# Model Answer Q2, 2015:

# Institution of Structural Engineers Chartered Membership Examination

STEEL
*for life*

BCSA

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# 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 2 from January 2015; 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 canopy and control room for a road toll barrier. Ten lanes, each 5 m wide and with 5 m between lanes were to be covered by a roof with an overall width of 105 m. Parallel to the road lanes, the canopy was to extend 20 m from a 10 m support zone, making the overall dimension 30 m. The high level control room was to be 10 m by 10 m, mid-way between the traffic lanes, aligned over the zone available for supports. A clear height of 5 m was required to the underside of the canopy, and 20 m to the underside of the control room. The general arrangement is shown in Figure 1.
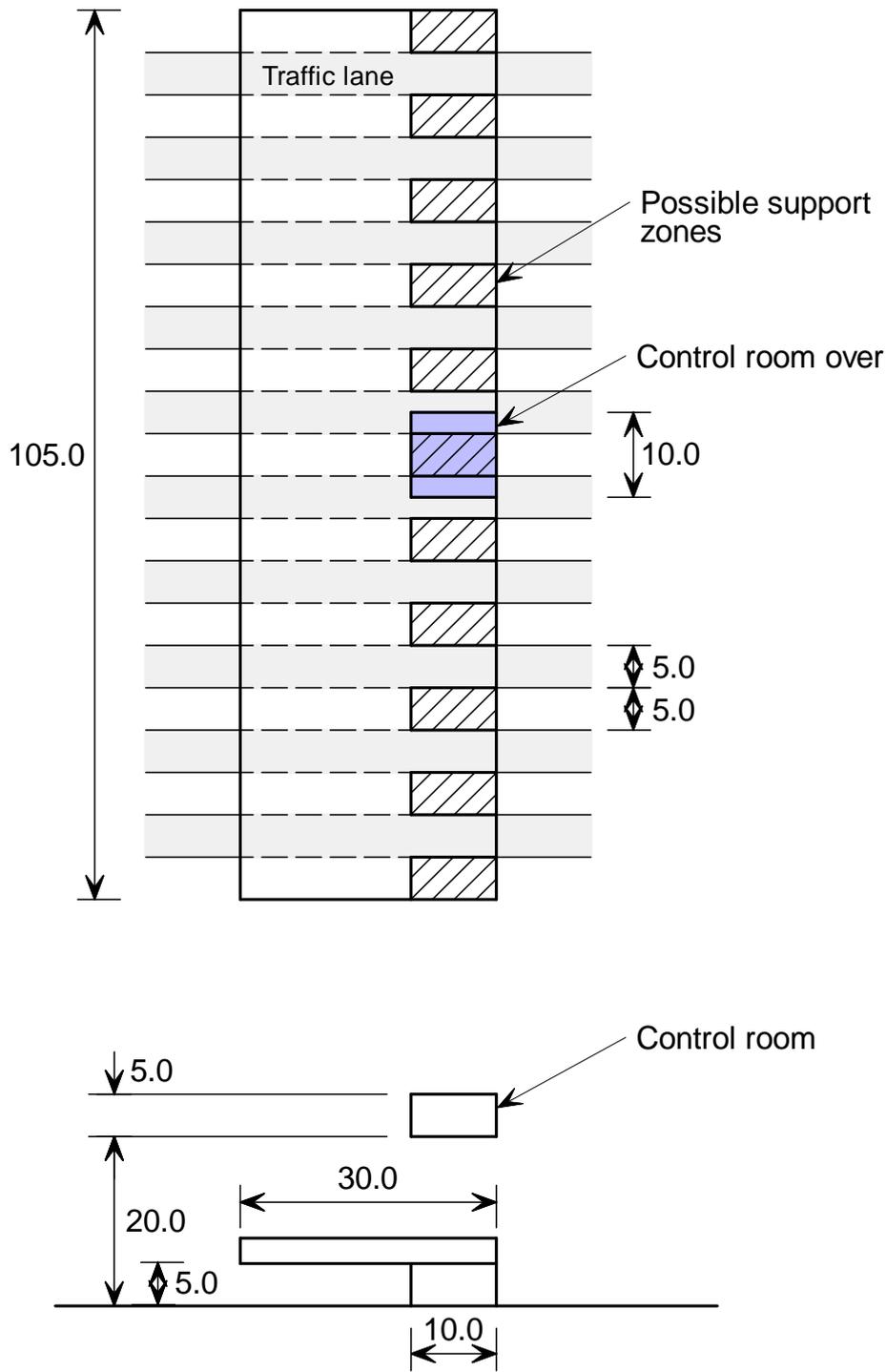
Figure 1     General Arrangement

Supports were only permitted in the 5 m gap between lanes, within the 10 m zone shown in Figure 1.

Although the supports were protected from impact, the brief required that the structure must withstand the removal of a single interior support, under partial loading conditions.

Only three road lanes could be closed at any one time during construction.

## 1.2 Loading

The following loads were specified:

Roof                          1.0 kN/m$^2$

Control room floor        4 kN/m$^2$

Basic wind speed of 46 m/s based on a 3 second gust, or a mean hourly wind speed of 23 m/s.

## 1.3 Ground conditions

Ground conditions varied at each side of the carriageway. Borehole 1 was located at the North end of the roof, and borehole 2 at the South (as drawn).

Borehole 1:

Ground level to – 0.4 m          Asphalt

0.4 m – 5.0 m                         Silty clay C = 50 kN/m$^2$

Below 5.0 m                           Stiff clay C = 100 kN/m$^2$

Borehole 2:

Ground level to – 0.4 m          Asphalt

0.4 m – 3.0 m                         Gravel with traces of silt, N = 15

Below 3.0 m                           Rock, allowable safe bearing pressure 2000 kN/m$^2$

Ground water at 2 m below ground in both locations.

## 1.4 Section 1b modification

The client wishes to move the control room to the opposite side of the roof.

# 2    CALCULATIONS

5

<table>
<tr><td rowspan="4"><strong>SCI</strong><br>Steel Knowledge<br><br>Silwood Park, Ascot, Berks SL5 7QN<br>Telephone: (01344) 636525<br>Fax: (01344) 636570<br><br><strong>CALCULATION SHEET</strong></td><td colspan="4">Title</td></tr>
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<tr><td rowspan="2">Client</td><td>Made by EDY</td><td colspan="2">Date Nov 17</td></tr>
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# SECTION 1(a) – ALTERNATIVE SOLUTIONS
# SCHEME 1

Scheme 1 is comprised of steel trusses at 10 m centres, supporting the cantilever roof. Each truss is supported within the permitted 10 m zone. Perpendicular to the carriageway, secondary trusses are provided at the cantilever tips and at the supports.

The control room is supported on a 10 m square steel tower. The tower columns are supported on transfer trusses at canopy roof level, spanning over the carriageway below and supported in the permitted zones.

Trusses are designed as pin ended, allowing articulation to accommodate any differential settlement of supports across the site.

Trusses perpendicular to the carriageway are designed as two span under the accidental loading conditions, and thus designed to support the structure if a single column is removed.

Uplift on the foundations is the critical load condition. In clay, tension piles are provided to resist uplift by skin friction. In rock, tension bars are grouted into the rock.



View on A

Plan on canopy roof
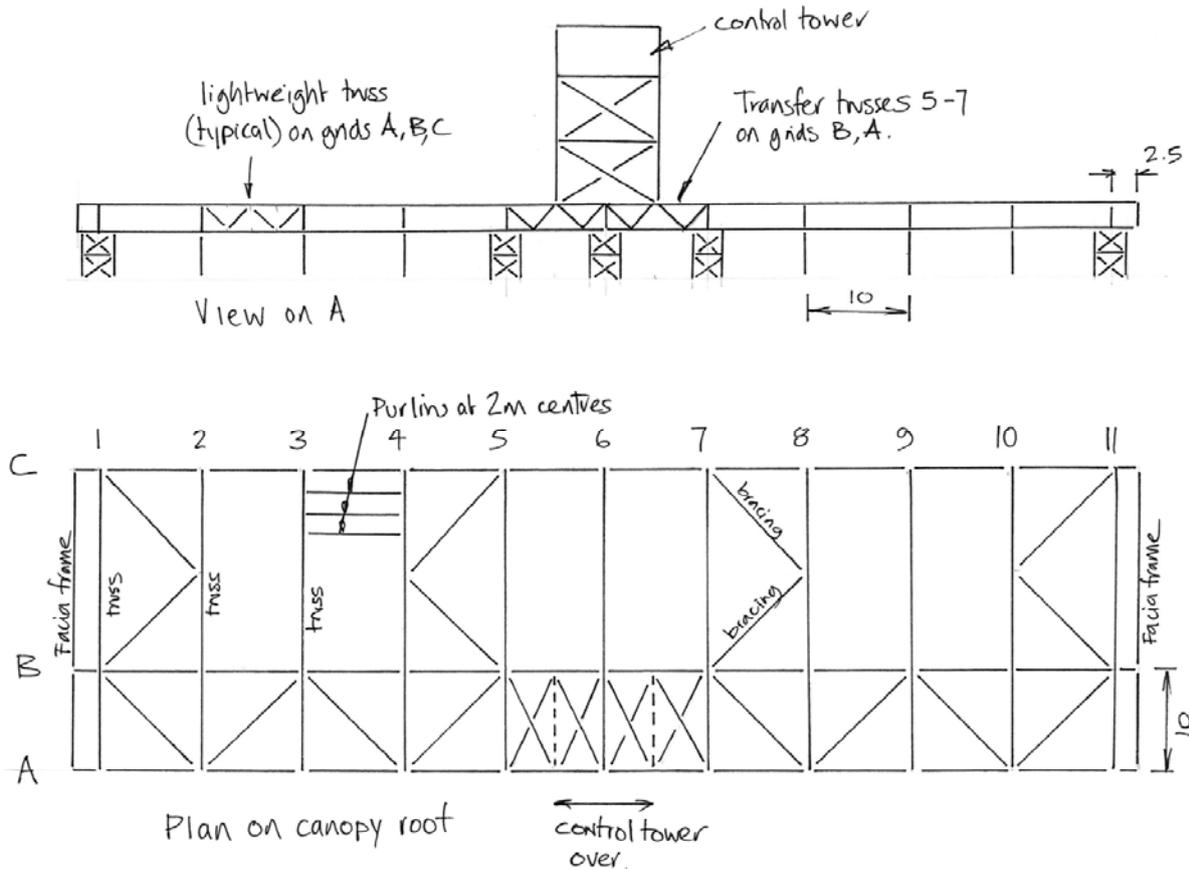
The key to this question is the cantilever of the canopy roof, compounded by the lateral forces on the control room tower, which will produce uplift on the foundations.

The quite different ground conditions across the site demand alternative arrangements for the clay and for the rock.

Structurally, the modest challenge is that the supports to the control room tower are located between supports (i.e. over a carriageway). A transfer arrangement is required if the control tower is to be supported at the four corners.

A narrow tower could be provided, becoming wider under the control room itself, but this would be tall and narrow, so not an efficient solution.

The different ground conditions mean that differential settlement across the site must be accommodated – and mentioned in the submission.

## Framing arrangement

The roof will be clad in profiled metal sheeting, spanning 2 m between purlins. The purlins span 10 m to the primary roof canopy trusses.

### Roof canopy trusses

The primary roof canopy trusses are 30 m long, supported on a braced support system within the 10 m permitted zone. The trusses have a 1 in 60 fall to the truss tip, to aid drainage.

Under gravity conditions, the bottom chord of the cantilever will be in compression. Lateral restraints will be required at appropriate intervals. In the wind uplift case, the purlins restrain the compression chord.

### Transverse trusses

Transverse trusses will be provided on lines A, B and C (see diagram) to provide stability at the support locations and at the tips of the cantilever. These trusses do not carry primary loads in normal conditions, but will be designed to carry the resulting load if one support is removed.

### Load transfer – canopy roof

Vertical loads are carried by the primary roof canopy trusses to the columns within the 10 m permitted support zone. Depending on the load combination, the columns will experience compression or tension, meaning the foundations must be designed for both cases.

Loads parallel to the carriageway are carried by the trusses to the vertical bracing within the permitted support zone and thus to the foundations.

Loads perpendicular to the carriageway will be carried by plan bracing in the line of the truss chords to bracing towers provided in five of the eleven permitted support zones. Within these towers, diagonal bracing takes the lateral loads to the foundations.

### Frame stability – canopy roof

Plan bracing is provided in each end bay and two further intermediate bays to stabilise the structure under loads perpendicular to the carriageway. Plane bracing is also provided over the full width of the structure, between Trusses A and B, to accommodate loads parallel to the carriageway and to ensure the roof behaves as a diaphragm.

### Control Tower

The control tower is a simple clad "box" supported on a braced tower. All four sides of the tower are braced. A simple floor of precast slabs could be provided, with supporting beams simply supported. A simple clad roof is envisaged, to suit the architectural details.

At the canopy roof level, due to the arrangement of the carriageways, the control tower steelwork must be supported by transfer trusses which span to adjacent supports. At this level, plan bracing is provided to transfer the horizontal loads to the adjacent braced bays.

### Control tower – load transfer

Vertical loads are carried by the four corner columns to the transfer trusses, and then to vertical supports in the permitted support zones. Lateral loads are carried by vertical bracing to the canopy roof level transfer trusses. At that level, horizontal bracing transfers the horizontal loads to vertical bracing provided in both directions within the permitted support zones.

10 m is a relatively long span. A hollow section will be used for the purlin, primarily so that lateral torsional buckling under uplift (where the member would be unrestrained) is not a design consideration.

The transverse trusses (perpendicular to the carriageway) would normally carry no load – the exceptions are the trusses carrying the column loads from the control tower. However, the brief demands that a support be removed, so the trusses must function as double span, or as a cantilever, and this will be the design condition.

Generally, examiners are keen to see issues of load transfer and stability addressed, so it may be a good discipline to formally describe these.

Detailed arrangements around the control tower are not seen as significant, so are largely ignored in the submission.

## Control tower – frame stability

The tower is braced on all four sides. Lateral loads are transferred to the plan bracing, and then to the adjacent vertical bracing. Relatively small vertical loads mean that second order effects are not likely to be significant.

## Foundations

The two boreholes indicate quite different ground conditions at each end of the site. In clay, bored piles will be provided, resisting the uplift by skin friction alone. In the download condition, the load will be resisted by skin friction and by end bearing. Differential settlement will be accommodated by pinned connections within the superstructure.

Where rock is indicated at shallow depth, short bored piles will be used to transfer compression. To resist tension, bars will be grouted into the underlying rock.

## SCHEME 2

Scheme 2 provides a similar arrangement of steelwork for the canopy roof, but quite different support arrangements. In this scheme, tension members are provided from high level on the control tower to support the roof canopy steelwork mid-way between the end of the canopy (grids 1 and 14) and the support tower. The general arrangement is shown in the following figure..



Part Elevation

Cross-section



Plan

Control room over
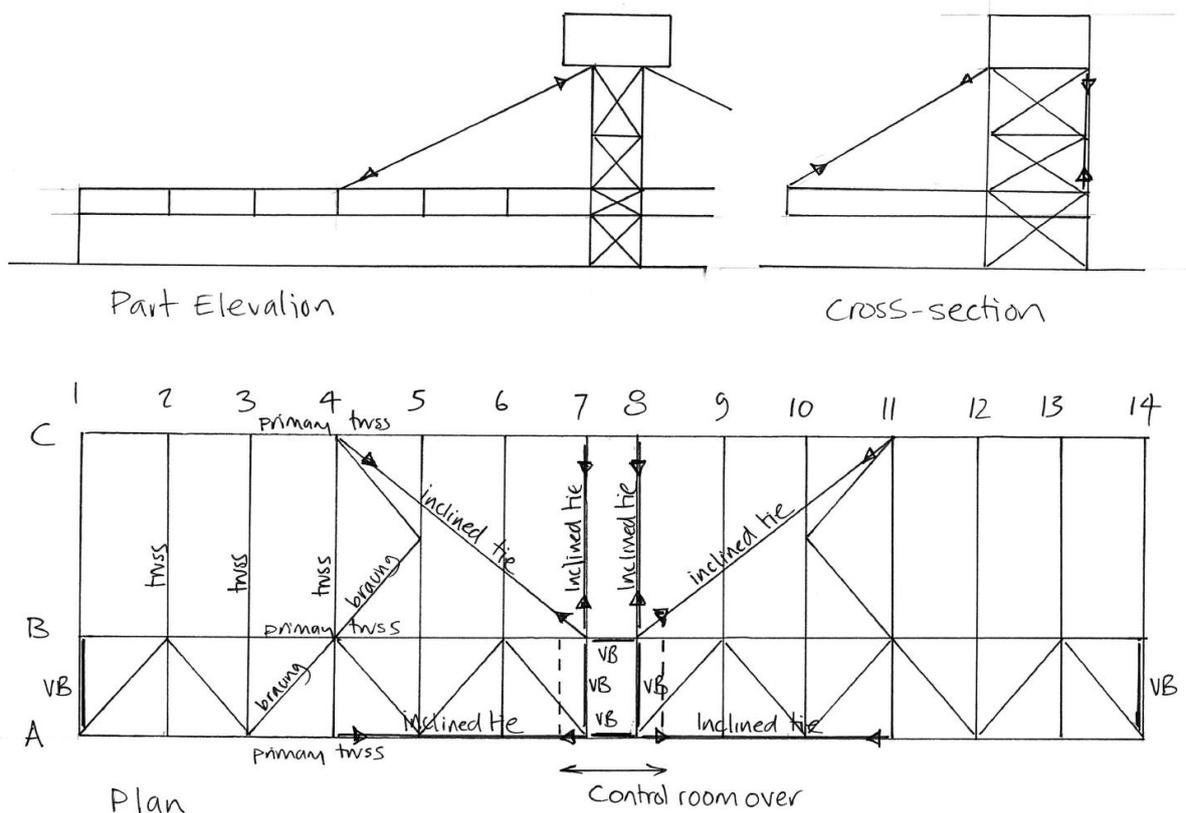
No intermediate supports or foundations are needed.

Primary trusses span are provided on grids A, B and C, with a mid-span support from inclined tension members anchored to the tower. Secondary trusses are provided parallel to the carriageway, from grid A to C. The secondary trusses are not aligned with the carriageways below. Purlins span 8.33 m between the secondary trusses to support the cladding at 2 m centres.

In scheme 2, the spacing of trusses parallel to the carriageway is no longer constrained by the lane spacing.

In scheme 2, a narrow tower is proposed, meaning no transfer trusses are needed.

In any scheme with inclined members, the resulting forces in the rest of the structure must be mentioned and managed in design. Here, additional compression is introduced.

With minimal supports, erection will be more challenging.

The tension in the ties produces compression in the primary truss chords, which must be included in the design. The inclined tie to grid C produces compression in the secondary truss chord, which must be resisted by a substantial horizontal truss between grids A and B, which spans from the ends of the canopy to the support tower.

In this scheme, the control room is supported on a narrow tower (5 m x 10 m) which is located within the permitted support zone between carriageway lanes.

The support tower is subject to high loads, and asymmetric loading conditions – including torque on plan, so will be heavily braced with diagonal members on all faces down to the foundations.

Because the tower carries a large proportion of all the forces applied to the structure, the foundations will be substantial.

## Load transfer – canopy roof

In the gravity loadcase, purlins carry the load on the sheeting to the secondary trusses. The secondary trusses are supported by primary trusses on grids A, B and C. The primary trusses span from the outside of the canopy roof to the support tower, with an intermediate support from an inclined tie, midway along the truss

The tie member is only effective in tension (the gravity loadcase), so in the wind uplift loadcase, the primary trusses must span 50 m from the edge of the roof to the support tower.

Load parallel to the carriageway is carried by the plan truss between grids A and B to the vertical bracing at the ends of the canopy and at the support tower.

Load perpendicular to the carriageway is carried by the primary trusses to the vertical bracing in the support tower.

## Load transfer – control room

The vertical loads from the control room are small. The more significant loads are from the component of the tie force that the columns must carry. All lateral loads will be carried by bracing on all four sides of the tower to the foundations.

## Frame stability

Frame stability is provided by the vertical bracing at the end of the canopy, and by vertical bracing in both directions at the support tower. The support tower is relatively narrow, so the resulting forces will be large.

## Foundations

Foundations are only required at the ends of the canopy and under the support tower. The foundations at the ends of the canopy will not be significant. The foundations under the support tower must resist large vertical forces and large horizontal forces, so will be substantial. Similar arrangements to scheme 1 are proposed – bored piles if clay and tension bars if rock.

## Scheme Selection

Although scheme 2 has greater architectural merit than scheme 1, the majority of force is concentrated on the support tower leading to an inefficient design. Because the inclined ties are ineffective in the uplift loadcase, the primary trusses will be substantial, and may be competent in the gravity loadscase even without the inclined ties, rendering the ties structurally redundant. Fewer foundations in scheme 2 are offset by the necessity for much larger foundations, leading to increased cost.

The narrow tower will lead to significant forces in the bracing, and tower legs, and in the foundations. The possibility of asymmetric loading would also be considered.

As ties do not work in compression, the primary trusses perpendicular to the carriageway will need to be substantial (and uplift is the more onerous design condition). Thus the trusses would be capable in the 'download' case without the ties – making them redundant.

The structure is inefficient as the ties are effectively redundant. They do limit deflection in the gravity loadcase.

The inclined ties result in additional compression in several members, adding complexity to the design

The structural connections between the canopy roof and the control room support tower are likely to produce some movement and possibly vibration from the fluctuating wind loads on the canopy roof, which may lead to discomfort. The control room is more isolated in scheme 1 so any dynamic effects are reduced.

Temporary works will be more involved in scheme 2, to support the primary trusses until the ties and the plan bracing is complete. In comparison, the scheme 1 steelwork can be erected in self-contained, stable stages. Scheme 2 is more sensitive to asymmetric loading.

The structural redundancy demanded by the brief is readily provided in scheme 1.

Both schemes recognise that differential deflection across the canopy roof is possible; both have sufficient flexibility to accommodate any anticipated movement

Scheme 1 is selected, for three primary reasons:

1. Scheme 1 is more robust in form.

2. Scheme 2 demands more temporary works.

3. The foundations for Scheme 2 will be much more expensive.

## SECTION 2 – DESIGN CALCULATIONS

(for scheme 1)

Preliminary design calculations completed in accordance with BS EN 1993-1-1.

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

All steelwork is S355, unless noted otherwise.

### Roof canopy

### Loading

Mean hourly wind speed is associated with BS 6399

No altitude given. Assume 150 m asl. so $c_{alt}$ 1.15

$v_s = 1.15 \times 23 = 26.5$ m/s

Assume country terrain, close to sea (most onerous)

Then from BS 6399 Table 4, $s_b = 1.96$ at 30 m height

$v_e = 26.5 \times 1.96 = 51.9$ m/s

$q_s = 0.613 \times 51.92 \times 10\text{-}3 = 1.65$ kN/m$^2$

BS 6399 Table 13 for overall coefficients (net)

Download = +0.2; uplift = -1.2

Download = 0.2 × 1.65 = 0.33 kN/m$^2$

Uplift = -1.2 × 1.65 = -1.98 kN/m$^2$

Assume permanent actions = 0.4 kN/m$^2$

Assume snow load = 0.6 kN/m$^2$

Imposed load (from brief) = 1.0 kN/m$^2$

In the scheme selection, consider structure, foundations, cost, aesthetics, erection, programme and risk.

It is not clear how the demand to remove one support can be realised in scheme 2.

For the candidate, an equally important reason is that scheme 1 is straightforward to design.

A 3 second gust is compatible with CP2-ChV. Wind data compatible with the Eurocode is not provided in the brief.

Not enough data is given for a thorough assessment to BS 6399, so assumptions must be made about altitude, distance from the sea, etc.

Note that in the Eurocode, the imposed load on a roof is separate to the snow load. Notably, the imposed load is not combined with wind or snow. An assumption about the snow load must be made, as this is not given in the brief.

## Design combinations of actions

Download

Permanent + imposed; $1.35 \times 0.4 + 1.5 \times 1.0 = 2.04$ kN/m$^2$

Permanent, snow + wind (down);

$1.35 \times 0.4 + 1.5 \times 0.6 + 1.5 \times 0.5 \times 0.33 = 1.68$ kN/m$^2$

Uplift

$1.0 \times 0.4 - 1.5 \times 1.98 = -2.57$ kN/m$^2$.

## Purlin for roof canopy

10 m span, 2 m spacing; critical load is uplift

Bending moment $= 2.57 \times 2 \times 102/8 = 64$ kNm

Adopt 150 × 150 × 6.3 SHS (68.2 kNm)

## Primary roof truss

Uplift on one truss $= -2.57 \times 10 = -25.7$ kN/m

Download on one truss $= 2.04 \times 10 = 20.4$ kN/m
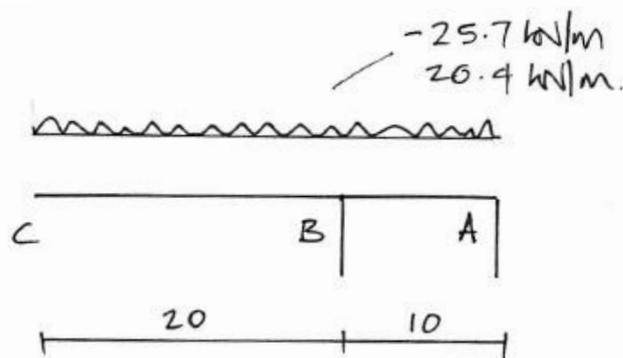


Uplift moment at B $= 25.7 \times 20 \times 10 = 5140$ kNm
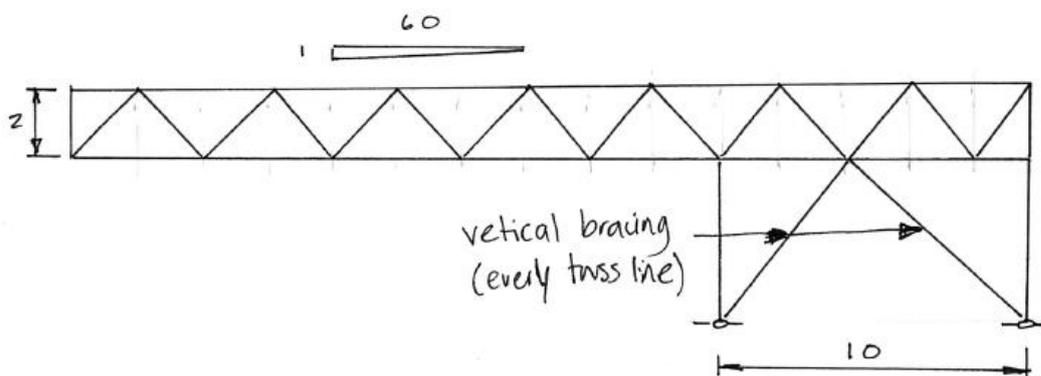


With a 2 m deep truss, chord force $= 5140/2 = 2570$ kN

In uplift, compression in the top chord, with restraints from purlins at 2 m centres.

Adopt 200 × 200 × 10 SHS (2570 kN on 2 m)

Download moment at B $= 20.4 \times 20 \times 10 = 4080$ kN

SHS has been chosen to avoid LTB being a design consideration. No deflection limits are needed at this stage.

Two design cases are considered – and uplift is more onerous.

Although the compression in the bottom chord will be smaller, as the loading is reduced, restraints are only provided at 4 m centres (compared to 2 m centres on the top chord) so this design condition must also be verified.

Chord force = 4080/2 = 2040 kN

Say restraints at every other purlin, so every 4 m

200 × 200 × 10 provides 2280 kN on 4 m, OK

Taking moments about A under uplift conditions:

-25.7 × 30 × 15 = VB × 10;  VB = -1156.5 kN (tension)

VA = -25.7 × 30 + 1156.5 = 385.5 kN (compression)

Shear at B = 20 × 25.7 = 514 kN
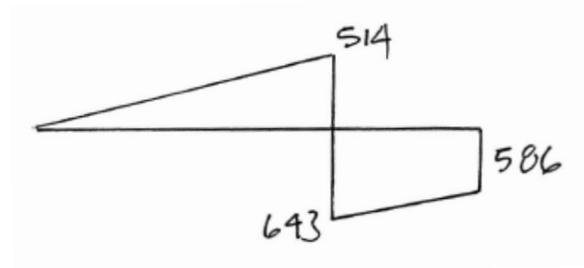


Shear force diagram

Most heavily loaded internal is subject to shear of 643 kN

Diagonal component = 643 × $2^{0.5}$ = 909 kN

Length = $(2 \times 2^2)^{0.5}$ = 2.82 m

Adopt 160 × 160 × 5 SHS (961 kN on 3 m)

Under download conditions, axial compression in column at B

= 1156.5 × 20.4/25.7 = 918 kN (compression)

Assuming that one support is removed, the compression increases by 50% (the load from the missing column is shared by the adjacent supports).
Design load in column at B = 1.5 × 918 = 1377 kN compression, or

1.5 × 1156.5 = 1735 kN tension

Column length is 5 m

Adopt 200 × 200 × 10  (2040 kN compression on 5 m; 2660 kN tension)

## Vertical bracing (not under control room)

Assume a 2 m deep facia to roof canopy.

Assume pressure coefficient on facia of 1.2 (net)

Horizontal force at vertical bracing = 1.2 × 1.65 × 2 × 10 = 39.6 kN

Force in bracing = 39.6 × $2^{0.5}$ = 56 kN

Length of compression bracing = $(2 \times 5^2)^{0.5}$ = 7.1 m

Adopt 114 × 5 CHS (99.3 kN on 7 m)

## Secondary truss on Grid C

Truss ties cantilever tips – nominal loading, so fabricate from 150 × 150 × 6.3 SHS chords and 100 × 100 × 6.3 internals.

This assumes that the internal is at 45°

The internal member has been selected to be relatively wide, to minimise the likelihood of punching through the face of the chord. In a full design, it is essential that the joint resistances are checked in accordance with BS EN 1993-1-8.

The secondary trusses on C are merely framing.

## Control room tower design

Vertical loads:

Roof say 0.5 kN/m$^2$ permanent

1.0 kN/m$^2$ imposed (from brief)

Floor say 3.0 kN/m$^2$ permanent

4 kN/m$^2$ imposed (from brief)

Design value of vertical load

= 1.35 × (0.5 + 3) + 1.5 × (1 + 4) = 12.2 kN/m$^2$

Axial load in each of the four corner columns = 12.2 × 10 × 10 / 4 = 306 kN

EHF will be taken as 2.5% of the factored vertical loads, recognising this is sufficient to ensure that second order effects are small enough to be ignored.

EHF at control room floor level = 12.2 × 10 × 10 × 2.5/100 = 30.5 kN

## Wind actions

Take overall coefficient as 1.2

Design value of wind actions

= 1.65 × 1.2 × 10 × 20 × 1.5 = 743 kN
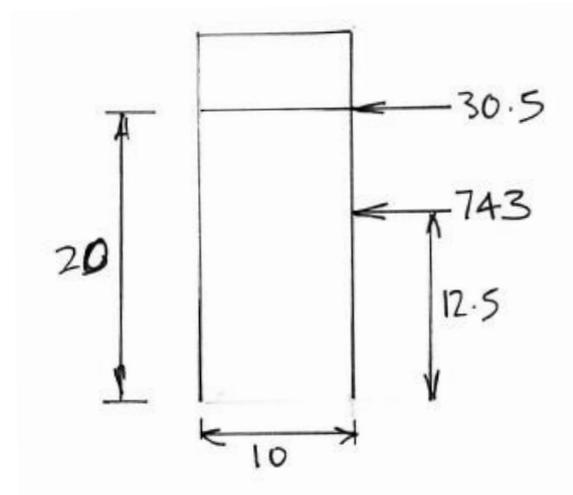


Force in column due to EHF and wind

= (30.5 × 25 + 743 × 12.5)/(10 × 2) = 503 kN

Total force in each column = 503 + 306 = 809 kN

Unrestrained length = 6.5 m

Adopt 200 × 200 × 6.3 (953 kN on 7 m)

EHF are normally 0.5% of the factored vertical loads. However, the use of 2.5% is considered sufficient to ensure that if frame stability were considered, $\alpha_{cr} > 10$ and second order effects would be small enough to be ignored.

## Vertical bracing in tower

Shear at base (each side) = (30.5 + 743)/2 = 386 kN

A single member in compression would be 12 m long, so try crossed flats in tension only.

386 kN will require 4 No. M24 bolts (4 × 136 = 544 kN)

Holes for M24 are 26 mm

Try 200 × 12 flat, S275

Resistance = (200 – 2 × 26) × 12 × 275 × $10^{-3}$ = 488 kN, OK

## Primary truss design (Grids 4 – 8, supporting the control tower)

Normal case: the load from the tower above is midway between grids.
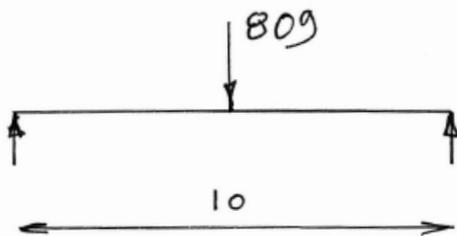


Bending moment = 809 × 10/4 = 2022 kNm

In the accidental case of a support being removed, the truss must span 20 m. Both columns from the tower above must be carried by the truss, together with the canopy loads. In the accidental case, the design is completed with 1/3 the imposed load (stated in the brief).

As it is an accidental case, the permanent actions will not be factored, following the guidance in BS EN 1990.

From the tower, the reduced vertical load (1/3 imposed load), but assuming the full design value of the wind load and EHF is given by:

503 + [ (0.5 + 3) + (1/3) ×(1 +4)] × 10 × 10 / 4 = 632 kN

From the canopy, the accidental load

= 0.4 + (1/3) × 1.0 = 0.73 kN/m²

The vertical load at the support would therefore be:

0.73 × 10 × 1156.5/25.7 = 328 kN
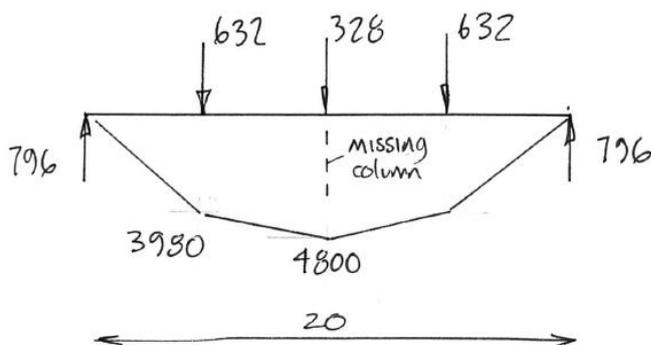
A compression member could be used – but is inconvenient as the length falls outside the range in the Blue Book.

A simple check is completed on the net area, using the yield strength. The Eurocode allows the resistance to be based on gross area × yield, or net area × 0.9 × ultimate, whichever is lower.

This is the "normal" case. It will be compared with the accidental condition, to see which gives the higher moment.

This first accidental condition assumes the missing support is on grid 6

Bending moment under load = 796 × 10 – 632 × 5 = 4800 kNm

An alternative scenario is when the truss must act as a cantilever:
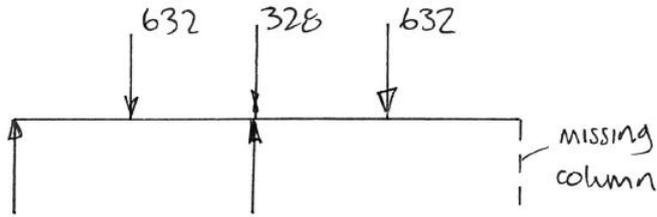


Moment at support = 632 × 5 = 3160 kNm

Maximum moment = 4800 kN; force in truss chord = 4800/2 = 2400 kN

To reduce the out of plane buckling length, a secondary truss providing a diaphragm will be introduced midway between grids. The bracing arrangement will be modified to provide restraint to the trusses on grids A and B at these diaphragm positions.
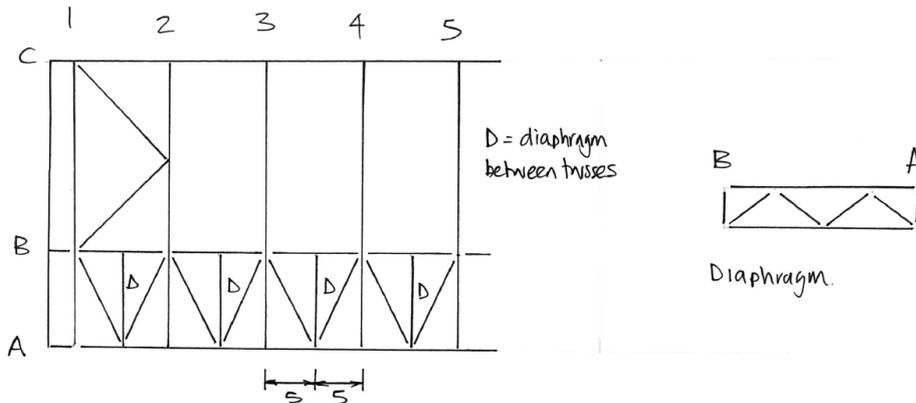


For the chords, adopt 200 × 200 × 16 SHS (3060 kN on 5 m)

Maximum shear is 769 kN.  Force in end diagonal = 769 × $2^{0.5}$ = 1125 kN

Length is 3.53 m

Adopt 160 × 160 × 8 SHS (1310 kN on 4 m)

Force in support column = [632 × (5+15) +328 × 10]/10 = 1592 kN

200 × 200 × 10 is OK (2040 kN on 5 m)

### Primary truss (grids 1 – 4 and 8 to 11)

Design case is when one support is missing:
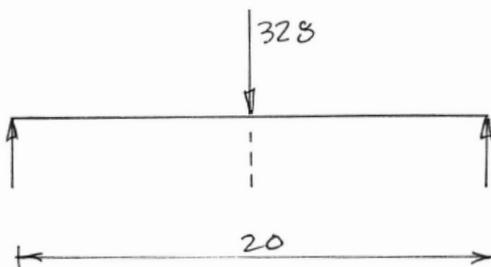


Midspan moment = 328 × 20/4 = 1640 kNm

This second accidental condition assumes that the missing column is on grid 7

This design covers the trusses that do not support the tower – in the normal condition they are merely framing the structure together, so the accidental case is the design condition to be verified.

This condition assumes an intermediate support is missing.
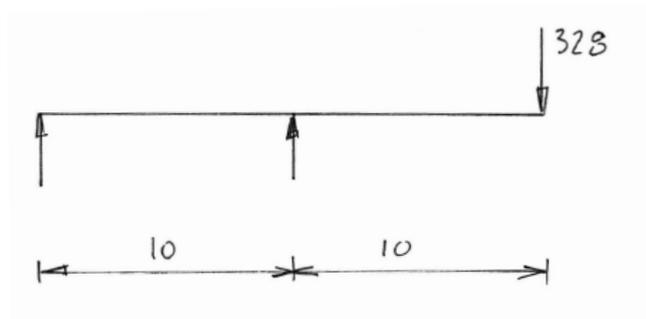
Moment at column = 328 × 10 = 3280 kNm

This is the scenario if the support on grid 1 or 11 were removed.

Under the maximum moment, the force in the chord = 3280/2 = 1640 kN

The chord is restrained at 5 m

Adopt 200 × 200 × 8 SHS (1670 kN on 5 m)

Maximum shear is 328 kN

Force in internal = 328 × $2^{0.5}$ = 464 kN

Length = 3.53 m

Adopt 160 × 160 × 6.3 SHS (1050 kN on 4 m)

## Plan Bracing

Plan bracing will be needed at top chord level and at bottom chord level. At bottom chord level, the assumed restraints to the chords are at 4 m centres, so the plan bracing will be arranged to suit. Plan bracing is provided in four bays.  Between the braced bays, ties are provided to restrain the bottom chords of the trusses.

## Vertical Bracing (North-South direction)

Perpendicular to the carriageway, the plan bracing carries lateral loads to the five braced towers. The loads in this direction are small — by inspection bracing of

150 × 150 × 6.3 will be adequate.

## Foundation design

## Column supports (not to tower)

Design value of vertical load from superstructure
= 1377 kN compression, or 1735 kN tension

Key design criteria is tension resistance. Foundations to resist both tension and compression are required. Two alternative designs will be undertaken, reflecting the different ground conditions at each end of the site. The beneficial weight of the pilecap will be taken into account when considering uplift.

## In clay (South end)

## Tension (1735 kN from superstructure)

The piles will be designed neglecting the contribution of the top 5 m of asphalt and silty clay (as the silty clay has a low strength). In tension, skin friction will be utilised. In compression, skin friction and end bearing will be utilised if necessary.

From Table 3.25 (Brooker), a firm to stiff clay has an undrained shear strength of approximately 75 kN/m$^2$.

This condition assumes that an end support is missing.

Plan bracing need only be nominal. Four braced bays and wind loads only mean the forces are small.

Foundations are the key design challenge. Two loading conditions exist – foundations that support the tower, and those that do not. Two quite different ground conditions have to be assessed – on clay, or over rock at relatively shallow depth.

Try four piles, each 750 mm diameter. $3d$ spacing = 2250 mm

Overall size of square pilecap = 4.25 m

Depth of pilecap = $(1/3)(8 \times 750 - 600)$ = 1800 mm

Weight of pilecap = $4.25 \times 4.25 \times 1.8 \times 22.5$ = 731 kN

Net uplift, applying a factor of 0.9 to the self weight of the pilecap

= $1735 - 0.9 \times 731$ = 1077 kN

With a pile group of $2 \times 2$ piles, the efficiency factor is 0.8 (Table 3.28, Brooker)

Based on an undrained shear strength of 75 kN/m$^2$, the adhesion factor is 0.7, from Figure 3.14 (Brooker) with soft clay overlying stiff clay.

Tomlinson recommends a 50% reduction in resistance for piles resisting uplift by skin friction (page 296).

The factor of 2.0 is taken from the UK NA to EN 1997-1 for bored piles (Table A.NA.7)

Assuming 750 mm diameter piles, the length in the stiff clay is at least:

$20 \times 0.75$ = 15 m

Circumference = $\pi d = \pi \times 0.75$ = 2.36 m

Ultimate resistance of pile group

= $4 \times 0.8 \times 0.7 \times 75 \times 2.36 \times 15 \times 0.5/2.0$ = 1487 kN, > 1077 kN, OK

## Compression (1377 kN)

In compression, the resistance is much larger, as the 0.5 factor is not applied, and end bearing would be included. 1487 kN > 1377 kN, OK

## Over rock (North end)

Rock is only 3 m below ground, so anchoring to the rock is proposed for the tension loadcase, and utilising piles bearing on the rock for the compression loadcase. Tension bars will be grouted into the rock, passing through the pile casing, before completing the pile.
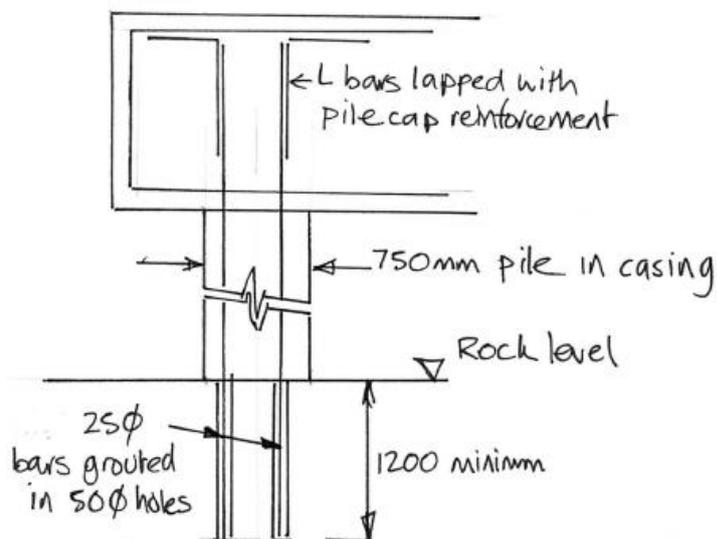
Some iteration is needed before a reasonable solution can be identified.  More piles are less efficient because of the group effect, but the pilecap is larger – and heavier, which reduces the net uplift.

Reinforced Concrete Designer's Handbook; Reynolds and Steedman. C&CA, 1981. Table 195.


0.9 factor from EN 1990 when considering resistance to uplift.




Foundation Design and Construction; Tomlinson, Pitman Publishing, 1995

There appears to be differences of view on the factor to be applied – so the Eurocode is used.




This includes the 0.5 factor used when assessing skin friction in uplift.


End bearing has been neglected. Even if the 0.5 factor for skin friction under uplift is used, the resistance is satisfactory in compression.

Tension (1077 kN net, assuming same pilecap dimensions)

Try 25 mm bars, with $f_y$ = 500 N/mm²

Bar area = $\pi \times 25^2/4$ = 491 mm²

Number of bars needed = $1077 \times 10^3/(500 \times 491)$ = 4.4 bars (use minimum 8: 2 per pile)

Maintaining the pile group of 4 piles, 750 mm diameter, try two 25 mm bars in each pile (8 bars total)

Load per bar = 1077 / 8 = 134.6 kN

Resistance of bar = $491 \times 500 \times 10^{-3}$ = 246 kN, > 134.6 kN, OK

Bond strength of bar to grout say 2 N/mm² (Tomlinson, page 296)

Bar circumference = $\pi d = \pi \times 25$ = 78.5 mm

Length of anchorage in grout = $134.6 \times 10^3 / (2 \times 78.5)$ = 857 mm

Allow embedment in rock of 1200 mm (top of rock may be fissured)

Bond strength of grout to rock say 1 N/mm² (Tomlinson, page 297)

Assume a 50 mm hole. Circumference = $\pi d = \pi \times 50$ = 157 mm

Shear between grout and rock = $134.6 \times 10^3/(1200 \times 157)$

= 0.71 N/mm² < 1, OK.

## Compression (1377 kN)

End bearing area of each pile = $\pi \times 0.75^2/4$ = 0.442 m².

Resistance of pile group in compression

= $4 \times 0.442 \times 2000$ = 3536 kN > 1377 kN, OK

## Foundations under the tower – Grids 5, 6, 7

The ground conditions at the tower location are not known – additional site investigation is needed before a solution can be chosen. Both alternatives (clay or rock) will be considered.

## Loads from tower

The maximum tension will result from permanent vertical loads only in the tower combined with EHF and wind. The EHF calculated previously will not be modified.

Permanent actions on tower = (0.5 + 3) = 3.5 kN/m²

Compression force in column = $3.5 \times 10 \times 10 / 4$ = 87.5 kN

Maximum tension in tower column = 503 – 87.5 = 415.5 kN

The maximum tension on the foundation is the summation of the tension in the tower leg and the tension due to the roof canopy loading (see previous)

Maximum tension on foundation = 1735 + 415 = 2150 kN

Net uplift = 2150 – 0.9 × 731 = 1492 kN

Maximum compression on foundation = 1377 + 809 = 2186 kN

## If founded above clay:

As previously calculated. Tension resistance of the pile group = 1487 kN. Compared to 1492 kN, this is satisfactory.

Some conservatism in the embedded length.

The EHF could be reduced, as in this case they may be based on the permanent actions only.

In compression, resistance of pile group, neglecting end bearing

= 4 × 0.8 × 0.7 × 75 × 2.36 × 15 × 0.5/2.0
= 2974 kN, > 2186 kN, OK

## If founded above rock:

In tension, as previous, 8 bars total proposed

Load per bar = 1492/8 = 187 kN

Resistance of bar = 246 kN, OK

Length of anchorage in grout = $187 \times 10^3 / (2 \times 78.5) = 1191$ mm

Allow embedment in rock of 1500 mm (top of rock may be fissured)

Shear between grout and rock = $187 \times 10^3/(1500 \times 157)$

 = 0.79 N/mm$^2$ < 1, OK

In compression, as previously calculated, resistance of the pile group

= 3536 kN, > 2186 kN, OK


## Pile Reinforcement

Adopt 4% reinforcement

0.4/100 × 442000 = 1768 mm$^2$

Adopt 6 H 20 = 1880 mm$^2$

Adopt H 10 links at 300 mm spacing

From Brooker

# 3    DRAWINGS

35

The following sheets reproduce the A3 drawings prepared for the scheme.
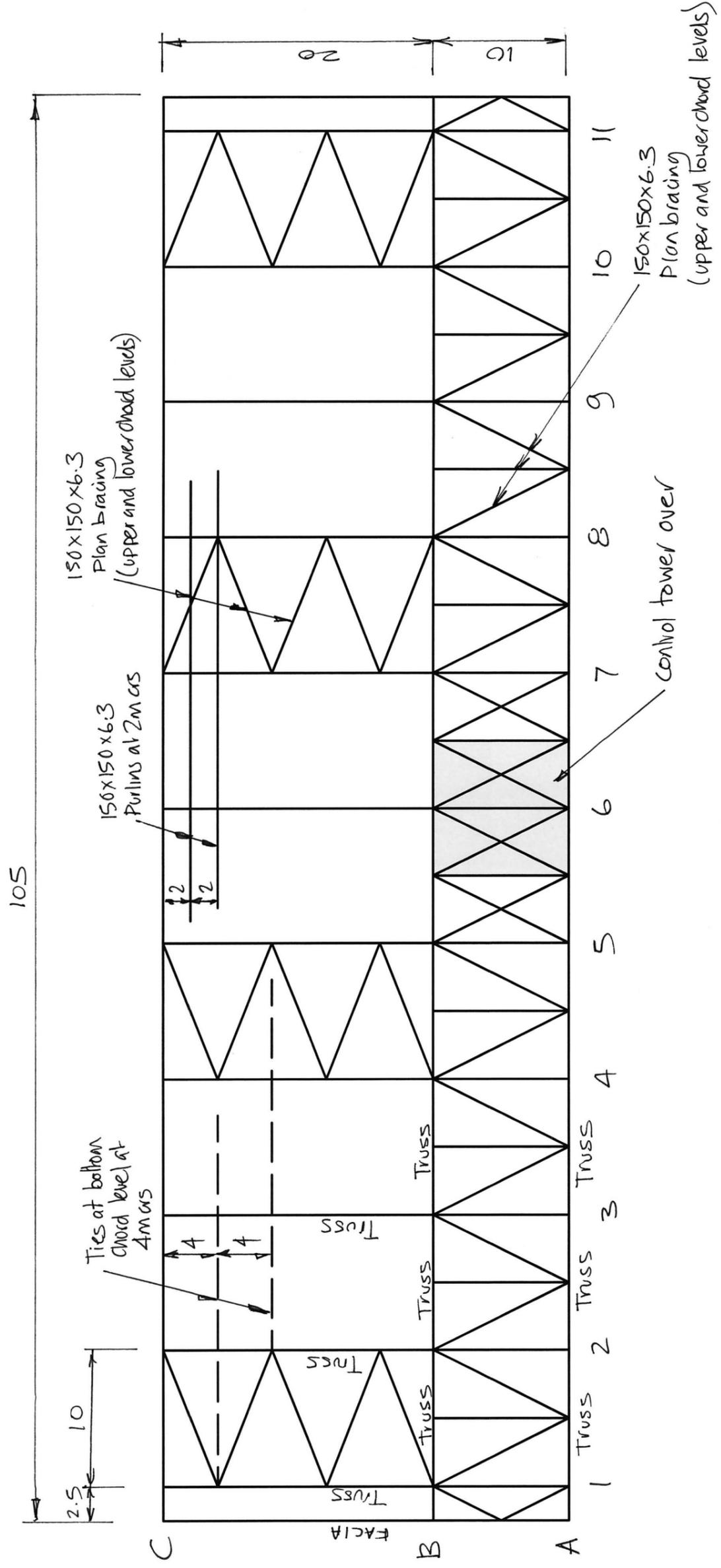
PLAN ON TOLL BARRIER ROOF

Labels within figure:

150×150×6.3 Plan bracing (upper and lower chord levels)

150×150×6.3 Plan bracing (upper and lower chord levels)

150×150×6.3 Purlins at 2m crs

Ties at bottom chord level at 4m crs

control tower over

Truss 1, Truss 2, Truss 3, Truss 4, 5, 6, 7, 8, 9, 10, 11

Truss, Truss, Truss, Truss, Truss, Truss

FACIA

A, B, C

105, 10, 2.5, 20, 10, 4, 4, 2, 2

ELEVATIONS AND SECTIONS

**PART ELEVATION ON GRID A**

200×200×8 chords
160×160×6.3 internals

200×200×16 chords
160×160×8 internals

200×200×8 chords
160×160×6.3 internals

Trusses on grids A & B

Control room tower

5m clear

10 6 4

Q R S

5 7 8 11
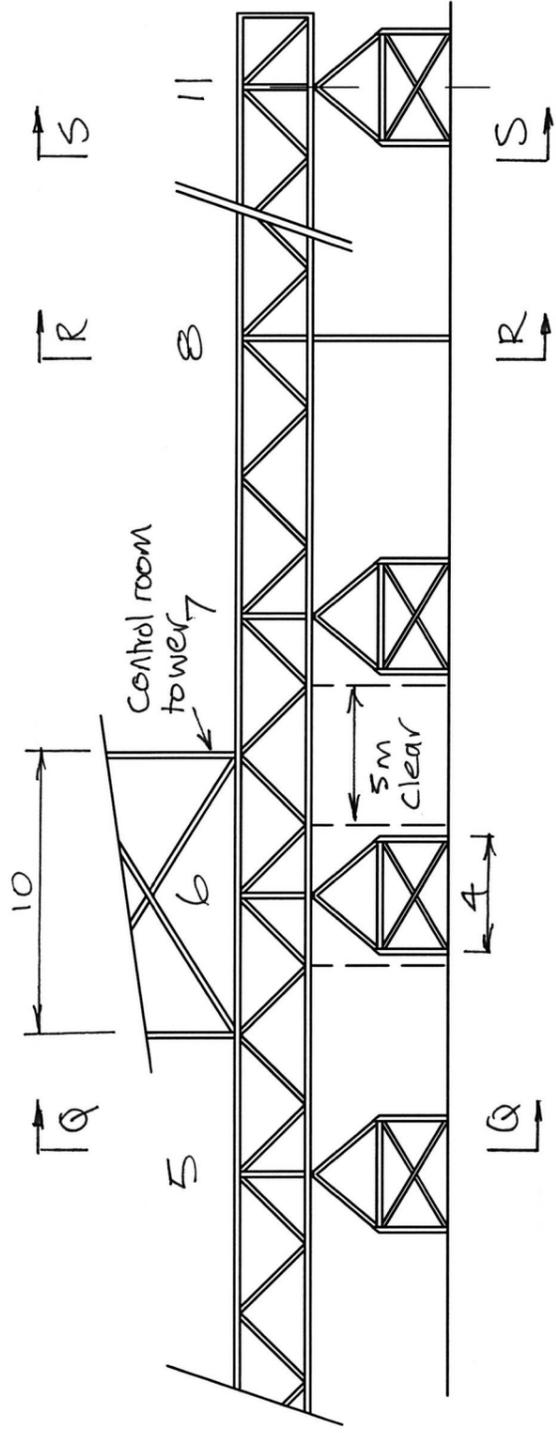
**SECTION S-S**
TYPICAL FOR GRIDS 1, 5-7, 11
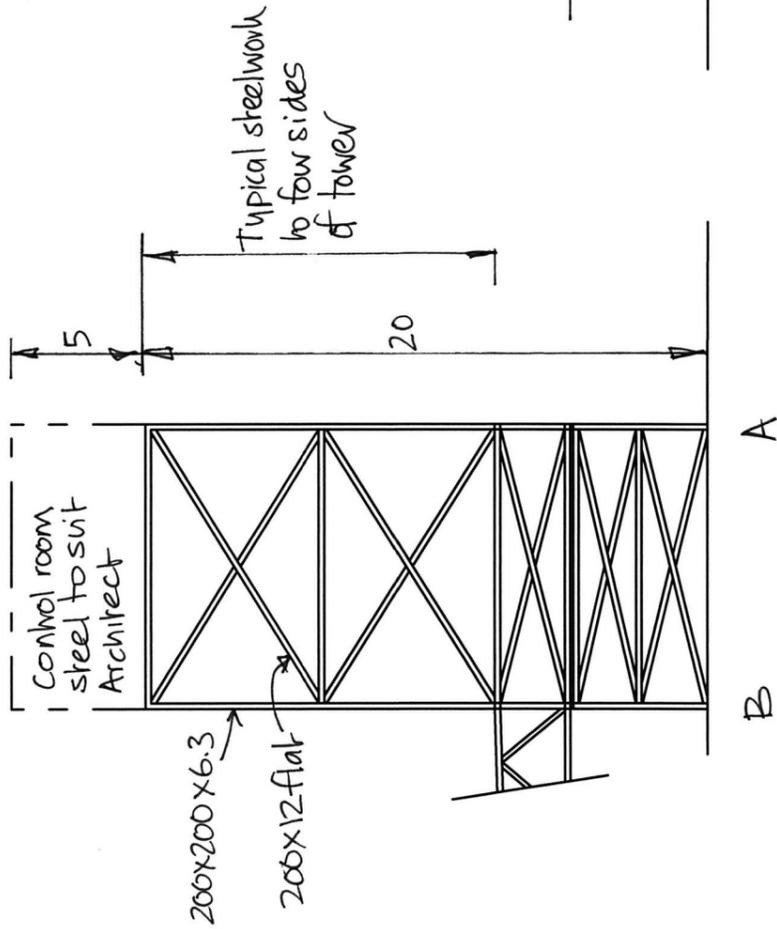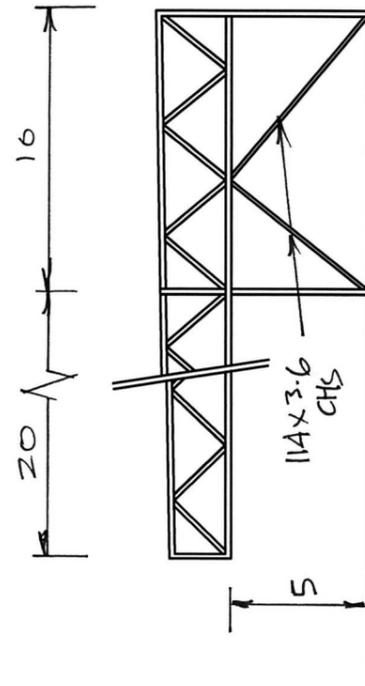
**SECTION R-R**
TYPICAL FOR GRIDS 2-4, 8-10

114×3.6 CHS

16

20

5

**SECTION Q-Q**

Control room steel to suit Architect

Typical steelwork to four sides of tower

200×200×6.3

200×12 flat

5

20

B A

Bars lapped to pile cap reinforcement

750 mm ⌀ casing

2 N° 25⌀ bars per pile grouted into 50⌀ sockets in rock

1.8m

3m

Rock level
1200 mm (not gnds 5,6,7)
1500 mm (if gnds 5,6,7 above rock)

2 N° 25⌀ bars per pile

1000    2250    1000

PLAN ON PILE CAP

TYPICAL CROSS SECTION

FOUNDED ABOVE ROCK
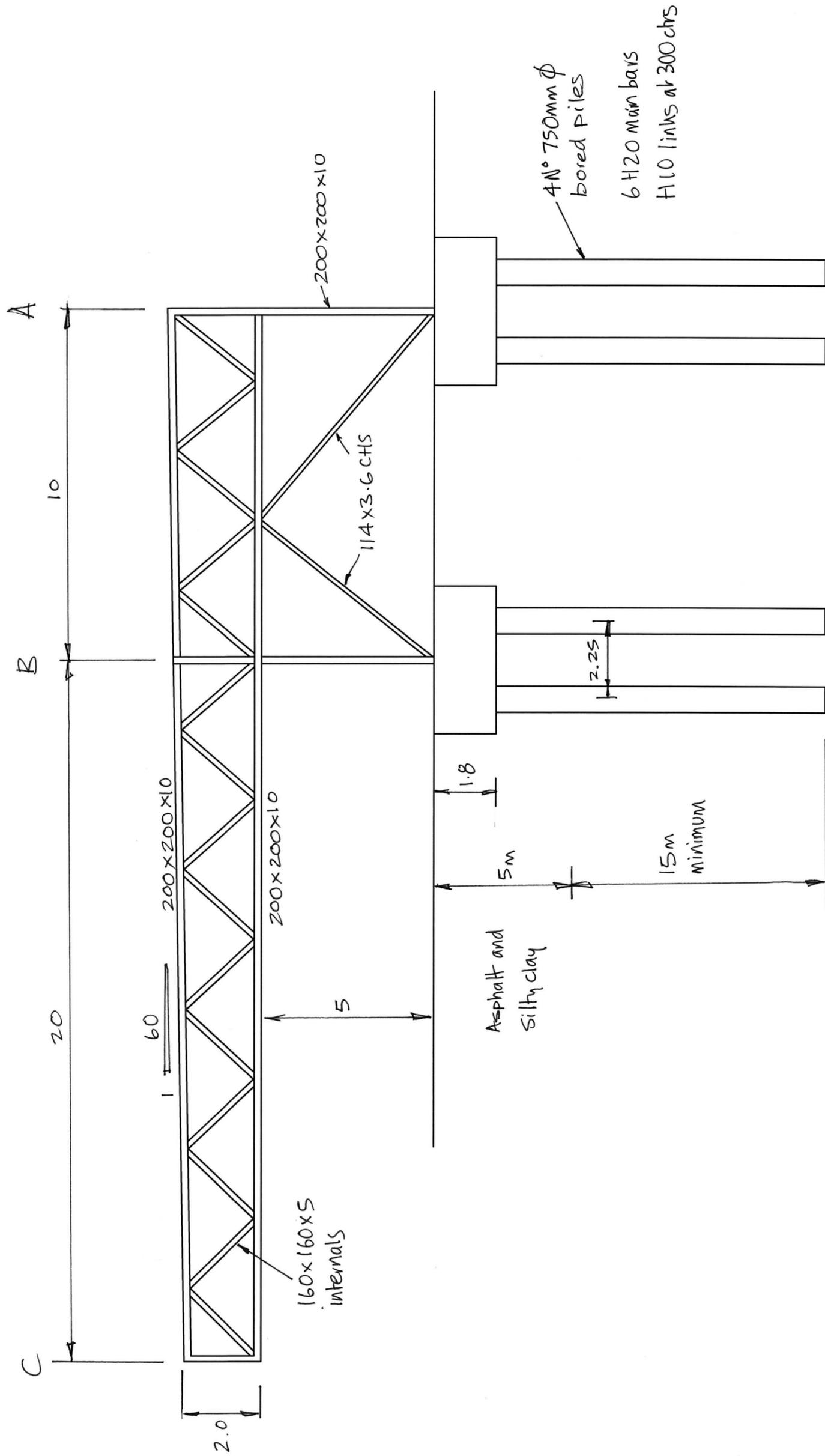
TYPICAL CROSS SECTION
FOUNDED ON CLAY

Labels visible in drawing:
- A
- B
- C
- 10
- 20
- 60
- 1
- 2.0
- 5
- 5m
- 1.8
- 2.25
- 15m minimum
- 200×200×10
- 200×200×10
- 200×200×10
- 160×160×5 internals
- 114×3.6 CHS
- Asphalt and Silty clay
- 4N° 750mm ø bored piles
- 6 H20 main bars
- H10 links at 300 ctrs

# 4    METHOD STATEMENT

Preliminaries
- The site should be secured to prevent unauthorised access by member of the public.
- As there is working at height, full PPE must be worn, including fall arrest equipment and safe methods of work must be established and followed.
- Additional ground investigation must be carried out to determine which locations are founded on rock, and which are founded on clay. This will necessitate lane closures of the two adjacent carriageways during the site investigation works.

The first three lanes of the carriageway must be secured, with appropriate barriers to protect the construction team, and to prevent the construction team accidentally straying into a 'live' carriageway. Systems of safe access must be established. This procedure must be repeated as the construction proceeds across the full carriageway in stages.

Sequence of work
- Lane closures provided between grids 1 and 4.
- Verify that existing asphalt (assumed to be a carriageway construction) is adequate for construction plant.
- Install bored pile foundations, grids 1 – 3. If the location is over rock, bore to rock level and install casing. Drill 50 mm holes in rock to prescribed depth, before grouting 25 mm bars in position. Reinforce piles and complete.
- Complete pile caps, grids 1 to 3.
- Erect braced tower on grid 1 and erect supports and bracing on grids 2 and 3.
- Erect roof canopy trusses, grids 1 – 3 and primary trusses on grids A, B and C, with associated plan bracing at upper and lower chord levels. Note that this part of the structure is stable, without the need for additional temporary bracing.
- Erect purlins grids 1 – 3 and roof cladding.
- Repeat the previous steps, closing three lanes at a time. Each portion of the structure is stable.
    - Complete piling (and tension bar installation if founded on rock).
    - Complete pile caps.
    - Erect steelwork, bracing and cladding.
- After completion of the roof canopy, close the carriageway, grids 5 – 7, so the steelwork for the control tower may be erected.
- Erect the control tower steelwork, installing the crossed flat bracing as erection proceeds.
- Install the flooring to the tower, cladding etc

# 5    PROGRAMME

41

The construction programme is shown on the following page, which assumes the necessary additional site investigation has been completed.

The programme is by necessity lengthy, as the brief stipulates that only three lanes of the carriageway may be closed at any one time, meaning that the works must be completed as a linear progression.

# Construction Programme

| Activity | Week Duration |
|---|---|
| Initial works | 2 |
| Establish site and verify adequacy of hard standing | 2 |
| | |
| Grids 1, 2 and 3 | 6 |
| Construct piles/ tension bars | 3 |
| Complete pile caps | 2 |
| Erect steekwork | 1 |
| Cladding | 1 |
| | |
| Grids 4, 5 and 6 | 6 |
| Construct piles/ tension bars | 3 |
| Complete pile caps | 2 |
| Erect steekwork | 1 |
| Cladding | 1 |
| | |
| Grids 7, 8 and 9 | 6 |
| Construct piles/ tension bars | 3 |
| Complete pile caps | 2 |
| Erect steekwork | 1 |
| Cladding | 1 |
| | |
| Grids 10, 11 | 5 |
| Construct piles/ tension bars | 2 |
| Complete pile caps | 2 |
| Erect steekwork | 1 |
| Cladding | 1 |
| | |
| Control room tower | 3 |
| Erect steelwork | 1 |
| Floors, cladding etc | 2 |

Week: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28

# 6 CLIENT LETTER

43

A Designer
Ascot, Berkshire SL5 7QN

31 December 2017

Dear Sir,

RE: Revised location for the control room.

With reference to your requirement to revise the position of the control room, we have the following comments:

1. The revised position of the control room is eccentric to the support locations. To achieve this, our outline proposal is to support the control room by a pair of 5 m deep trusses extending as cantilevers from the control room tower. Access to the control room would be via the tower and the cantilever.

2. The steelwork needed to support the revised position of the control room is perfectly feasible. The design must be revised, with increased section sizes and additional steelwork introduced, but the revised position can be accommodated.

3. The eccentricity of the control room (and the cantilever trusses) increases the overturning moment considerably, which will have a significant effect on the foundations.

4. The overturning moment must be resisted by tension in the foundations. The additional forces will increase the length of the piles needed in clay, and increase the requirement for tension bars grouted into the rock, depending on the ground conditions under the tower.

5. The additional works in the foundations will add significantly to the construction time and cost of the scheme. The additional steelwork is a more modest addition to the erection program.

6. If the proposed changes are implemented, the control room will be located at the end of a steel cantilever, supported on a steel tower. The flexibility of the superstructure in the proposed configuration may mean that control room personnel perceive some movement on windy days, and some vibration depending on the traffic conditions. It is recommended that a dynamic vibration analysis be completed to

assess these effects. If necessary, additional steelwork to stiffen the structure can be introduced.

The proposed changes will result in:

1. Additional design costs.
2. Additional costs to assess the dynamic behavior of the proposed cantilevered arrangement.
3. Significant additional cost and time required to construct the new foundations.
4. Additional costs for the steelwork and cladding to the cantilever from the tower to the control room.

Yours sincerely

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