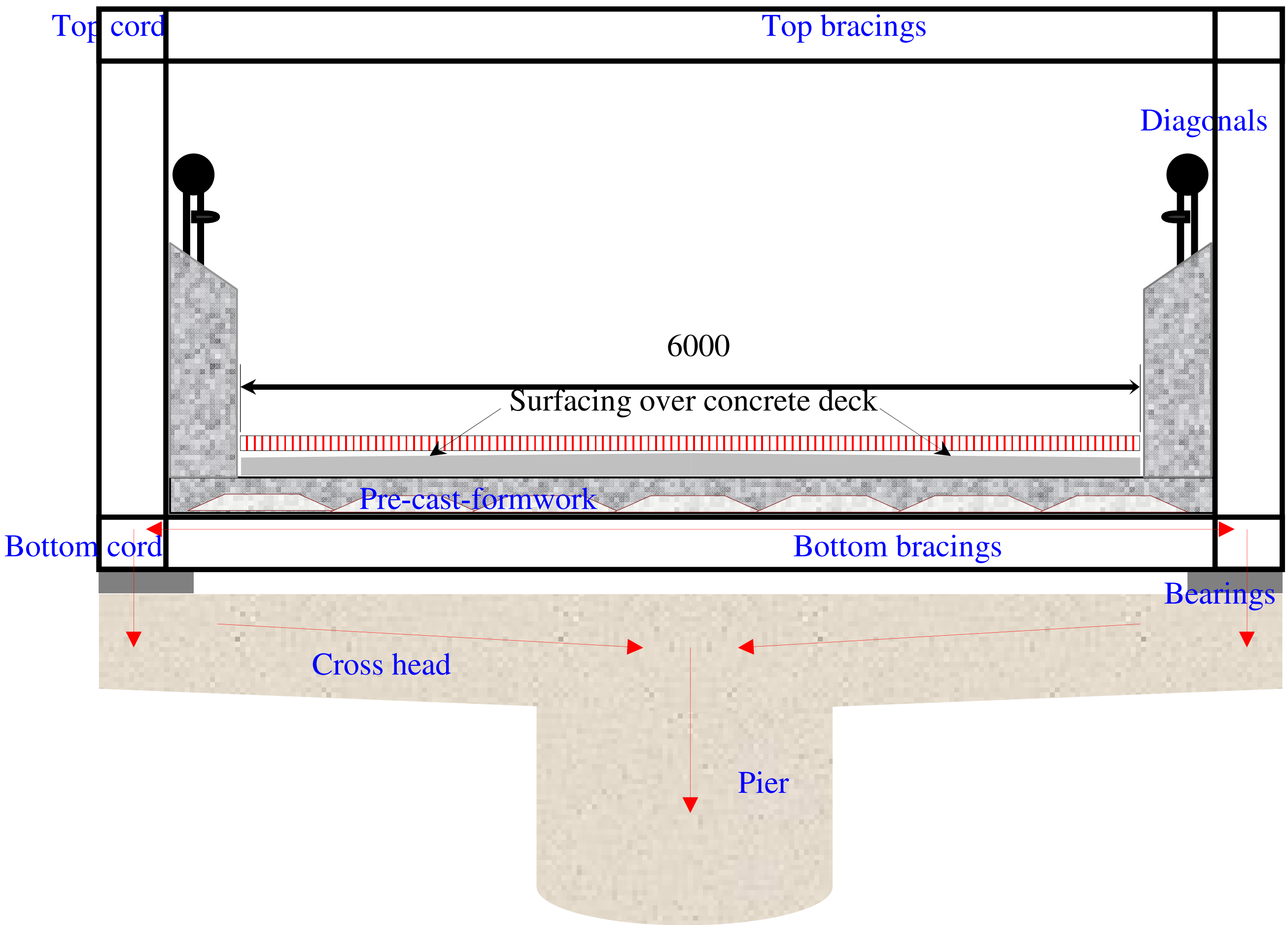


Figure F: Typical cross section of the solution 2 with load & load path

This is another simple possible solution for the given problem. In the table below it is shown how it overcomes the above mentioned constrains:

Constraints as listed	How the solution overcomes them
Least / no disruption Prefabricated / Pre-cast form of deck Cast in situ deck on permanent formwork	Transportation of prefabricated Warren truss individually, erecting them in place by tying them each other at top and bottom using transverse members and casting of deck slab on top of the permanent formwork shown above will minimise the disruption.
Detailing to accommodate 30 degree skew Superstructure's behaviour due to the skew. Superstructure to be cantilevered / overhanged No load to be transferred to the Commercial building.	Trusses are simply supported on piers at the central reserve & either end of carriageway. The trough shape concrete deck is cantilevered to the either end with appropriate detail at the connection with ramp / commercial building will overcome all these constrains easily.
Limited space for substructure & foundation Lighter superstructure is more appropriate to avoid skew effect on the substructure.	Single circular pier with cross head supporting two trusses at either side on bearings and the concrete deck on prefabricated formwork will be considerably light superstructure and stiff enough to deal with the constrains listed.
6m wide foot bridge and 4m wide ramp Omission of central support using a longer superstructure will be extremely difficult. Transport of prefabricated superstructure shall be planned since design stage. Erection of the major components of the superstructure shall be considered in design.	Two pairs of trusses and one pair of them over each carriageway is enough for 6m wide foot bridge. 4m wide concrete ramp supported on intermediate piers joining the footbridge RC deck with a triangular wedge, is a very simple arrangement for joining slope with skew deck. Best use of central reserve pier support Transportation and erection of Warren girders shall not be difficult for this solution.

<u>Comparison and selection of the more appropriate solution</u>

Three important points stand out for option 1 over 2:

- Least disruption to the road under.
- Easier to transport and erect in place
- Much simpler detail for connection with ramp.

Important points for the letter to client:

- Pier near commercial building cannot be used, but without a support the bridge cannot be built as a cantilever from central reserve support.
- Instead of single cylindrical pier with cantilevered crossheads a pair of piers at least 16m apart will have to support the superstructure by its monolithic crosshead in between.

The letter should discuss the impact of these two important points to the design and nothing else.

Calculation:

For the chosen option calculation is required for principal structural elements;

Standard 250mm thick deck slab with B20 -150c/c T&B both direction can easily be considered for this particular case based on minor calculation or even engineering judgement is acceptable. Standard plate girders or even rolled sections at high end of the steel section table can easily be demonstrated as capable of carrying quarter of entire load with little amount of calculation (using appropriate references to the available information from various guidance notes etc). Since the structure is continuous, so hogging moment will govern the section design. Therefore neglecting composite effect will not be grossly uneconomical as the section has to satisfy the requirements in combined effect of bending and shear. Hence the calculation should demonstrate the need.

Though wind may not be governing but minimum calculation is needed in line with the question.

Calculation for the sizing of substructure, foundation and ramp are also equally important, which are often forgotten. For this particular solution other than working out of the ramp geometry minimum amount of calculation or at least a design statement is necessary for its sizing purpose.

Drawings:

As mentioned in the question the answer script must include general arrangement plans, sections and elevations to show the dimensions, layout and disposition of the structural elements and critical details for estimation purpose. For this extremely simple solution other than plan, elevation and section, it is desirable that the expansion joint with the commercial building and the triangular slab to act as a transition between the deck slab and ramp must be included. Detailed three dimensional views of both the proposed solutions are provided in the appendix.

Method of Statement and outline construction programme:

The detailed method statement for the safe construction of the footbridge and its ramp and an outline construction programme should include various stages of construction and the anticipated time required for each of them. This can be done by putting bullet points accompanied with free hand sketches and a bar chart, but health and safety aspect of each activity and reasonable understanding of the time involved for respective activity has to be well demonstrated.

For example in this chosen solution:

- Approval in Principle & designers risk assessment followed by the detailed design and preparation of fabrication / construction drawings should be an activity in the beginning of the project, which is often forgotten.
- Prior to any construction activity the site preparation and enabling works along with the site mobilisation with adequate fence to the construction area for safe construction is equally important.
- On completion of the project hand over of the structure with as built drawings and Health & Safety file to the owner client should not be ignored in the method of statement.

Appendix 1: Three dimensional view of the proposed two solutions.

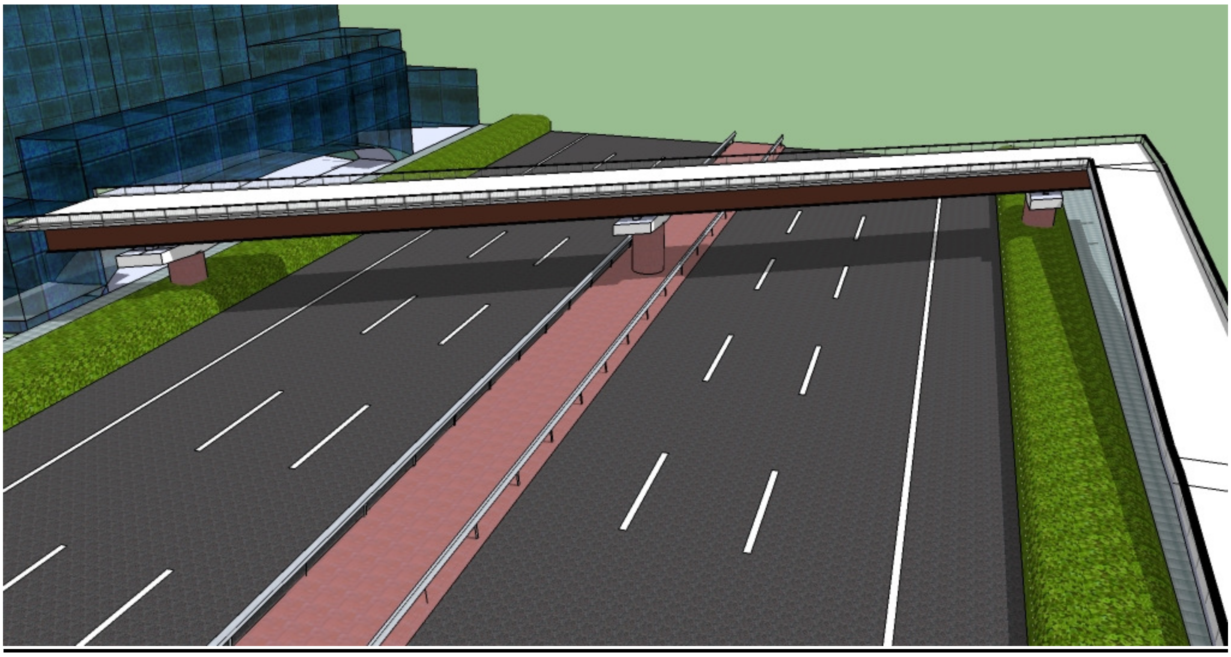


Figure 1A: Three dimensional view of the bridge and ramp made of RC slab on steel girders

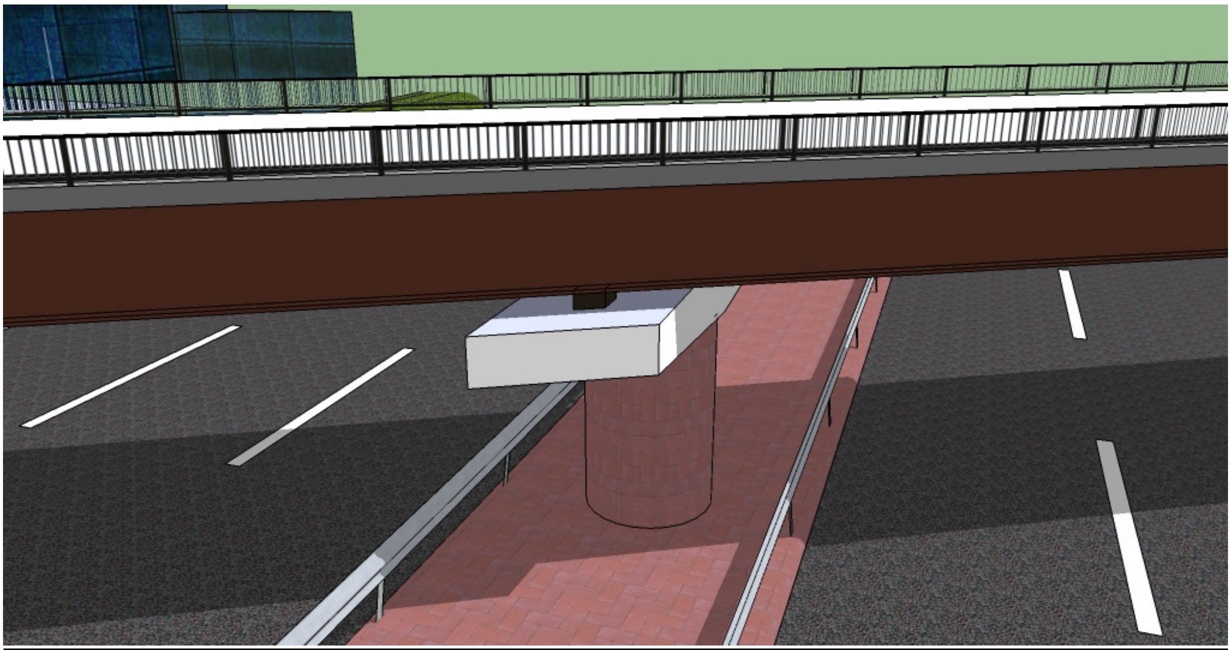


Figure 1B: Three dimensional view of the bridge deck slab on steel girders, bearings and parapets

SOLUTION1

Figure 2A: Three dimensional view of the bridge concrete deck on through warren truss & ramp

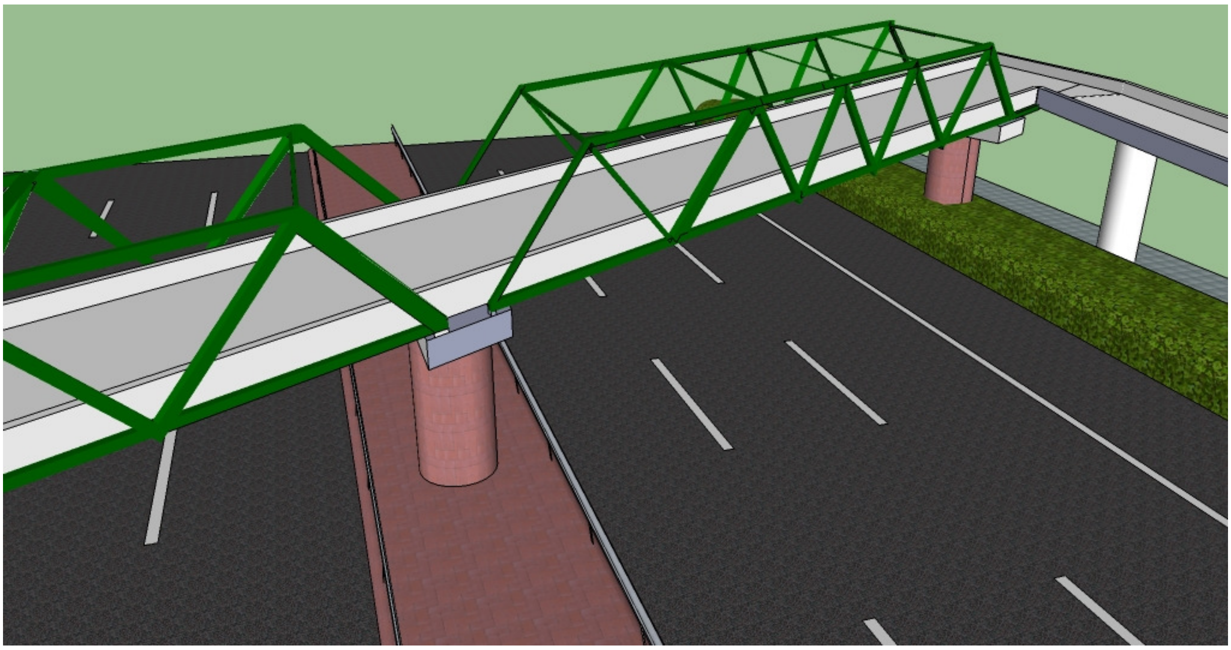


Figure 2B: Three dimensional view of the bridge deck on steel truss, bearings & ramp connection

SOLUTION2

Possible solution to past CM examination question

Question 4 - April 2010

Cityscape Development

by Bob Wilson

Question 4. Cityscape Development

Client's Requirements

1. A new landmark building on an open site to offer a variety of shopping and entertainment venues and to provide a panoramic view of the city; see Figure Q4.
2. Other than the four service cores, no vertical or inclined structural elements are permitted between levels 1 and 2. Not more than one internal column is permitted in each compartment above level 2. No structural elements are to be constructed outside the enclosure walls of the square compartments.
3. A minimum clear internal headroom of 4.0 m is to be provided to the floors on levels 2 and 3, with a structure-free ceiling zone of 0.3m depth. A minimum clear headroom of 9.0m is to be provided to the unenclosed area on level 1. There is no restriction on the overall roof height.
4. The minimum fire resistance period required for structural elements is 2 hours.

Imposed Loading

5. Roof 2.0 kN/m^2
All floors 5.0 kN/m^2

Site Conditions

6. The site is level and is located in a coastal area near the sea. Basic wind speed is 40m/s based on a 3 second gust; the equivalent mean hourly wind speed is 20m/s.
7. Ground Conditions

Ground level – 8.0m	Soft coastal reclamation
8.0m – 15.0m	Sand and gravel. N varies from 10 to 20
Below 15.0m	Rock. Compressive strength 5,000kN/m^2

Ground water was encountered at 1.0m below ground level

Omit from consideration

8. Detailed design of lifts and staircases inside the service cores.

SECTION 1 (50 marks)

a. Prepare a design appraisal with appropriate sketches indicating two distinct and viable solutions for the proposed structure including the foundations. Indicate clearly the functional framing, load transfer and stability aspects of each scheme. Identify the solution you recommend, giving reasons for your choice. (40 marks)

b. After the design has been completed, the client advises that he wishes to have a skylight 6.0m diameter in the roof of each compartment. Write a letter to your client recommending the suitable location and explaining how this may be achieved. (10 marks)

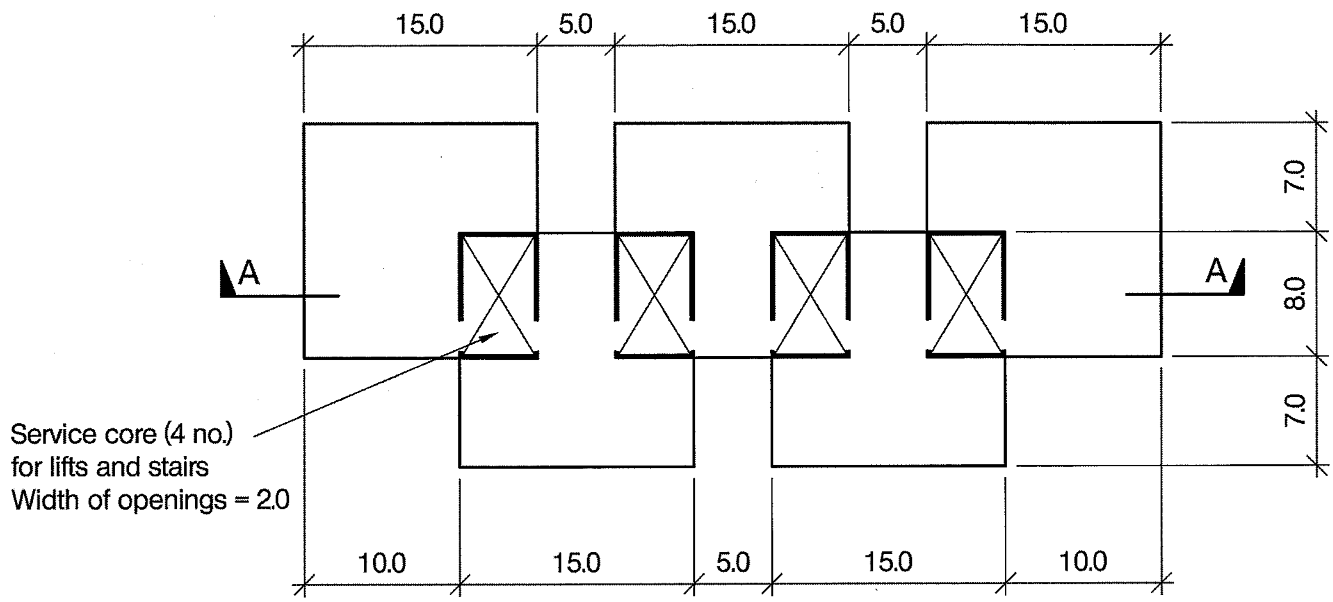
SECTION 2 (50 marks)

For the solution recommended in Section 1(a):

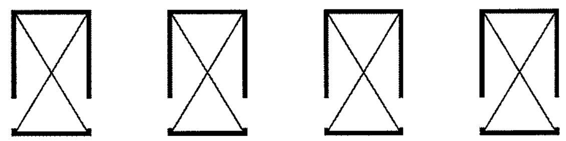
c. Prepare sufficient design calculations to establish the form and size of all the principal structural elements including the foundations. (20 marks)

d. Prepare general arrangement plans, sections and elevations to show the dimensions, layout and disposition of the structural elements and critical details for estimating purposes. (20 marks)

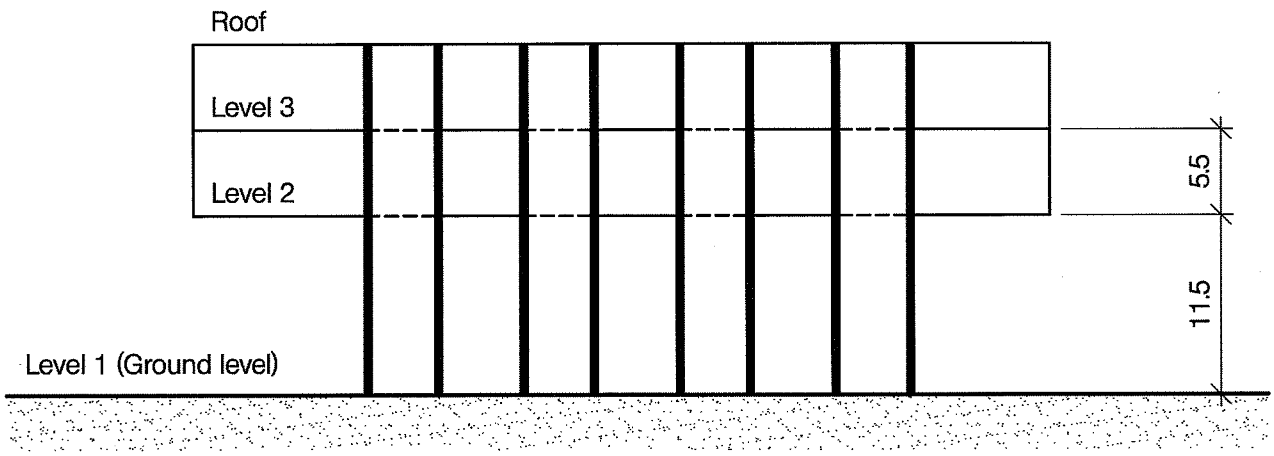
e. Prepare a detailed method statement for the safe construction of the building and an outline construction programme. (10 marks)



PLAN ON LEVELS 2 AND 3



PLAN ON LEVEL 1



SECTION A-A

NOTE: All dimensions are in metres

FIGURE Q4

QUESTION 4 / 2010 - CITYSCAPE DEVELOPMENT

The following need to be included in your answer:

The site conditions state that the ground conditions from ground level to -8.0m is "soft coastal reclamation". Rock is found below -15m. The top 8.0m of soil appears to be unsuitable for bearing foundations, so all the load must be conveyed to the rock using piles. Two different pile types are: large-diameter bored caisson piles and bottom-driven shell piles with an insitu reinforced core.

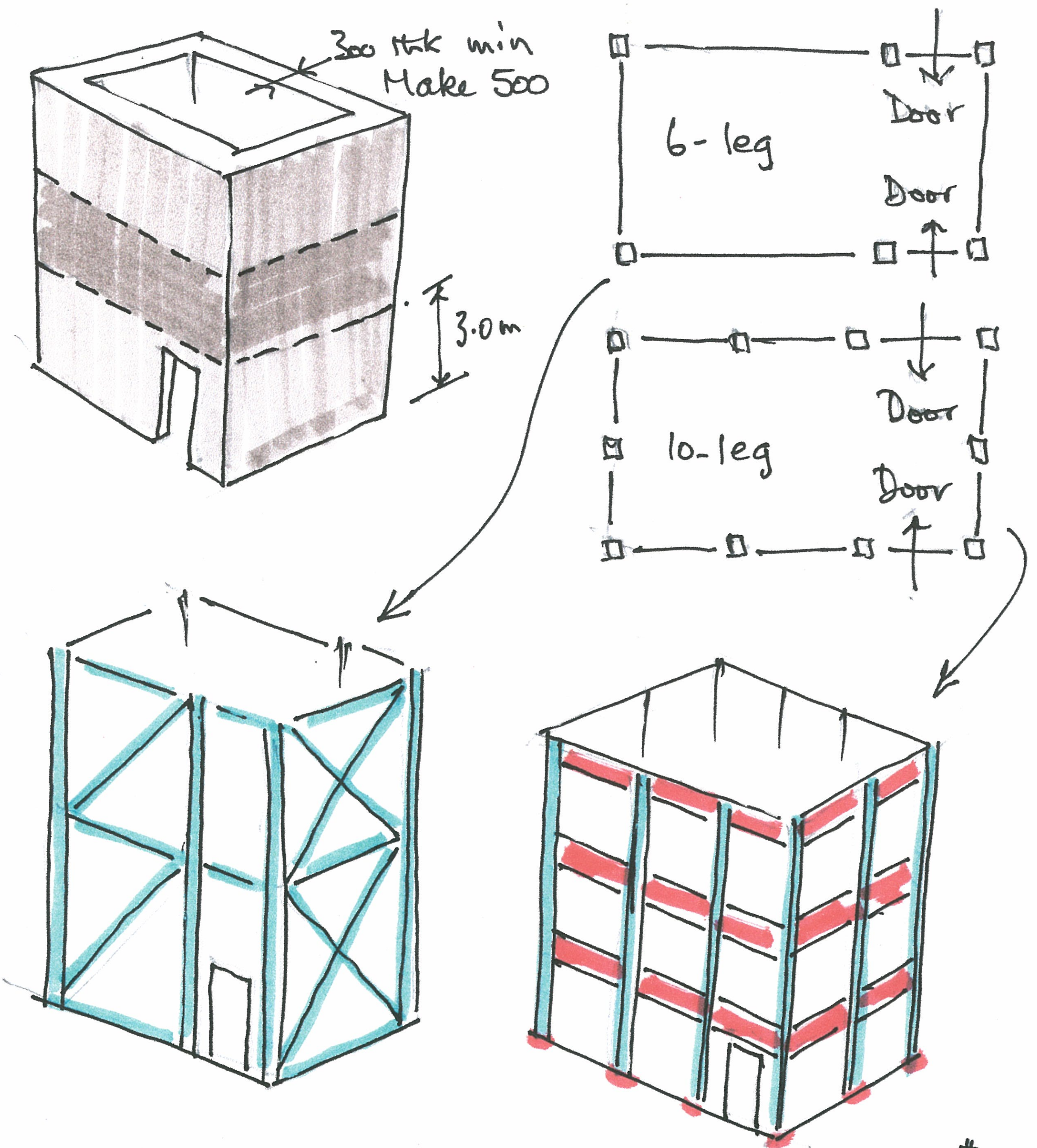
Each caisson pile will carry a large load and may be under-reamed so that the bearing stress on the rock does not exceed an allowable value, say 5000 kN/m² divided by a F.O.S of 3 equals 1700 kN/m². Each core will need, say, 5 piles: each 1200 mm. diameter.

Each shell pile can carry between 500 and 1000 kN (600 mm diameter pile) and so approximately 50 number piles will be needed. The piles would be driven to "refusal".

The pile system would be capped by a reinforced concrete raft because

in each case the pilecaps would overlap.
In the case of the caisson piles the raft thickness would be determined by "punching shear" and so would be approximately 1500 to 2000 mm thick.
For the larger spread of the shell-pile system bending would have an influence with deflections being critical. A suitably stiff raft would be a cellular raft perhaps 3.0 or 4.0m thick.
The choice of system could be decided by the weight of the piling equipment on the soft coastal reclamation. Both schemes will need a working platform comprising geotextile over the site topped with, say, 600 mm thick layer of selected granular material. The choice would be between a smaller number of large-diameter piles bored using heavy equipment, or a larger number of small-diameter piles bottom-driven by a lighter rig using a heavy steel mandrel.

The four cores are a major feature of the development and will require an appropriate finish (cladding). My first choice is reinforced concrete 300 mm thick cast in traditional formwork in a series of lifts, each approximately 3.0 m. high. My second choice is a framework of structural steel comprising either 6 or 10 leg layouts with either diagonal bracing or moment-resisting connections.

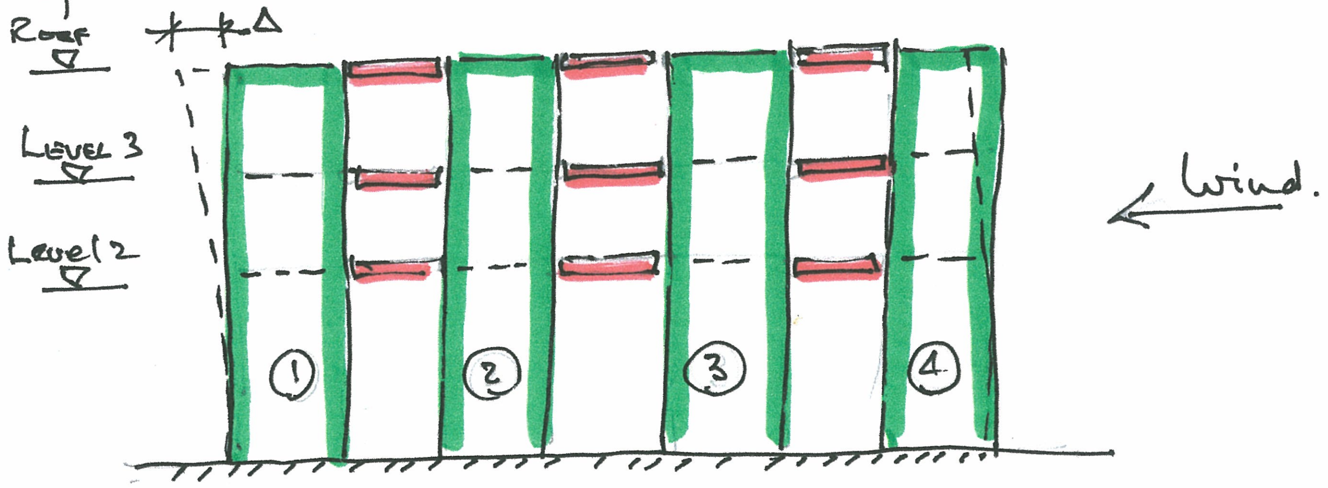


Stanchions will probably be 356 x 406 UC 393#
in Grade 355 steel.

The 120 minute fire-resistance period will require concrete walls not less than 300 mm thick with concrete cover of 50 mm. The steelwork will require a protective covering of concrete or sprayed-on intumescent foam as well as external cladding. The internal appearance may also require a suitable cladding.

All claddings and finishes need to be durable enough to resist the corrosive, salt-laden atmosphere of the coastal area near the sea (Site Conditions No. 6). Concrete surfaces shall be impregnated with Silane. Exposed steel surfaces shall be painted with an epoxy-based, 5-coat protection.

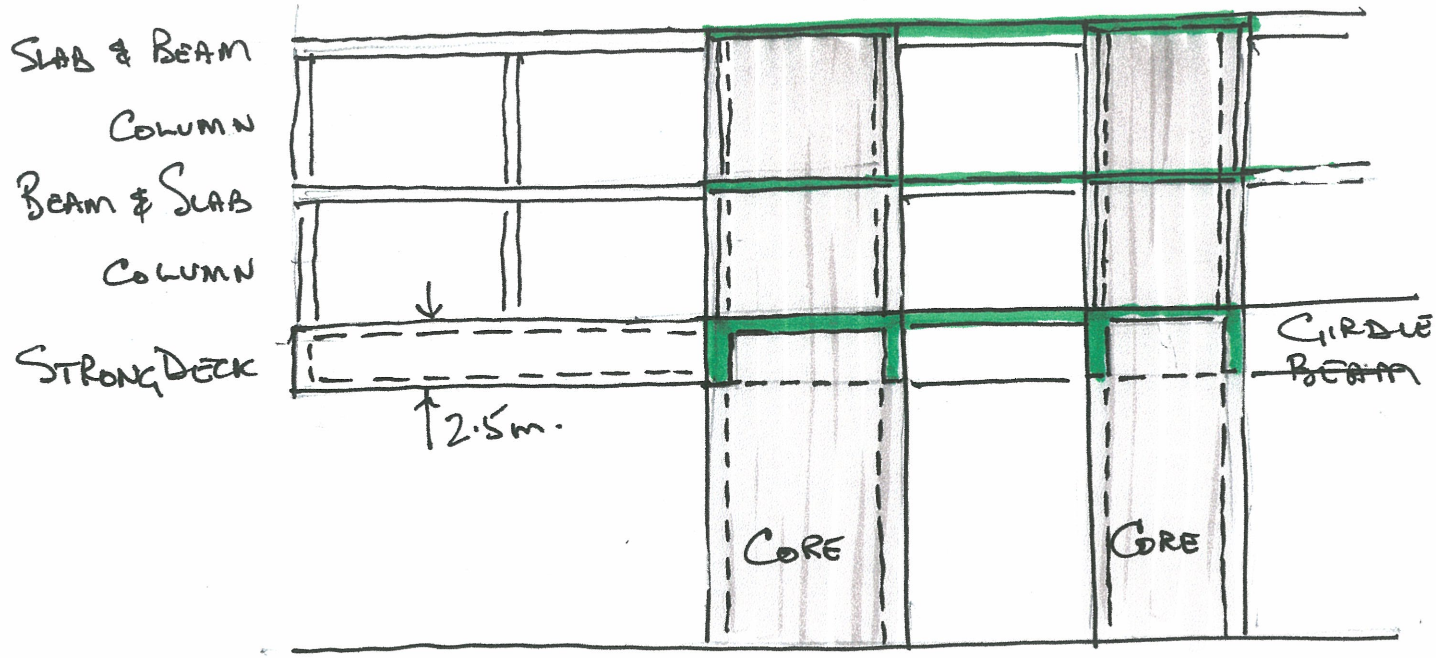
The 4 cores must be made to act together and not sway independently under wind load. "Girdle" beams will be provided at each floor level and at roof level.



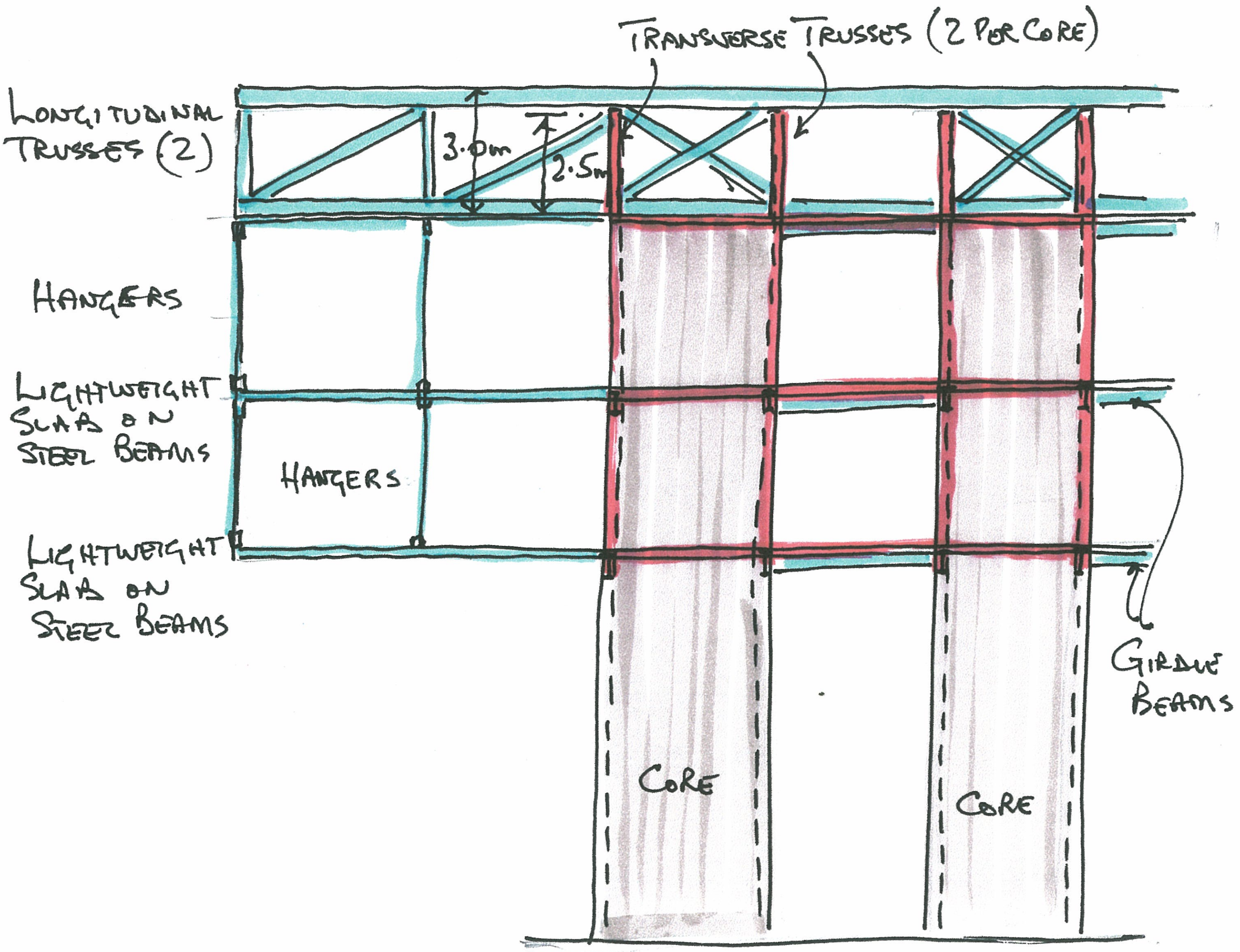
Two superstructure options are considered:

(i) A reinforced-concrete "Strong Deck" at Level 2 with either reinforced concrete framing and floors standing on it or steel framing and lightweight floors.

(ii) A steel girder or truss system at Roof level with steel-framed lightweight floors hung from the principal roof members on steel hangers.



"CONCRETE" OPTION.



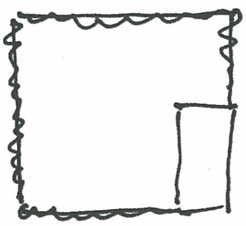
"STEEL" OPTION

Client's Requirements N°1 requires a panoramic view of the city. Consequently the external walls of the superstructure will be windows. Two options will be considered:-

(i) traditional framing – vinyl-coated aluminium – supporting toughened glass double glazing

(ii) top-hung toughened-glass curtain walling.

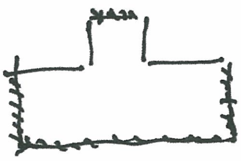
Glass weighs 25 kN/m³ [Fiona Cobb – Structural Engineer's Pocket Book], so double-glazed patent glazing – including framing – will weigh approximately $25 \times 0.025 = 0.625$ kN/m².



$(15+15+10+7) \times 0.625 \times 11.0 \quad = \quad 325$ kN

TWO THUS

Ht $= 5.5 + 5.5 = 11.0$



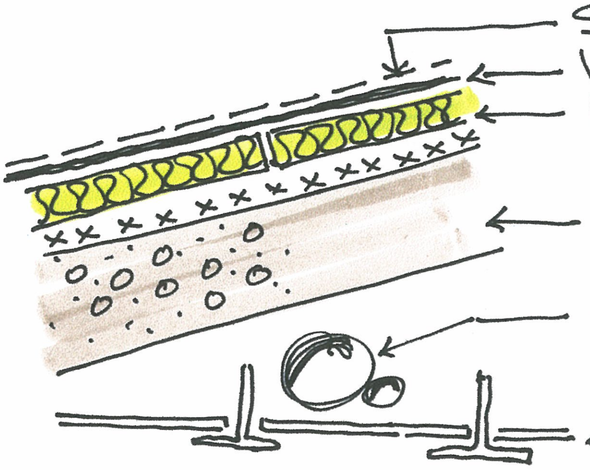
$(5+7+7+15) \times 0.625 \times 11.0 \quad = \quad 234$ kN

THREE THUS

TOTAL LOAD OF GLAZING $\quad 325 + 325 + 234 + 234 + 234 = \underline{1352}$ kN
(which is roughly 135 tonnes)

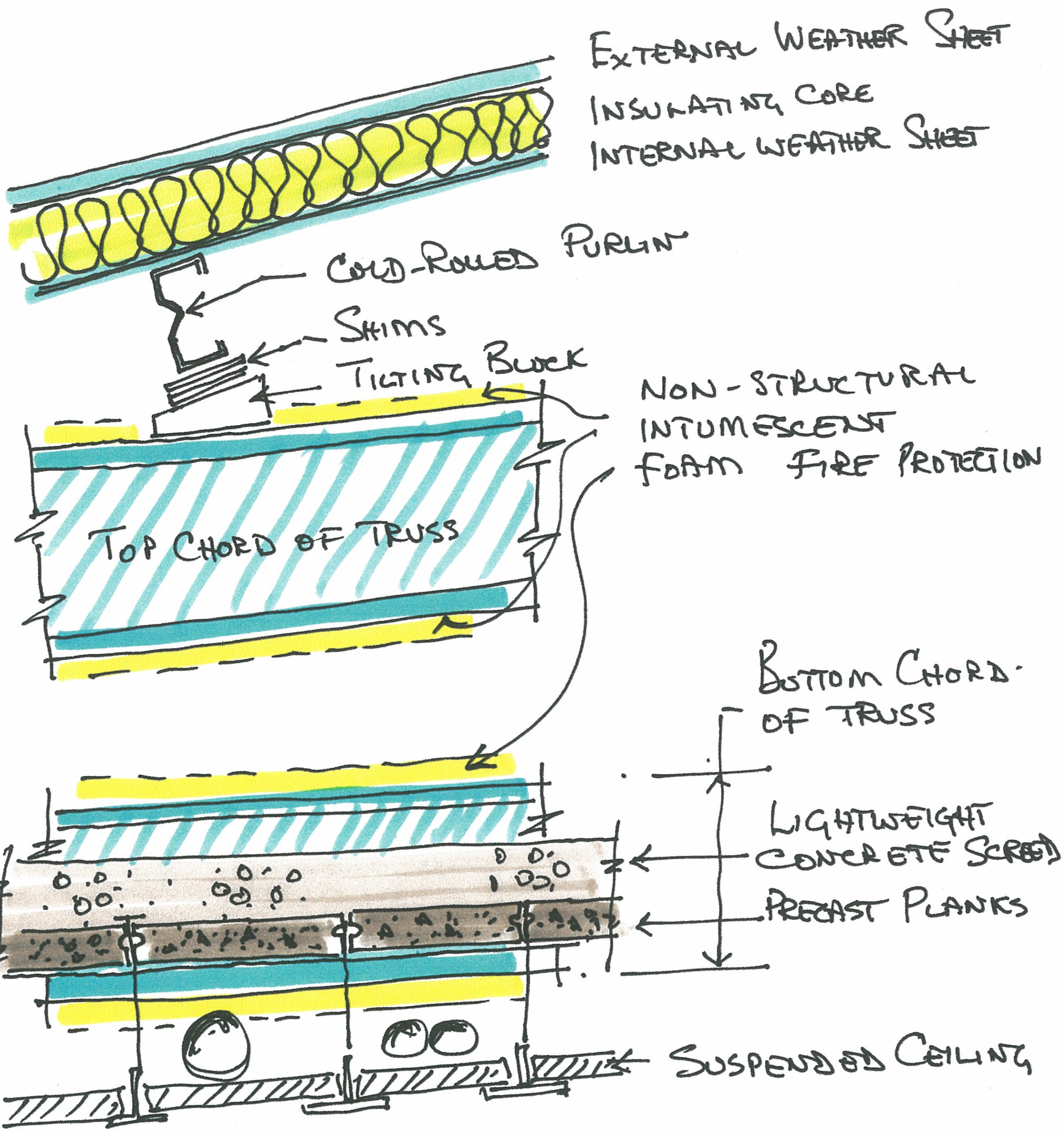
The patent glazing is carried through the building and fitted from within: the curtain walling is lifted into place from outside the building.

In the concrete option the roof would comprise: a reinforced concrete, lightweight aggregate slab, laid to falls, cast insitu on traditional formwork. Casting to falls with a constant thickness will reduce the amount of a relatively expensive material, reduce dead weight and will not require screeding to falls. The roof insulation would be laid over the slab and waterproofed with a high-quality membrane (similar but better quality than roofing felt).



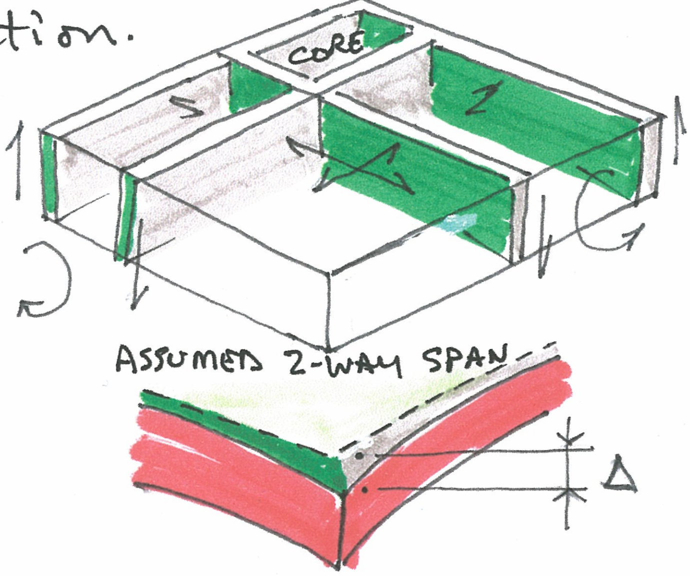
For 120 minute fRp Slab to be 125 thk 35 mm cover

In the steel option the roof would enclose the upstanding steel trusses with sloping areas – in the manner of a Mansard roof. Drainage falls would be "forced" using packing under the cold-rolled purlins. The purlins would be clad with profilled-steel panels – external weather sheet, internal weather sheet with the insulation core sandwiched between. The floor of the roof space would be formed with precast-concrete planks topped with a lightweight screed that would provide lateral support to the bottom chords of the trusses (in compression because of the cantilever action). The service space would be above a suspended ceiling.



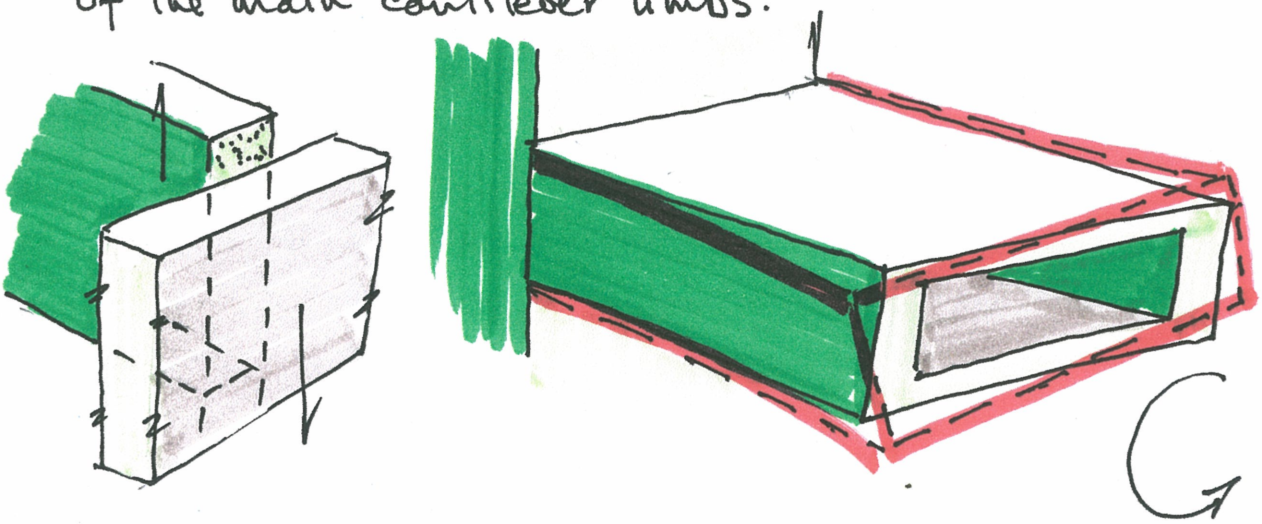
The unit weights of either of these can be calculated or an equivalent concrete slab, say, 300 thick — $25\,kN/m^3 \times 0.30 = 7.5\,kN/m^2$.
All roof drainage would be disposed of internally [garland gutters, etc] to prevent water cascading off the façade.

The floors for the concrete option would naturally be reinforced concrete. At level 2 the slabs would be cast integral with the deep (2500mm) downstand beams at this level. However, they do not contribute to the strength of the beams because of the cantilever action. The overhanging portion of the two end compartments requires special attention.



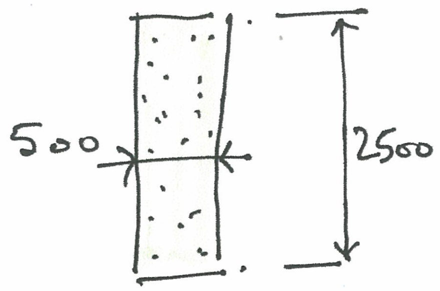
EDGE BEAMS BEND AND DEFLECT.
THE DEFLECTION IS COMMON TO BOTH BEAMS. IF THE BEAMS HAVE THE SAME DIMENSIONS THE LOAD CARRIED BY EACH BEAM $\propto I/L$

The shear is resisted by the cross-section of the main cantilevers. The torsion is resisted by the box-section of the main cantilever limbs.



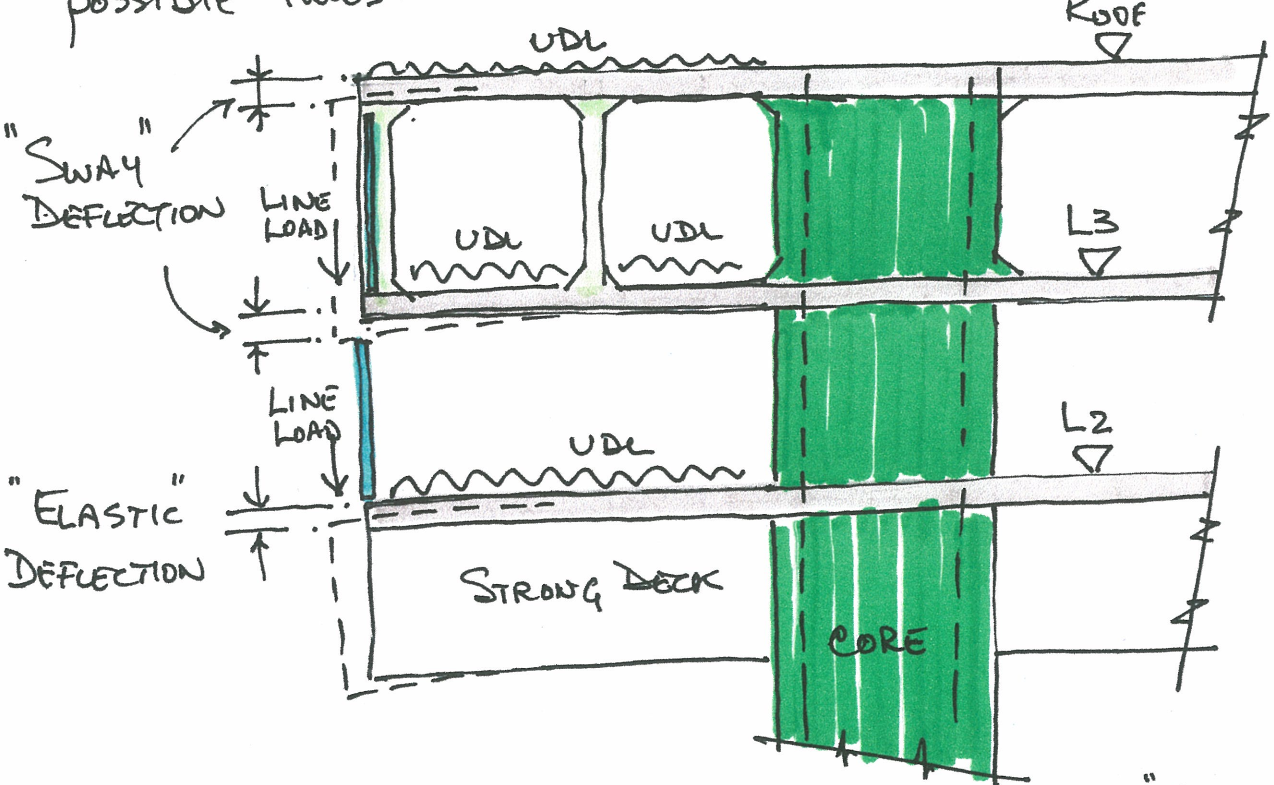
This is a simplification of the complex behaviour of this part of the superstructure but will allow investigation of the structural adequacy of the available section in Section 2c (to follow) — if appropriate.

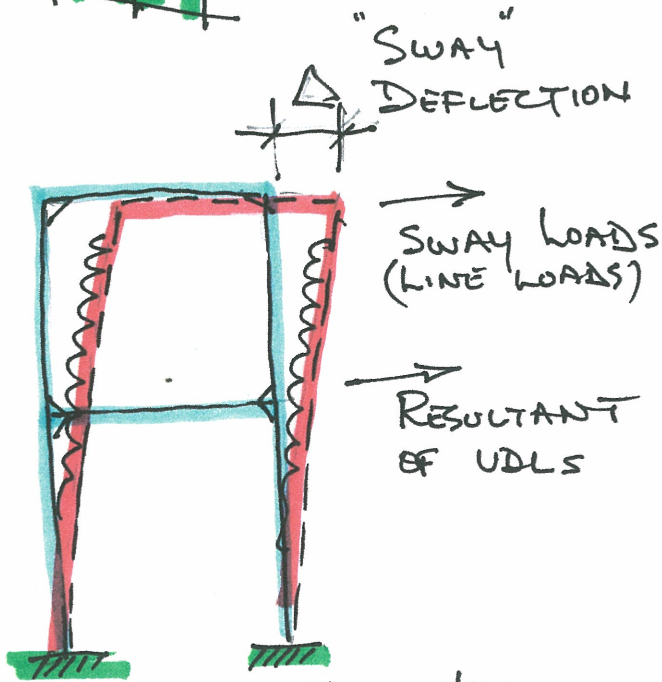
The available beam section is:



The width matches the thickness of the core walls.

Removal of Level 3 and Roof loading may be possible thus:





The L3 and Roof "compo" might be modelled thus, as a two-storey portal frame with fixed bases subject to a sway load

However, this analysis will take more time than I have available at present.

The recommended scheme draws upon features from the basic schemes described earlier. It will comprise:

1. Caisson piles bearing on rock – The smaller number of piles can be located more directly below the loads and requires a smaller, thinner capping raft. Any uplift forces can be resisted by the underreamed enlargements at the foot of each pile.

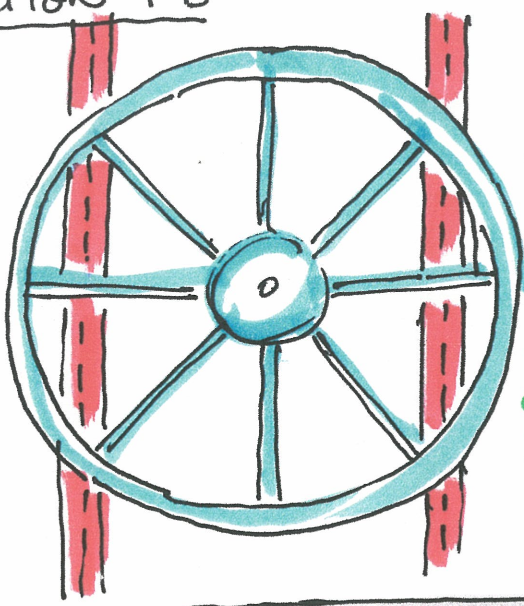
2. Reinforced concrete cores – The four walls of each core distribute the loads relatively evenly to the pile-cap raft. Concrete construction is also relatively easy and is intrinsically durable, fire-resisting and enclosing (no further cladding needed)

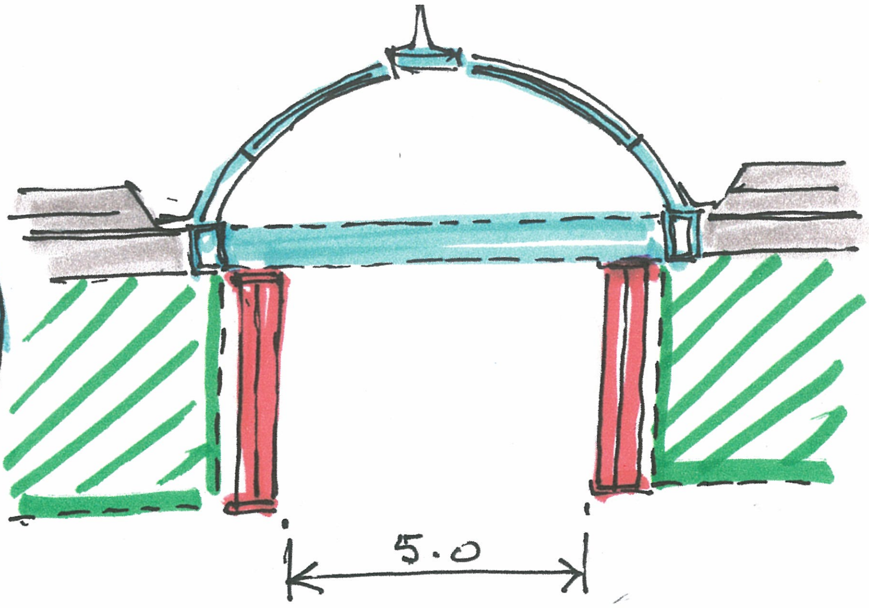
3. Reinforced concrete "strong deck" and girdle beams – although requiring substantial falsework and formwork the resulting structure ties and stabilizes the four cores at mid-height. It will also provide a platform on which to construct the Level 3 floor before it is winched up to its finished level.

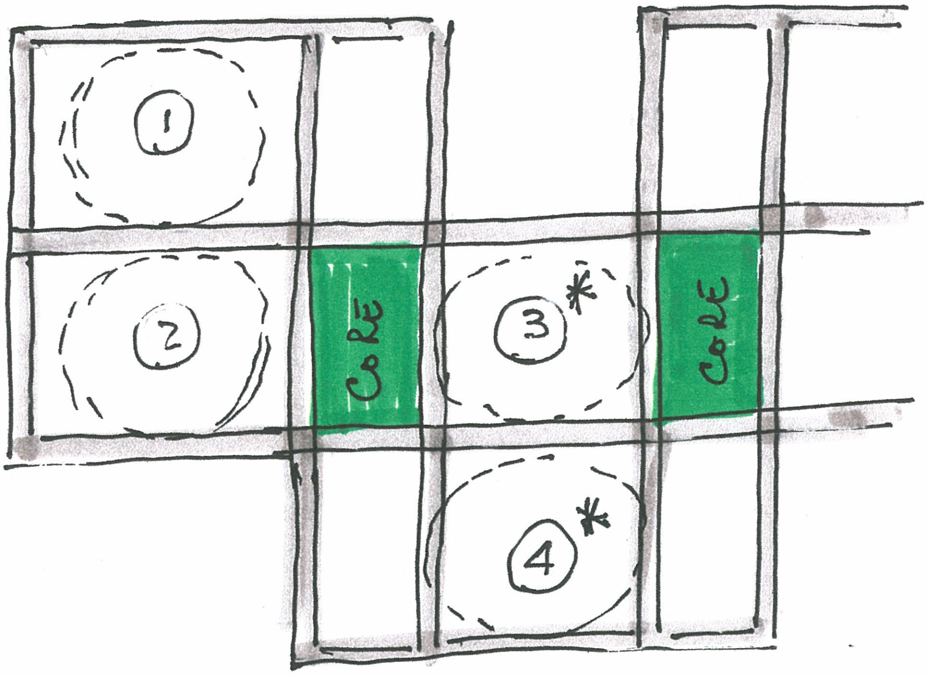
4. Structural steel roof-level trusses with perimeter hangers supporting Level 3 flooring

5. "Traditional" framed windows carried through the building and fitted from inside the finished structure.

SECTION 1b







To (address)
Your reference

From (address)
Date
Reference.

Dear Norman,

Cityscape Development - Possible Skylights

Further to your request for us to investigate the possible installation of 6.0m diameter skylights, I enclose three quick sketches illustrating my thoughts about the matter. The core dimensions dictate the positions of the roof trusses and these cannot easily be changed. I have indicated four positions where a large 6.0m diameter "hole" in the roof could be made. Numbers 3 and 4 (marked with an asterisk) are the tightest, only having 5.0m clear between the steels.

Each skylight would have a ring beam that would straddle a pair of supporting beams. The glazing bars might arch up, as shown, to meet at a central boss or, alternatively, form a pyramid. The skylight dome would be fitted into the weatherproof layer (see sheet 8 of this text) with appropriate gutters and flashing. The steelwork might be left exposed (as shown) or enclosed. This will create a "well" through the depth of the roof trusses.

As your adviser I must admit to being luke warm to the suggestion. Experience has shown that skylights often leak, especially circular ones, because of the difficulties of weatherproofing the joints between the glass panes and between the skylight and the main weatherproof roofing. There are several unavoidable movements, shrinkage and deflection, that take place at gutter level. Because these skylights are large and heavy these movements will be significant and cause maintenance problems.

May I ask you to consider a greater number of smaller rectangular skylights or even the bold move to have a glass roof? An example is the Queen Elizabeth II Great Court at the British Museum.

Yours sincerely,

The pages above have taken me about 3½ hours to write [approximately 4 minutes per mark]. You may wish to award "marks" out of 50. Part 1a is out of 40; Part 1b is out of 10. I would not expect 100% because several matters could be improved: equally I would not expect to fail!

Try re-writing the content and see how you get on. Where you think you can make improvements do so – but speed up so that you keep within the same time.

If you wish you can correspond with me on:

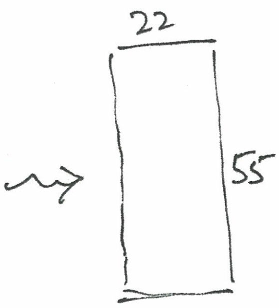
bob.wilson2@virgin.net.

The following calculations and drawings are some that I prepared for my marking. Hopefully any errors you may find are "small" ones. This applies to any examination answer. For this question you should consider (1) Stability (2) Foundations (3) Level 2 and (4) Roof. Each calculation includes loading, moments, etc. and section.

	Project Q4/2010	Sheet No 1
	Date	Name

Ref	Calculations	Output

WIND



Greater Horiz Distance / Lesser Horiz Distance

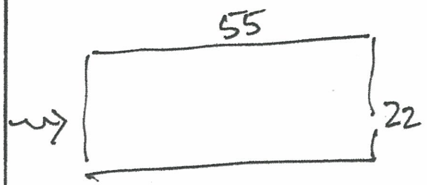
Normal Dim to wind / Dim in direction of wind

$\frac{\text{Height}}{\text{Breadth}}$ C_f

$\frac{55}{22} = 2.5$ | $\frac{22.5}{22} = 1$ | 1.05

$H = 22.5$

$\frac{55}{22} = 2.5$

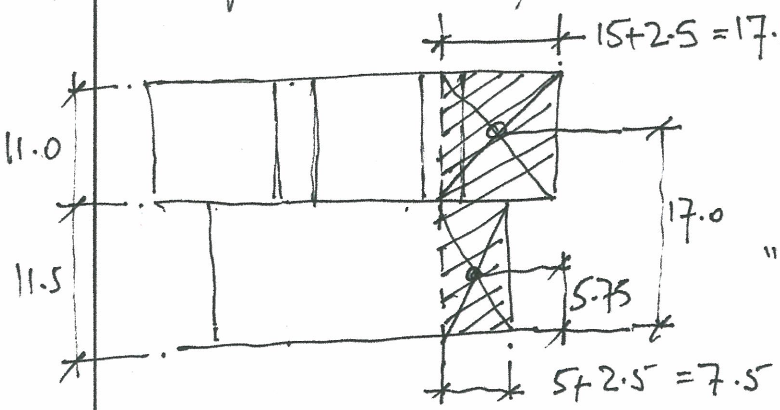


$\frac{22}{55} = 0.4$ | $\frac{22.5}{55} = 0.4$ | 0.75

$V_{3sec.\ gust} = 40\ m/sec$ Let $S_1 = 1.1$ $S_2 = 1.0$ Class C1 $S_3 = 1.2$

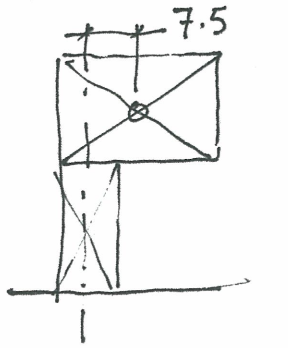
Design wind speed $= 40 \times 1.1 \times 1.0 \times 1.2 =$ 53 m/sec

$\therefore q_r = 1.72\ kN/m^2$ 1.72 kN/m²



WIND ON ONE CORE SHAFT "FACE ON"

NOTE ECCENTRICITY OF THE TOP AREA WRT ℄ OF SHAFT



$A_e = (11 \times 17.5) + (11.5 \times 7.5)$

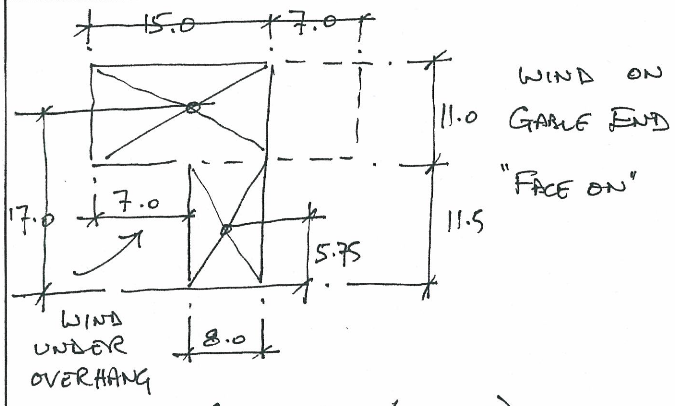
$= \underline{279\ m^2}$ WIND UNDER OVERHANG

Add (7.5×7.0) 52.5 m² $A_e = 332\ m^2$

Check

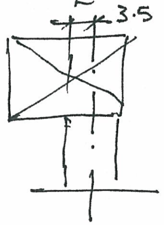
	Project Q4/2010	Sheet No 2
	Date	Name

Ref	Calculations	Output



WIND ON GABLE END "FACE ON"

NOTE THE ECCENTRICITY OF TOP AREA WRT ℄ SHAFT



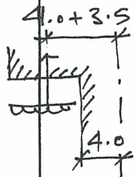
$A_e = (11 \times 15) + (11.5 \times 8)$

$= 257 + 77 = 334\ m^2$

Add. $(8.0 \times 10) = 80\ m^2$

$A_e = 414\ m^2$

OTM — SIDE WIND



$1.05 \times 1.72 \times (17.5 \times 11) \times 17 = 5910$

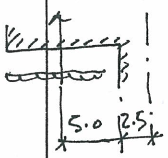
$1.05 \times 1.72 \times (7.5 \times 11.5) \times 5.75 = 896$

$1.05 \times 1.72 \times (52.5) \times 7.5 = 711$

$\underline{\underline{7517}}$

OTM SIDE $= \underline{\underline{7517\ kNm}}$

OTM — GABLE WIND



$0.75 \times 1.72 \times (11 \times 15) \times 17 = 3619$

$0.75 \times 1.72 \times (11.5 \times 8) \times 5.75 = 683$

$0.75 \times 1.72 \times (8.0 \times 10.0) \times 7.5 = 774$

$\underline{\underline{5076}}$

OTM GABLE $= \underline{\underline{5076\ kNm}}$

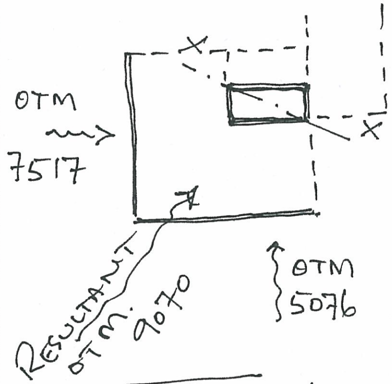
N.B. TORSION ON SHAFT FROM EACH WIND BUT OPPOSITE EFFECTS.

Check

	Project Q4/2010	Sheet No 3.
	Date	Name

Ref	Calculations	Output

Resultant of winds on each face — i.e. wind on weaker axis, the diagonal axis X-X.



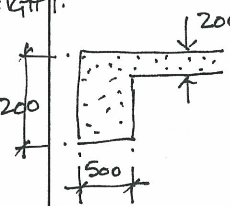
ASSUMING THAT SHAFT CAN RESPOND AS A SINGLE ELEMENT!

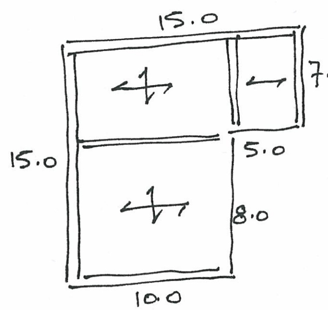
RESULTANT OTM $= 9070\ kNm$

UNFACTORED

$\sqrt{7517^2 + 5076^2} = (82\,271\,065)^{\frac{1}{2}} = \underline{\underline{9070\ kNm}}$

SELF WEIGHT.





kN/m^3

$25 \times 0.2 \times 15 \times 7 = 525$

$25 \times 0.2 \times 8 \times 10 = 400$

$25 \times 0.5 \times \underbrace{(10 + 10 + 7 + 7 + 15 + 15)}_{64} = 800$

$\underline{\underline{1725\ kN}}$

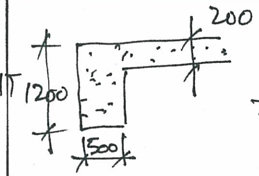
$Wt/m^2 = \dfrac{1725}{\underbrace{(15 \times 7) + (8 \times 10)}_{185}} = 9.32\ kN/m^2.$

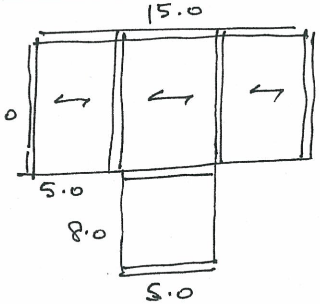
Check

	Project Q4/2010	Sheet No 4
	Date	Name

Ref | **Calculations** | **Output**

SELF WEIGHT





kN/m^3

$25 \times 0.2 \times 15 \times 7 =$ 525

$25 \times 0.2 \times 8 \times 5 =$ 200

$25 \times 0.5 \times \underbrace{(15+7+7+7+7+5+5)}_{53} =$ 663

1388 kN.

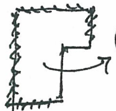
$$wt/m^2 = \frac{1388}{\underbrace{(15\times7)+(8\times5)}_{145}} = 9.57\ kN/m^2$$

CLADDING Glass 12mm thk + framing

Say $25 \times 0.025 = \underline{0.625}\ kN/m^2$

(i.e. 2 x Thickness of glass)

*Ht.



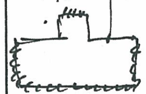
$(15+15+10+7) \times 0.625 \times 11.0 = 325$

$(5+7+7+15) \times 0.625 \times 11.0 = 234$

557 kN.

559

*Ht = 5.5 + 5.5 = 11.0

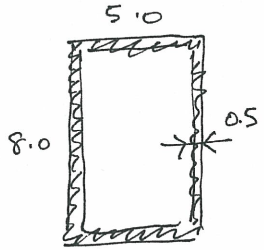


Check

	Project Q4/2010	Sheet No 5.
	Date	Name

Ref | **Calculations** | **Output**

SELF WT CORE



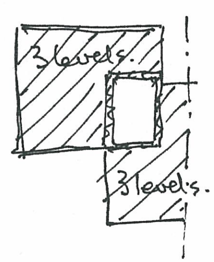
Ground to Roof
i.e. 22.5m ht.

$$25 \times 0.5 \times \overbrace{(8.0+8.0+5+5)}^{26} \times 22.5 = \underline{7313\ kN.}$$

kN/m^3

GRAVITY LOAD ON END CORE!

* Let Roof = Floor. ∴ 3 equal levels.



$3 \times 1725 = 5175$ } Beam + Slabs.

$3 \times \frac{1388}{2} = 2082$

$+ \quad 325$ } Cladding

$+ \ 234/2 = 117$

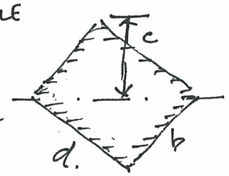
7699

Add core 7313

15012 kN UNFACTORED.

PROPERTIES OF RECTANGLE

AXIS OF MOMENTS ON DIAGONAL (ASSUMED)



$A = bd.$

$c = \frac{bd}{\sqrt{b^2+d^2}}$

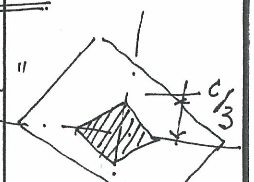
$I = \frac{b^3d^3}{6(b^2+d^2)}$

$Z = \frac{b^2d^2}{6\sqrt{b^2+d^2}}$

$r = \frac{bd}{\sqrt{6(b^2+d^2)}}$

$$\frac{m^2 \times m^2}{\sqrt{m^2}} = \frac{m^4}{m} = m^3$$

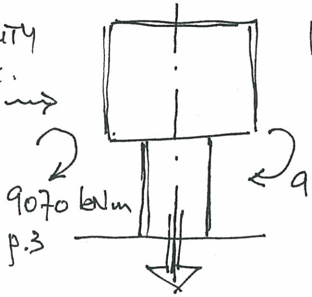
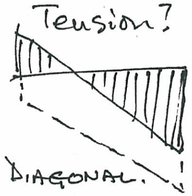
"KERN"



FORMULAE ASSUME SOLID SECTION RATHER THAN HOLLOW RECTANGLE

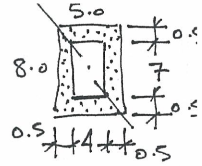
Check

Project Q4 /2010	Sheet No 6.
Date	Name

Ref	Calculations	Output
	$b = 5.0$	
	$d = 8.0$	
$\frac{64}{25}$ 89	$c = \dfrac{bd}{\sqrt{b^2+d^2}} = \dfrac{5 \times 8}{\sqrt{5^2+8^2}} = 4.24\,m$ 9.433	
	$\dfrac{c}{3}$ = limit of Kern $= \dfrac{4.24}{3} = \underline{\underline{1.41\,m}}$	
STABILITY CHECK. →	 9070 kNm p.3 / 9070 kNm / 15012 kN. Let +ve wind = −ve wind suction $OTM = 2 \times 9070 = \underline{18140\,kNm}$	
	$\dfrac{M}{W} = e = \dfrac{18140}{15012} =$	$1.208\,m = e.$
	$e < \dfrac{c}{3}$ $\quad 1.208 < 1.41$ − BUT CHECK	HOLLOW RECTANGLE
	Max stress diagram  Tension?	Compression?
	$Stress = \dfrac{W}{A} \pm \dfrac{M}{Z_{\text{AVAILABLE ON DIAGONAL}}}$	
$\frac{N}{mm^2}$	$\dfrac{15012 \times 10^3}{0.5 \times 26 \times 10^6} \pm \dfrac{18140 \times 10^6}{12.06 \times 10^9}$	Z AVAILABLE SEE PAGE 7
$\frac{1600}{56.6} \times 10^3$		$+2.658\,N/mm^2$
	$= 1.154\,N/mm^2 \pm 1.504\,N/mm^2$	$-0.364\,N/mm^2$ Check

18-

7



$$Z = \frac{b^2 d^2}{6\sqrt{b^2+d^2}} \qquad \frac{m^4}{m} = m^3.$$

$$Z_{AVAILABLE} = Z_{BIG} - Z_{SMALL}$$

$$\frac{5^2 \times 8^2}{6\sqrt{5^2+8^2}} \qquad\qquad \frac{4^2 \times 7^2}{6\sqrt{4^2+7^2}}$$

$$= \frac{25 \times 64}{6\sqrt{25+64}} \qquad\qquad = \frac{16 \times 49}{6\sqrt{16+49}}$$

$$= \frac{1600}{6\sqrt{89}} \qquad\qquad = \frac{784}{6\sqrt{65}}$$

(6×9.43) 56.6. $\qquad$ (6×8.06) 48.37

$$= \frac{1600}{56.6} \qquad\qquad \frac{784}{48.37}$$

$$= 28.27\,m^3 \qquad\qquad 16.21\,m^3$$

$$Z_{AVAILABLE} = \underline{\underline{12.06\,m^3}}$$

Project Q4/2010	Sheet No 8
Date	Name

Calculations

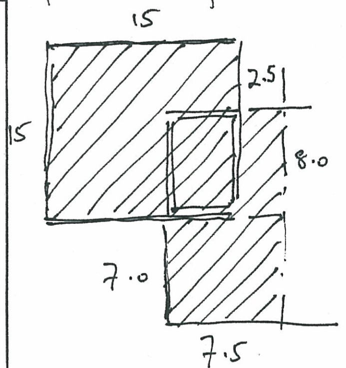
UNFACTORED LOADS ON FOUNDATIONS.

Imposed live on roof $= 2.0\ kN/m^2$
Imposed dead on roof (say) $= 1.0\ kN/m^2$
$\underline{\underline{3.0\ kN/m^2}}$

Imposed live on floor $= 5.0\ kN/m^2$
Imposed dead on floor (say) $= 2.0\ kN/m^2$
$\underline{\underline{7.0\ kN/m^2}}$

Roof/Floor area per shaft.

$15 \times 15 = 225$
$7 \times 7.5 = 52.5$
$2.5 \times 8.0 = 20$
$\underline{\underline{297.5\ m^2}}$



Imposed on roof $= 297.5 \times 3.0 =$ 892.5
Imposed on L3 $= 297.5 \times 7.0 =$ 2082.5
Imposed on L2 $= 297.5 \times 7.0 =$ 2082.5
$\underline{\underline{5057.5\ kN}}$

Output: UNFACTORED.

Check

Project Q4/2010	Sheet No 9
Date	Name

Calculations

Total load on pilecap: kN.
Imposed load 5057.5
Gravity load 15012.0
$\underline{\underline{20069.5}}$

Pile cap. say 21×13. $\times$ 2.0 thk.
$25 \times 2 \times 21 \times 13$ 13650.0
Assume 5 caisson piles
$5 \times 1.13\ m^2 \times 13 \times 25$ 1836.0
(15-2)

Total load on rock per shaft $\underline{\underline{35,556\ kN}}$

Let F.o.S = 3 to allow for crushing and settlement

Allowable bearing on rock $= \dfrac{5000}{3} =$

1.2 m ϕ caisson. $A = \dfrac{\pi \times 1.2^2}{4} = 1.13\ m^2$

Belled to 3ϕ $A = \dfrac{\pi \times 3.6^2}{4} = 10.18\ m^2$

Number of piles:
@ 1.2ϕ $\dfrac{35556}{1.13 \times 1667} = 19$

@ 3.6ϕ $\dfrac{35556}{10.18 \times 1667} = 3$

Output:

Total load on Rock $\underline{\underline{35,556\ kN}}$

Allowable on Rock
$\underline{1667\ kN/m^2}$

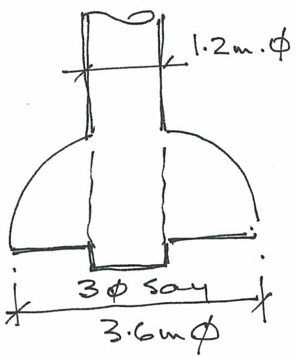
PROVIDE 5 PILES PER CORE — INCREASES F.O.S ON ROCK AND PROVIDES GOOD PILE CAP

Check

Project Q4/2010 **Sheet No** 10

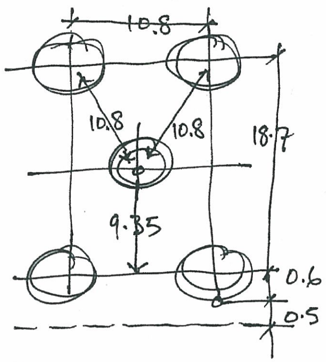
Date **Name**

Ref	Calculations	Output



Min spacing
$c/c = 3D$

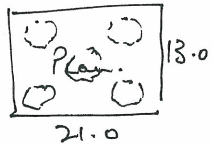
$3 \times 3.6 = 10.8$



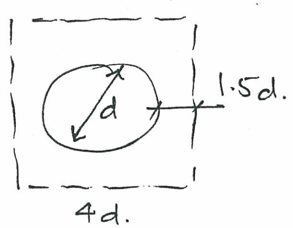
5.4 / 10.8 / 9.35

$18.7 + 2(0.6+0.5) = 21.0$

$10.8 + 2(0.6+0.5) = 13.0$



Punching shear



UNFACTORED DESIGN

BS 8110 3.7.7.4

Shear perimeter

$4 \times 4d =$

$4 \times 4 \times 1.2 = 19.2\,m. = b_v$

Let $v_c = 0.5\ N/mm^2$

$v = \dfrac{V}{b_v d}$ $\quad V_{ULT} = 1.5 \times 35\,556/5 = 1066.7\ kN.$ (FACTORED)

$\therefore d = \dfrac{V}{b_v v_c}$ $\quad \therefore d = \dfrac{1066.7 \times 10^3}{19.2 \times 10^3 \times 0.5} = 1111.0\,mm.$

Check

-20-

Project Q4/2010 **Sheet No** 11

Date **Name**

Ref	Calculations	Output

When $d = 1110\ mm$

Bar $\phi/2 = 20$

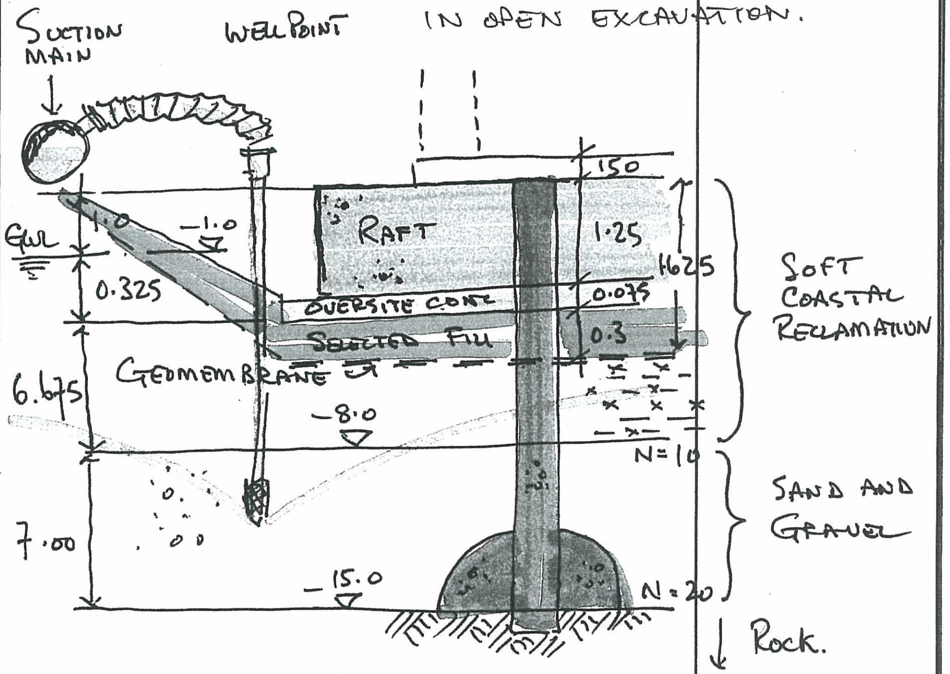
Cover. $= 75$

$1205\ mm = H$ min.

MAKE PILE CAP 1.25m. THICK ON 75mm OVERSITE CONCRETE.

* REMEMBER: GWL @ −1.0

* METHOD STATEMENT: REQUIRES SOME PUMPING TO CONTROL G.W. DURING CONSTRUCTION OF PILE CAPS IN OPEN EXCAVATION.

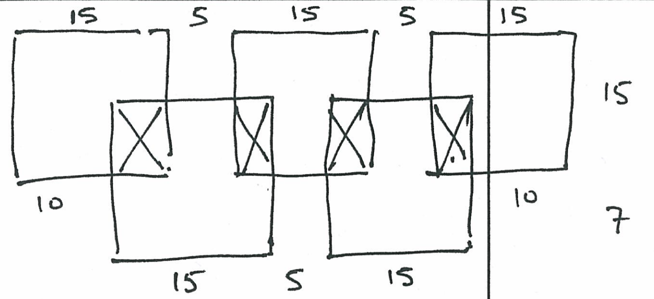


Check

Project Q4/2010 | Sheet No 100
Date | Name

Ref | Calculations | Output

STEEL CORES



Notional Floor/Roof:

$25\,\text{kN/m}^2 \times 0.2\ \text{thk.} \left[55 \times 22 - (2 \times 10 \times 7) - (3 \times 5 \times 7)\right]$

1210 − 140 − 105

$965\ \text{m}^2$

Deep metal deck with concrete topping reinforced with mesh 200 mm thk spans ~ 3.5 m. @ $5\ \text{kN/m}^2$ imposed load for a F.R.P of 2hrs = Clients Requirement #4

$25\ \text{kN/m}^2 \times 0.2\ \text{thk} \times 965\ \text{m}^2 = 48\,250\ \text{kN}$ — S.L.S.

ULT = $1.5 \times 48250 \times 3$ levels = 217,125 kN — ULT x3

Add 10% for steelwork and cladding = 21,713

238,838 kN.

Add imposed loads

Ult. = $1.5[(965 \times 2) + (965 \times 5 \times 2)] = 11\,580$ kN

(kN/m^2 Roof.) (kN/m^2 Floor)

Check

Project Q4/2010 | Sheet No 101
Date | Name

Ref | Calculations | Output

Total Ultimate load on 4 cores.

238 838
11 580
250 418 kN. MAX

Ult. load/core = $\frac{250418}{4} = 62\,605$ kN./CORE.

If. 6 legs/core $62605/6 = 10{,}434$ kN/leg.

if 10 legs/core $62605/10 = 6{,}260$ kN/leg.

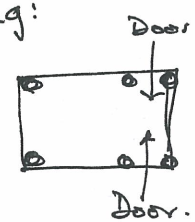
Let $p_c = 180\ \text{N/mm}^2$ or $217\ \text{N/mm}^2$

Estimated area of each column leg:

6-leg layout

$10^3 \times 10{,}434/180 = 57967\ \text{mm}^2 \equiv 580\ \text{cm}^2$

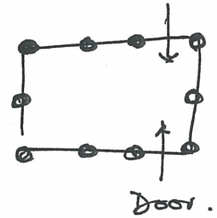
$10^3 \times 10{,}434/217 = 48083\ \text{mm}^2 \equiv 481\ \text{cm}^2$



10-leg layout

$10^3 \times 6260/180 = 34778\ \text{mm}^2 \equiv 348\ \text{cm}^2$

$10^3 \times 6260/217 = 28\,848\ \text{mm}^2 \equiv 289\ \text{cm}^2$

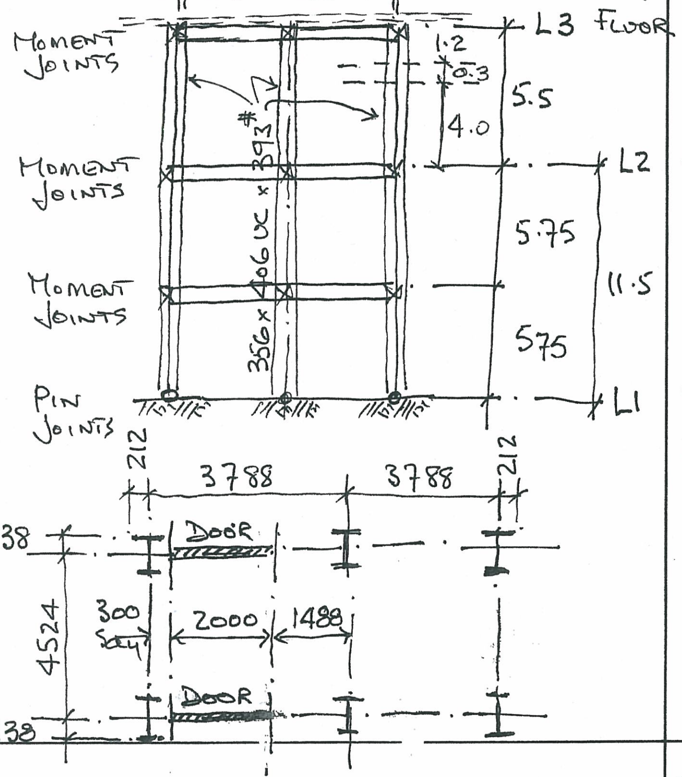


Check

Project	Q4 / 2010	Sheet No	102
Date		Name	

Ref	Calculations			Output
6x I	356 x 406 UC 393#	501 cm²	> 481	Grade 355.
	356 x 406 UC 467#	595 cm²	> 580	Grade 275
10x	305 x 305 UC 283#	360 cm²	> 348	Grade 275
	305 x 305 UC 240#	306 cm²	> 289	Grade 355
O	508 ϕ 20mm thk (10x)	307 cm²	> 289	Grade 355
□	400 x 400 SHS 20mm thk (10x)	300 cm²	> 289	Grade 355.

VIERENDEEL FRAMING



FLOOR

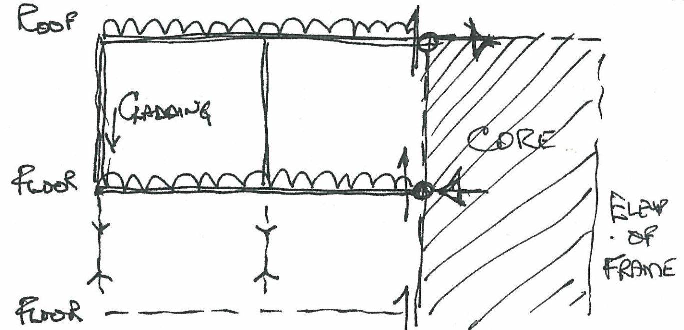
SELECT!
6x 356x406UC 393#

USE GRADE 355!

Check

Project		Sheet No	104
Date		Name	

Ref | Calculations | Output



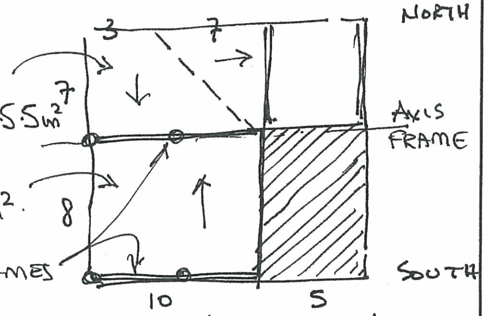
ASSUMED MODEL

ROOF/FLOOR AREA:

$(3+10)/2 \times 7 = 45.5\,m^2$

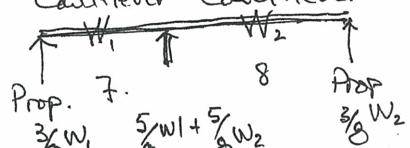
$8 \times 10 = 80\,m^2$

MAKE BOTH FRAMES THE SAME



↕ FLOORS SPAN NORTH – SOUTH

Propped Cantilever | Propped Cantilever



DISTRIBUTION OF PANEL LOADS.

ULT. ROOF/FLOOR WT $= (25 \times 0.2) \times 1.5 = \underline{\underline{7.5\,kN/m^2}}$

ULT IMPOSED ROOF $= 2 \times 1.5 = \underline{\underline{3.0}}\,kN/m^2$

FLOOR $= 5 \times 1.5 = \underline{\underline{7.5}}\,kN/m^2$

Check

	Project	Sheet No 105
	Date	Name

Ref | **Calculations** | **Output**

TOTAL ULT LOAD ON ROOF PANEL

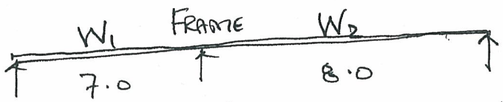
$45.5\,m^2 \times (7.5 + 3.0) = 477.75$ kN

$80.0\,m^2 \times (7.5 + 3.0) = 840.00$ kN.

TOTAL ULT LOAD ON FLOOR PANEL

$45.5\,m^2 \times (7.5 + 7.5) = 682.5$ kN

$80.0\,m^2 \times (7.5 + 7.5) = 1200.0$ kN.



Assuming $M_1 = M_2$ over Spine beam.

	$\frac{3}{8} \times W_1$	$\frac{5}{8} W_1$	$\frac{5}{8} W_2$	$\frac{3}{8} W_2$	
ROOF =	180	299	525	315	kN.
		824			
FLOOR =	256	427	750	450	kN.
		1177			

LOAD DISTRIBUTED ALONG FRAME:

ADD CLADDING + STEEL.

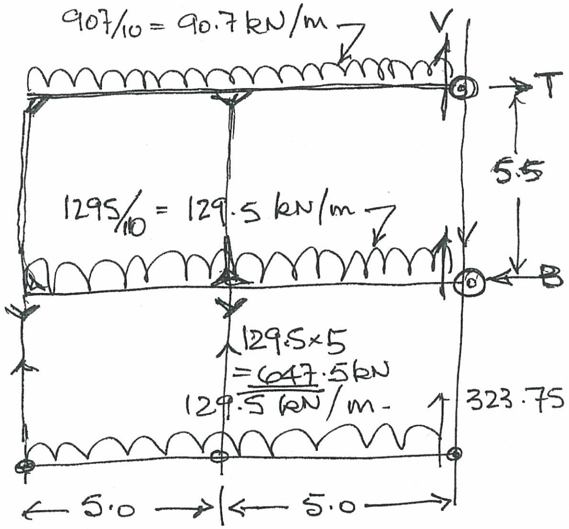
AT ROOF LEVEL = 824 kN + 83 (CLADDING & STEEL 10%) = 907 kN

AT FLOOR LEVEL = 1177 + 118 (CLADDING & STEEL 10%) = 1295 kN

Check

	Project	Sheet No 106
	Date	Name

Ref | **Calculations** | **Output**



129.5×2.5 = 323.75

ASSUMED MODEL OF LOADING ON MAIN MEMBERS

ASSUME SHEAR IS SHARED BY SUPPORTS A & B. = V

$2V = 907 + 1295 + 323.75 + 647.5 = 3173.25$ kN.

$\therefore V = 1587$ kN.

T=B Moments about B:

907 × 5	=	4535
1295 × 5	=	6475
323.75 × 10	=	3237.5
647.5 × 5	=	3237.5
		17485.0 / 5.5 = 3179 kN

kN

T: 1587, 3179

B: 1587, 3179

P.101. $p_c = 217\,N/mm^2$

$A_c = \frac{3179 \times 10^3}{217} = 14650\,mm^2 = 146.5\,cm^2$

$A = 306\,cm^2$

$P_r = 1680$ kN

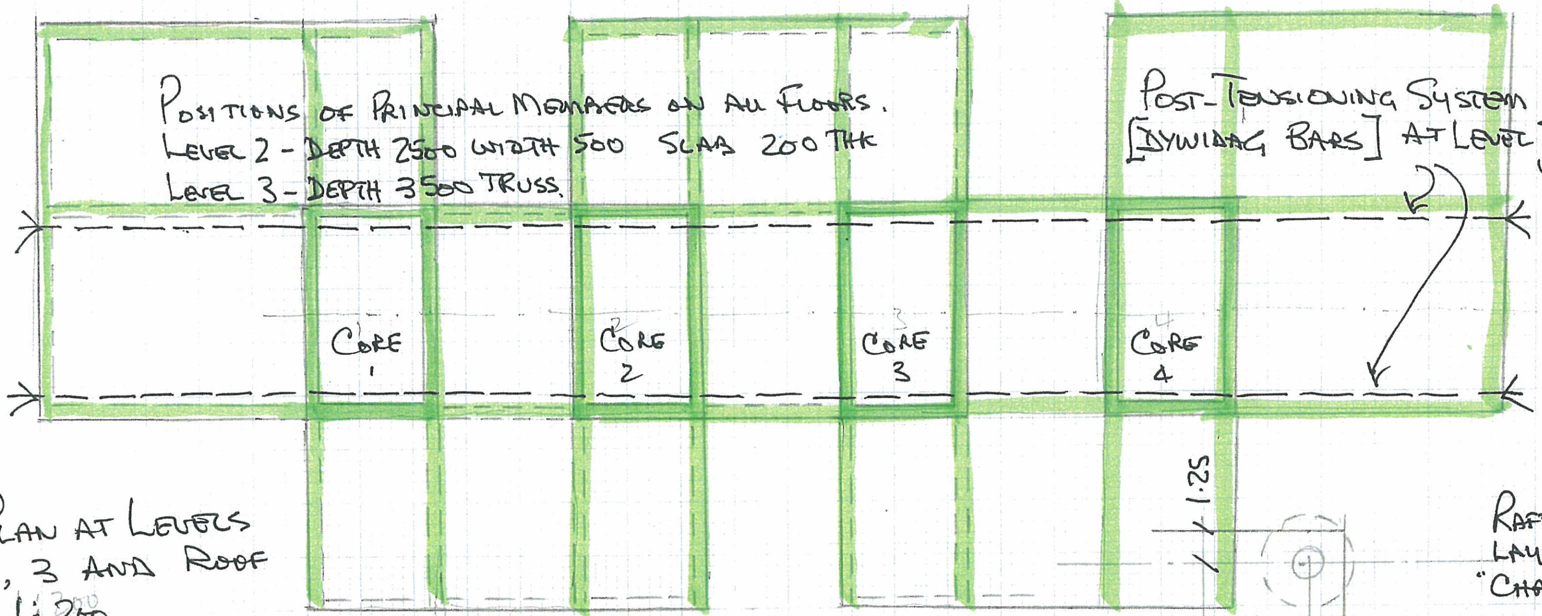
TRY 305×305 UC 204# GRADE 355

254 × 254 UC 167 A = 213 cm² P_r = 1150 kN P_{cr} = 3910

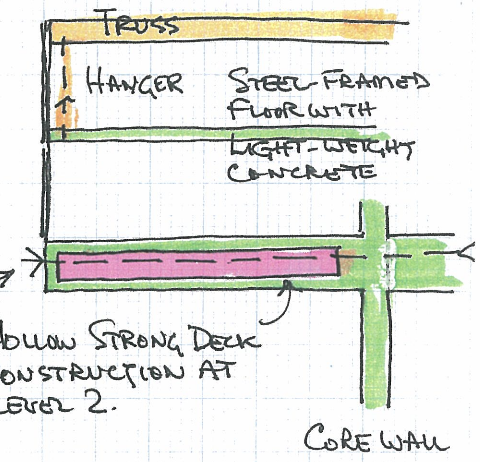
305 × 305 UC 240 A = 306 cm² P_r = 1680 kN P_{cr} = 6200

Check

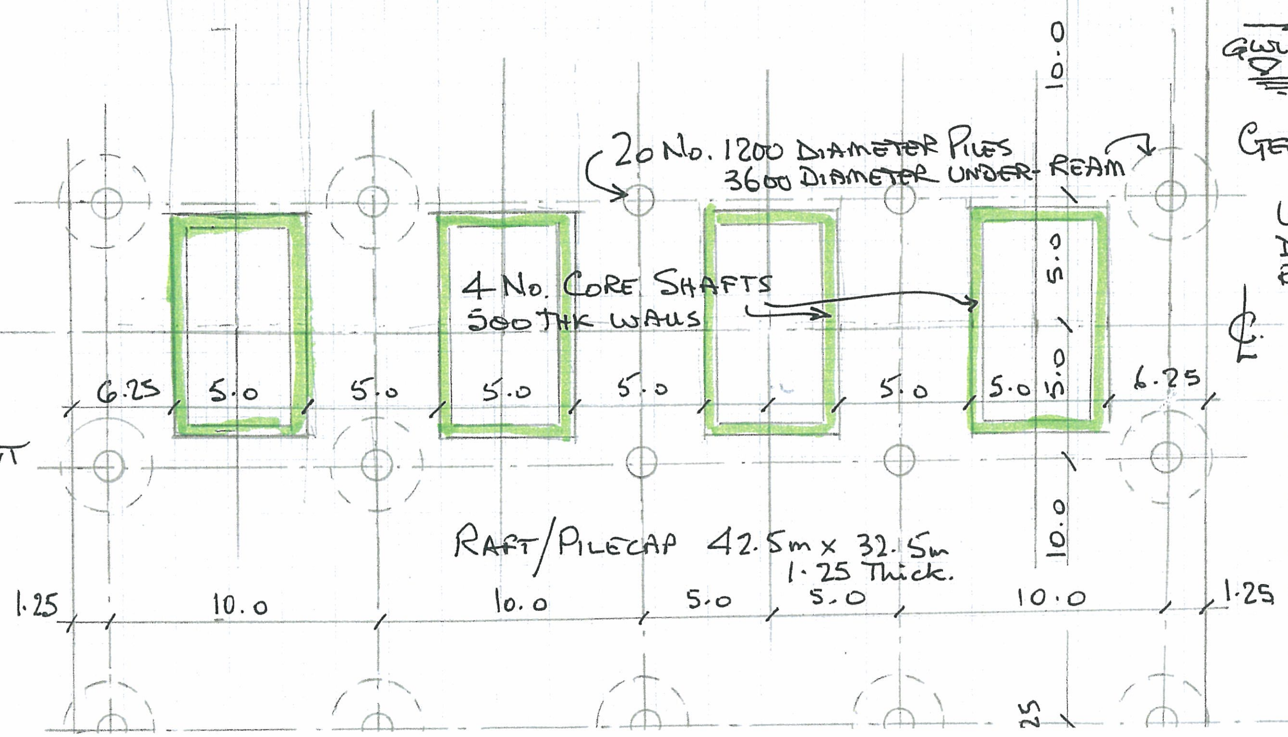
For Principal Dimensions See Figure Q3 [≡ Architect's Drawing].



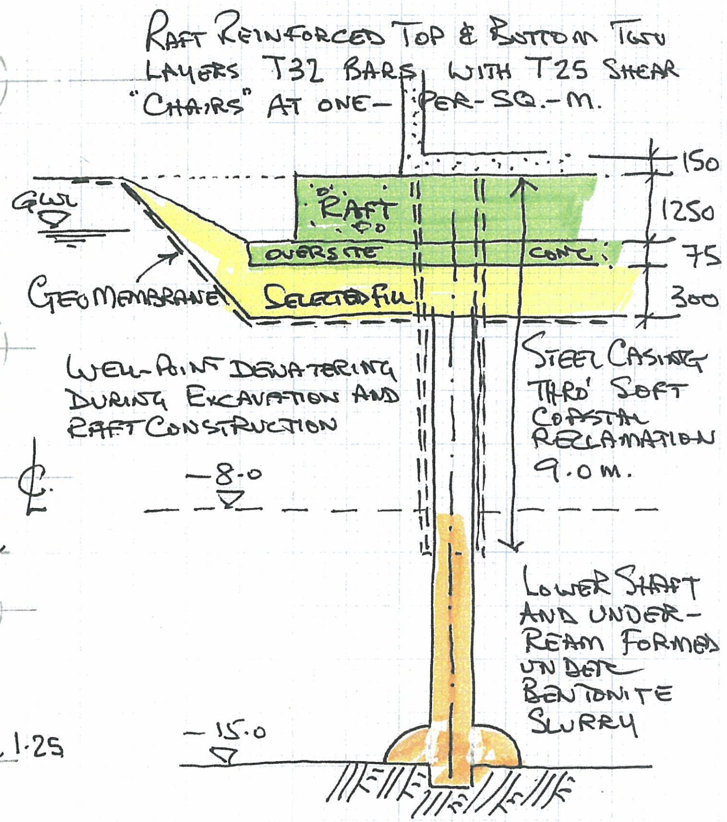
Plan At Levels 2, 3 And Roof 1:200



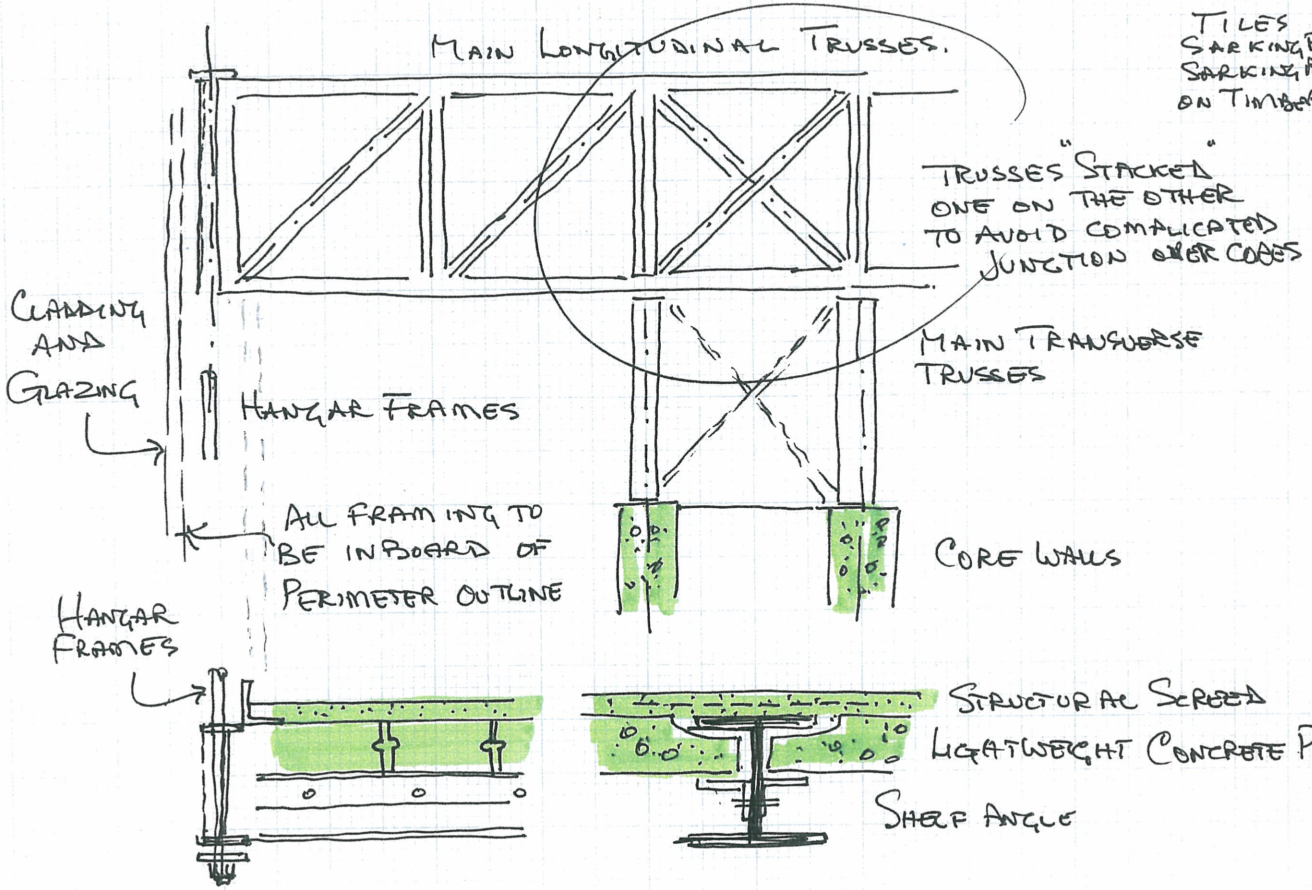
Typical Section

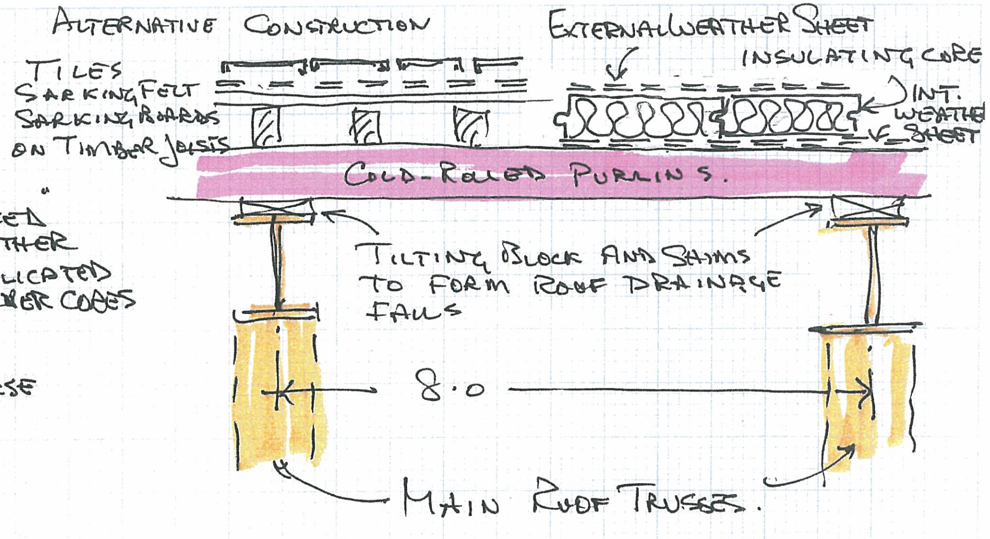


Plan At Level 1 With Pile Layout 1:200

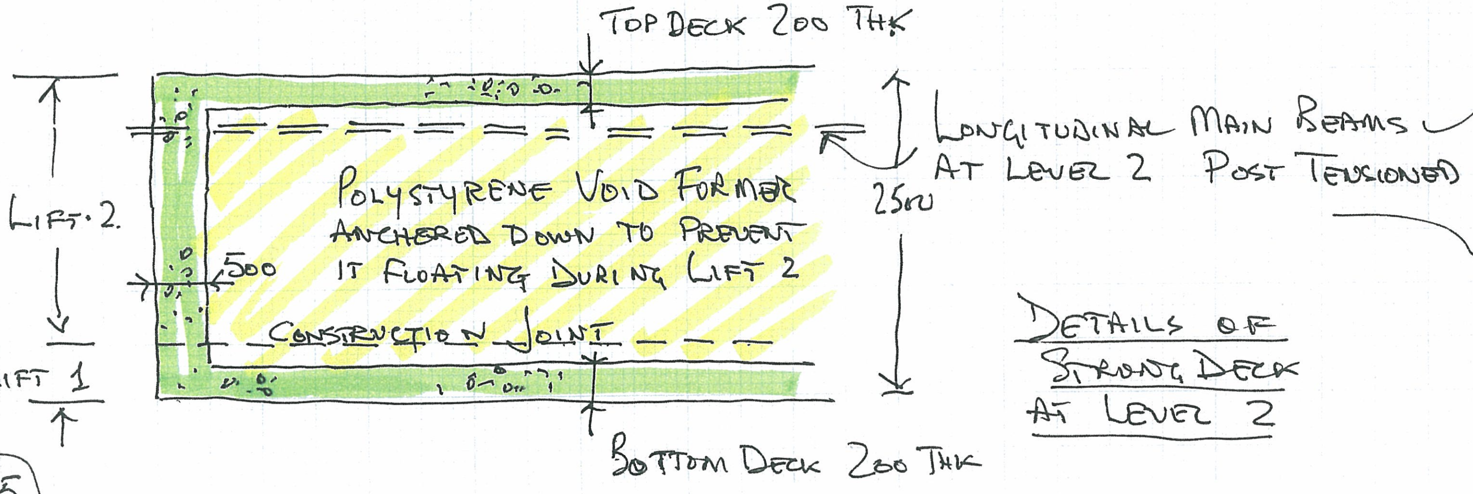


Detail - Pile And Raft

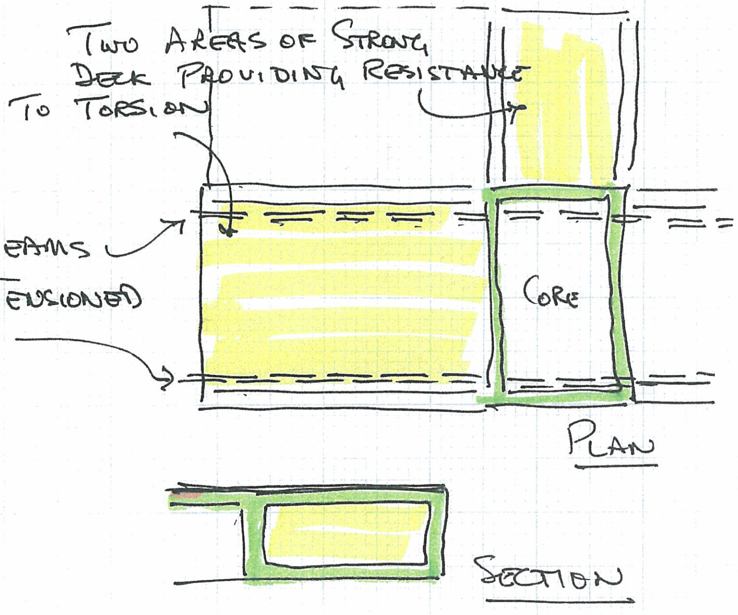




Detail of Roof Cladding



Details of Strong Deck at Level 2



Pages 24 and 25 — the drawings comprising Section 2d and worth 20 marks — took me about 2 hours to draw and sketch. My plan allowed me 4 × 20 = 80 minutes and in fact I took about 120 — i.e. 40 minutes too long. At this stage of the examination I would not be able to recover this "lost" time and I would probably not have been able to finish Section 2e – Method Statement and Programme, worth 10 marks in a time of 40 minutes!

I spent too long on the pile layout which was drawn to scale. However, I need to demonstrate my competence at communicating by drawing in a formal manner. I consider myself to be experienced at formal manual drawing. This demonstrates clearly how practiced you need to be to score enough in this part of the examination!

Section 2e Items in the Bill of Quantities and Specification need to be provided for:

1. A working platform over the whole site of 300 thk hardcore laid on geomembrane to allow piling to commence

2. Before excavation for the pilecap raft the

groundwater level must be lowered by well-point pumping.

(3) The excavation shall be lined with geomembrane, a layer of selected fill and blinded with 75 thk oversite concrete – See General Arrangement Drawing

(4) The pilecap raft shall be cast as a single continuous pour to its full thickness of 1.25 m. No construction joints will be allowed. Shrinkage cracks shall be prevented by applying "Thermal curing" to control the core temperature during cooling.

(5) The reinforced concrete cores shall be cast in stages with construction joints and using starter bars. In the interests of safety the continuity of the reinforcement may be achieved using screwed tension couplings.

(6) The level 2 "Strong Deck" and "Girdle Beams" shall be constructed off falsework raised to the appropriate levels. Support may be taken off the top of the pilecap raft. Additional support beyond the area of the raft may be necessary. No differential movement between the two types of support is allowed.

(7) similar clauses highlighting construction practice would be prepared for:

- transport and erection of the roof-level trusses
- assembly of the level 3 framing and slabs on top of the level 2 strong deck
- the winching of the level 3 deck using the hangers from the main trusses
- connections to the four cores

etc.

The outline programme requires some assumptions to be made. I have assumed that the cost is £10m and that £0.5m worth of work can be done every month: the job will last 20 months.

If the structure takes half this time (10 months) because furnishing the building (electrical, heating, lighting, air conditioning, decorating and cleaning. etc will take the rest of the time) I estimate that the following activities – on the critical path – will take

- piling – 1 month
- excavation + raft pilecap – 1 month
- 4 cores to Level 2 – 1 month
- falsework, formwork, Level 2 strongdeck - 2 month
- 4 cores to roof level – 1 month
- Erect roof level steelwork and assemble Level 3 floors upon strongdeck – 1½ month

  post-tension Level 2 girdle beams – ½ month
- winch Level 3 floors to level. – 1 month
- make building weathertight – 1 month

Construction to weathertight Total = <u>10 month</u>