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Model Answer Q1, 2016:

Institution of Structural Engineers Chartered Membership Examination

SCI, Silwood Park, Ascot,
Berkshire, UK. SL5 7QN
Tel: +44 (0)1344 636525
Fax: +44 (0)1344 636570

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For information on publications, telephone direct: +44 (0) 1344 636505
or Email: publications@steel-sci.com

For information on courses, telephone direct: +44 (0) 1344 636500
or Email: education@steel-sci.com

Email: reception@steel-sci.com

World Wide Web site: http://www.steel-sci.org

FOREWORD

This document has been prepared to assist candidates preparing for the Institution of Structural Engineers chartered membership examination. It forms one of a series of answers, demonstrating steel solutions.

The document was prepared by Ed Yandzio and David Brown of the Steel Construction Institute (SCI), with valuable input from Tom Cosgrove of the British Constructional Steelwork Association (BCSA) and Owen Brooker of Modulus.

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Contents

1 THE QUESTION

A range of past papers is available from the Institution of Structural Engineers at the following url: https://www.istructe.org/membership/examination/papers-etc

The question addressed by this model answer is Question 1 from July 2016; it is recommended that the full question is reviewed.

The question requires:

- Development of two distinct and viable schemes
- A recommendation on the scheme to be adopted
- Design calculations for the principle structural elements
- General arrangement plans, sections and elevations
- A method statement
- An outline construction programme
- A letter to the client advising on the implications of the change specified in section 1b

1.1 General arrangement

The challenge posed by the question was a new five-storey office and underground car park with the cross-section and plan as shown in Figure 1.

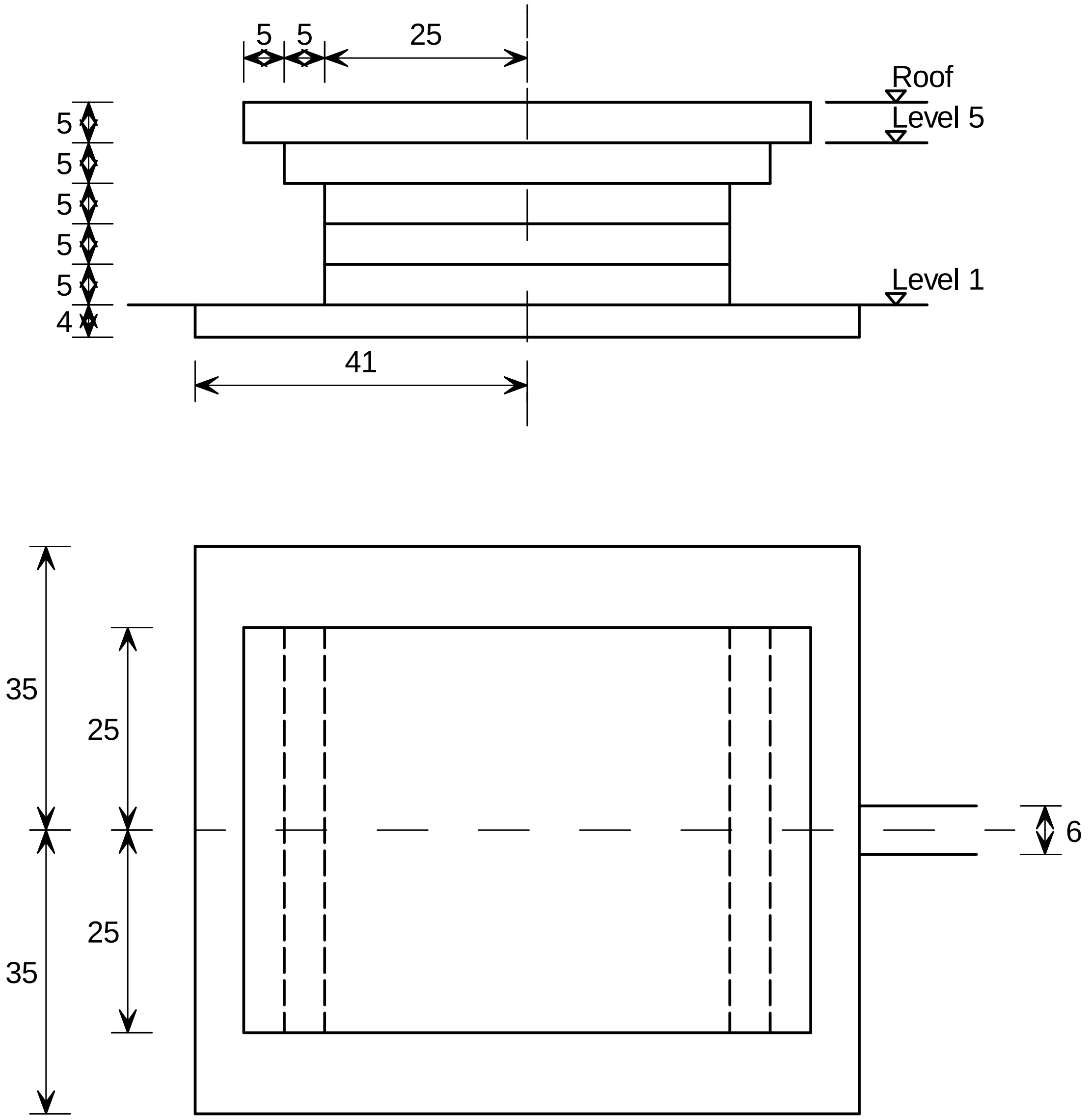

Figure 1 General Arrangement

Four core areas were required, each 5 m × 5 m.

External columns to support fully glazed elevations were required at a minimum spacing of 5 m. Internal columns were to be kept to a minimum and to suit the underground car park layout.

At least 200 car parking spaces were required, each 5 m × 2.5 m. The circulation aisles were to be 6 m minimum.

The specified clear heights and floor levels meant that the floor construction depth was limited to 2 m for the offices and 1 m for the car park.

1.2 Loading

The following loads were specified:

Roof	1.5 kN/m^2
Floors	5 kN/m^2
Car park	2.5 kN/m^2

Basic wind speed of 40 m/s based on a 3 second gust, or a mean hourly wind speed of 20 m/s.

1.3 Ground conditions

Ground level to – 1 m	Topsoil or fill
1 m – 5 m	Sand and gravel; N = 10
5 m – 9 m	Sand and gravel; N = 35
Below 9 m	Sandstone; characteristic compressive strength 1500 kN/m^2

Ground water at 5 m below ground.

1.4 Section 1b modification

The client wishes to lower the car park by 1 m, adding 1 m of topsoil on the car park roof. In addition, 0.3 m or topsoil is to be added to the office roof.

2 CALCULATIONS

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Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 636525
Fax: (01344) 636570

CALCULATION SHEET

Job No.		Sheet 1 of 40	Rev
Title			
Subject			
Client	Made by	Date	
	Checked by	Date	

SECTION 1(a) – ALTERNATIVE SOLUTIONS

Both schemes are braced steel-framed solutions, with an identical internal column layout dominated by the car park arrangement. The key difference between the schemes is the approach to the cantilever areas of the superstructure.

Car park layout

The 70 m car park dimension leads naturally to 4 rows of double bays (4 × 10 m = 40 m) plus 5 aisles of 6 m (5 × 6 m = 30 m), which total 70 m.

If columns are placed midway between the 4 rows of parked cars, the external line of columns is at 24 m, appropriate to support the superstructure above, where the elevation is at 25 m from the structure centreline.

If columns are placed at 10 m intervals, which is a convenient multiple of car spaces at 2.5 m, the external line of columns falls at 25 m from the centreline, which is appropriate to support the elevations.

Car park spaces

If each row of cars contains 28 bays, (28 × 2.5 m = 70 m), two circulation spaces of 6 m form the basement dimension of 82 m.

The total number of potential car parking bays = 28 × 8 = 224.

8 bays will be lost to the four cores, meaning 216 bays are provided, exceeding the minimum 200 specified.

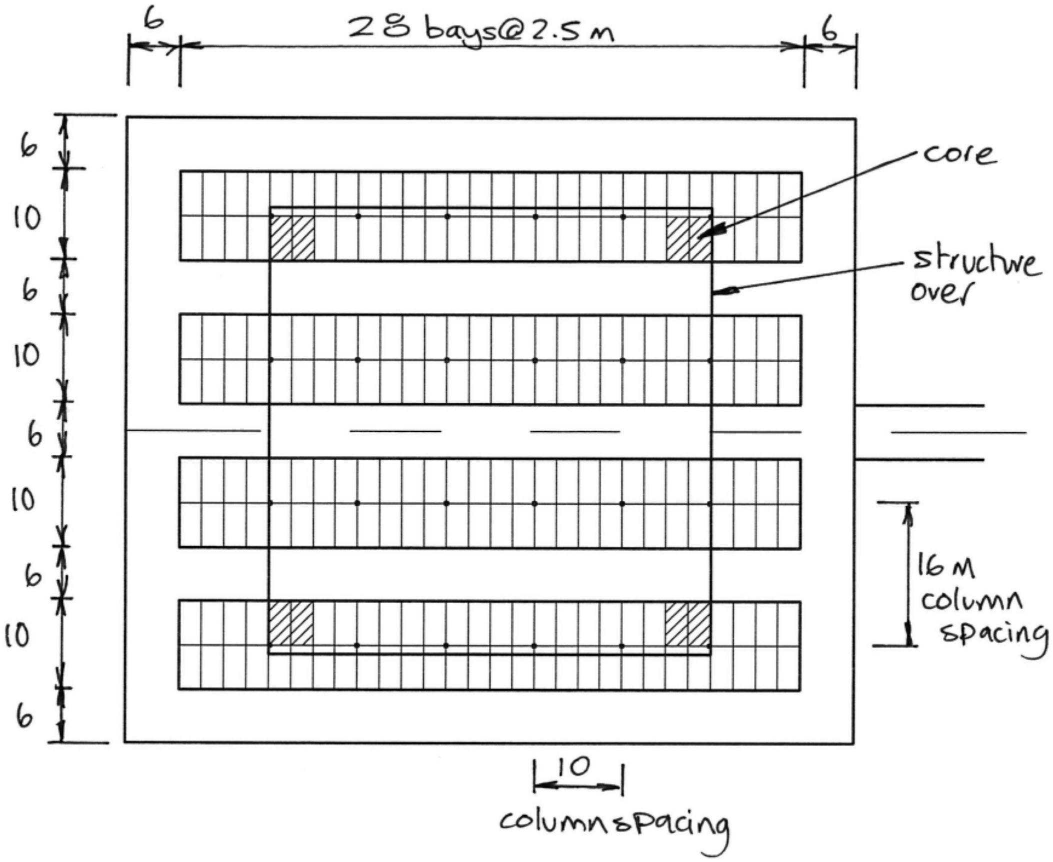
The resulting 16 m × 10 m column grid satisfies the requirement to minimise the number of columns.

The resulting car park layout and column location is shown over page.

The more challenging part of this question is to develop two schemes that offer significantly different solutions. There appears to be one layout of the car parking in the basement which suits the specified dimensions. With the clear requirement that columns in the car park area must be minimised, it appears that the preferred column layout at basement level is the same for both schemes.

Alternative schemes must therefore consider alternative arrangements for the superstructure and foundations. Generally, simply specifying different grid layouts is not considered to be of significance.

The proposed layout of cars appears to fit the basement dimensions well.



Scheme 1

The general arrangement of the superstructure is secondary beams spanning 16 m to primary beams spanning 10 m between columns. The secondary beams are proposed as composite cellular beams, supporting a concrete composite slab and permanent profiled steel decking.

The cantilevered parts of the superstructure will be supported by a plate girders at level 5. The primary roof beams will be supported by columns above the plate girder; the level below will be hung from the plate girder. The plate girder will be continuous over the line of the external columns, with a back span attached to an internal column. This internal column will be subject to uplift from the cantilever spans.

The car park will be formed with a propped sheet pile wall, with the level 1 structure being designed to carry the prop forces. The car park will have a concrete floor slab. Temporary ties will be provided to the sheet pile walls (in lieu of the permanent props) during construction.

Vertical bracing will be arranged around the four core areas to provide stability in both directions and resist lateral forces.

The roof will be formed from composite cladding, supported by secondary steelwork

Foundations to the main columns will be steel H bearing piles, driven to the rock, and pile caps.

The 5 m x 5 m cores appear ready-made to fill a pair of car spaces. There is no limitation on where the cores may be situated, so locating these at the extreme corners of the superstructure is appropriate.

The line of columns on the North and South are 1 m inside the line of the elevations. It is not considered that this is significant; cantilever brackets and wind posts can be placed to support the cladding.

16 m span composite secondary beams are orthodox – cellular beams would be common though deep rolled sections or plate girders would be possible. Cellular beams cannot readily be designed by hand, so design charts will be used later.

Secondary beams at 3.33 m spacing are also quite orthodox. No detailed design of the slab will be done, other than to recognise that the slab would be around 130 mm thick.

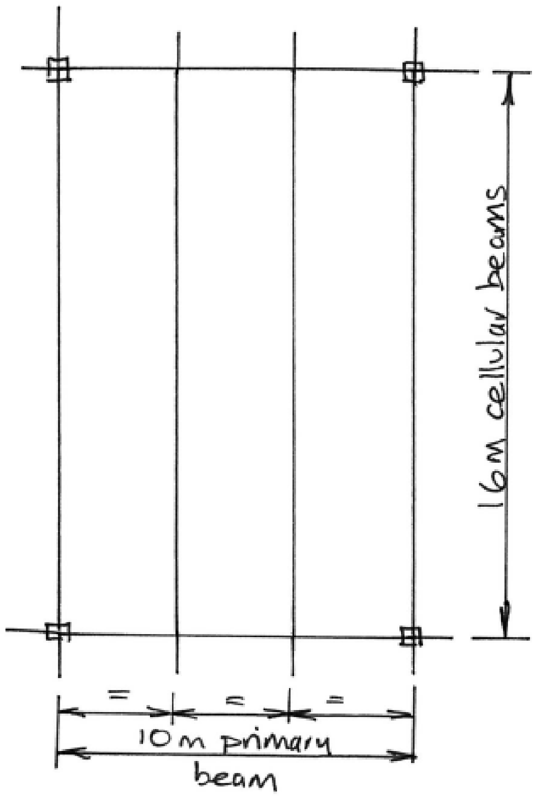
The key structural challenge is the support of the overhanging parts of the structure. In Scheme 1, a plate girder is proposed, continuous over the column on the elevation. The cantilever of 10 m and overall length of 20 m are both reasonable. The plate girder must be shallow enough to fit in the 2.0 m construction depth.

Various solutions could be proposed for the basement walls, including excavation and a conventional retaining wall. The ground water level must be considered as it would affect a conventional excavation. A sheet pile wall is relatively common – with SCI publication P275 covers this form of construction.

The rock at lower level presents an opportunity, although the water table would pose challenges to any excavated or bored solution.

Framing arrangement - superstructure

Generally 16 m secondary cellular beams, with nominally pinned connections to 10 m span simply supported secondary beams. All floor beams are composite. A typical floor arrangement is shown below.



Typical floor beam arrangement

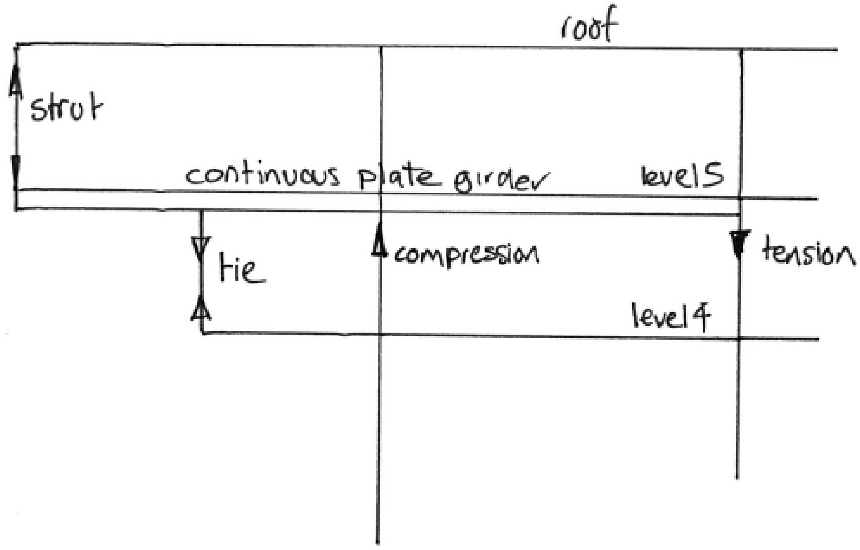
At roof level, the secondary beams are spaced at 5 m centres. Standard cold-rolled purlins at approximately 1.8 m centres support the roof cladding.

Intermediate wind posts will support the cladding on the elevations.

The general arrangement of the structural scheme to support the overhanging parts of the structure is shown over page.

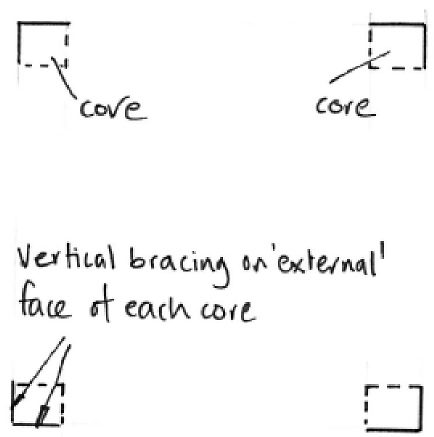
10 m secondary beams are orthodox, so no particular challenge

There are no special requirements for the roof construction at this stage, so a conventional clad structure is proposed.



General arrangement of the cantilever storeys

Stability and resistance to lateral loads is provided by vertical bracing at each of the four cores. The vertical bracing is arranged on the two 'external' sides of each core, as shown below.



Arrangement of vertical bracing

Foundations and car park structure

The external walls of the car park are to be constructed from a sheet pile wall. This will be propped at ground level, utilising the floor/roof slab at that level. The columns to the superstructure will be supported on driven steel piles below the basement floor slab. A typical cross section is shown over page.

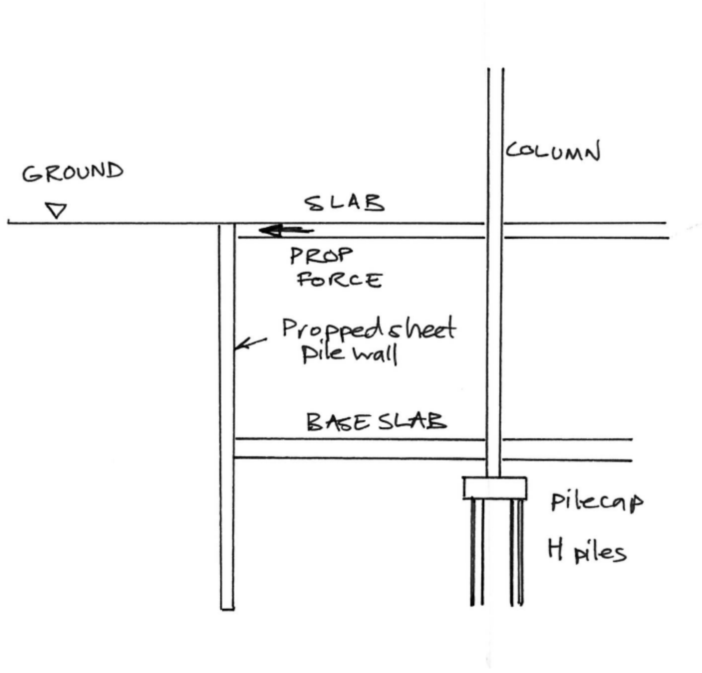
Due to the proximity of the water table, the sheet piles will be sealed by welding after installation and excavation. A sealing membrane will be fixed to the piles via a "puddle" plate, before laying the base slab. Uplift of the basement slab will be considered but is not anticipated to be a design condition.

The important issues here are the transfer of vertical load, as the lateral loads will be managed by the four cores.

Two key points are the potential uplift on the internal column, and the additional compression on the external column. The forces in the strut and tie are small and do not present a significant challenge.

Some comment about drainage is important here, as the water table is close to basement level. More elaborate measures could be taken, but some dampness is permitted in car parks. Sheet piles could be sealed in other ways, so welding is only one method

High water tables would mean that uplift should be considered, and flotation during construction. This is not a design consideration in this scheme.



Typical cross section through basement

Temporary support is needed to the basement walls, before excavation is commenced. Sheet pile anchors and ties will be installed outside the area of the basement, and outside the area of influence of the basement piles. Ties will be installed to walings connected to be pile walls, and only removed once the roof structure to the car park is completed.

Load transfer

Vertical loads on roof cladding and on the floor slabs are carried by simply supported secondary steel members to the primary beams. The primary beams are connected to columns, with nominally pinned connections. Cantilevered sections of the structure are supported by plate girders on the primary grids. The plate girders are continuous with a back span connected to an internal column. The column will be verified for the tension induced by the cantilever loading.

All lateral loads are connected via the floor and roof diaphragms to the vertical bracing arranged around the four cores. Lateral loads are conveyed to the piles under the basement.

Lateral loads from the sheet pile wall are supported at base level by a ground slab, and at ground level by the roof slab of the car park. The basement excavation is propped at car park roof level by the beams in each direction, which will be verified for the axial force from the basement wall.

Differential settlement of the superstructure with respect to the basement walls is anticipated. Joints at the perimeter of the superstructure will be provided to allow articulation of the car park roof slab.

Key point here is the temporary stability of the sheet pile wall, which is propped in the final arrangement. A cantilever sheet pile wall would be simpler, but the piles would be much longer, and clash with the rock. To avoid uncertainty with the design of the piles, a propped solution is preferred. Sheet pile walls are not easy to design by hand, but design charts allow a quick comparison between propped and unpropped solutions, and specify the prop force.

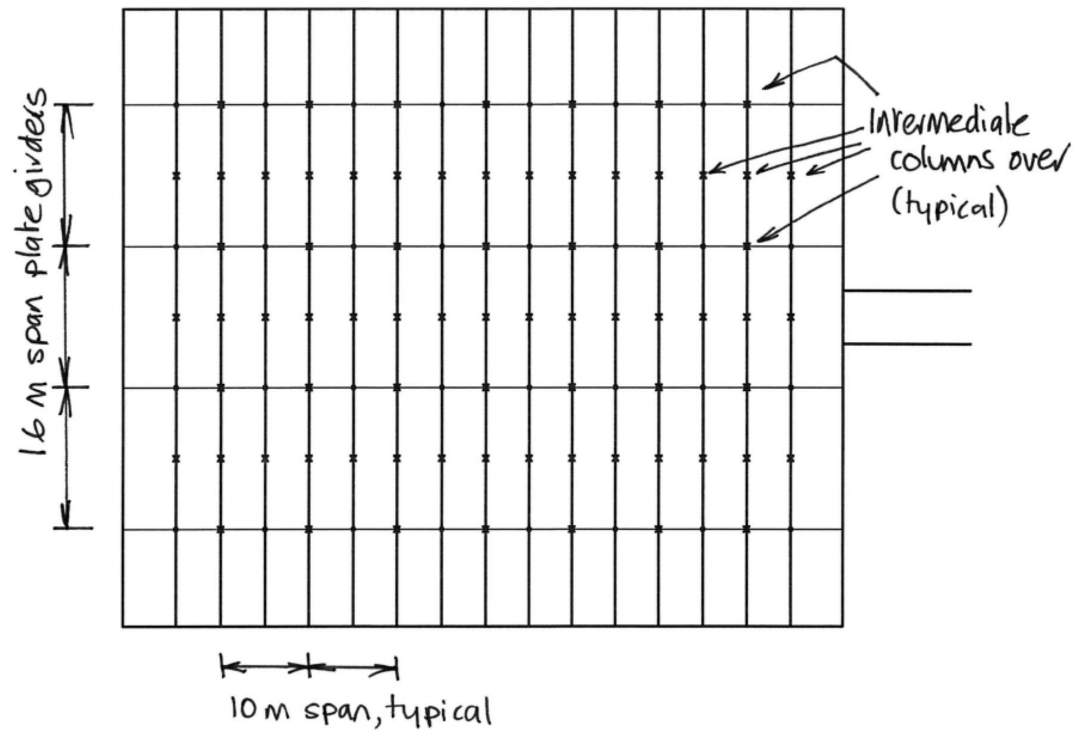
Describing load transfer is a good discipline, as it demands consideration of vertical and lateral loads.

Candidates should always be aware of differential settlement – which is likely to appear in most questions. The solution is likely to be measures that accommodate the settlement, rather than trying to overcome the problem.

SCHEME 2

Scheme 2 involves additional columns to form a grid of 8 m × 5 m within the superstructure, leading to smaller beams and columns.

To meet the Client's objective of minimal columns in the car park area, the 16 m × 10 m grid previously determined is proposed, meaning that at ground level, a transfer structure is required to carry loads from internal columns to those through the basement. In the 16 m direction, plate girders are proposed, as the span in relatively large with a central point load. In the 10 m direction, deep rolled sections are proposed. The arrangement at ground level is shown below.



Scheme 2 – layout at ground level showing plate girders to support intermediate columns above.

In Scheme 2 the overhanging parts of the superstructure are supported by inclined members back to the lower floors, as shown in the general arrangement over page.

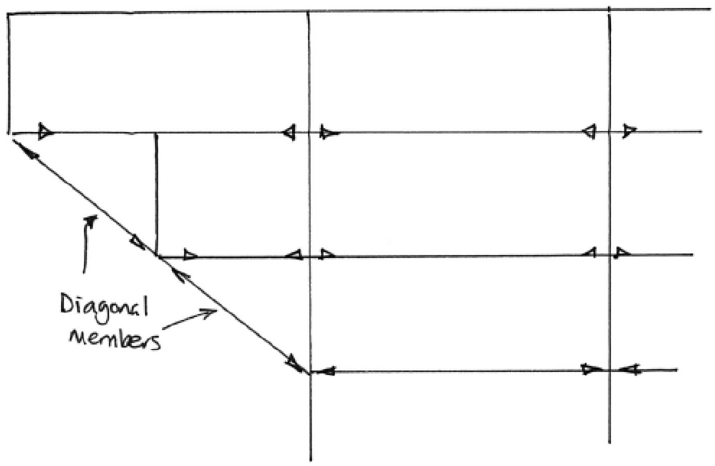
Although a simple grid change is generally not considered significant, the change necessitates a transfer structure in order to maintain the 16 × 10 m grid within the car park. For many steel designers, the 8 × 5 m grid proposed here is unnecessary, as 16 × 10 is quite orthodox.

The most significant opportunity for a substantially different scheme comes in the support for the overhanging sections of the structure. The brief is silent on the need to maintain the cross sectional profile of the structure, so propping the overhangs is an admissible alternative.

A more elegant solution would be to provide a storey height Vierendeel truss between level 5 and the roof, and to hang the level 4 overhang from it. This was considered at length, but it was concluded that the “cantilever” aspect of the truss solution made the scheme rather similar to Scheme 1.

Pragmatically, although Vierendeel trusses can be readily analysed by hand (See the Steel Designer’s Manual for an example), it is much easier to propose a propped solution.

A Vierendeel truss would be relatively expensive to fabricate.



General arrangement of supports to the overhanging structure

The diagonal members, in compression under 'gravity' loading, induce a tension in the horizontal members at levels 5 and 4. Members and connections at this level will need to be verified for this tension. Members and connections at level 3 will be verified for loads including the compression induced by the diagonal members.

In Scheme 2, lateral loads are carried by the floor diaphragms to the vertical bracing at the cores, and thus to the foundations.

The car park structure remains identical to Scheme 1. Modest alternatives could be proposed, but a propped cantilever sheet pile solution is considered to be the most efficient solution. After assessing the column loads a piled solution has been identified as the only appropriate solution – pad foundations are not possible if the 10 m × 16 m grid of columns in the car park is retained.

A concrete retaining wall to the perimeter of the car park was considered, but not preferred because of the water table level. The excavations for the retaining wall may need dewatering during construction, which is not necessary with a sheet piled retaining wall.

Scheme selection

Scheme 1 has been selected as the preferred option.

The primary reason for the selection of Scheme 1 is the external appearance of the superstructure. Although the design brief does not preclude the diagonal members proposed in Scheme 2, it is felt that the probable intent to emphasise the overhanging nature of the upper storeys would be compromised if Scheme 2 was selected.

The key point here is that inclined members always have a components in the orthogonal directions, which must be recognised in design.

A potential weakness of this model answer is that Scheme 2 could be considered insufficiently different to Scheme 1. It is felt that the constraints of the car parking in the basement, really drive a single preferred solution for that level; differences in the schemes must be identified in other areas.

The additional columns in Scheme 2 lead to shorter primary beams throughout the superstructure, and lighter weight columns, but increases the piece count to be erected, making the construction programme longer. The increased number of columns reduces the internal flexibility that a large grid spacing provides. The necessity to provide a transfer structure at ground level adds significantly to cost.

The column-free space that Scheme 1 provides with a 10 m × 16 m grid is preferred. The 16 m span members are not unorthodox.

Neither scheme involve any particular difficulties in erection.

The Scheme 1 basement structure is straightforward, involving familiar construction processes, and making efficient use of the structure provided as a roof to the car park as a prop to the piles.

Stability is provided by orthodox bracing arrangements, with no onerous constraints.

SECTION 2 – DESIGN CALCULATIONS

(for Scheme 1)

Preliminary design completed in accordance with EN 1993-1-1

Design combinations of actions determined in accordance with expression 6.10 of BS EN 1990

All steelwork is S355

Roof steelwork

Roof loading

Say 0.30 kN/m^2 permanent

Say 0.15 kN/m^2 services

1.5 kN/m^2 variable (from brief)

ULS loading = 1.35(0.3+0.15) + 1.5(1.5) = 2.86 kN/m^2

Secondary roof members span 16 m, at 5 m centres

Bending moment = 2.86 × 5 × 16^2/8 = 458 kNm

Try 457 × 191 × 67 (M_b = 478 kNm at 2 m with C_1 = 1.0)

Unfactored variable action = 1.5 kN/m^2

$$\text{deflection} = \frac{5\times5\times1.5\times16000^4}{384\times210000\times29400\times10^4} = 103 \text{ mm}$$

Allowable = 16000/200 = 80 (not a brittle finish)

Adopt 533 × 210 × 82 I_{yy} = 47500× 10^4 mm^4

deflection = 64 mm, OK

Primary roof members span 10 m with a central point load from the secondary member.

An opportunity to comment on any construction challenges, including temporary works.

S355 being the commonly available grade of steel for rolled sections.

Roof beams are simply supported. A cellular beam could have been selected as an alternative.

Point load = 2.86 × 5 × 16 = 229 kN

Bending moment = 229 × 10/4 = 572 kNm

Adopt 533 × 210 × 82 (M_b = 580 kNm on 5 m with C_1 = 1.77)

Floor steelwork

The following calculations cover typical floors 2 to 5. The selected members will be verified for the additional forces at level 1, and revised sections specified if necessary.

Floor loading

Take typical permanent action for a composite slab as 3.5 kN/m^2

(typical for a 130 deep slab, trapezoidal decking; 3.33 m span is reasonable)

Variable action = 5.0 kN/m^2 (from the brief)

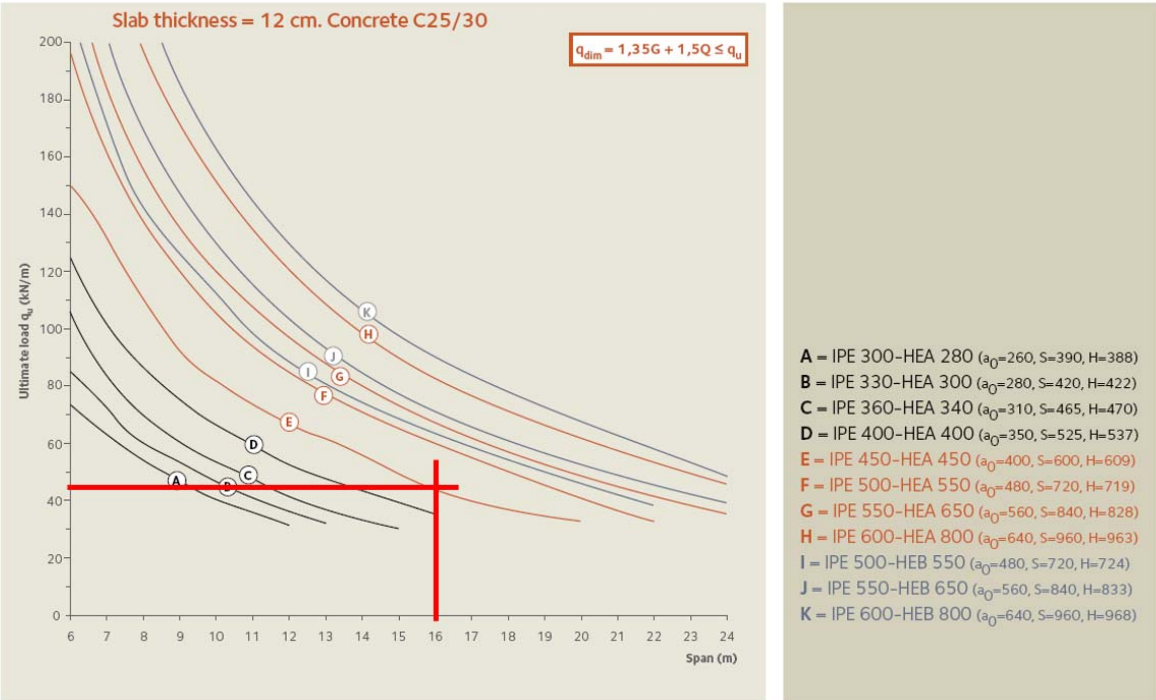
Allow 0.8 kN/m^2 for partitions

Secondary floor beams (16 m span)

Secondary floor beams span 16 m, at 3.33 m centres

ULS load on beam = (1.35 (3.5) + 1.5 (5 + 0.8))× 3.3
= 44.7 kN/m

Design chart 10: Steel-Concrete composite section - starting sections IPE & HEA-B. S = 1.5 a_0 - Grade S355



Adopt solution E: 600 mm deep cellular beam, 400 mm holes at 600 mm centres. Top member IPE 450 and bottom HEA 450

No calculations of permanent actions; this value is typical for composite slabs.

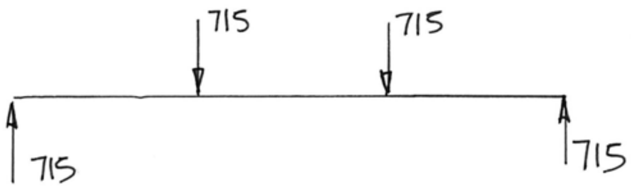
It is not possible to design cellular beams quickly by hand, so design charts are a reasonable alternative. The charts in the Arcelor Mittal guide cover composite floor beams, so are ideal. The specified solutions are given as “European” section sizes. At the preliminary stage, it would be acceptable to specify equivalent UK members.

As is typical with composite fabricated sections the bottom flange is larger than the top.

Primary floor beams (10 m span)

Primary beams are loaded from secondary beams at 1/3 points

ULS load from secondary beam = 44.7 × 16 = 715 kN



Forces on a 10 m span primary beam

Maximum bending moment = 715 × 3.33 = 2381 kNm

Adopt 838 × 292 × 226
(M_b = 2620 kNm on 4 m with C_1 = 1.0)

Columns

Columns will be spliced immediately above level 3

Column loading

Typical internal column loaded area = 10 × 16 = 160 m^2

ULS load from roof = 2.86 × 160 = 458 kN

ULS load from each floor
= (1.35(3.5) + 1.5(5.8)) × 160 = 2148 kN

Lower level columns

Axial load at base = 458 + 5 × 2148 = 11200 kN

Buckling length say 5 m (allows for floor construction depth; an effective buckling length could be 0.85 × system length)

At this stage the load will be assumed to be axial only

Adopt 356 × 406 × 340 UC (N_{bz} = 11300 kN on 5 m)

This column will be used from level -1 to level 3

Upper level columns

Above level 3, Axial load = 458 + 2 × 2148 = 4754 kN

Adopt 305 × 305 × 198 UC (N_{bz} = 5740 kN on 5 m)

Plate girder at Level 5

A simple design will be adopted, assuming that the bending moment is carried by the flanges and the shear in the web. The plate girder will have through-deck studs welded on site to the top flange, meaning that the member is fully restrained in sagging regions of the bending moment diagram, Deep secondary beams are provided which should mean that lateral torsional buckling is not a design check in the hogging lengths, though this would need to be verified.

These primary beams could have been designed as fully restrained, but with a beam of this size and a short length between restraints, the difference in resistance is small.

Columns have been designed for an axial load only. There would be a nominal moment from floor beams if the shear reactions differed, but this is not the case for an internal column.

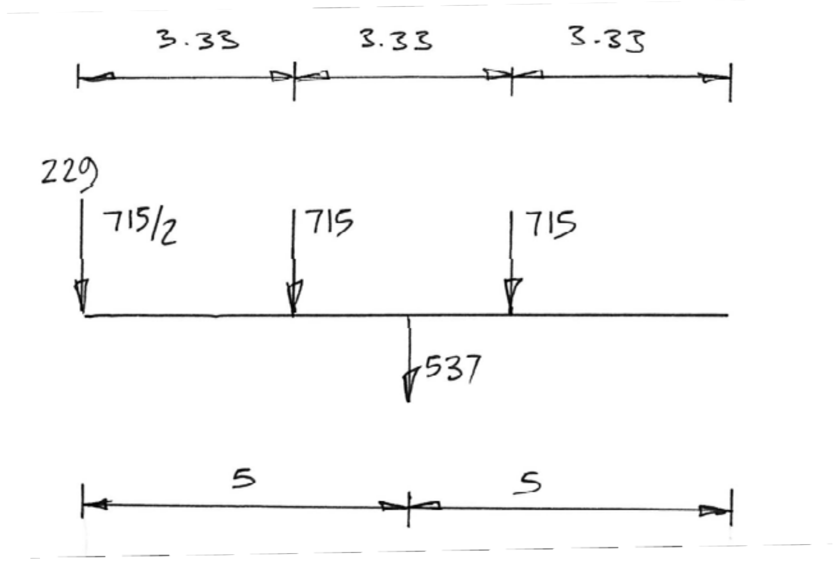
The columns on the elevations will experience a nominal moment, and additional axial load from the overhanging parts of the structure. Generally, the external columns are loaded from half the area compared to internal columns and at this stage, it has been assumed that the same columns will be adopted throughout.

The simple approach of allocating the moment to the flanges is appropriate at this stage.

Point load at tip of cantilever from the roof
$= 2.86 \times 16 \times 5 = 229$ kN

Hanger load from level 4
$= (1.35(3.5) + 1.5(5.8)) \times 2.5 \times 16 = 537$ kN

Point loads from each secondary beam at level 5 = 715 kN



Forces on the plate girders

Maximum bending moment

$= 537 \times 5$
$+ 715 \times (3.33 + 6.66)$
$+ (358 + 229) \times 10$

$= 15700$ kNm

Assume the flanges carry the bending moment

Assume a 1.5 m deep plate girder (within the 2.0 m construction depth)

Say 1.45 m between flange centroids

Force in flange = 15700/1.45 = 10828 kN

Assuming S355 steel and $f_y = 335$ N/mm^2

Then area required $= 10828 \times 10^3 / 335 = 32322$ mm^2

Try 650×50 flange (32500 mm^2)

Check flange classification
$c/t = 320/50 = 6.4$

Class 1 limit $= 9\varepsilon$ and $\varepsilon = 0.84$

So Class 1 limit $= 9 \times 0.84 = 7.56$

So flange would be Class 1

Anticipating a relatively thick flange, the “design” strength has been reduced.

Web buckling will dominate the web resistance

Shear (at support)

= 229 + 715 × 2.5 + 537 = 2554 kN

From BS EN 1993-1-5;

k_τ = 5.5 approximately

Try 15 mm web, then $\sigma_E = 190000\left(\dfrac{15}{1500-50-50}\right)^2 = 21.8$

$$\bar{\lambda}_w = 0.76\sqrt{\frac{f_y}{k_\tau \times \sigma_E}} = 0.76\sqrt{\frac{355}{5.5\times 21.8}} = 1.3$$

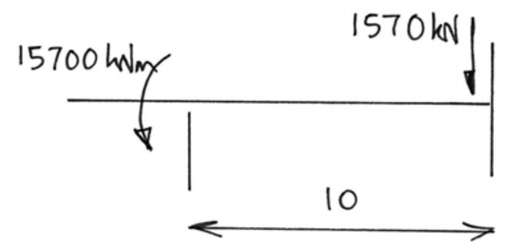
then $\chi_w = \dfrac{0.83}{1.3} = 0.64$

Web resistance $= \dfrac{0.64\times 355\times 1400\times 15}{\sqrt{3}}\times 10^{-3} = 2755$ kN

A plate girder, 1500 mm deep, with flanges 650 × 50 and 15 mm web will be satisfactory. Web stiffening will be required, notably at points of load application, and where the plate girder is continuous over the intermediate support.

Equilibrium/Uplift check

Assuming <u>no</u> load on the backspan (conservative), the uplift on the internal column is 15700/10 = 1570 kN



Equilibrium of the plate girder

Considering permanent actions <u>only</u> applied to the column, and factoring these by 0.9 as they are relieving, the vertical force at the base is:

160 × (0.3 + 5 × 3.5) × 0.9 = 2563 kN

Thus the permanent actions alone means that there is no uplift at the base, although the splice between column sections will need to be checked for tension.

If web buckling were to be avoided, the web would be very (unreasonably) thick. The following calculations make some assessment of the shear resistance calculated in accordance with BS EN 1993-1-5

Candidates would be advised to review the design rules for web buckling, so that suitable approximations may be made, as has been done in these calculations.

Usually, it might be expected that the question was arranged that uplift would be a design consideration. Here, assessing only the permanent actions, and amplifying them by 0.9 because they are ‘relieving’, no uplift occurs.

Connections between plate girder and columns

At the intermediate column, a cap and baseplate detail will be adopted, with stiffeners to the plate girder web in line with the column flanges.

At the internal column, uplift force = 1570 kN

Assuming M24 Class 8.8 bolts, number of bolts needed
= 1570/136 = 11.5, so minimum 6 pairs M24 adopted.

Column splice design

Under gravity loading, a bearing splice will be detailed. This splice must carry any potential uplift from the reaction of the plate girder, with only two floors providing resistance at the splice level.

Tensile force at the splice
$= 1570 - 160 \times 3.5 \times 2 \times 0.9 = 562$ kN

This force will be readily accommodated by standard splice details. Say 3 pairs of M24 bolts on each flange

Resistance $= 6 \times 136 = 816$ kN

Say each flange plate is 300×20

Net area $= 20 \times (300 - 2 \times 26) = 4960$ mm^2

Resistance of two flange plates
$= 2 \times 345 \times 4960 \times 10^{-3} = 3422$ kN, OK

Check of external column

The external columns carry the additional load from the overhanging portions of the structure, but in contrast to the internal column previously designed, generally only carry load from half on the floor area previously assessed.

By inspection, the additional load from the overhanging portions of the structure is more than offset by the reduction in load from the remaining floors. Therefore, at this stage, the same column sizes will be used throughout. In the final design, the nominal moments from the beam reactions would be considered in the verification of the external columns.

Lateral loads

The hourly mean of 20 m/s specified in the brief is appropriate for use with BS 6399.

Assume that all factors, including the direction factor S_d, are 1.0

Then $V_s = 20$ m/s

From Table 4, assume $S_b = 2.0$ (conservative)

Then $V_e = 2 \times 20 = 40$ m/s

pressure $q_s = 0.613 \times 40^2 \times 10^{-3} = 0.98$ kN/m^2

A simple assessment to see if a reasonable connection may be envisaged.

Although there is no uplift at the base, tension is possible at the splice, so this is verified. The potential tension is not significant and readily accommodated by an orthodox splice detail.

The values quoted are not consistent with the Eurocodes, so BS 6399 is preferred. Some assumptions are made in order to determine a reasonable wind load.

From Table 5a, *B/D* = 1 (approximately) so take net pressure coefficient as 1.2

Area of larger elevation

= 50 × 15
+ 60 × 5
+ 70 × 5
=1400 m^2

Characteristic wind force = 1400 × 1.2 × 0.98 = 1646 kN

Equivalent horizontal forces

The ULS load on a floor = 1.35(3.5) + 1.5(5.8) = 13.4 kN/m^2

Based on the ULS loads on the roof and floors, the EHF are:

At the roof; 1/200 × 50 × 70 × 2.86 = 50 kN

At level 5; 1/200 × 50 × 70 × 13.4 = 235 kN

At level 4; 1/200 × 50 × 60 × 13.4 = 201 kN

At level 3,2,1; 1/200 × 50 × 50 × 13.4 = 168 kN

Base shear

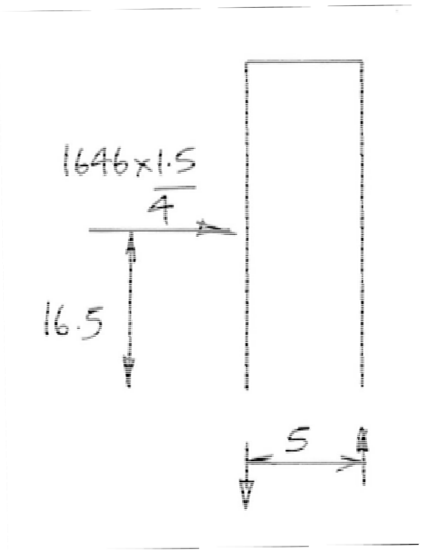
The total base shear, assuming both the wind and vertical loads to be acting simultaneously is therefore:

1.5 × 1646 + 50 + 235 + 201 + 3 × 168 = 3459 kN

Because the building is symmetrical with identical bracing in each core, the load can be assumed to be shared equally between cores.

Shear at the base of each core = 3459/4 = 865 kN

Check for uplift at the base of the cores under wind loading



Assessment of overturning moment

Uplift = 1646/4 × 1.5 × 16.5/5 = 2037 kN

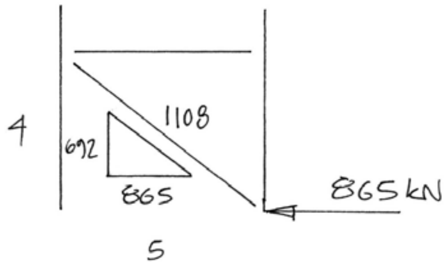
Because the calculations are directed to the base shear and design of the vertical bracing, overall force coefficients are appropriate.

In Eurocode design, unlike design to BS 5950, the EHF are applied in combination with the design lateral loads. Strictly, when the floor loads and wind loads act in combination, one of them would be the ‘accompanying’ action, and would be reduced by the appropriate ψ factor. For simplicity here the EHF and lateral loads have been combined with no ψ factor; this is conservative.

From previous calculations, the downward load on an internal column, when factored by 0.9 based on permanent actions only, is 2563 kN

Thus there is no net uplift at the base of the bracing

Diagonal bracing



Vertical bracing force

Force in bracing = 1108 kN; bracing length = 6.5 m

Try 219 × 10 CHS (N_b = 1180 kN on 7 m)

Frame stability

Frame stability should be verified, under the EHF. With well-ordered bracing, as is the case in this structure, frame stability is unlikely to be critical.

If the bracing is designed for EHF of 2.5%, rather than 0.5%, second-order effects will be small enough to be ignored.

The EHF based on 0.5% are
50 + 235 + 201 + 3 × 168 = 990 kN

Based on 2.5%, the total would be 4590 kN, or 1238 kN per bracing stack.

The force in the diagonal is 1238/865 × 1108 = 1585 kN

Adopt 244 × 12.5 CHS (N_b = 1890 kN on 7 m)

Basement wall design

A propped sheet pile wall is proposed.

With ground conditions given as N = 10 over the height of the wall, a friction angle of 30° may be assumed

Ground water is at the level of the inside of the basement, which is “Condition B” in the piling handbook.

Figure 8.23 covers tied (propped) walls, condition B and a friction angle of 30°

Bracing could be reduced in size at higher levels, but this finesse is not needed at this stage.

An analysis to calculate lateral deflections under the EHF is not realistic – three solutions are possible:

Conclude by inspection that the frame is not sensitive to second order effects

Design the bracing to a maximum utilisation of (say) 85% to allow for some second order effects

Design the bracing for a larger value of EHF

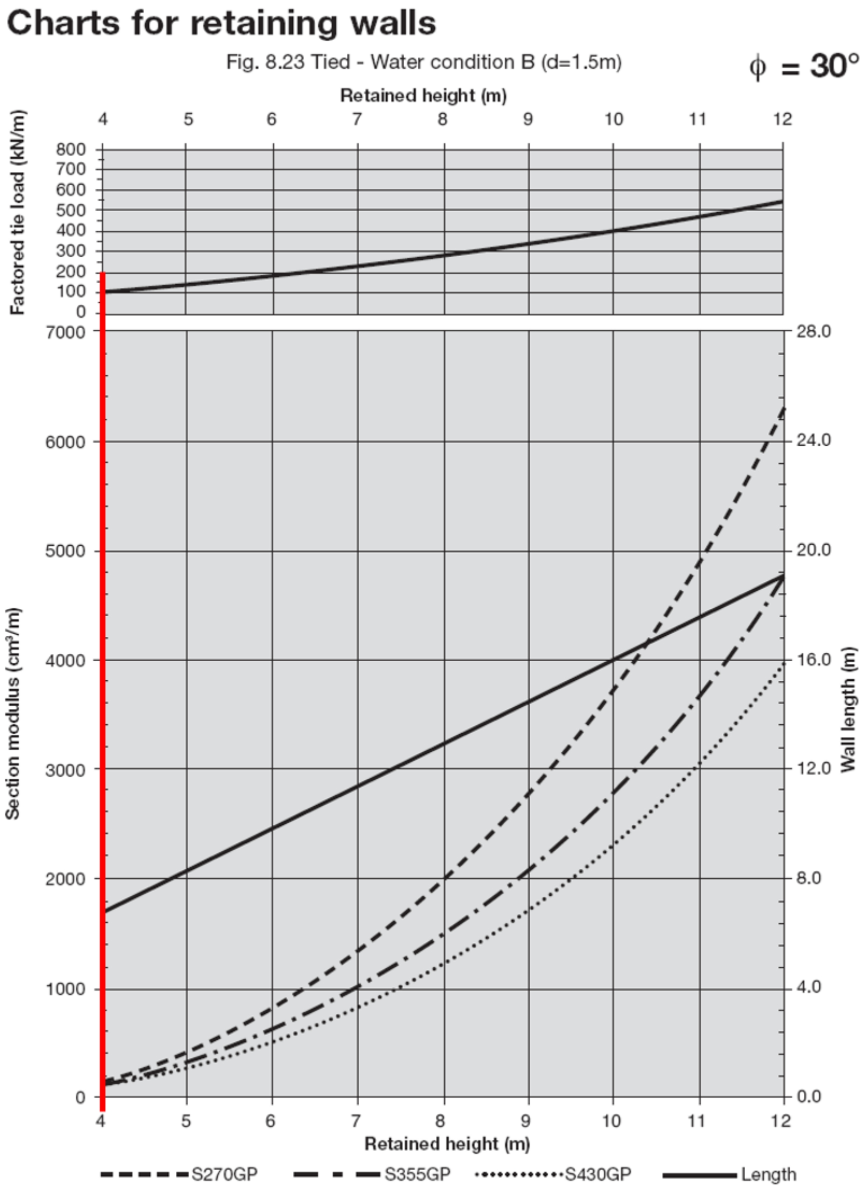
Design charts are the only reasonable way to produce a preliminary design. The Piling Handbook provides a range of situations, friction angles and water table levels.

No friction angle is provided – this is a reasonable value.

Chart 8.23 shows:
piles of 7 m length would be appropriate

the tie force is 100 kN/m length of wall

piles of a section modulus of 200 cm^3/m would be appropriate in S355GP steel



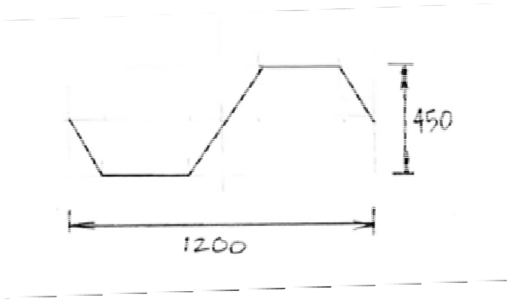
Pile retaining wall design chart

PU 8R provides 775 cm^3/m and would be appropriate for the ground conditions

The prop force of 100 kN/m will be used to design horizontal waling members around the basement wall, and to check the design of the floor beams at level 1 which must carry this force.

The pile length means that the pile toe does not hit the rock – a cantilever solution would, so introduces complications.

The design situation is relatively modest, with only a small pile section required.



Pile cross section

Base slab

Because the basement is a car park, some water leakage and damp patches are allowed. Detailing at the pile/slab interface will be orthodox.

A 175 mm thick base slab will be provided, with A393 mesh, laid over 150 mm compacted sub-base. Movement joints will not be provided, to minimise water ingress.

Column foundation design

ULS load on an internal column (see previous) = 11200 kN

Approximate SLS load = 11200/1.4 = 8000 kN

Assume four driven steel piles down to rock

Allowable bearing pressure on rock = 1500 kN/m^2 = 1.5 N/mm^2 as given in the brief.

SCI Guide P335 (H-Pile Design Guide) notes that an allowable bearing pressure for steel pile design is not appropriate – the resistance of a pile is often yield of the section.

Assuming 9 piles per pile cap, the load per pile is 8000/9
= 890 kN

Try 305 × 305 × 186 UBP, Area = 237 cm^2

Stress at pile tip = $890 \times 10^3 / 237 \times 10^2$ = 38 N/mm^2

This stress is well below yield (355 N/mm^2) and from P335 is reasonable for the sandstone identified in the brief.

Adopt 9No. 305 × 305 × 186 UBP

Assume piles at 900 mm centres, pile cap say 450 mm deep

Reinforcement top and bottom, lapped with reinforcement site welded to column. Additional bars installed through holes site drilled in pile top after cutting pile to length.

These details are from SCI publication “Steel intensive basements”

The allowable bearing pressure is appropriate for pad foundations, but not for driven steel piles. The SCI guides point out that a much higher value may be assumed in design, which has been done here.

Pile support to beams

Around the basement, beams are supported on the basement external wall. Sheet piles could be driven to the sandstone to support this load, or H section piles could be driven at those locations, as a "king pile". This latter solution will be adopted.
The beam end reaction (from a primary beam) = 715 kN

SLS end reaction is approximately 715/1.4 = 510 kN

Assuming one 305 × 305 × 186 UBP, the stress at the pile tip
$= 510 \times 10^3/237 \times 10^2 = 21.5\ \text{N/mm}^2$

This stress is low, so one 305 × 305 × 186 UBP is adopted at each primary beam support location on the basement wall.

Waling to perimeter piles

The tie force is 100 kN/m. Assuming a continuous waling, the maximum bending moment $= wl^2/12 = 100 \times 16^2/12 = 2133$ kNm

Assuming two members are used to form the waling, the bending moment in each is 1067 kNm. There is no buckling, as the waling is attached to the piles, so Adopt twin 610 × 229 × 113
(M_{cy} = 1130 kNm)

Temporary support to the basement piles

Temporary support is required to the basement piles until the internal struts are installed. Small sections of short sheet piles will be driven outside the basement perimeter, and tie rods installed to the waling members before excavation commences. Once the roof to the car park is installed, the temporary works may be removed.

Consideration of differential settlement

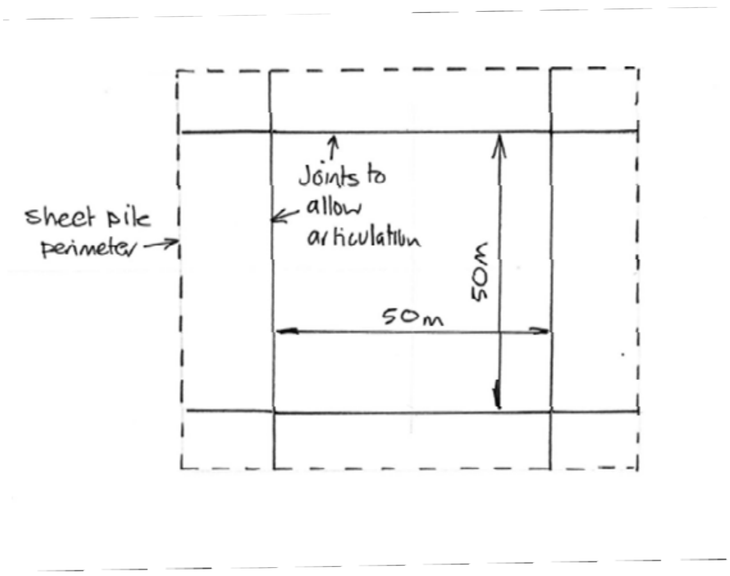
It is certain that there will be differential settlement between the internal steelwork and the perimeter to the basement. This will be managed by detailing construction joints in the car park roof with flexible filler around the perimeter of the superstructure. The beams spanning from the internal columns to the perimeter will rotate at their supports, since these connections are nominally pinned.

At the basement wall, the end of the beam must be supported. One approach would be to calculate the resistance of the sheet pile wall. Here, it is assumed that an individual additional H pile is driven to support each beam.

The waling design assumes a lateral UDL, and waling members that are continuous. The 16 m span between props produces a large moment, but this is considered preferable than introducing additional members within the floor grid to reduce the spacing of the props.

Although no design is completed here, the paragraph demonstrates that the temporary situation has been considered.

As previously mentioned, differential settlement is likely to be a feature of every question.



Articulation joints in the slab at level 1

Verification of car park roof beams for combined axial load and bending

At the car park roof level, the beams will carry both moment and axial force due to them acting as struts across the basement.

The design bending moment in the 10 m span primary floor beams is 2381 kNm; the Lateral torsional buckling resistance of the selected 838 × 292 × 226 UB is 2620 kNm

The primary floor beams are at 16 m spacing, so the axial load is 100 × 16 = 1600 kN

The primary floor beams will not buckle in the minor axis, since there are intermediate floor beams connected to the floor slab, which acts as a diaphragm.

In the major axis, N_{by} = 8690 kN

Verifying combined axial load and bending:

$\frac{1600}{8690}+\frac{2381}{2620}=1.09$ Larger beams must therefore be specified for the car park roof level

Try 838 × 292 × 251

Overall depth = 859 + 130 = 989 mm, OK

M_b = 2970 kNm (on 4m)

N_{by} = 9780 kN (on 10 m)

Verifying combined axial load and bending:

$$\frac{1600}{9780}+\frac{2381}{2970}=0.97$$

Adopt 838 × 292 × 251 S355

A simple interaction has been used to assess the beam, which needs to be increased to accommodate the axial force

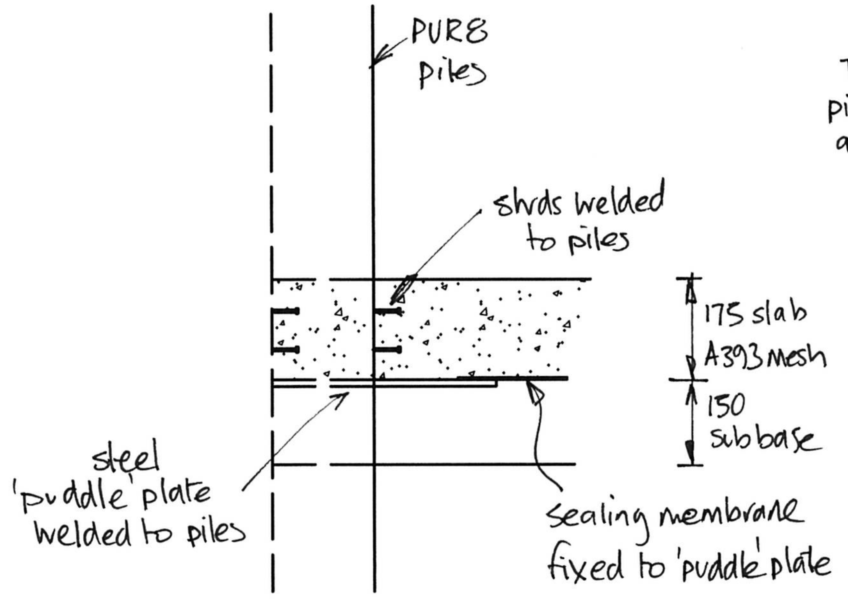
The available construction depth of 1 m must be respected

The secondary beams (cellular beams) at the car park roof level will also need verifying for the axial load. The concern here is modest, as the axial load is only 333 kN, since the secondary beams are spaced at 3.33 m. It is likely that slightly larger members will be needed at the car park roof level to accommodate the axial forces.

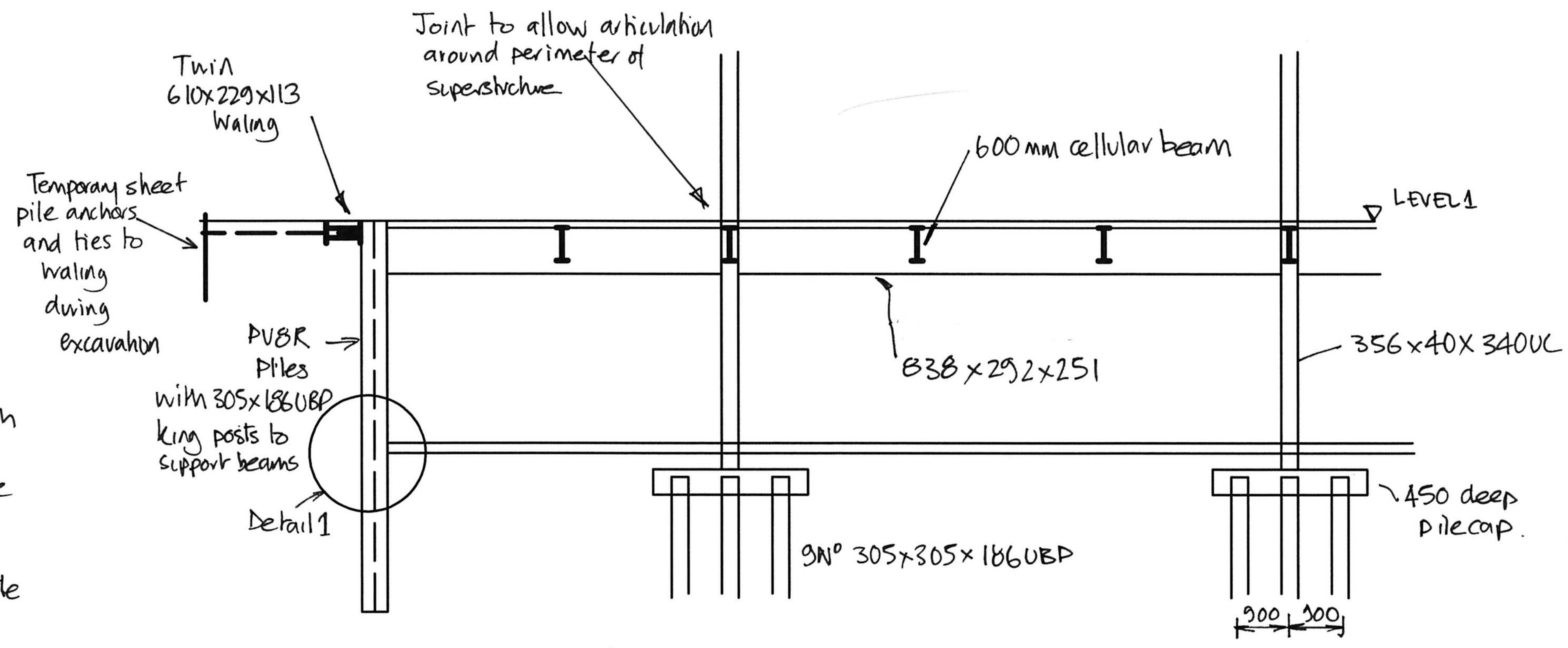
No design of a cellular beam with axial force has been attempted.

3 DRAWINGS

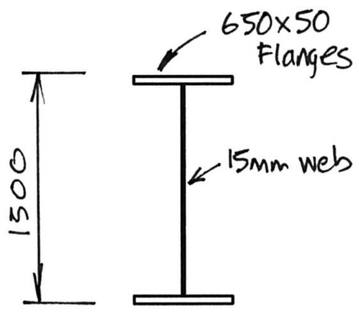
The following sheets reproduce the A3 drawings prepared for the scheme.



DETAIL 1



BASEMENT AND FOUNDATION DETAILS



SECTION A - A

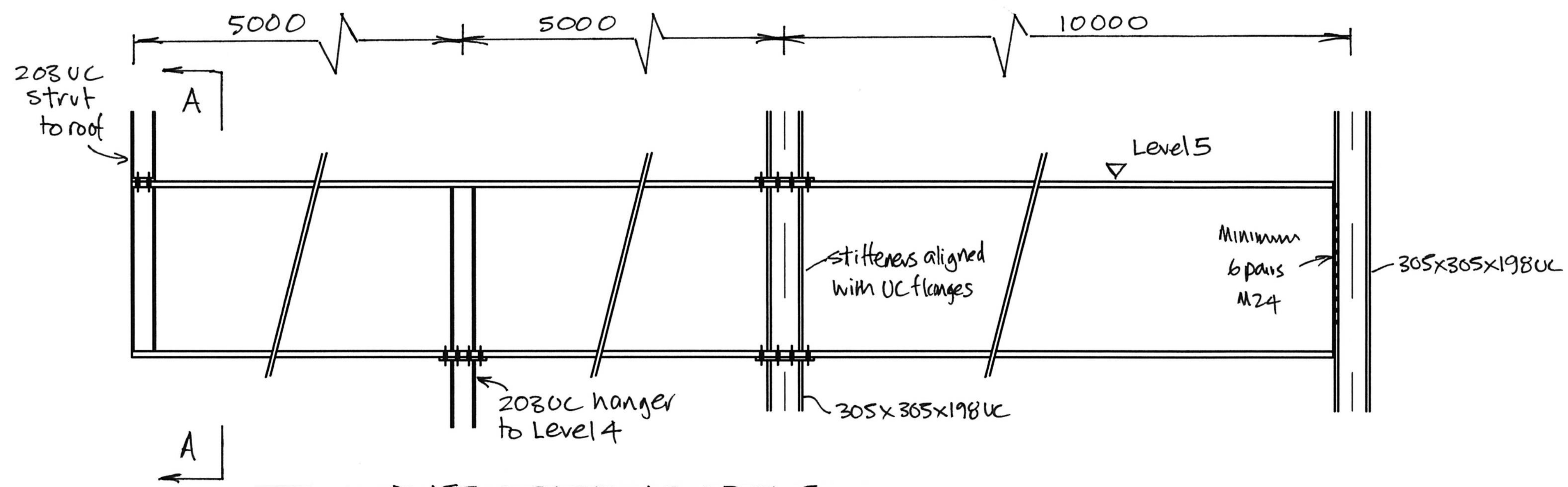
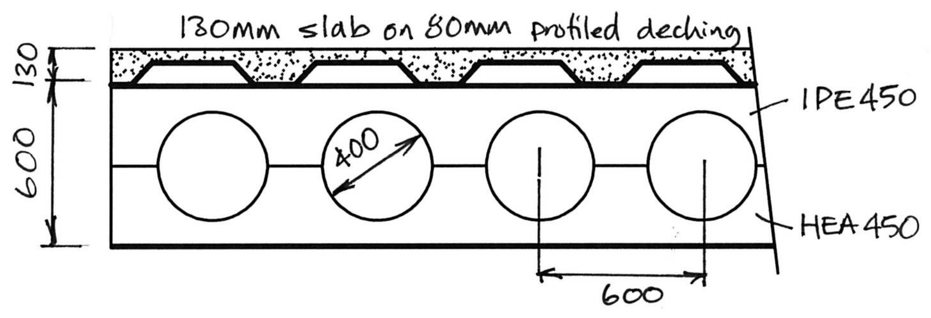


PLATE GIRDER AT LEVEL 5



CELLULAR BEAM AND SLAB

PRIMARY CONSTRUCTION DETAILS

Dimensions in m UNO
All steel S355

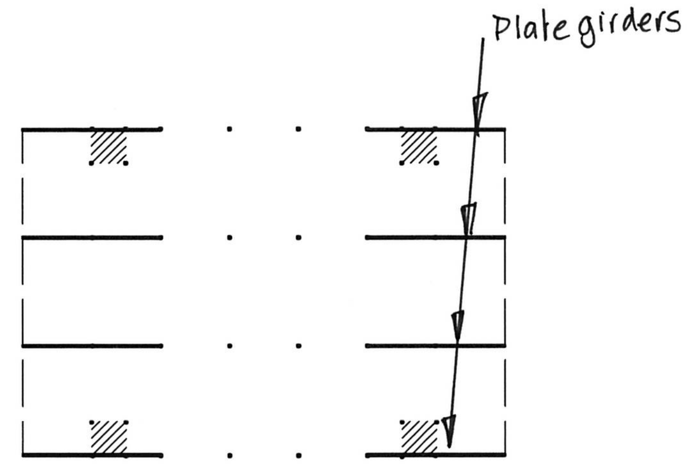
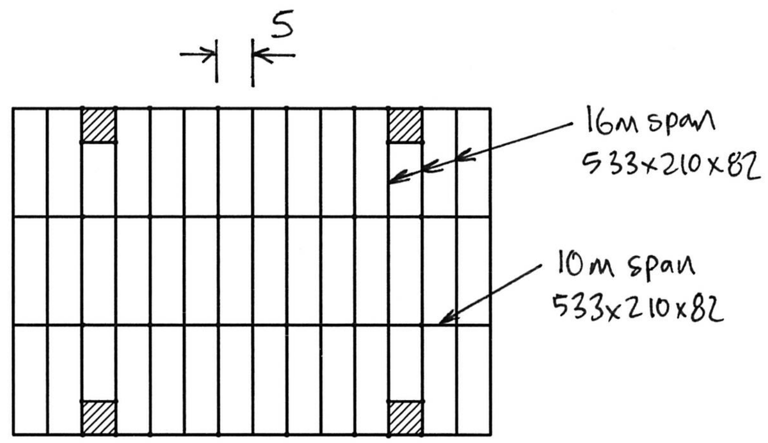
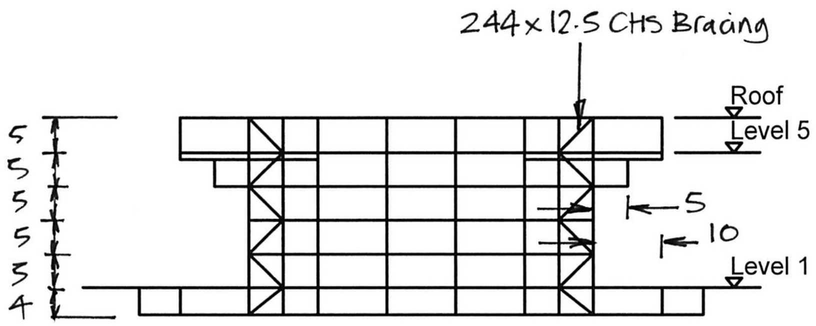


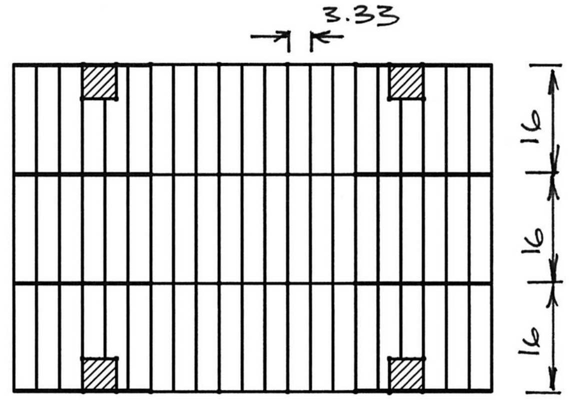
PLATE GIRDERS AT LEVEL 5



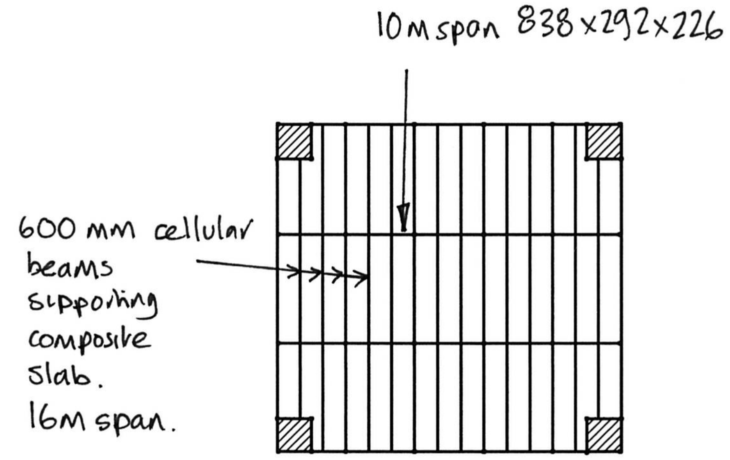
PLAN AT ROOF LEVEL



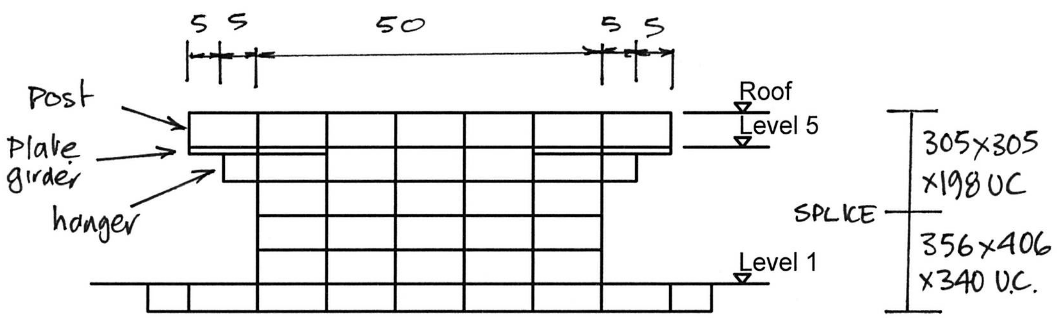
SECTION AT BRACED CORES



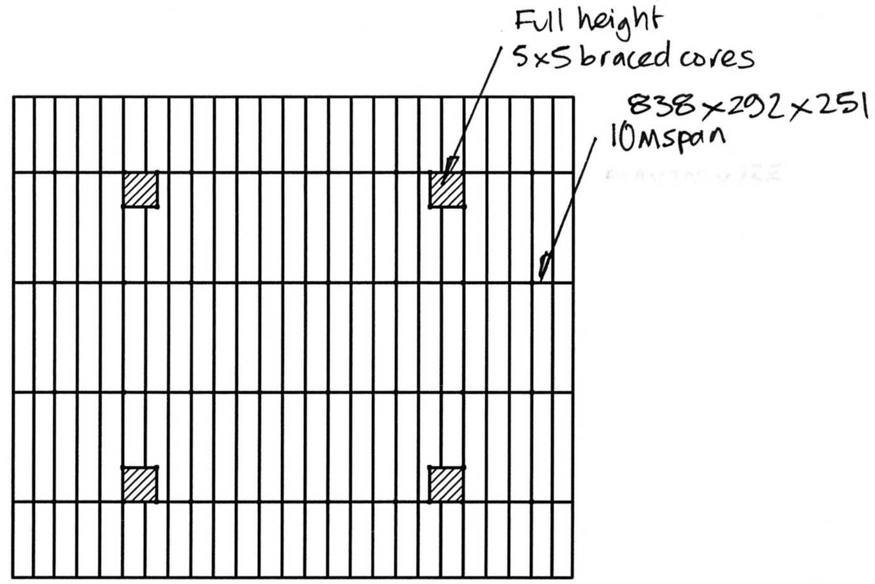
PLAN AT LEVEL 5



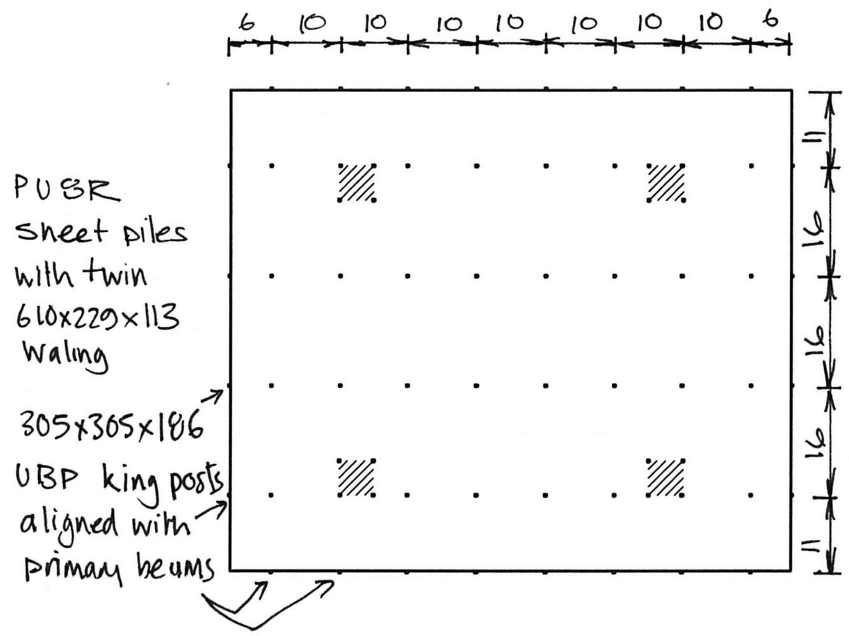
PLAN AT LEVELS 2,3



CROSS SECTION



PLAN AT LEVEL 1



FOUNDATION PLAN

PLANS AND SECTIONS

4 METHOD STATEMENT

Preliminaries

- The site should be secured to prevent unauthorised access by member of the public.
- As there is much working at height, full PPE must be worn, including fall arrest equipment and safe methods of work must be established and followed.

Sequence of work

- Hard standing is to be provided along the line of the retaining wall, so that the sheet piles may be installed.
- Sheet piles are to be installed on the line of the retaining wall, on the four sides of the basement.
- Temporary anchors are to be installed at intervals around the outside of the sheet pile wall, to tie back the sheet piles before excavation. The temporary anchors are to consist of 3 sheet plies for each anchor
- Install waling beam around the permanent sheet piles, tie and anchor back to the temporary anchor points.
- Excavate the basement down to underside of blinding. As part of this operation, dewatering equipment is to be installed inside the line of the piles. Modest dewatering is anticipated.
- Provide hard standing at the locations of the bases of the columns, such that the driven H piles can be installed.
- Install the H piles under each pile cap. Cut to length, weld and drill for the pile cap reinforcement.
- Construct the pile caps.
- Clear hardstanding, lay and compact 150 mm sub base
- Weld waterproofing plate to piles, lay and attach the waterproofing membrane.
- Construct the basement slab.

Steelwork erection

- Erect the columns and bracing to the core areas.
- Working away from the cores, erect the Level 1 beams.
- Lay the profiled decking and reinforcement, and concrete the Level 1 slab.
  - o Profiled steel sheet is laid on the floor beams and shot fired to the steelwork.
  - o Shear studs are through-deck welded to the supporting steel beams
  - o Edge strip is fixed around the perimeter of the floor slab, and any openings.
  - o Pumped concrete is cast to the specified thickness, and finished.

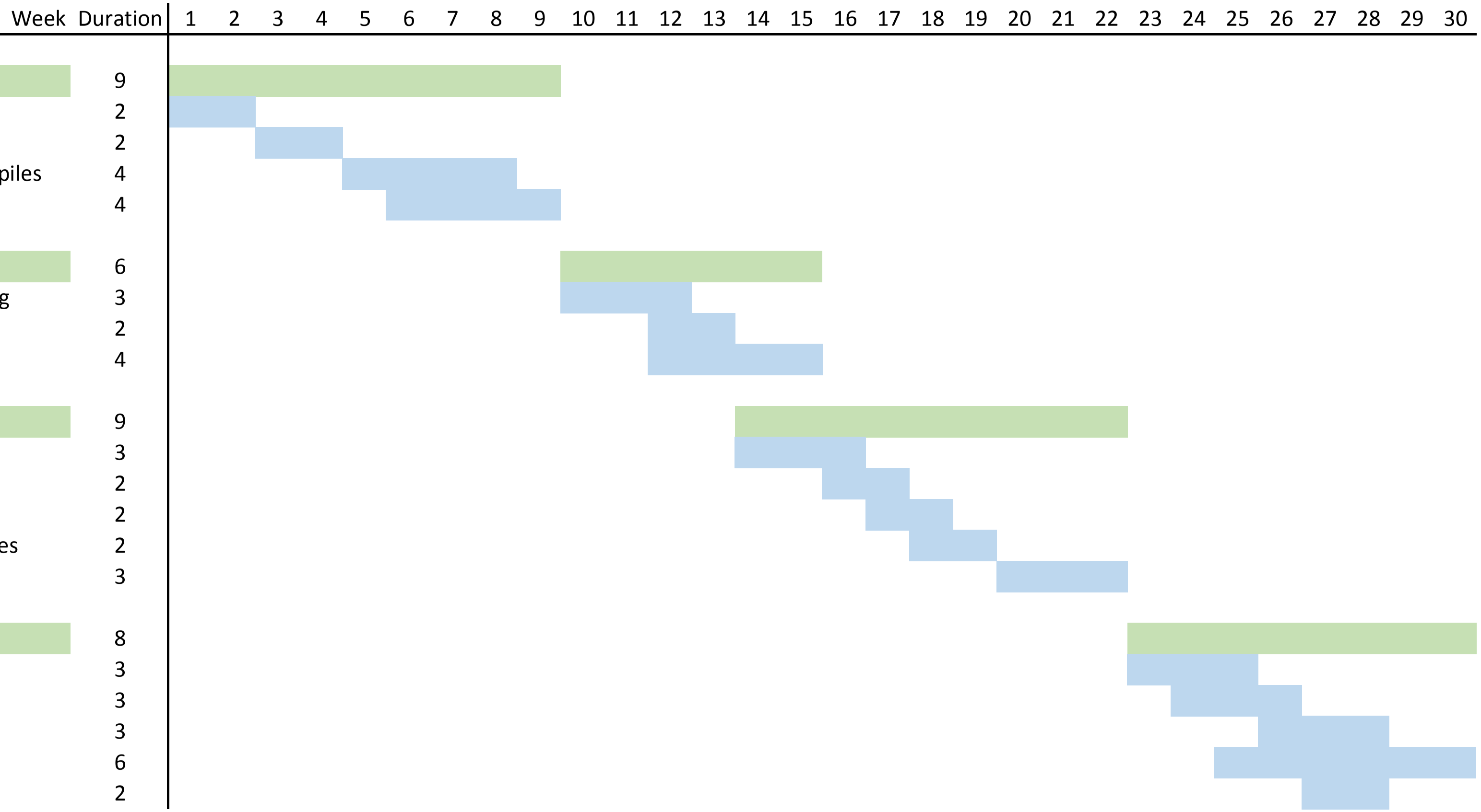
- The temporary ties to the sheet pile wall may be removed.Continue to erect floor beams, decking and construct the slab for Levels 2, 3 and 4. This involves splicing and erecting the upper lengths of columns. Vertical bracing to the cores to be installed in advance of floor level steelwork. Intermediate cladding support columns to the elevations to be installed as erection proceeds.
- Install the plate girders on Level 5, connected back to the internal columns.
- Complete the floor slab on Level 5, initially casting the slab within the building footprint, and only then casting the slab in the overhanging areas.
- Install the tie to Level 4 overhang, and complete the Level 4 construction.
- Install the roof steelwork, secondary steelwork and cladding.
- Cranes and mobile elevating working platforms (MEWPS) to operate within and adjacent to the footprint of the building.

5 PROGRAMME

The construction programme is shown on the following page

Construction Programme

Activity	Duration
Initial works	9
Establish site	2
Provide hard standing	2
Install sheet piles and temporary anchor piles	4
Install walings and ties back to anchors	4
Excavation and H Piles	6
Excavate basement and install dewatering	3
Lay hard standing for driven piles	2
Install H Piles	4
Pile caps and basement base slab	9
Cut piles and construct pile caps	3
Sub-base preparation and blinding	2
Welding of waterproofing detail	2
Waterproofing membrane and seal to piles	2
Construct base slab	3
Steelwork erection	8
Steelwork	3
Decking	3
Floor concrete	3
Cladding to elevations	6
Roof cladding	2



6 CLIENT LETTER

A Designer
Ascot, Berkshire SL5 7QN

30 October 2017

Dear Sir,

RE: Addition of landscaping to the car park roof and to the office roof

With reference to your requirement to add 1.0 m of topsoil to the car park roof and 0.3 m of topsoil to the office roof, we have the following comments:

1. The overall form of the structure will not change, though additional steelwork will be required to accommodate the increased loads. Additional design time will be needed to verify the members under the new loads, and to design alternatives where necessary.
2. The roof structure will need significant modification. The lightweight cladding panels will need to be replaced by a concrete slab, approximately 130 mm thick, to support the landscaping. The steelwork supporting the roof will need to be increased in size, and more members will be required – the roof construction will become very similar to the intermediate floor levels.
3. The columns between the roof and Level 1 will need to be increased in size, though this will not be significant.
4. The support structure for the overhanging parts of the building will need to be checked. Minor modifications to the member sizes are likely, but this will not be significant.
5. The landscaping on the roof of the car park is very significant, adding considerable weight to the car park roof construction. Additional steel beams will be required in order to reduce the span of the slab at that level and to keep the beam size within the construction depth constraints.
6. The additional landscaping surcharges the ground outside the basement walls. This will increase the forces in the level 1 steelwork, which acts as a prop to the basement wall. It is not expected that the sheet piles will need to be modified.

7. The base of the car park must be lowered, bringing it below the water table. More comprehensive dewatering will be necessary during construction. The waterproofing details will require modification to ensure no water enters the basement.
8. Because additional steel members are required, the erection program will lengthen. The slab required to support the landscaping on the roof will extend the construction program by approximately two weeks.
9. The column size between level 1 and level -1 will need to increase significantly; our recommendation is that the column is spliced immediately above level 1, so the heavier section is limited in length.

The proposed changes will result in

1. Additional design costs.
2. Additional cost and time required to construct the new foundations.
3. Additional cost and time required to construct the roof slab.
4. Increased costs of steelwork, notably at the roof, the lower lengths of column and level 1 steelwork.

Yours sincerely

Steel Construction Institute

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