6190 Ady Smith
Hilton Scout Hut Extension
Structural Calculations

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Project				Job Ref.	
Hilton Scout Headquarter Extension				6190	
Section				Sheet no./rev.	
Contents				1	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GB	16/04/2020	HD	16/04/2020		

Contents:

EAST EXTENSION PORTAL FRAME DESIGN

LOADING

The design has been carried out by assuming the following loads.

Dead Load

- Roof Cladding (0.20 kN/m2 incl. Purlins)

Imposed Load

- Roof (0.6 kN/m2)

Wind Load

See MasterPort calculations for wind load.

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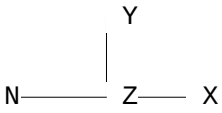

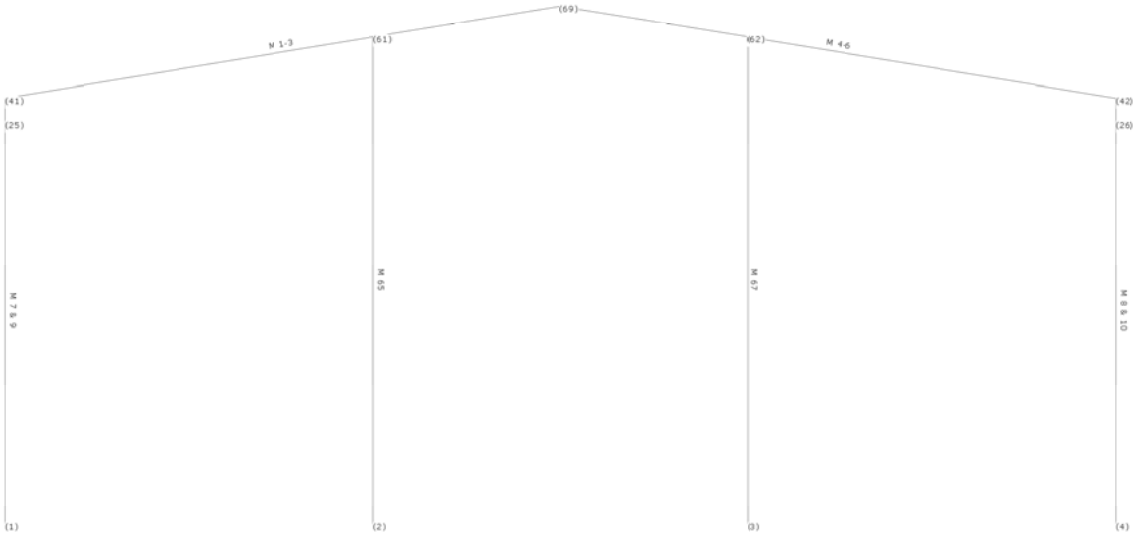

Load Case 001 : 1.25 (Dead+Services) + 1.5 Live/Snow
Frame Geometry - (Grid Line : A - A) - Front View

Not to Scale

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MasterPort Data File
Loading Cases and Load Combination

Load Group Labels

Load Group UT	Unity Load Factor (All Cases)
Load Group D1	Dead Load
Load Group D2	Services
Load Group L0	Uniform Snow/live load
Load Group L1	Live Load
Load Group W1	Wind on Side (Top Values) --> (Fetches 305 to 35 Degrees)
Load Group W2	Wind on Side (Bottom Values) --> (Fetches 215 to 305 Degrees)
Load Group W3	Wind on Gable (Fetches 125 to 215 Degrees)
Load Group P1	Wind on Side (Top Values) --> with Internal Pressure Cpi = 0.2
Load Group P3	Wind on Gable with Internal Pressure Cpi = 0.2
Load Group S1	Wind on Side (Top Values) --> with Internal Suction Csi = 0.3
Load Group S3	Wind on Gable with Internal Suction Csi = 0.3

Load Case 001 : 1.25 (Dead+Services) + 1.5 Live/Snow
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1

Load Case 002 : Dead + Services + Live/Snow (Service)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 1.00 L0 + 1.00 L1

Load Case 003 : Live/Snow Only (Service)
Load Combination + 1.00 UT + 1.00 L0 + 1.00 L1

Load Case 004 : (Sway Stability)
Load Combination + 1.00 UT
Notional Loads Apply horizontal notional loads at 0.0 degrees from X axis equal to 0.5% of the factored vertical loads in case 1

Load Case 005 : 1.25 (Dead+Services) + 1.5 Live/Snow + Notional -->
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1
Notional Loads Apply horizontal notional loads at 0.0 degrees from X axis equal to 0.5% of the factored vertical loads in case 5

Load Case 006 : 1 Dead + 1.5 Side Wind W1 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 W1

Load Case 007 : 1 Dead + 1.5 Side Wind P1 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 P1

Load Case 008 : 1 Dead + 1.5 Side Wind S1 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 S1

Load Case 009 : 1.25(Dead + Serv) + 1.5Side Wind W1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 W1

Load Case 010 : 1.35(Dead + Serv) + 0.75Side Wind W1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 W1

Load Case 011 : 1.25(Dead + Serv) + 1.5Side Wind P1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 P1

Load Case 012 : 1.35(Dead + Serv) + 0.75Side Wind P1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 P1

Load Case 013 : 1.25(Dead + Serv) + 1.5Side Wind S1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 S1

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Load Case 014 : 1.35(Dead + Serv) + 0.75Side Wind S1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 S1

Load Case 015 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 W1

Load Case 016 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 W1

Load Case 017 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind P1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 P1

Load Case 018 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind P1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 P1

Load Case 019 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind S1 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 S1

Load Case 020 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind S1 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 S1

Load Case 021 : 1 Dead + 1.5 Side Wind W2 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 W2

Load Case 022 : 1.25(Dead + Serv) + 1.5Side Wind W2 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 W2

Load Case 023 : 1.35(Dead + Serv) + 0.75Side Wind W2 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 W2

Load Case 024 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W2 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 W2

Load Case 025 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W2 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 W2

Load Case 026 : 1 Dead + 1.5 Side Wind W3 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 W3

Load Case 027 : 1 Dead + 1.5 Side Wind P3 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 P3

Load Case 028 : 1 Dead + 1.5 Side Wind S3 (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.50 S3

Load Case 029 : 1.25(Dead + Serv) + 1.5 Side Wind W3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 W3

Load Case 030 : 1.35(Dead + Serv) + 0.75Side Wind W3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 W3

Load Case 031 : 1.25(Dead + Serv) + 1.5 Side Wind P3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 P3

Load Case 032 : 1.35(Dead + Serv) + 0.75Side Wind P3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 P3

Load Case 033 : 1.25(Dead + Serv) + 1.5 Side Wind S3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 S3

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Load Case 034 : 1.35(Dead + Serv) + 0.75Side Wind S3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 0.75 S3

Load Case 035 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 W3

Load Case 036 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 W3

Load Case 037 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind P3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 P3

Load Case 038 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind P3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 P3

Load Case 039 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind S3 (Local Wind)
Load Combination + 1.00 UT + 1.25 D1 + 1.25 D2 + 1.50 L0 + 1.50 L1 + 0.75 S3

Load Case 040 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind S3 (Local Wind)
Load Combination + 1.00 UT + 1.35 D1 + 1.35 D2 + 1.05 L0 + 1.05 L1 + 0.75 S3

Load Case 041 : (Dead+Services) + Live/Snow + Notional --> (Service)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1

Load Case 042 : 1 Dead + Side Wind W1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 W1

Load Case 043 : 1 Dead + Side Wind P1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 P1

Load Case 044 : 1 Dead + Side Wind S1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 S1

Load Case 045 : (Dead + Serv) + Side Wind W1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W1

Load Case 046 : (Dead + Serv) + Side Wind W1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W1

Load Case 047 : (Dead + Serv) + Side Wind P1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 P1

Load Case 048 : (Dead + Serv) + Side Wind P1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 P1

Load Case 049 : (Dead + Serv) + Side Wind S1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 S1

Load Case 050 : (Dead + Serv) + Side Wind S1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 S1

Load Case 051 : (Dead + Serv) + Live/Snow + Side Wind W1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W1

Load Case 052 : (Dead + Serv) + Live/Snow + Side Wind W1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W1

Load Case 053 : (Dead + Serv) + Live/Snow + Side Wind P1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 P1

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Load Case 054 : (Dead + Serv) + Live/Snow + Side Wind P1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 P1

Load Case 055 : (Dead + Serv) + Live/Snow + Side Wind S1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 S1

Load Case 056 : (Dead + Serv) + Live/Snow + Side Wind S1 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 S1

Load Case 057 : 1 Dead + Side Wind W2 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 W2

Load Case 058 : (Dead + Serv) + Side Wind W2 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W2

Load Case 059 : (Dead + Serv) + Side Wind W2 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W2

Load Case 060 : (Dead + Serv) + Live/Snow + Side Wind W2 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W2

Load Case 061 : (Dead + Serv) + Live/Snow + Side Wind W2 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W2

Load Case 062 : 1 Dead + Side Wind W3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 W3

Load Case 063 : 1 Dead + Side Wind P3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 P3

Load Case 064 : 1 Dead + Side Wind S3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 0.50 S3

Load Case 065 : (Dead + Serv) + Side Wind W3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W3

Load Case 066 : (Dead + Serv) + Side Wind W3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 W3

Load Case 067 : (Dead + Serv) + Side Wind P3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 P3

Load Case 068 : (Dead + Serv) + Side Wind P3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 P3

Load Case 069 : (Dead + Serv) + Side Wind S3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 S3

Load Case 070 : (Dead + Serv) + Side Wind S3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.50 S3

Load Case 071 : (Dead + Serv) + Live/Snow + Side Wind W3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W3

Load Case 072 : (Dead + Serv) + Live/Snow + Side Wind W3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 W3

Load Case 073 : (Dead + Serv) + Live/Snow + Side Wind P3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 P3

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Load Case 074 : (Dead + Serv) + Live/Snow + Side Wind P3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 P3

Load Case 075 : (Dead + Serv) + Live/Snow + Side Wind S3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 S3

Load Case 076 : (Dead + Serv) + Live/Snow + Side Wind S3 (Service) (Local Wind)
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2 + 0.70 L0 + 0.70 L1 + 0.50 S3

Load Case 077 : D1 Dead Load
Load Combination + 1.00 UT + 1.00 D1

Load Case 078 : D1 Dead Load plus D2 Services
Load Combination + 1.00 UT + 1.00 D1 + 1.00 D2

Load Case 079 : L0 Uniform Snow/live load plus L1 Live Load
Load Combination + 1.00 UT + 1.00 L0 + 1.00 L1

Load Case 080 : W1 Wind on Side (Top Values) --> (Local Wind)
Load Combination + 1.00 UT + 1.00 W1

Load Case 081 : W2 Wind on Side (Bottom Values) --> (Local Wind)
Load Combination + 1.00 UT + 1.00 W2

Load Case 082 : W3 Wind on Gable (Local Wind)
Load Combination + 1.00 UT + 1.00 W3

Member Properties and Loading

```
MEMBER 1  (4.207 m)
  M  N41 to N61       203x133 UB 25 [S 275] ß +000.00
   A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Rafter 1 of Bay  1
   D1 UDLY -000.200             [ kN/m ]
   D2 UDLY -000.000             [ kN/m ]
   L0 UDLY -000.600             [ kN/m ]
   W1 PTRN +000.727 0.000 1.183 +000.727    [Co/m,m,m,Co/m]
   P1 PTRN +000.828 0.000 1.183 +000.828    [Co/m,m,m,Co/m]
   S1 PTRN +000.583 0.000 1.183 +000.583    [Co/m,m,m,Co/m]
   W1 PTRN +000.250 1.183 6.335 +000.250    [Co/m,m,m,Co/m]
   P1 PTRN +000.348 1.183 6.335 +000.348    [Co/m,m,m,Co/m]
   S1 PTRN +000.104 1.183 6.335 +000.104    [Co/m,m,m,Co/m]
   W2 PTRN -000.043 0.000 1.183 -000.043    [Co/m,m,m,Co/m]
   W2 PTRN -000.043 1.183 6.335 -000.043    [Co/m,m,m,Co/m]
   W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
   P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
   S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
   W3 UDLN +000.211             [ kN/m ]
   P3 UDLN +000.326             [ kN/m ]
   S3 UDLN +000.041             [ kN/m ]
   W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
   P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
   S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
MEMBER 2  (0.017 m)
  M  N61 to N65       203x133 UB 25 [S 275] ß +000.00
   A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Rafter 1 of Bay  1
   D1 UDLY -000.200             [ kN/m ]
   D2 UDLY -000.000             [ kN/m ]
   L0 UDLY -000.600             [ kN/m ]
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```
    W1 PTRN +000.727 0.000 1.183 +000.727    [Co/m,m,m,Co/m]
    P1 PTRN +000.828 0.000 1.183 +000.828    [Co/m,m,m,Co/m]
    S1 PTRN +000.583 0.000 1.183 +000.583    [Co/m,m,m,Co/m]
    W1 PTRN +000.250 1.183 6.335 +000.250    [Co/m,m,m,Co/m]
    P1 PTRN +000.348 1.183 6.335 +000.348    [Co/m,m,m,Co/m]
    S1 PTRN +000.104 1.183 6.335 +000.104    [Co/m,m,m,Co/m]
    W2 PTRN -000.043 0.000 1.183 -000.043    [Co/m,m,m,Co/m]
    W2 PTRN -000.043 1.183 6.335 -000.043    [Co/m,m,m,Co/m]
    W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
    P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
    S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
    W3 UDLN +000.211             [ kN/m ]
    P3 UDLN +000.326             [ kN/m ]
    S3 UDLN +000.041             [ kN/m ]
    W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
    P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
    S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
MEMBER 3  (2.111 m)
  M  N65 to N69       203x133 UB 25 [S 275] ß +000.00
    A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
    E 210.0E6           G 80.77E6
    UT Spacing 02.725 [Multiply AllLoads]
        Rafter 1 of Bay  1
    D1 UDLY -000.200             [ kN/m ]
    D2 UDLY -000.000             [ kN/m ]
    L0 UDLY -000.600             [ kN/m ]
    W1 PTRN +000.727 0.000 1.183 +000.727    [Co/m,m,m,Co/m]
    P1 PTRN +000.828 0.000 1.183 +000.828    [Co/m,m,m,Co/m]
    S1 PTRN +000.583 0.000 1.183 +000.583    [Co/m,m,m,Co/m]
    W1 PTRN +000.250 1.183 6.335 +000.250    [Co/m,m,m,Co/m]
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    W2 PTRN -000.043 0.000 1.183 -000.043    [Co/m,m,m,Co/m]
    W2 PTRN -000.043 1.183 6.335 -000.043    [Co/m,m,m,Co/m]
    W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
    P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
    S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
    W3 UDLN +000.211             [ kN/m ]
    P3 UDLN +000.326             [ kN/m ]
    S3 UDLN +000.041             [ kN/m ]
    W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
    P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
    S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
MEMBER 4  (4.207 m)
  M  N42 to N62       203x133 UB 25 [S 275] ß +180.00
    A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
    E 210.0E6           G 80.77E6
    UT Spacing 02.725 [Multiply AllLoads]
        Rafter 2 of Bay  1
    D1 UDLY -000.200             [ kN/m ]
    D2 UDLY -000.000             [ kN/m ]
    L0 UDLY -000.600             [ kN/m ]
    W1 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
    P1 PTRN +000.624 5.152 6.335 +000.624    [Co/m,m,m,Co/m]
    S1 PTRN +000.380 5.152 6.335 +000.380    [Co/m,m,m,Co/m]
    W1 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
    P1 PTRN +000.315 0.000 5.152 +000.315    [Co/m,m,m,Co/m]
    S1 PTRN +000.070 0.000 5.152 +000.070    [Co/m,m,m,Co/m]
    W2 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
    W2 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
    W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
    P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
    S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
    W3 UDLN +000.211             [ kN/m ]
    P3 UDLN +000.326             [ kN/m ]
    S3 UDLN +000.041             [ kN/m ]
    W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
    P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
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    S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
MEMBER 5  (0.017 m)
  M  N62 to N66       203x133 UB 25 [S 275] ß +180.00
   A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Rafter 2 of Bay  1
   D1 UDLY -000.200             [ kN/m ]
   D2 UDLY -000.000             [ kN/m ]
   L0 UDLY -000.600             [ kN/m ]
   W1 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
   P1 PTRN +000.624 5.152 6.335 +000.624    [Co/m,m,m,Co/m]
   S1 PTRN +000.380 5.152 6.335 +000.380    [Co/m,m,m,Co/m]
   W1 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
   P1 PTRN +000.315 0.000 5.152 +000.315    [Co/m,m,m,Co/m]
   S1 PTRN +000.070 0.000 5.152 +000.070    [Co/m,m,m,Co/m]
   W2 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
   W2 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
   W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
   P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
   S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
   W3 UDLN +000.211             [ kN/m ]
   P3 UDLN +000.326             [ kN/m ]
   S3 UDLN +000.041             [ kN/m ]
   W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
   P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
   S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
MEMBER 6  (2.111 m)
  M  N66 to N69       203x133 UB 25 [S 275] ß +180.00
   A 31.96E-4          Iy 2341E-8          Iz 308.5E-8         It 5.96E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Rafter 2 of Bay  1
   D1 UDLY -000.200             [ kN/m ]
   D2 UDLY -000.000             [ kN/m ]
   L0 UDLY -000.600             [ kN/m ]
   W1 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
   P1 PTRN +000.624 5.152 6.335 +000.624    [Co/m,m,m,Co/m]
   S1 PTRN +000.380 5.152 6.335 +000.380    [Co/m,m,m,Co/m]
   W1 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
   P1 PTRN +000.315 0.000 5.152 +000.315    [Co/m,m,m,Co/m]
   S1 PTRN +000.070 0.000 5.152 +000.070    [Co/m,m,m,Co/m]
   W2 PTRN +000.525 5.152 6.335 +000.525    [Co/m,m,m,Co/m]
   W2 PTRN +000.216 0.000 5.152 +000.216    [Co/m,m,m,Co/m]
   W3 PTRN +000.278 2.957 6.335 +000.278    [Co/m,m,m,Co/m]
   P3 PTRN +000.393 2.957 6.335 +000.393    [Co/m,m,m,Co/m]
   S3 PTRN +000.107 2.957 6.335 +000.107    [Co/m,m,m,Co/m]
   W3 UDLN +000.211             [ kN/m ]
   P3 UDLN +000.326             [ kN/m ]
   S3 UDLN +000.041             [ kN/m ]
   W3 PTRN +000.397 0.000 2.957 +000.397    [Co/m,m,m,Co/m]
   P3 PTRN +000.513 0.000 2.957 +000.513    [Co/m,m,m,Co/m]
   S3 PTRN +000.227 0.000 2.957 +000.227    [Co/m,m,m,Co/m]
MEMBER 7  (4.530 m)
  M  N1 to N25        254x146 UB 31 [S 275] ß +000.00
   A 39.67E-4          Iy 4414E-8          Iz 448.4E-8         It 8.55E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Column  1
   UT PartFix 20.00 +++ ---    (0/10/20)
   W1 UDLX +000.302             [ kN/m ]
   P1 UDLX +000.258             [ kN/m ]
   S1 UDLX +000.503             [ kN/m ]
   W2 UDLX +000.302             [ kN/m ]
   W3 UDLX -000.519             [ kN/m ]
   P3 UDLX -000.725             [ kN/m ]
   S3 UDLX -000.439             [ kN/m ]
   W1 UDLZ -000.371             [ kN/m ]
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    P1 UDLZ -000.515             [ kN/m ]
    S1 UDLZ -000.320             [ kN/m ]
    W2 UDLZ -000.371             [ kN/m ]
    W3 UDLZ +000.271             [ kN/m ]
    P3 UDLZ +000.228             [ kN/m ]
    S3 UDLZ +000.456             [ kN/m ]
MEMBER 8  (4.530 m)
  M  N4 to N26        254x146 UB 31 [S 275] ß +180.00
   A 39.67E-4          Iy 4414E-8          Iz 448.4E-8         It 8.55E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Column  2
   UT PartFix 20.00 +++ ---    (0/10/20)
   W1 UDLX +000.150             [ kN/m ]
   P1 UDLX +000.274             [ kN/m ]
   S1 UDLX +000.029             [ kN/m ]
   W2 UDLX +000.150             [ kN/m ]
   W3 UDLX +000.519             [ kN/m ]
   P3 UDLX +000.725             [ kN/m ]
   S3 UDLX +000.439             [ kN/m ]
   W1 UDLZ -000.231             [ kN/m ]
   P1 UDLZ -000.350             [ kN/m ]
   S1 UDLZ -000.154             [ kN/m ]
   W2 UDLZ -000.231             [ kN/m ]
   W3 UDLZ +000.271             [ kN/m ]
   P3 UDLZ +000.228             [ kN/m ]
   S3 UDLZ +000.456             [ kN/m ]
MEMBER 9  (0.270 m)
  M  N25 to N41       254x146 UB 31 [S 275] ß +000.00
   A 39.67E-4          Iy 4414E-8          Iz 448.4E-8         It 8.55E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Column  1
   UT PartFix 20.00 +++ ---    (0/10/20)
   W1 UDLX +000.302             [ kN/m ]
   P1 UDLX +000.258             [ kN/m ]
   S1 UDLX +000.503             [ kN/m ]
   W2 UDLX +000.302             [ kN/m ]
   W3 UDLX -000.519             [ kN/m ]
   P3 UDLX -000.725             [ kN/m ]
   S3 UDLX -000.439             [ kN/m ]
   W1 UDLZ -000.371             [ kN/m ]
   P1 UDLZ -000.515             [ kN/m ]
   S1 UDLZ -000.320             [ kN/m ]
   W2 UDLZ -000.371             [ kN/m ]
   W3 UDLZ +000.271             [ kN/m ]
   P3 UDLZ +000.228             [ kN/m ]
   S3 UDLZ +000.456             [ kN/m ]
MEMBER 10  (0.270 m)
  M  N26 to N42       254x146 UB 31 [S 275] ß +180.00
   A 39.67E-4          Iy 4414E-8          Iz 448.4E-8         It 8.55E-8
   E 210.0E6           G 80.77E6
   UT Spacing 02.725 [Multiply AllLoads]
       Column  2
   UT PartFix 20.00 +++ ---    (0/10/20)
   W1 UDLX +000.150             [ kN/m ]
   P1 UDLX +000.274             [ kN/m ]
   S1 UDLX +000.029             [ kN/m ]
   W2 UDLX +000.150             [ kN/m ]
   W3 UDLX +000.519             [ kN/m ]
   P3 UDLX +000.725             [ kN/m ]
   S3 UDLX +000.439             [ kN/m ]
   W1 UDLZ -000.231             [ kN/m ]
   P1 UDLZ -000.350             [ kN/m ]
   S1 UDLZ -000.154             [ kN/m ]
   W2 UDLZ -000.231             [ kN/m ]
   W3 UDLZ +000.271             [ kN/m ]
   P3 UDLZ +000.228             [ kN/m ]
```

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   S3 UDLZ +000.456              [ kN/m ]
```

MEMBER 65 ✸✸✸ ✸⚐⚐ (5.487 m)

```
  M  N2 to N61          178x102 UB 19 [S 275] ß +090.00
   A 24.26E-4            Iy 1357E-8          Iz 137.6E-8          It 4.41E-8
   E 210.0E6             G 80.77E6
       Gable Column
   W1 UDLZ -001.483               ( kN/m )
   P1 UDLZ -002.151               ( kN/m )
   S1 UDLZ -001.135               ( kN/m )
   W2 UDLZ -001.483               ( kN/m )
   W3 UDLZ +001.412               ( kN/m )
   P3 UDLZ +001.187               ( kN/m )
   S3 UDLZ +002.374               ( kN/m )
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MEMBER 67 ✸✸✸ ✸⚐⚐ (5.487 m)

```
  M  N3 to N62          178x102 UB 19 [S 275] ß +090.00
   A 24.26E-4            Iy 1357E-8          Iz 137.6E-8          It 4.41E-8
   E 210.0E6             G 80.77E6
       Gable Column
   W1 UDLZ -001.373               ( kN/m )
   P1 UDLZ -002.022               ( kN/m )
   S1 UDLZ -001.006               ( kN/m )
   W2 UDLZ -001.373               ( kN/m )
   W3 UDLZ +001.412               ( kN/m )
   P3 UDLZ +001.187               ( kN/m )
   S3 UDLZ +002.374               ( kN/m )
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MEMBER 91 (5.845 m)
M N41 to N57 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6
Roof Bracing

MEMBER 92 (5.845 m)
M N42 to N58 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6
Roof Bracing

MEMBER 95 (5.845 m)
M N57 to N65 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6
Roof Bracing

MEMBER 96 (5.845 m)
M N58 to N66 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6
Roof Bracing

MEMBER 99 (5.845 m)
M N65 to N70 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6

Roof Bracing

MEMBER 100 (5.845 m)
M N66 to N70 114.3x3.2 CHS 8.77 [Grade 43] ß +000.00
A 11.17E-4 Iy 172.5E-8 Iz 172.5E-8 It 344.9E-8
E 210.0E6 G 80.77E6
Roof Bracing

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Nodal Deflections Serviceability (Maximum Values)

Node	Nodal Displacements (mm)				Nodal Rotations (Degrees)			
	δX→	δY↑	δZ↗	δXYZ	ΦX→	ΦY↑	ΦZ↗	ΦXYZ
25	17.59	0.04	-1.39	17.64	0.24	-0.11	-0.15	0.31
26	17.56	-0.04	1.12	17.59	-0.17	0.11	-0.16	0.26
41	18.25	0.04	0.31	18.26	0.25	-0.11	-0.13	0.30
42	18.28	-0.04	0.31	18.28	-0.17	0.11	-0.15	0.25
62	18.30	-0.12	6.74	19.50	1.35	-0.12	0.03	1.36
65	18.28	-0.12	6.77	19.50	1.36	0.12	0.03	1.36
69	18.28	-0.47	11.08	21.38	0.79	0.00	-0.01	0.79

Nodal Deflections Serviceability (Minimum Values)

Node	Nodal Displacements (mm)				Nodal Rotations (Degrees)			
	δX→	δY↑	δZ↗	δXYZ	ΦX→	ΦY↑	ΦZ↗	ΦXYZ
25	-0.17	-0.03	1.12	1.13	-0.17	0.08	-0.01	0.19
26	0.04	0.04	-0.94	0.94	0.15	-0.06	0.02	0.16
41	-0.06	-0.04	-0.23	0.24	-0.17	0.09	-0.01	0.19
42	0.01	0.04	-0.24	0.24	0.15	-0.06	0.02	0.17
62	-0.01	0.08	-2.86	2.86	-0.45	0.05	0.00	0.45
65	-0.05	0.06	-2.88	2.88	-0.39	-0.04	0.00	0.39
69	0.00	0.18	-4.16	4.16	-0.24	0.00	0.00	0.24

Member Forces Ultimate (Maximum Values)

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
1	41	7.68T	-0.05	-9.49	0.07	24.40	0.13	10.82	0.23	4.56
	61	8.25T	0.05	-11.94	0.07	-9.93	0.23	@ 0.745	@ 4.206	@ 1.621
3	65	7.79T	0.04	9.05	0.02	-9.81	0.22	10.82	0.23	4.56
	69	8.06T	-0.04	3.81	0.02	2.47	0.22	@ 0.745	@ 4.206	@ 1.621
4	42	7.68T	0.05	9.52	-0.10	-25.98	0.17	8.41	-0.27	4.47
	62	8.25T	-0.05	8.35	-0.10	8.32	-0.27	@ 4.224	@ 4.206	@ 1.588
6	66	8.57T	-0.04	6.94	0.02	8.41	-0.22	8.41	-0.27	4.47
	69	8.84T	0.04	-2.50	0.02	2.47	-0.22	@ 4.224	@ 4.206	@ 1.588
7	1	9.55C	0.00	8.68	5.06	0.00	0.00	18.33	6.09	16.02
	25	10.24T	0.00	7.60	-4.47	23.44	1.34	@ 4.213	@ 2.401	@ 2.401
8	4	14.28C	0.00	-6.84	4.48	0.00	0.00	-5.81	5.39	13.78
	26	12.55C	0.00	7.60	-3.96	-24.70	1.18	@ 1.993	@ 2.401	@ 2.401
9	25	10.24T	0.00	7.60	-4.47	23.44	1.34	18.33	6.09	16.02
	41	10.32T	0.00	8.40	-5.04	24.04	-0.17	@ 4.213	@ 2.401	@ 2.401
10	26	12.55C	0.00	7.60	-3.96	-24.70	1.18	-5.81	5.39	13.78
	42	12.45C	0.00	8.40	-4.47	-25.77	0.19	@ 1.993	@ 2.401	@ 2.401
65	2	23.25C	0.00	-9.77	0.00	0.00	0.00	-13.40	0.00	14.75
	61	21.96C	0.00	9.77	0.00	0.00	0.00	@ 2.744	@ 0.000	@ 2.744
67	3	16.91C	0.00	-9.77	0.00	0.00	0.00	-13.40	0.00	14.75
	62	15.63C	0.00	9.77	0.00	0.00	0.00	@ 2.744	@ 0.000	@ 2.744

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Member Forces Ultimate (Minimum Values)

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
1	41	1.77C	0.02	6.30	-0.04	-2.62	-0.11	-9.93	-0.15	0.15
	61	0.57T	-0.02	4.20	-0.04	2.30	-0.15	@ 4.207	@ 4.206	@ 3.199
3	65	2.84C	-0.01	-3.27	0.00	2.03	-0.07	-9.93	-0.15	0.15
	69	1.86C	0.01	-0.31	0.00	-0.44	-0.07	@ 4.207	@ 4.206	@ 3.199
4	42	3.20C	-0.02	-6.36	0.04	5.93	-0.03	-5.77	0.15	0.15
	62	2.56C	0.02	-7.80	0.04	-5.77	0.15	@ 4.207	@ 4.206	@ 3.199
6	66	2.84C	0.01	-4.22	0.00	-4.94	0.07	-5.77	0.15	0.15
	69	1.86C	-0.01	1.31	0.00	-0.44	0.07	@ 4.207	@ 4.206	@ 3.199
7	1	8.86T	0.00	-5.87	-4.48	0.00	0.00	-5.81	-5.39	0.49
	25	7.82C	0.00	-0.95	3.96	-3.25	-1.18	@ 1.993	@ 2.401	@ 2.763
8	4	8.86T	0.00	-0.72	-3.43	0.00	0.00		-4.11	0.61
	26	10.24T	0.00	-4.07	3.05	4.01	-0.85		@ 2.401	@ 2.763
9	25	7.82C	0.00	-0.95	3.96	-3.25	-1.18	-5.81	-5.39	0.49
	41	7.72C	0.00	-1.23	4.47	-3.45	0.07	@ 1.993	@ 2.401	@ 2.763
10	26	10.24T	0.00	-4.07	3.05	4.01	-0.85		-4.11	0.61
	42	10.32T	0.00	-4.04	3.44	6.17	-0.07		@ 2.401	@ 2.763
65	2	6.08T	0.00	8.85	0.00	0.00	0.00	12.14	0.00	0.00
	61	7.10T	0.00	-8.85	0.00	0.00	0.00	@ 2.744	@ 0.000	@ 0.000
67	3	11.21T	0.00	8.32	0.00	0.00	0.00	11.42	0.00	0.00
	62	12.24T	0.00	-8.32	0.00	0.00	0.00	@ 2.744	@ 0.000	@ 0.000

Support Reactions Serviceability (Maximum Values)

Node	Directional Reactions (kN)			Moment Reactions (kN.m)		
	Rx→(kN)	Ry↑(kN)	Rz↗(kN)	Mx→(kN.m)	My↑(kN.m)	Mz↗(kN.m)
1	-5.067	-8.841	-6.883	0.648	0.061	5.535
2	0.000	12.458	4.069	0.000	0.000	0.000
3	0.000	12.056	-3.874	0.000	0.000	0.000
4	-4.002	9.153	-6.883	-0.469	0.065	5.236
Total	-99.201	485.666	-32.250	-1.378	0.014	113.675

Support Reactions Serviceability (Minimum Values)

Node	Directional Reactions (kN)			Moment Reactions (kN.m)		
	Rx→(kN)	Ry↑(kN)	Rz↗(kN)	Mx→(kN.m)	My↑(kN.m)	Mz↗(kN.m)
1	2.850	7.506	5.808	-0.469	-0.053	-0.678
2	0.000	-5.339	-3.874	0.000	0.000	0.000
3	0.000	-7.373	3.767	0.000	0.000	0.000
4	-0.172	-8.841	4.937	0.405	-0.013	0.129
Total	0.000	-148.422	0.000	0.007	0.000	0.000

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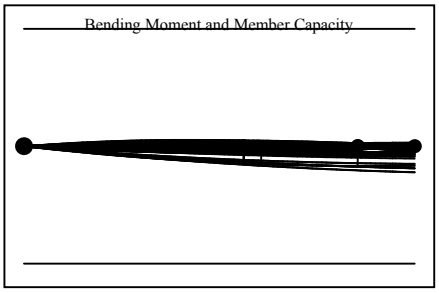
Axial with Moments (Member)
Column 1 : Members 7 & 9 (A\1)
Between 2.700 and 4.100 m, in Load Case 7

Member Loading and Member Forces

Loading Combination : 1 UT + 1 D1 + 1.5 P1

```
UT Spacing 02.725 [Multiply AllLoads]
UT PartFix 20.00 +++ ---   (Mt My Mz)
P1 UDLX +000.258             [ kN/m ]
P1 UDLZ -000.515             [ kN/m ]
```



Member Forces in Load Case 7 and Maximum Deflection from Load Case 81

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	1	4.52T	0.00	6.77	5.06	0.00	0.00		6.09	6.58
	41	5.99T	0.00	1.71	-5.04	20.34	0.05		@ 2.401	@ 2.492

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 275, 0, 20.34, 6.09	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	3.92 / 259.813 =	0.015	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 393.1/1	108.103 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.622 / 398.98 =	0.002	Low Shear
$M_{c.z.Rd} = f_y.W_{p'.z}/\gamma_{M0}$	275 x 94.1/1	25.878 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	39.67x275/1 (No bearing / block tearing design)	1090.925 kN	
$n = N_{Ed}/N_{pl.Rd}$	-5.985 / 1090.925 =	0.005	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	393.1, 16.364, 0.005	393.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	393.1 x 275/1	108.103 kN.m	
$W_{pl.N.z}$ = Fn($W_{pl.z}$, A_{vz}, n)	94.1, 25.129, 0.005	94.1 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	94.1 x 275/1	25.878 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})^{\alpha}+(M_{z.Ed}/M_{N.z.Rd})^{\beta}$	$(14.427/108.103)^2+(5.994/25.878)^1$=	0.249	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2,ψ)	14.4, 18.9, 0.764	1.123	Not Loaded
C_{mLT}=Max(0.6+0.4ψ, 0.4)	M = 18.88,ψ = 0.764	0.906	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 5.99,ψ = 0.512	0.805	Table B.3
C_{my}=Max(0.6+0.4ψ, 0.4)	M = 20.34,ψ = 0.000	0.6	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 1.4 =	1.4 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.123, 1.400, 448.4, 8.552, 0.06588, 210000	676.495 kN.m	
$\lambda_{LT}=\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 393.1 x 275 / 676.495	0.400	
γ_{LT}= Fn(λ_{LT},λ_{LT5950})	0.400, 0.413	1.000	Curve d
γ_{LT}.mod = Fn($\gamma_{I.T}$,λ_{LT},k_c,f)	1.000, 0.400, 0.955, 0.985	1.000	6.3.2.3
$M_{b.Rd} = \chi$ $W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 393.1 x 275≤ 108.103 =	108.103 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\gamma_y.N_{Rk}/\gamma_{M1})$	0 / 999.973	0.000	OK
$U_{N.z} = N_{Ed}/(\gamma_z.N_{Rk}/\gamma_{M1})$	0 / 974.186	0.000	OK
$U_{M.y} = M_{y.Ed}/(\gamma_{LT}.M_{..Rk}/\gamma_{M1})$	15.247 / 108.103	0.141	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	5.811 / 25.878	0.225	OK
$k_{yy}=C_{my}\{1+(\lambda_v-0.2)U_{N.y}\}$		0.600	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.805	
k_{yz}=0.6 k_{zz}		0.483	
k_{zy}=0.6 k_{yy}		0.360	

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$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.600x0.141+0.483x0.225	0.193	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+0.360x0.141+0.805x0.225	0.231	OK

Deflection Check - Load Case 81

In-span $\delta \leq$ Span/360	6.58$\leq$ 4800 / 360	6.58 mm	OK

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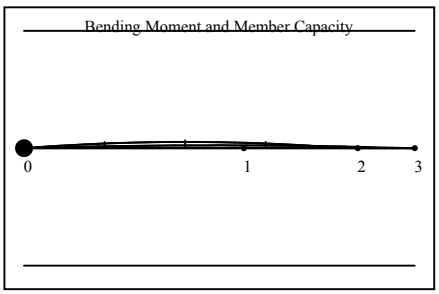
Appendix-BB Stability (Member) : Column 1 : Members 7 & 9 (A\1) Between 0.000 and 3.960 m, in Load Case 31

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 P3

```
UT Spacing 02.725 [Multiply AllLoads]
UT PartFix 20.00 +++ ---   (Mt My Mz)
P3 UDLX -000.725             [ kN/m ]
P3 UDLZ +000.228             [ kN/m ]
```



Lateral and Torsional Restraints

Side rails @ 2.7, 4.1 and 4.8 m
No stays are provided

Member Forces in Load Case 31

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	1	7.91T	0.00	-5.87	-2.22	0.00	0.00	-5.81	-2.66	0.00
	41	9.75T	0.00	8.36	2.25	5.97	0.06	@ 1.993	@ 2.356	@ 2.492

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 275, -9.75, 5.8, 2.65	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Tension Side Lateral Restraint Spacing Check, Lm

L_m= fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	9.7, 39.7,1.13, 393.1, 8.6	1.534m	
L_m< s	1.534<2.700 - Effect of tension side lateral restraints ignored		

Compression Resistance N.b.Rd

$\lambda z = \sqrt{A.fy/Ncrz}$	√39.67x275/592.64	1.358	
$N_{b.z.Rd}$ = Area.χ.f_y/γ_{M1}	39.67x0.4x275/10/1 =	436.525 kN	Curve b
i_s= Fn(i_y, i_z, a)	105.5, 33.6, 150.8	110.7 mm	
N_{crT}= Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 448, 9, 0.066, 3960, 0, 111	1273.6 kN	
$\lambda T = \sqrt{A.fy/NcrT}$	√39.67x275/ 10 /1273.65	0.925	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	39.67x0.645x275/10/1 =	703.400 kN	Curve b

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(L_t, E, I_z, I_T, I_w)	3.960, 210, 448.4, 8.552, 0.06588	96.195 kN.m	
M_{cr} = C_1• M_{cr0}	1.127 • 96.2	108.412 kN.m	
$\lambda_{LT}=\sqrt{W.f_y/M_{cr}}$	√ 393.1 x 275 / 676.495	0.999	
γ_{LT}= Fn(λ_{LT},ϕ_{LT},β)	0.999, 0.976, 0.750	0.701	Curve b
γ_{LT}.mod = Fn(γ_{LT},λ_{LT},k_c,f)	0.701, 0.999, 0.942, 0.973	0.720	6.3.2.3
$M_{b.Rd} = \chi W_{pl.y}.f_y$	0.720 x 393.1 x 275	77.810 kN.m	

Combined Axial and Bending 6.62

k_{zy}	1- {0.1/(C_{mLT}-0.25)}• $N_{Ed}/N_{b,z,Rd}$	1.002	
$N_{Ed}/N_{b,z,Rd}+k_{zy}.M_{y,Ed}/M_{b,Rd}$	9.745 / 436.525 + 1.002 x 5.808 / 77.81 =	0.097	OK

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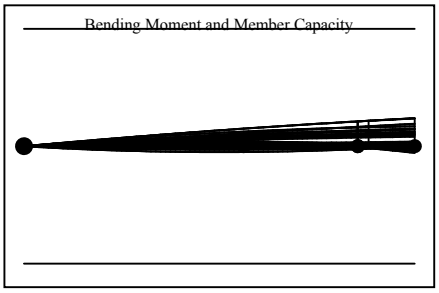
Axial with Moments (Member)
Column 2 : Members 8 & 10 (A\4)
Between 4.100 and 4.800 m, in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
UT PartFix 20.00 +++ ---   (Mt My Mz)
W2 UDLX +000.150             [ kN/m ]
W2 UDLZ -000.231             [ kN/m ]
```



Member Forces in Load Case 22 and Maximum Deflection from Load Case 82

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	4	14.18C	0.00	-6.84	-2.25	0.00	0.00		-2.68	4.66
	42	12.35C	0.00	-3.90	2.28	-25.77	0.09		@ 2.356	@ 2.446

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 275, 14.18, 25.77, 2.68	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	4.327 / 259.813 =	0.017	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 393.1/1	108.103 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	1.623 / 398.98 =	0.004	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 94.1/1	25.878 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	39.67 x 275/1 =	1090.925 kN	
$n = N_{Ed}/N_{pl.Rd}$	12.351 / 1090.925 =	0.011	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	393.1, 16.364, 0.011	393.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	393.1 x 275/1	108.103 kN.m	
$W_{pl.N.z}$ = Fn($W_{pl.z}$, A_{vz}, n)	94.1, 25.129, 0.011	94.1 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	94.1 x 275/1	25.878 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})^{\alpha}+(M_{z.Ed}/M_{N.z.Rd})^{\beta}$	$(22.895/108.103)^2+(1.28/25.878)^1$=	0.094	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x4.8 =	4.8	
$\lambda y = \sqrt{A.fy/Ncr}$	√39.67x275/3970.99	0.524	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	39.67x0.917x275/10/1 =	999.973 kN	Curve a
Lez = Kz.Lz	1x0.7 =	0.7	
$\lambda z = \sqrt{A.fy/Ncrz}$	√39.67x275/18966.57	0.24	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	39.67x0.986x275/10/1 =	1075.429 kN	Curve b
Let = Kt.Lx	1x4.8 =	4.8	
$\lambda T = \sqrt{A.fy/NcrT}$	√39.67x275/1046.82	1.021	
$N_{b.T.Rd}$= $Area.\chi.fy/\gamma_{M1}$	39.67x0.584x275/10/1 =	636.934 kN	Curve b

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, $M_{2,\psi}$)	-22.9, -25.8, 0.889	1.054	Not Loaded
C_{mLT}=Max(0.6+0.4$_{\psi}$, 0.4)	M = -25.77,$_{\psi}$ = 0.889	0.955	Table B.3
C_{mz}=Max(0.6+0.4$_{\psi}$, 0.4)	M = -1.28,$_{\psi}$ = -0.065	0.574	Table B.3
C_{my}=Max(0.6+0.4$_{\psi}$, 0.4)	M = -25.77,$_{\psi}$ = 0.000	0.6	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 0.7 =	0.7 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.054, 0.700, 448.4, 8.552, 0.06588, 210000	2452.575 kN.m	
$\lambda_{LT} = \sqrt{}$ W.f_y/M_{cr}	√ 393.1 x 275 / 2452.575	0.210	
γ_{LT}= Fn(λ_{LT},λ_{LT5950})	0.210, 0.210	1.000	Curve d
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	1.000, 0.210, 0.955, 0.985	1.000	6.3.2.3

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$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 393.1 x 275 ≤ 108.103 =	108.103 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\gamma_y.N_{Rk}/\gamma_{M1})$	14.184 / 999.973	0.014	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	14.184 / 1075.429	0.013	OK
$U_{M.y} = M_{y.Ed}/(\gamma_{LT}.M_{..Rk}/\gamma_{M1})$	23.472 / 108.103	0.217	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	1.054 / 25.878	0.041	OK
$k_{yy}=C_{my}\{1+(\lambda_v-0.2)U_{N.y}\}$		0.603	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.573	
$k_{yz}=0.6\ k_{zz}$		0.344	
$k_{zy}=0.6\ k_{yy}$		0.362	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.014+0.603x0.217+0.344x0.041	0.159	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.013+0.362x0.217+0.573x0.041	0.115	OK

Deflection Check - Load Case 82

In-span $\delta \leq$ Span/360	4.66 ≤ 4800 / 360	4.66 mm	OK

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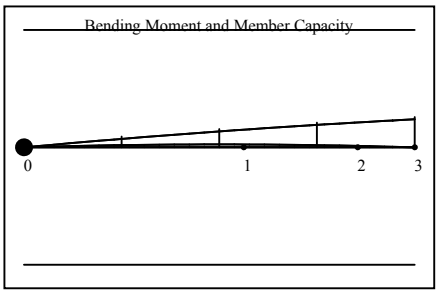
Appendix-BB Stability (Member) : Column 2 : Members 8 & 10 (A\4) in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
UT PartFix 20.00 +++ ---   (Mt My Mz)
W2 UDLX +000.150             [ kN/m ]
W2 UDLZ -000.231             [ kN/m ]
```



Lateral and Torsional Restraints

Side rails @ 2.7, 4.1 and 4.8 m
No stays are provided

Member Forces in Load Case 22

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	4	14.18C	0.00	-6.84	-2.25	0.00	0.00		-2.68	0.00
	42	12.35C	0.00	-3.90	2.28	-25.77	0.09		@ 2.356	@ 2.446

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 275, 14.18, 25.77, 2.68	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Tension Side Lateral Restraint Spacing Check, Lm

L_m= fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	14.2, 39.7,1.16, 393.1, 8.6	1.549m	
L_m< s	1.549<2.700 - Effect of tension side lateral restraints ignored		

Compression Resistance N.b.Rd

$\lambda z = \sqrt{A.fy/Ncrz}$	√39.67x275/403.37	1.646	
$N_{b.z.Rd}$ = Area.χ.f_y/γ_{M1}	39.67x0.294x275/10/1 =	320.506 kN	Curve b
i_s= Fn(i_y, i_z, a)	105.5, 33.6, 150.8	110.7 mm	
N_{crT}= Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 448, 9, 0.066, 4800, 0, 111	1046.8 kN	
$\lambda T = \sqrt{A.fy/NcrT}$	√39.67x275/ 10 /1046.82	1.021	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	39.67x0.584x275/10/1 =	636.934 kN	Curve b

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(L_t, E, I_z, I_T, I_w)	4.800, 210, 448.4, 8.552, 0.06588	71.948 kN.m	
M_{cr} = C_1• M_{cr0}	1.750 • 71.9	125.894 kN.m	
$\lambda_{LT}=\sqrt{}$ W.f_y/M_{cr}	√ 393.1 x 275 / 2452.575	0.927	
γ_{LT}= Fn(λ_{LT},ϕ_{LT},β)	0.927, 0.912, 0.750	0.744	Curve b
γ_{LT}.mod = Fn(γ_{LT},λ_{LT},k_c,f)	0.744, 0.927, 0.756, 0.882	0.844	6.3.2.3
$M_{b.Rd}$ =χ $W_{pl.y}$.f_y	0.844 x 393.1 x 275	91.216 kN.m	

Combined Axial and Bending 6.62

k_{zy}	1- {0.1λ_T/(C_{mLT}-0.25)}• $N_{Ed}/N_{b,z,Rd}$	0.994	
$N_{Ed}/N_{b,z,Rd}$+k_{zy}.$M_{y,Ed}/M_{b,Rd}$	14.184 / 320.506 + 0.994 x 25.774 / 91.216 =	0.325	OK

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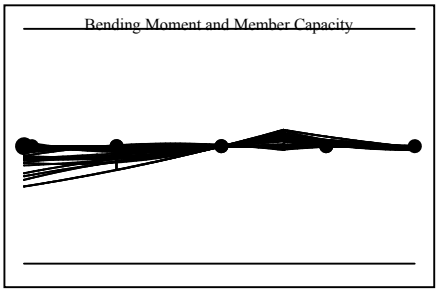
Axial with Moments (Member)
Rafter 1 of Bay 1 : Members 1-3 (A\1-2)
Between 1.500 and 3.200 m, in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
D1 UDLY -000.200              [ kN/m ]
D2 UDLY -000.000              [ kN/m ]
W2 PTRN -000.043 0.000 1.183 -000.043
W2 PTRN -000.043 1.183 6.335 -000.043
```



Member Forces in Load Case 22 and Maximum Deflection from Load Case 81

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	41	4.51T	-0.01	-5.72	-0.01	24.24	0.12	-9.93	-0.02	1.68
	69	3.41T	-0.02	3.80	0.02	0.76	0.03	@ 4.207	@ 4.224	@ 1.577

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, 0, 24.24, 0.12	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	7.435 / 203.402 =	0.037	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.007 / 329.914 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{p^1.z}/\gamma_{M0}$	275 x 70.9/1	19.498 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	31.96x275/1 (No bearing / block tearing design)	878.9 kN	
$n = N_{Ed}/N_{pl.Rd}$	-5.188 / 878.9 =	0.006	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	257.7, 12.811, 0.006	257.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	257.7 x 275/1	70.868 kN.m	
$W_{pl.N.z}$ = Fn($W_{pl.z}$, A_{vz}, n)	70.9, 20.779, 0.006	70.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	70.9 x 275/1	19.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})^{\alpha}+(M_{z.Ed}/M_{N.z.Rd})^{\beta}$	$(14.371/70.868)^2+(0.11/19.498)^1$=	0.047	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, $M_{2,\psi}$)	14.4, 0.1, 0.006	1.744	Not Loaded
C_{mLT}=Max(0.6+0.4$_{\psi}$, 0.4)	M = 14.36,$_{\psi}$ = 0.006	0.602	Table B.3
C_{mz}=Max(0.6+0.4$_{\psi}$, 0.4)	M = 0.11,$_{\psi}$ = 0.891	0.956	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h= 24.24, M_s= 0.39,$_{\psi}$ = 0.031,α_s= 0.016	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 1.7 =	1.7 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.744, 1.700, 308.5, 5.964, 0.02933, 210000	416.998 kN.m	
$\lambda_{LT}=\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 257.7 x 275 / 416.998	0.412	
γ_{LT}= Fn(λ_{LT},λ_{LT5950})	0.412, 0.525	0.995	Curve b
γ_{LT}.mod = Fn($\gamma_{I.T.}$,λ_{LT},k_c,f)	0.995, 0.412, 0.757, 0.915	1.000	6.3.2.3
$M_{b.Rd} = \chi$ $W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 257.7 x 275≤ 70.868 =	70.868 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\gamma_y.N_{Rk}/\gamma_{M1})$	0 / 671.812	0.000	OK
$U_{N.z} = N_{Ed}/(\gamma_z.N_{Rk}/\gamma_{M1})$	0 / 722.373	0.000	OK
$U_{M.y} = M_{y.Ed}/(\gamma_{LT}.M_{..Rk}/\gamma_{M1})$	14.371 / 70.868	0.203	OK
$U_{M.z} = M_{z.Ed}/(M_{Z.Rk}/\gamma_{M1})$	0.11 / 19.498	0.006	OK
$k_{yy}=C_{my}\{1+(\lambda_v-0.2)U_{N.y}\}$		0.400	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.956	
k_{yz}=0.6 k_{zz}		0.574	
k_{zy}=0.6 k_{yy}		0.240	

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$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.400x0.203+0.574x0.006	0.084	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+0.240x0.203+0.956x0.006	0.054	OK

Deflection Check - Load Case 81

In-span $\delta \leq$ Span/360	1.68≤ 6335 / 360	1.68 mm	OK

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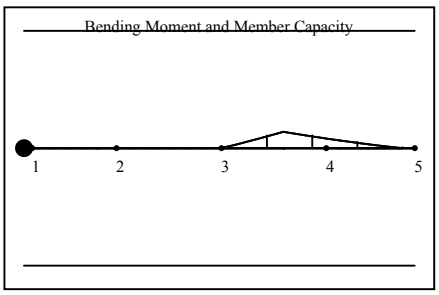
Appendix-BB Stability (Member) : Rafter 1 of Bay 1 : Members 1-3 (A\1-2) Between 3.208 and 6.141 m, in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
D1 UDLY -000.200              [ kN/m ]
D2 UDLY -000.000              [ kN/m ]
W2 PTRN -000.043 0.000 1.183 -000.043
W2 PTRN -000.043 1.183 6.335 -000.043
```



Lateral and Torsional Restraints

Purlins @ 1.5, 3.2, 4.9 and 6.335 m
Stay @ 6.335 m

Member Forces in Load Case 22

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	41	4.51T	-0.01	-5.72	-0.01	24.24	0.12	-9.93	-0.02	0.00
	69	3.41T	-0.02	3.80	0.02	0.76	0.03	@ 4.207	@ 4.224	@ 1.577

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, -5.19, 9.93, 0.12	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Tension Side Lateral Restraint Spacing Check, Lm

L_m= fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	5.2, 32.0,1.13, 257.7, 6.0	1.631m	
s < L_m	Tension side lateral restraints must be spaced <= to 1.631m in this portion.		

Compression Resistance N.b.Rd

i_s= Fn(i_y, i_z, a)	85.6, 31.1, 121.9	152.2 mm	
N_{crT}= Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 309, 6, 0.029, 2933, 122, 152	990.2 kN	
$\lambda T = \sqrt{A.fy/NcrT}$	√31.96x275/ 10 /990.16	0.942	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	31.96x0.634x275/10/1 =	557.268 kN	Curve b

Modification Factor for Non-Linear Moment Gradients Cn BB.3.3.2

R_1=($M_{y.Ed1}$+ a N_{Ed})/(f_y.$W_{pl.y}$)	0.633/(.001x 275x257.7) ≥ 0	0.009	
R_2=($M_{y.Ed2}$+ a N_{Ed})/(f_y.$W_{pl.y}$)	7.817/(.001x 275x257.7) ≥ 0	0.110	
R_3=($M_{y.Ed3}$+ a N_{Ed})/(f_y.$W_{pl.y}$)	7.759/(.001x 275x257.7) ≥ 0	0.109	
R_4=($M_{y.Ed4}$+ a N_{Ed})/(f_y.$W_{pl.y}$)	3.883/(.001x 275x257.7) ≥ 0	0.055	
R_5=($M_{y.Ed5}$+ a N_{Ed})/(f_y.$W_{pl.y}$)	0.633/(.001x 275x257.7) ≥ 0	0.009	
R_S-R_E	0.110 - 0.009 ≥ 0	0.101	
C_n=Fn(All above)	0.009, 0.110, 0.109, 0.055, 0.009, 0.101	1.147	BB.14

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(N_{crT}, i_s, a)	990.2, 0.152, 0.122	94.042 kN.m	
$M_{cr0,Lim}$ = fn(s, E, I_z, I_T, I_w)	0.000, 210, 308.5, 5.964, 0.02933	Infinity kN.m	
M_{cr0} = Min(M_{cr0}, $M_{cr0,Lim}$)	Min(0.094, Infinity)	94.042 kN.m	
c=fn(h_{max}/h_{min}, L_h/L_y, h/t_f)	1, 1, 26.051	1	BB.17
M_{cr} = c^2 • C_n• M_{cr0}	1.000^2 • 1.147 • 94.0	107.883 kN.m	
λ_{LT}=√ W.f_y/M_{cr}	√ 257.7 x 275 / 416.998	0.810	
γ_{LT}= Fn(λ_{LT},ϕ_{LT},β)	0.810, 0.816, 0.750	0.811	Curve b
γ_{LT}.mod = Fn(γ_{LT},λ_{LT},k_c,f)	0.811, 0.810, 0.942, 0.971	0.836	6.3.2.3
$M_{b.Rd}$ =χ $W_{pl.y}$.f_y	0.836 x 257.7 x 275	59.216 kN.m	

Combined Axial and Bending 6.62

k_{zy}	1- {0.1/(C_{mLT}-0.25)}• N_{Ed}/$N_{b,z,Rd}$	1.001	
N_{Ed}/$N_{b,z,Rd}$+k_{zy}.$M_{y,Ed}$/$M_{b,Rd}$	5.188 / 557.268 + 1.001 x 9.933 / 59.216 =	0.177	OK

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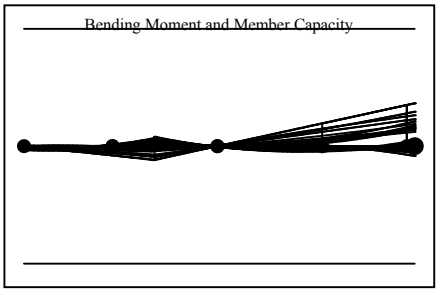
Axial with Moments (Member)
Rafter 2 of Bay 1 : Members 4-6 (A\4-2)
Between 0.127 and 1.500 m, in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
D1 UDLY -000.200              [ kN/m ]
D2 UDLY -000.000              [ kN/m ]
W2 PTRN +000.525 5.152 6.335 +000.525
W2 PTRN +000.216 0.000 5.152 +000.216
```



Member Forces in Load Case 22 and Maximum Deflection from Load Case 81

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	42	2.32C	-0.01	8.25	-0.08	-25.98	0.15	8.13	-0.17	1.88
	69	4.45T	-0.01	-2.50	0.02	0.76	-0.03	@ 4.224	@ 4.206	@ 1.631

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, 2.32, 25.98, 0.17	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	8.237 / 203.402 =	0.04	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.078 / 329.914 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{p'.z}/\gamma_{M0}$	275 x 70.9/1	19.498 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	31.96x275/1 (No bearing / block tearing design)	878.9 kN	
$n = N_{Ed}/N_{pl.Rd}$	-4.454 / 878.9 =	0.005	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	257.7, 12.811, 0.005	257.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	257.7 x 275/1	70.868 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	70.9, 20.779, 0.005	70.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	70.9 x 275/1	19.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})^{\alpha}+(M_{z.Ed}/M_{N.z.Rd})^{\beta}$	$(24.928/70.868)^2+(0.144/19.498)^1$=	0.131	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x6.335 =	6.335	
$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{31.96x275/1209.03}$	0.852	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.764x275/10/1 =	671.812 kN	Curve a
Lez = Kz.Lz	1x1.373 =	1.373	
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{31.96x275/3391.82}$	0.509	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.88x275/10/1 =	773.799 kN	Curve b
Let = Kt.Lx	1x6.335 =	6.335	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{31.96x275/763.36}$	1.073	
$N_{b.T.Rd}$= $Area.\chi.fy/\gamma_{M1}$	31.96x0.552x275/10/1 =	484.724 kN	Curve b

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2,ψ)	24.9, 13.7, 0.550	1.263	Not Loaded
C_{mLT}=Max(0.6+0.4ψ, 0.4)	M = 24.92,ψ = 0.550	0.82	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0.14,ψ = 0.257	0.703	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h= 25.98, M_s= 0.27,ψ = -0.029,α_s= 0.010	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 1.373 =	1.373 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.263, 1.373, 308.5, 5.964, 0.02933, 210000	447.914 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{257.7 x 275 / 447.914}$	0.398	
χ_{LT}= Fn(λ_{LT},λ_{LT5950})	0.398, 0.431	1.000	Curve d
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	1.000, 0.398, 0.970, 0.989	1.000	6.3.2.3

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$M_{b.Rd} = \chi W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 257.7 x 275 ≤ 70.868 =	70.868 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\gamma_y.N_{Rk}/\gamma_{M1})$	2.322 / 671.812	0.003	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	2.322 / 773.799	0.003	OK
$U_{M.y} = M_{y.Ed}/(\gamma_{LT}.M_{..Rk}/\gamma_{M1})$	24.928 / 70.868	0.352	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.144 / 19.498	0.007	OK
$k_{yy}=C_{my}\{1+(\lambda_v-0.2)U_{N.y}\}$		0.401	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.704	
$k_{yz}=0.6\ k_{zz}$		0.422	
$k_{zy}=0.6\ k_{yy}$		0.241	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.003+0.401x0.352+0.422x0.007	0.148	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.003+0.241x0.352+0.704x0.007	0.093	OK

Deflection Check - Load Case 81

In-span $\delta \leq$ Span/360	1.88 ≤ 6335 / 360	1.88 mm	OK

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Appendix-BB Stability (Member) : Rafter 2 of Bay 1 : Members 4-6 (A\4-2) Between 0.127 and 3.201 m, in Load Case 22

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 W2

```
UT Spacing 02.725 [Multiply AllLoads]
D1 UDLY -000.200              [ kN/m ]
D2 UDLY -000.000              [ kN/m ]
W2 PTRN +000.525 5.152 6.335 +000.525
W2 PTRN +000.216 0.000 5.152 +000.216
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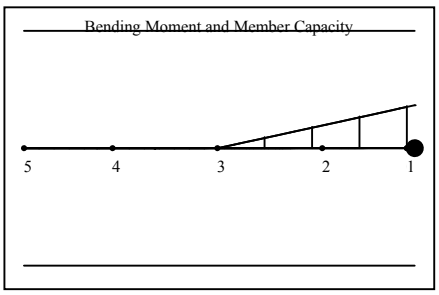

Lateral and Torsional Restraints

Purlins @ 1.5, 3.2, 4.9 and 6.335 m
Stay @ 6.335 m

Member Forces in Load Case 22

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	42	2.32C	-0.01	8.25	-0.08	-25.98	0.15	8.13	-0.17	0.00
	69	4.45T	-0.01	-2.50	0.02	0.76	-0.03	@ 4.224	@ 4.206	@ 1.631

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, -4.45, 25.98, 0.17	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Tension Side Lateral Restraint Spacing Check, Lm

L_m= fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	4.5, 32.0,1.26, 257.7, 6.0	1.824m	
L_m> s	1.824>1.700 - Effect of tension side lateral restraints considered		

Compression Resistance N.b.Rd

i_s= Fn(i_y, i_z, a)	85.6, 31.1, 121.9	152.2 mm	
N_{crT}= Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 309, 6, 0.029, 3074, 122, 152	920.1 kN	
$\lambda T = \sqrt{A.fy/NcrT}$	√31.96x275/ 10 /920.05	0.977	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	31.96x0.611x275/10/1 =	537.391 kN	Curve b

Modification Factor for Linear Moment Gradients Cm BB.3.3.1

N_{crE}= Fn(E, I_z, L_t)	210, 309, 3074	676.7 kN	
$\eta = N_{crE}/ N_{crT}$	676.7 / 920.1	0.735	
βt=Fn(M1,M2)	0.0, 0.0	0.000	
C_m=Fn(η,βt,B_0,B_1,B_2)	0.735, 0.000, 0.532, 0.366, 0.104	1.880	BB.13

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(N_{crT}, i_s, a)	920.1, 0.152, 0.122	87.383 kN.m	
$M_{cr0,Lim}$ = fn(s, E, I_z, I_T, I_w)	1.700, 210, 308.5, 5.964, 0.02933	239.142 kN.m	
M_{cr0} = Min(M_{cr0}, $M_{cr0,Lim}$)	Min(0.087, 0.239)	87.383 kN.m	
c=fn(h_{max}/h_{min}, L_h/L_y, h/t_f)	1, 1, 26.051	1	BB.17
M_{cr} = c^2 • C_m• M_{cr0}	1.000^2 • 1.880 • 87.4	164.307 kN.m	
$\lambda_{LT}=\sqrt{W.f_y/M_{cr}}$	√ 257.7 x 275 / 447.914	0.657	
γ_{LT}= Fn(λ_{LT},ϕ_{LT},β)	0.657, 0.705, 0.750	0.891	Curve b
γ_{LT}.mod = Fn(γ_{LT},λ_{LT},k_c,f)	0.891, 0.657, 0.756, 0.883	1.000	6.3.2.3
$M_{b.Rd} = \chi W_{pl.y}.f_y$	1.000 x 257.7 x 275	70.868 kN.m	

Combined Axial and Bending 6.62

k_{zy}	1- {0.1/(C_{mLT}-0.25)}• $N_{Ed}/N_{b,z,Rd}$	0.999	
$N_{Ed}/N_{b,z,Rd}+k_{zy}.M_{y,Ed}/M_{b,Rd}$	4.454 / 537.391 + 0.999 x 24.928 / 70.868 =	0.360	OK

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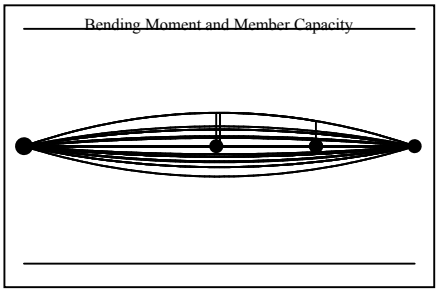
Axial with Moments (Member)
Gable Column : Member 65 (A\2)
Between 2.700 and 4.100 m, in Load Case 33

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 S3

S3 UDLZ +002.374 (kN/m)



Member Forces in Load Case 33 and Maximum Deflection from Load Case 81

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
65	2	3.36C	0.00	-9.77	0.00	0.00	0.00	-13.40 @ 2.744	0.00 @ 0.000	6.14 @ 2.744
	61	2.08C	0.00	9.77	0.00	0.00	0.00			

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.41, 30.58, 275, 3.36, 13.4, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.037 / 156.396 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{-l.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	24.26 x 275/1 =	667.15 kN	
$n = N_{Ed}/N_{pl.Rd}$	2.081 / 667.15 =	0.003	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	171.3, 9.85, 0.003	171.3 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	171.3 x 275/1	47.108 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})^{\alpha}+(M_{z.Ed}/M_{N.z.Rd})^{\beta}$	$(13.398/47.108)^2+(0)^1$=	0.081	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x5.487 =	5.487	
$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{}$24.26x275/934.04	0.845	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	24.26x0.769x275/10/1 =	513.039 kN	Curve a
Lez = Kz.Lz	1x1.4 =	1.4	
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{}$24.26x275/1455.06	0.678	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	24.26x0.796x275/10/1 =	531.199 kN	Curve b
Let = Kt.Lx	1x5.487 =	5.487	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{}$24.26x275/687.89	0.985	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	24.26x0.607x275/10/1 =	404.755 kN	Curve b

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, $M_{2,\psi}$)	-13.4, -10.1, 0.756	1.128	Not Loaded
C_{mLT}=Max(0.6+0.4$_{\psi}$, 0.4)	M = -13.4,$_{\psi}$ = 0.756	0.902	Table B.3
C_{mz}=Max(0.6+0.4$_{\psi}$, 0.4)	M = 0,$_{\psi}$ = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4$_{\psi}$, 0.4)	M = 0,$_{\psi}$ = 1.000	1	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 1.4 =	1.4 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.128, 1.400, 137.6, 4.408, 0.009848, 210000	160.795 kN.m	
$\lambda_{LT}=\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 171.3 x 275 / 160.795	0.541	
γ_{LT}= Fn(λ_{LT},λ_{LT5950})	0.541, 0.560	0.943	Curve b
γ_{LT}.mod = Fn($\gamma_{I.T}$,λ_{LT},k_c,f)	0.943, 0.541, 0.942, 0.975	0.967	6.3.2.3
$M_{b.Rd} = \chi$ $W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.967 x 171.3 x 275≤ 47.108 =	45.567 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	3.362 / 513.039	0.007	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	3.362 / 531.199	0.006	OK

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$U_{M.y} = M_{y.Ed}/(\gamma_{LT}.M_{..Rk}/\gamma_{M1})$	13.398 / 45.567	0.294	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 11.44	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_v-0.2)U_{N.y}\}$		1.004	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.005	
$k_{yz}=0.6\ k_{zz}$		0.603	
$k_{zy}= 1- \{0.1/(C_{mLT}-0.25)\}U_{N.z}$		0.999	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.007+1.004x0.294+0.603x0.000	0.302	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.006+0.999x0.294+1.005x0.000	0.300	OK

Deflection Check - Load Case 81

In-span$\delta \leq$ Span/360	6.14$\leq$ 5487 / 360	6.14 mm	OK

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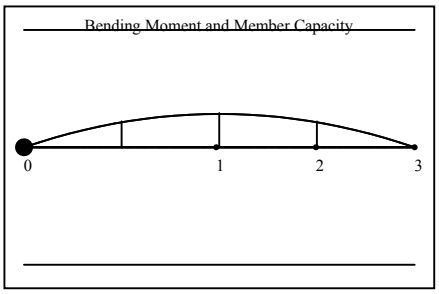
Appendix-BB Stability (Member) : Gable Column : Member 65 (A\2) in Load Case 33

Member Loading and Member Forces

Loading Combination : 1 UT + 1.25 D1 + 1.25 D2 + 1.5 S3
S3 UDLZ +002.374 (kN/m)



Lateral and Torsional Restraints

Side rails @ 2.7, 4.1 and 5.487 m
No stays are provided

Member Forces in Load Case 33

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
65	2	3.36C	0.00	-9.77	0.00	0.00	0.00	-13.40 @ 2.744	0.00 @ 0.000	0.00 @ 2.744
	61	2.08C	0.00	9.77	0.00	0.00	0.00			

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.41, 30.58, 275, 3.36, 13.4, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Tension Side Lateral Restraint Spacing Check, Lm

L_m= fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	3.4, 24.3,1.13, 171.3, 4.4	1.403m	
L_m< s	1.403<2.700 - Effect of tension side lateral restraints ignored		

Compression Resistance N.b.Rd

λ_z =√A.fy/Ncrz	√24.26x275/94.73	2.656	
$N_{b.z.Rd}$ = Area.χ.f_y/γ_{M1}	24.26x0.125x275/10/1 =	83.328 kN	Curve b
i_s= Fn(i_y, i_z, a)	74.8, 23.8, 106.7	78.5 mm	
N_{crT}= Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 138, 4, 0.010, 5487, 0, 78	687.9 kN	
λ_T =√A.fy/NcrT	√24.26x275/ 10 /687.9	0.985	
$N_{b.T.Rd}$= Area.χ.fy/γ_{M1}	24.26x0.607x275/10/1 =	404.757 kN	Curve b

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(L_t, E, I_z, I_T, I_w)	5.487, 210, 137.6, 4.408, 0.009848	20.037 kN.m	
M_{cr} = C_1• M_{cr0}	1.127 • 20.0	22.582 kN.m	
λ_{LT}=√ W.f_y/M_{cr}	√ 171.3 x 275 / 160.795	1.444	
γ_{LT}= Fn(λ_{LT},ϕ_{LT},β)	1.444, 1.460, 0.750	0.452	Curve b
γ_{LT}.mod = Fn(γ_{LT},λ_{LT},k_c,f)	0.452, 1.444, 0.942, 0.995	0.454	6.3.2.3
$M_{b.Rd}$ =χ $W_{pl.y}$.f_y	0.454 x 171.3 x 275	21.397 kN.m	

Combined Axial and Bending 6.62

k_{zy}	1- {0.1/(C_{mLT}-0.25)}• N_{Ed}/$N_{b,z,Rd}$	0.999	
N_{Ed}/$N_{b,z,Rd}$+k_{zy}.$M_{y,Ed}$/$M_{b,Rd}$	3.362 / 83.328 + 0.999 x 13.398 / 21.397 =	0.666	OK

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Strut and Tie (Member)
Roof Bracing : Member 96 (3\B-A)

Classification and Effective Area (EN 1993: 2006)

Section (8.77 kg/m)	114.3x3.2 CHS 8.77 [Grade 43]		
Class = Fn(D/t,f_y)	35.72, 275	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 5-40		

Strut by One Row of Bolts or Equivalent Weld (Case 33)

λ = Fn(Lx,Ly,rx,ry)	5.845,5.845,3.93,3.93	148.72	OK
$N_{b.Rd}$ = Area.f_c	11.17x81.23/10 =	90.731 kN	Curve a
F (Strut)/$N_{b.Rd}$	10.528 / 90.731	0.116	OK

Axially Loaded Member in Tension : 6.2.3 (Case 7)

Tc = A_g.fy/γ_{M0}	11.17x275/1 (No bearing / block tearing design)	307.175 kN	
F (Tie)/Tc	10.561 / 307.175	0.034	OK

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Elastic Critical Load Factor

Load Case 001 : 1.25 (Dead+Services) + 1.5 Live/Snow
α_{cr} From Buckling Analysis 14.993

Load Case 005 : 1.25 (Dead+Services) + 1.5 Live/Snow + Notional -->
α_{cr} From Buckling Analysis 14.993

Load Case 006 : 1 Dead + 1.5 Side Wind W1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 007 : 1 Dead + 1.5 Side Wind P1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 008 : 1 Dead + 1.5 Side Wind S1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 009 : 1.25(Dead + Serv) + 1.5Side Wind W1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 010 : 1.35(Dead + Serv) + 0.75Side Wind W1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 011 : 1.25(Dead + Serv) + 1.5Side Wind P1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 012 : 1.35(Dead + Serv) + 0.75Side Wind P1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 013 : 1.25(Dead + Serv) + 1.5Side Wind S1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 014 : 1.35(Dead + Serv) + 0.75Side Wind S1 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 015 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 016 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 017 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind P1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 018 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind P1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 019 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind S1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 020 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind S1 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 021 : 1 Dead + 1.5 Side Wind W2 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 022 : 1.25(Dead + Serv) + 1.5Side Wind W2 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 023 : 1.35(Dead + Serv) + 0.75Side Wind W2 (Local Wind)
α_{cr} From Buckling Analysis 29.993

Load Case 024 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W2 (Local Wind)
α_{cr} From Buckling Analysis 14.993

Load Case 025 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W2 (Local Wind)

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α_{cr}	From Buckling Analysis	14.993
Load Case 026 : 1 Dead + 1.5 Side Wind W3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 027 : 1 Dead + 1.5 Side Wind P3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 028 : 1 Dead + 1.5 Side Wind S3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 029 : 1.25(Dead + Serv) + 1.5 Side Wind W3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 030 : 1.35(Dead + Serv) + 0.75Side Wind W3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 031 : 1.25(Dead + Serv) + 1.5 Side Wind P3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 032 : 1.35(Dead + Serv) + 0.75Side Wind P3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 033 : 1.25(Dead + Serv) + 1.5 Side Wind S3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 034 : 1.35(Dead + Serv) + 0.75Side Wind S3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 035 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind W3 (Local Wind)		
α_{cr}	From Buckling Analysis	14.993
Load Case 036 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind W3 (Local Wind)		
α_{cr}	From Buckling Analysis	14.993
Load Case 037 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind P3 (Local Wind)		
α_{cr}	From Buckling Analysis	14.993
Load Case 038 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind P3 (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 039 : 1.25(Dead + Serv) + 1.5 Live/Snow + 0.75 Side Wind S3 (Local Wind)		
α_{cr}	From Buckling Analysis	14.993
Load Case 040 : 1.35(Dead + Serv) + 1.05 Live/Snow + 0.75 Side Wind S3 (Local Wind)		
α_{cr}	From Buckling Analysis	14.993
Load Case 077 : D1 Dead Load		
α_{cr}	From Buckling Analysis	29.993
Load Case 078 : D1 Dead Load plus D2 Services		
α_{cr}	From Buckling Analysis	29.993
Load Case 079 : L0 Uniform Snow/live load plus L1 Live Load		
α_{cr}	From Buckling Analysis	29.993
Load Case 080 : W1 Wind on Side (Top Values) --> (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 081 : W2 Wind on Side (Bottom Values) --> (Local Wind)		
α_{cr}	From Buckling Analysis	29.993
Load Case 082 : W3 Wind on Gable (Local Wind)		
α_{cr}	From Buckling Analysis	29.993

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Sway Stability Summary

Load Case	α_{cr}	Sway Amp†	Plastic LF	Method	From Case	P-Δ ON	Verdict
001 : 1.25 (Dead+Services) + 1.5 L	14.99	1.000	N.A	Buckling Anal.	001	No	OK
005 : 1.25 (Dead+Services) + 1.5 L	14.99	1.000	N.A	Buckling Anal.	005	No	OK
006 : 1 Dead + 1.5 Side Wind W1 (L	29.99	1.000	N.A	Buckling Anal.	006	No	OK
007 : 1 Dead + 1.5 Side Wind P1 (L	29.99	1.000	N.A	Buckling Anal.	007	No	OK
008 : 1 Dead + 1.5 Side Wind S1 (L	29.99	1.000	N.A	Buckling Anal.	008	No	OK
009 : 1.25(Dead + Serv) + 1.5Side	29.99	1.000	N.A	Buckling Anal.	009	No	OK
010 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	010	No	OK
011 : 1.25(Dead + Serv) + 1.5Side	29.99	1.000	N.A	Buckling Anal.	011	No	OK
012 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	012	No	OK
013 : 1.25(Dead + Serv) + 1.5Side	29.99	1.000	N.A	Buckling Anal.	013	No	OK
014 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	014	No	OK
015 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	015	No	OK
016 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	016	No	OK
017 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	017	No	OK
018 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	018	No	OK
019 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	019	No	OK
020 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	020	No	OK
021 : 1 Dead + 1.5 Side Wind W2 (L	29.99	1.000	N.A	Buckling Anal.	021	No	OK
022 : 1.25(Dead + Serv) + 1.5Side	29.99	1.000	N.A	Buckling Anal.	022	No	OK
023 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	023	No	OK
024 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	024	No	OK
025 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	025	No	OK
026 : 1 Dead + 1.5 Side Wind W3 (L	29.99	1.000	N.A	Buckling Anal.	026	No	OK
027 : 1 Dead + 1.5 Side Wind P3 (L	29.99	1.000	N.A	Buckling Anal.	027	No	OK
028 : 1 Dead + 1.5 Side Wind S3 (L	29.99	1.000	N.A	Buckling Anal.	028	No	OK
029 : 1.25(Dead + Serv) + 1.5 Side	29.99	1.000	N.A	Buckling Anal.	029	No	OK
030 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	030	No	OK
031 : 1.25(Dead + Serv) + 1.5 Side	29.99	1.000	N.A	Buckling Anal.	031	No	OK
032 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	032	No	OK
033 : 1.25(Dead + Serv) + 1.5 Side	29.99	1.000	N.A	Buckling Anal.	033	No	OK
034 : 1.35(Dead + Serv) + 0.75Side	29.99	1.000	N.A	Buckling Anal.	034	No	OK
035 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	035	No	OK
036 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	036	No	OK
037 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	037	No	OK
038 : 1.35(Dead + Serv) + 1.05 Liv	29.99	1.000	N.A	Buckling Anal.	038	No	OK
039 : 1.25(Dead + Serv) + 1.5 Live	14.99	1.000	N.A	Buckling Anal.	039	No	OK
040 : 1.35(Dead + Serv) + 1.05 Liv	14.99	1.000	N.A	Buckling Anal.	040	No	OK

Second-Order Elastic Analysis has been used in the following cases

None

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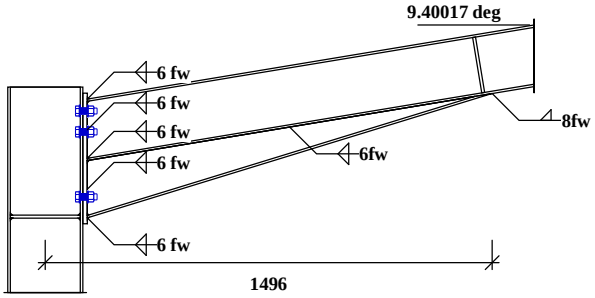

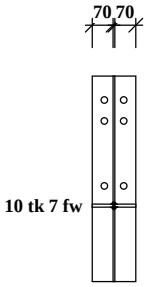

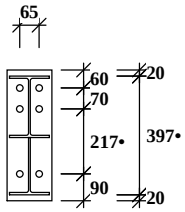

Plates S 275
Beam 203x133 UB 25 [S275]
Haunch 203x133 UB 25 [S275]

Column 254x146 UB 31 [S275]
Top 40 above top flange

End-Plate 437 x 150 x 15 mm (8 kg)
6 No. M20 Grade 8.8 Bolts in 22 mm holes
Haunch Stiff 10 mm with 7 FW

Eaves joint at : B1 - Level 1 : Rafter 1 of Bay 1 : Members 11-12 (B\1-2)
Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)
Loading Case 001 : 1.25 (Dead+Services) + 1.5 Live/Snow
Basic Data

Integrated Applied Forces at Column/Right Rafter Interface

Right Rafter Forces M, Fvr, Fr	76.5 kNm, 39.9 kN, 24.0 kN
Resultant Forces M, Fv, F	76.5 kNm, 43.3 kN, 17.1 kN
Load directions	Top of Joint in Tension, Rafter moving Down and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Joints SCI Green Book P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

Column-254x146UB31 [28]	D=251.4, B=146.1, T=8.6, t=6.0, r=7.6, py=275		
Rafter-203x133UB25 [28]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Haunch-203x133UB25 [28]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	170.9 kN.m, 379.0 kN, 1183.1 kN	Mc = 170.9 kN.m	OK

Summary of Results (Unity Ratios)

Moment Capacity 78.6 kNm (for 2 rows of bolts) (Modified Applied Mom. M_{mod}=71.5 kNm)		0.91	OK
Shear Capacity		0.15	OK
Flange Welds	0.74	0.74	OK
Web Welds	0.68, 0.11	0.68	OK
Haunch Welds	0.38, 0.07	0.38	OK
Column Compression stiff Web Weld	0.24	0.24	OK
End of Haunch Compression Zone	0.02, 0.02	0.02	OK

Step 1: Tension Zone

Basics

Bm/Plt b_p, W, t_{wb}, S_w, m_p, e_p, n_p	150.0, 65, 5.7, 6, 24.9, 42.5, 31.1		
Column B_c, t_{wc} r_c, m_c, e_c, n_c	146.1, 6.0, 7.6, 23.4, 40.6, 29.3		
$F_{t,Rd}=k_2 \bullet f_{ub} \bullet A / \gamma_{m^2}$	0.90•800•245/1.25	141.1 kN	
$F_{T,3,Rd}=\sum F_{t,Rd}$	2•141.1	282.2 kN	

BOLT ROW 1

Column Flange row 1 only

m, e, e_x	23.4, 40.6, 80.0		
$l_{eff,cp}$=min(Circle, Indiv End)	min(147.2, 233.6)	147.2 mm	T2.2 (b)
$l_{eff,ncp}$=min(Corner2 (b), Side (e))	min(152.2, 144.4)	144.4 mm	T2.2 (e)

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Mode 1 $l_{eff,1}=min(l_{eff,cp}, l_{eff,cnp})$	min(147.2, 144.4)	144.4 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	144.4	144.4 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	144.4•8.6•8.6•275.0/4/1.0	734.1 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	144.4•8.6•8.6•275.0/4/1.0	734.1 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•23.42•29.28-9.25•(23.42+29.28)	883.8	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /883.8$	(8•29.28-2•9.25)•734.1/883.8	179.2 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•734.1 + 29.28•2•141.1)/(23.42 + 29.28)	184.7 kN	
$F_{T,Rd} =min(F_{T,Rd}$ mode1,2,3)	min(179.2, 184.7, 282.2)	179.2 kN	

Column Web Tension row 1 only

$\omega=fn(l_{eff}\ t_w, A_{vc}, \beta)$	fn(144.4, 6.0, 1636, 1.00)	0.86	
$F_{t,wc,Rd} =\omega \cdot l_{eff,1} \cdot t_{wc} \cdot f_{y,wc}/\gamma_{M0}$	0.86•144.4•6.0•275/1.00	203.9 kN	

End Plate row 1 only

m, e, mx, α	24.9, 42.5, 27.3, 6.7		
l_{eff} modes	$l_{eff,cp}$ = 156.1, $l_{eff,ncp}$ = 165.5		T2.2 (c)
Mode 1 $l_{eff,1}=min(l_{eff,cp}, l_{eff,cnp})$	min(156.1, 165.5)	156.1 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	165.5	165.5 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	156.1•15.0•15.0•275.0/4/1.0	2415.2 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	165.5•15.0•15.0•275.0/4/1.0	2560.1 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•24.85•31.06-8.25•(24.85+31.06)	1082.5	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /1082.5$	(8•31.06-2•8.25)•2415.2/1082.5	517.6 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•2560.1 + 31.06•2•141.1)/(24.85 + 31.06)	248.4 kN	
$F_{T,Rd} =min(F_{T,Rd}$ mode1,2,3)	min(517.6, 248.4, 282.2)	248.4 kN	

Beam Web Tension row 1 only

$F_{t,wb,Rd} =l_{eff,1} \cdot t_{wb} \cdot f_{y,wb}/\gamma_{M0}$	156.1•5.7•275/1.00	244.7 kN	
Potential resistance of Bolt Row 1	$F_{t1,Rd}$	179.2 kN	Mode 1

BOLT ROW 2

Column Flange row 2 only

m, e	23.4, 40.6		
l_{eff} modes	$l_{eff,cp}$ = 147.2, $l_{eff,ncp}$ = 144.4		T2.2 (e)
Mode 1 $l_{eff,1}=min(l_{eff,cp}, l_{eff,cnp})$	min(147.2, 144.4)	144.4 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	144.4	144.4 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	144.4•8.6•8.6•275.0/4/1.0	734.1 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	144.4•8.6•8.6•275.0/4/1.0	734.1 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•23.42•29.28-9.25•(23.42+29.28)	883.8	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /883.8$	(8•29.28-2•9.25)•734.1/883.8	179.2 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•734.1 + 29.28•2•141.1)/(23.42 + 29.28)	184.7 kN	
$F_{T,Rd} =min(F_{T,Rd}$ mode1,2,3)	min(179.2, 184.7, 282.2)	179.2 kN	

Column Web Tension row 2 only

$\omega=fn(l_{eff}\ t_w, A_{vc}, \beta)$	fn(144.4, 6.0, 1636, 1.00)	0.86	
$F_{t,wc,Rd} =\omega \cdot l_{eff,1} \cdot t_{wc} \cdot f_{y,wc}/\gamma_{M0}$	0.86•144.4•6.0•275/1.00	203.9 kN	

End Plate row 2 only

m, e	24.9, 42.5		
l_{eff} modes	$l_{eff,cp}$ = 156.1, $l_{eff,ncp}$ = 152.5		T2.2 (e)
Mode 1 $l_{eff,1}=min(l_{eff,cp}, l_{eff,cnp})$	min(156.1, 152.5)	152.5 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	152.5	152.5 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	152.5•15.0•15.0•275.0/4/1.0	2359.4 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	152.5•15.0•15.0•275.0/4/1.0	2359.4 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•24.85•31.06-8.25•(24.85+31.06)	1082.5	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /1082.5$	(8•31.06-2•8.25)•2359.4/1082.5	505.6 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•2359.4 + 31.06•2•141.1)/(24.85 + 31.06)	241.2 kN	
$F_{T,Rd} =min(F_{T,Rd}$ mode1,2,3)	min(505.6, 241.2, 282.2)	241.2 kN	

Beam Web Tension row 2 only

$F_{t,wb,Rd} =l_{eff,1} \cdot t_{wb} \cdot f_{y,wb}/\gamma_{M0}$	152.5•5.7•275/1.00	239.1 kN	

Column Flange rows 1 to 2 combined

l_{eff},cp(Row 1)=PI•m	PI•23.42	73.6 mm	
l_{eff},nc(Row 1)=2m+0.625e	2•23.42+0.625•40.55	72.2 mm	
l_{eff},cp(Row 2)=PI•m	PI•23.42	73.6 mm	
l_{eff},nc(Row 2)=2m+0.625e	2•23.42+0.625•40.55	72.2 mm	
$l_{eff,cp}=l_{eff,cp(R1)}+2P+l_{eff,cp(R2)}$	73.6 + 2•70.0 + 73.6	287.2 mm	
$l_{eff,ncp}=l_{eff,ncp(R1)}+P+l_{eff,ncp(R2)}$	72.2 + 70.0 + 72.2	214.4 mm	
$l_{eff,1}=min(l_{eff,cp}, l_{eff,ncp})$	Min(287.2, 214.4)	214.4 mm	
$l_{eff,2}=l_{eff,ncp}$	214.4	214.4 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	214.4•8.6•8.6•275.0/4/1.0	1090.0 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	214.4•8.6•8.6•275.0/4/1.0	1090.0 kN.mm	

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$2•m•n-e_w•(m+n)$	2•23.42•29.28-9.25•(23.42+29.28)	883.8	
$F_{T,1,Rd}=(8•n-2•e_w)•M_{pl,1}$ /883.8	(8•29.28-2•9.25)•1090.0/883.8	266.0 kN	
$F_{T,2,Rd}=(2•M_{pl,2}+n•Nb•F_{t,Rd})/(m+n)$	(2•1090.0 + 29.28•4•141.1)/(23.42 + 29.28)	355.0 kN	
$F_{T,Rd}$ =min($F_{T,Rd}$ mode1,2,3)	min(266.0, 355.0, 564.5)	266.0 kN	
$F_{t2,Rd}=F_{T,Rd}-F_{t,Rd}1,1$	266.0 - 179.2	86.9 kN	

Column Web Tension rows 1 to 2 combined

ω=fn(l_{eff} t_w, A_{vc},β)	fn(214.4, 6.0, 1636, 1.00)	0.74	
$F_{t,wc,Rd}=\omega•l_{eff,1}•t_{wc}•f_{y,wc}/\gamma_{M0}$	0.74•214.4•6.0•275/1.00	263.4 kN	
$F_{t2,Rd}=F_{T,Rd}-F_{t,Rd}1,1$	263.4 - 179.2	84.3 kN	

End-Plate rows 1 to 2 combined

$l_{eff,cp}$(Row 1)=PI•m	PI•24.85	78.1 mm	
$l_{eff,nc}$(Row 1)=α•m-(2m+0.625e)	6.66•24.85-(2•24.85+0.625•42.50)	89.2 mm	
$l_{eff,cp}$(Row 2)=PI•m	PI•24.85	78.1 mm	
$l_{eff,nc}$(Row 2)=2m+0.625e	2•24.85+0.625•42.50	76.3 mm	
$l_{eff,cp}=l_{eff,cp(R1)}+2P+l_{eff,cp(R2)}$	78.1 + 2•70.0 + 78.1	296.1 mm	
$l_{eff,ncp}=l_{eff,ncp(R1)}+P+l_{eff,ncp(R2)}$	89.2 + 70.0 + 76.3	235.5 mm	
$l_{eff,1}$=min($l_{eff,cp}$, $l_{eff,ncp}$)	Min(296.1, 235.5)	235.5 mm	
$l_{eff,2}=l_{eff,ncp}$	235.5	235.5 mm	
$M_{pl,1}=l_{eff,1}•t_f^2•p_y/4/\gamma_{M0}$	235.5•15.0•15.0•275.0/4/1.0	3642.9 kN.mm	
$M_{pl,2}=l_{eff,2}•t_f^2•p_y/4/\gamma_{M0}$	235.5•15.0•15.0•275.0/4/1.0	3642.9 kN.mm	
$2•m•n-e_w•(m+n)$	2•24.85•31.06-8.25•(24.85+31.06)	1082.5	
$F_{T,1,Rd}=(8•n-2•e_w)•M_{pl,1}$ /1082.5	(8•31.06-2•8.25)•3642.9/1082.5	780.7 kN	
$F_{T,2,Rd}=(2•M_{pl,2}+n•Nb•F_{t,Rd})/(m+n)$	(2•3642.9 + 31.06•4•141.1)/(24.85 + 31.06)	443.9 kN	
$F_{T,Rd}$ =min($F_{T,Rd}$ mode1,2,3)	min(780.7, 443.9, 564.5)	443.9 kN	
$F_{t2,Rd}=F_{T,Rd}-F_{t,Rd}1,1$	443.9 - 179.2	264.8 kN	

Beam Web Tension rows 1 to 2 combined

$F_{t,wb,Rd}=l_{eff,1}•t_{wb}•f_{y,wb}/\gamma_{M0}$	235.5•5.7•275/1.00	369.2 kN	
$F_{t2,Rd}=F_{T,Rd}-F_{t,Rd}1,1$	369.2 - 179.2	190.0 kN	
Potential resistance of Bolt Row 2	$F_{t2,Rd}$	84.3 kN	Col Web

Step 1C Plastic distribution Limit

T_p< d/1.9•√(f_{ub}/f_{yp})	15.0 < 20/1.9•√(800.0/275)	15.0 <= 18.0	Plastic
T_{fc}< d/1.9•√($f_{ub}/f_{y,fc}$))	8.6 < 20/1.9•√(800.0/275)	8.6 <= 18.0	Plastic
$F_{t\ 1,Rd}$ < 1.9 $F_{t,rd}$	179.2 < 1.9•141.1		Plastic
Classification	Plastic Deformation occurs.	Use Plastic distribution	

Potential Tension Capacity

Sigma $F_{ti,Rd}$	179.2 + 84.3 kN	263.4 kN	

Step 2 & 6B: Compression Zone

Web Bearing

n =min(5,2+0.6•B_e/(t_{fc}+s))	min(5, 2 + 0.6•230.0/16.2)	5.000	
$h_{\sim ff,c}=t_{fb}+2s_f+n(t_{fc}+s)+s_p$)	8.20+2•6.0+5.000(8.60+7.60)+29.00	130.2	
ω=fn(l_{eff}, t_w, A_{vc},β)	fn(130.2, 6.0, 1636, 1.00)	0.88	
$b_{sn}=b_{tfc}/2-t_{wc}$-snipe	146.1/2 - 6.0 - 10.0	57.0 mm	
$N_{c,Rd}$ =(2•b_{sn}•ts+$b_{eff,c}$•t_{wc})•$f_{y,wc}/\gamma_{M0}$	(2•57.0•10+130.2•6.0)•275/1.00	528.3 kN	

Web Buckling

$A_{s,eff}$ =2•b_{sg}•t_s+L_w•t_w	2•70.0•10.0 + 176.4•6.0	2458 mm²	
$I_s=t_s(2•b_{sg}+t_{wc})^3/12$	10.0(2•70.0 + 6.0)³/12	2593447 mm⁴	
i_s=√($I_S/A_{s,eff}$)	√(2593447/2458)	32.48 mm	
λ=(h_c-2•t_{fc})/i_s	(251.4 - 2•8.6)/32.48	0.08	
κ=min(1.0, 1/(ϕ+√(ϕ^2-λ^2)))	min(1.0, 1/(0.47 +√(0.47²-0.08²))	1.000	
$N_{b,Rd}=\kappa•A_{s,eff}•f_y/\gamma_{M1}$	1.00•2458.4•275/1.00	676.1 kN	

Beam Compression

Beam Compression Zone	Flange and Web in Compression Utilising 20% OverStressing		
Total Area Flange and Web	133.2•7.8 + 5.7•30.0	12.1 cm²	
$F_{c,fb,Rd}$	12.1•275•1.20	399.3 kN	

Step 3: Column Web Shear

$V_{wp,Rd}=0.9•f_{yc}•A_{vc}/(\gamma_{M0}•\sqrt{3})$	0.9•275•1636.4/(1.0•√3)	233.8 kN	

Potential Compression Capacity

$F_{c,Rd}$min	Min(528.3, 676.1, 399.3)	399.3 kN	OK

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Step 4: Moment Capacity

$F_{c,Rd}$ =Min($F_{c,wc,Rd}$, $F_{c,fc,Rd}$)	min(528.3, 399.3)	399.3 kN	
Shear limit F_{ri}=min($F_{t,Rd}$ Total, $V_{wp,Rd}$)	min(263.4, 233.8)	233.8 kN	
Fδ=$F_{t,Rd}$ -F_{ri}	263.4 - 233.8	29.6 kN	

Final Bolt Forces and Moment Capacities

Bolt row 2: M_{c2} =($F_{t2,Rd}$ -Fδ)• h_2	84.3 - 29.6 = 54.7•282.7	15.5 kN.m	
Bolt row 1 $M_{c,Rd}$1 =$F_{t1,Rd}$ •h_1	179.2•352.7	63.2 kN.m	
$M_{c,Rd}$		78.6 kN.m	
$M_{mod,Ed}$ =M-N_{Ed}•h_n	76.5 - 17.1•289.7	71.5 kN.m	OK
$F_{ri,Ed}$ for 2 rows	179.2, 54.7	233.8 kN	
$F_{ri,Ed}$ design=$F_{ri,Ed}$ •M_{Ed}/$M_{c,Rd}$'	233.8•71.5/78.6	212.7 kN	

Final Web Compression Zone Height

	Reducing Compression zone for applied moments.		
F red=$F_{c,beam}$-$F_{ri,Ed}$ design	399.3 - 212.7	186.6 kN	
h red=F red/t/p_y/1.2	186.6/5.7/275/1.2	99.2 mm	
h=max(0, h old-h red)	max(0,30.0 - 99.2)	0.0 mm	OK

Step 5: Shear Bolts

$F_{v,Rd}$=α_v•f_{ub}•A /γ_{M2}	0.6 • 800 • 245.0 / 1.25	94.1 kN	
Bearing $F_{b,Rd,End}$-End Plate, End	e1=60, e2=43, k_1=2.5,α_b=0.91, d=20, t=15, f_u=410	223.6 kN	
Bearing $F_{b,Rd}$-End Plate, Inner	p1=70, e2=43, k_1=2.5,α_b=0.81, d=20, t=15, f_u=410	199.4 kN	
Bearing $F_{b,Rd,End}$-Col Flange, End	e1=80, e2=41, k_1=2.5,α_b=1.00, d=20, t=8.6, f_u=410	141.0 kN	
Bearing $F_{b,Rd}$-Column Flange, Inner	p1=70, e2=41, k_1=2.5,α_b=0.81, d=20, t=8.6, f_u=410	114.3 kN	
$F_{v,Rd,Sh}$=Min(bearing, shear)	Min(199.4, 114.3, 94.1)	94.1 kN	
$F_{v,Rd,T}$=min($F_{b,Rd}$,0.28•shr)	Min(199.4, 114.3, 26.3)	26.3 kN	
$F_{v,Rd,TEnd}$=min($F_{b,Rd,End}$,0.28•shr)	Min(223.6, 141.0, 26.3)	26.3 kN	
Shear=Ns•$F_{v,Rd,Sh}$	2•94.1	188.2 kN	
Tension= (Nt-1)•$F_{v,Rd,T}$+ $F_{v,Rd,TEnd}$	2•26.3 + 2•26.3	105.4 kN	
$F_{v,Rd}$ Total=Shear+Tension	188.2 + 105.4	294 kN	OK

Steps 7&8: Welds

Beam $f_{vw,d}$=f_u/ (√3•β_w) /γ_{M2}	410.0 /√3 / 0.85 / 1.25	222.8 N/mm²	

Flange Tension Weld

$Ft_{,flng}$ =min(B•T•P_y, F_{r1}+F_{r2})	Min(133.2•7.8•275, 162.9 + 49.7)	212.7 kN	
$F_{vw,Rd}$ =2•K•0.7•t_s•L•$f_{vw,d}$	2•1.225•0.7•6•(133.2 - 2•0.7•6)•223	288.8 kN	OK

Flange Compression Weld

	Direct Bearing assumed. No check required		

Web Welds in Tension Zone

Web Weld OK if >= 0.71•T	6 >= 0.71•5.7	>= 4.1 mm	OK

Web Welds in Shear Zone

Lws=D-(T_t+T_b)-r_t-r_b-Lwt	397.2 - 16.1 - 7.6 - 7.6 - 151	215.2 mm	
F_{wCap}=2•0.7•ts•Lws•$f_{vw,d}$	2•0.7•6•215.2•223	406.8 kN	OK

Haunch Welds

F_h,Ed=min(M_h/(h_c-t_{fb}), b_b•t_{fb}•f_y)	Min(-28.1/(203.2 - 7.8), 133.2•7.8•275)	143.6 kN	
Haunch/Beam Flange area ratio,α	133.2•7.8/(133.2•7.8 + 133.2•7.8)	0.50	
F_{hnch}=F_h•min(0.5,α)	143.6•min(0.5, 0.50)	71.8 kN	
Fh_{comp}=(Ft+N)/Cos(θ)	212.7 + 17.1 / Cos(9.4)	232.9 kN	

Haunch End Weld

t=min(S_{hw}, t_f) •Cos((90-ThetaH1)/2)	min(8, 7.8)•Cos((90-7.6)/2)	5.9 mm	
K=√(3/(1+2•Cos((90-ThetaH1)/2)^2))	√(3/(1+2•Cos((90-7.6)/2)^2)	1.19	
End_{Cap}=K•t(B-2•w)•$f_{vw,d}$	1.186•5.9•(133.2 - 2•0.7•8)•223	189.1 kN	
End_{Cap}>= F_{hnch}	189.1 >= 71.8	71.8 kN	OK

Haunch Web Weld

Web force F_{wh}=Fh_{comp}-F_{hnch}	232.9 - 71.8	161.1 kN	
L_w =(H_l-D_c/2-T_{ep})/Cos(Theta)	(1496 - 251.4/2 - 15)/Cos(9.4)	1374.2 mm	
L_w=L_w -t_w-(T+t_{w1})/Sin(Theta1)	1374.2 - 6 -(7.8 + 6)/Sin(7.6)= 1374.2-6-103.7	1264.5 mm	
Web_{Cap}=2•0.7•t(L_w-2•t)•$f_{vw,d}$	2•0.7•6(1264.5 - 2•0.7•6)•223	2350.6 kN	
Web_{Cap}>= F_{wh}	2350.6 >= 161.1	161.1 kN	OK

Compression Stiffener Web Weld

Column $f_{vw,d}$=f_u/ (√3•β_w) /γ_{M2}	410.0 /√3 / 0.85 / 1.25	222.8 N/mm²	
F_{app}=F_t+N	212.7 + 17.1	229.8 kN	
F_{wCap}=2•2•0.7•t_s(D-2(T+r))•$f_{vw,d}$	2•2•0.7•7(251.4 - 2(8.6 + 7.6))•223	965.9 kN	OK

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Step 8: End of Haunch Compression Zone

Force Applied - Transverse to Beam Web

$F_{T,Ed} = F_{hnch} \bullet Tan(ThetaH1)$	71.8•Tan(7.6)	9.6 kN	

Web Bearing

$n = min(5, 2+0.6 \bullet B_e/(t_{fb}+s))$	min(5, 2 + 0.6•203.2/15.4)	5.000	
$h_{\sim ff,b} = t_{fb}+2s_f+n(t_{fb}+s)+s_p)$	7.80+2•0.0+5.000(7.80+7.60)+0.00	84.8	
$\omega = fn(l_{eff}, t_w, A_{vc}, \beta)$	fn(84.8, 6.0, 1281, 1.00)	0.92	
b_{sn}=Stiff L-snipe	60.0 - 10.0	50.0 mm	
$N_{b,Rd} = (2 \bullet b_{sn} \bullet ts + b_{eff,b} \bullet t_{wb}) \bullet f_{y,wb}/\gamma_{M0}$	(2•50.0•10+84.8•5.7)•275/1.00	407.9 kN	OK

Web Buckling

$A_{s,eff} = 2 \bullet b_{sg} \bullet t_s + L_w \bullet t_w$	2•60.0•10.0 + 168.1•5.7	2158 mm²	
$I_s = t_s(2 \bullet b_{sg}+t_{wb})^3/12$	10.0(2•60.0 + 5.7)³/12	1655101 mm⁴	
$i_s = \sqrt{(I_S/A_{s,eff})}$	√(1655101/2158)	27.69 mm	
$\lambda = (h_b - 2 \bullet t_{fb})/i_s$	(203.2 - 2•7.8)/27.69	0.08	
$\kappa = min(1.0, 1/(\phi + \sqrt{(\phi^2 - \lambda^2)}))$	min(1.0, 1/(0.47 +√(0.47²-0.08²))	1.000	
$N_{b,Rd} = \kappa \bullet A_{s,eff} \bullet f_y/\gamma_{M1}$	1.00•2158.0•275/1.00	593.5 kN	OK

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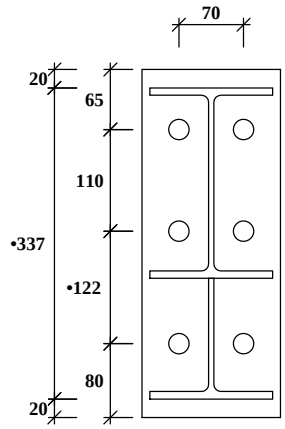

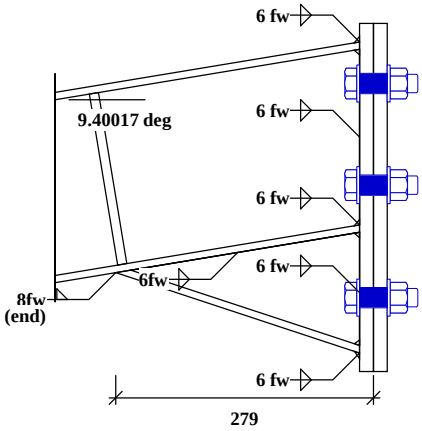

End-Plate 377 x 150 x 15 mm (7 kg)
6 No. M20 Grade 8.8 Bolts in 22 mm holes
Haunch Stiff 10 mm with 7 FW

Plates S 275
Beam 203x133 UB 25 [S275]
Haunch 203x133 UB 25 [S275

Apex joint at : N.70 - Level 1 : Rafter 1 of Bay 1 : Members 11-12 (B\1-2)
Beam to Beam End-Plated Connection to EC 3 (UK NAD)
Loading Case 001 : 1.25 (Dead+Services) + 1.5 Live/Snow
Basic Data

Integrated Applied Forces at End-plate Interface

Left Rafter Forces M, Fvr, Fr	-38.0 kNm, 2.4 kN, 16.7 kN
Resultant Forces M, Fv, F	-38.0 kNm, -0.3 kN, 16.9 kN
Load directions	Bottom of Joint in Tension, Rafter moving Up and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Joints SCI Green Book P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

Rafter-203x133UB25 [28]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Haunch-203x133UB25 [28]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	136.6 kN.m, 324.2 kN, 1088.2 kN	Mc = 136.6 kN.m	OK

Summary of Results (Unity Ratios)

Moment Capacity 108.2 kNm (for 2 rows of bolts) (Modified Applied Mom. M_{mod}=36.4 kNm)		0.34	OK
Moment Capacity 72.5 kNm (for the 1 rows of bolts required in the tension zone)		0.50	OK
Shear Capacity		0.00	OK
Beam Tension Stiffener at row 1	0.96, 0.93	0.96	OK
Flange Welds	0.47	0.47	OK
Web Welds	0.68, 0.00	0.68	OK
Haunch Welds	0.49, 0.41	0.49	OK

Step 1: Tension Zone

Basics

Bm/Plt b_p, W, t_{wb}, S_w, m_p, e_p, n_p	150.0, 70, 5.7, 6, 27.4, 40.0, 34.2	
$F_{t,Rd}=k_2 \cdot f_{ub} \cdot A / \gamma_{m^2}$	0.90•800•245/1.25	141.1 kN
$F_{T,3,Rd}=\sum F_{t,Rd}$	2•141.1	282.2 kN

BOLT ROW 1

End Plate row 1 only

m, e, m_{2U}, m_{2L},α,α'	27.4, 40.0, 47.2, 66.0, 5.9, 8.0

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l_{eff} modes	$l_{eff,cp}$ = 171.8, $l_{eff,ncp}$ = 221.1		T2.2 (d)
Mode 1 $l_{eff,1}=\min(l_{eff,cp}, l_{eff,cnp})$	min(171.8, 221.1)	171.8 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	221.1	221.1 mm	
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	171.8•15.0•15.0•275.0/4/1.0	2658.2 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	221.1•15.0•15.0•275.0/4/1.0	3419.7 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•27.35•34.19-8.25•(27.35+34.19)	1362.4	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /1362.4$	(8•34.19-2•8.25)•2658.2/1362.4	501.5 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•3419.7 + 34.19•2•141.1)/(27.35 + 34.19)	267.9 kN	
$F_{T,Rd}$ =min($F_{T,Rd}$ mode1,2,3)	min(501.5, 267.9, 282.2)	267.9 kN	
Beam Web Tension row 1 only			
$F_{t,wb,Rd} =l_{eff,1} \cdot t_{wb} \cdot f_{y,wb}/\gamma_{M0}$	171.8•5.7•275/1.00	269.4 kN	
Potential resistance of Bolt Row 1	$F_{t1,Rd}$	267.9 kN	Mode 2
BOLT ROW 2			
End Plate row 2 only			
$M_{pl,1} =l_{eff,1} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	165.3•15.0•15.0•275.0/4/1.0	2556.6 kN.mm	
$M_{pl,2} =l_{eff,2} \cdot t_f^2 \cdot p_y/4/\gamma_{M0}$	165.3•15.0•15.0•275.0/4/1.0	2556.6 kN.mm	
$2 \cdot m \cdot n - e_w \cdot (m+n)$	2•27.35•34.19-8.25•(27.35+34.19)	1362.4	
$F_{T,1,Rd}=(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1} /1362.4$	(8•34.19-2•8.25)•2556.6/1362.4	482.3 kN	
$F_{T,2,Rd}=(2 \cdot M_{pl,2} + n \cdot Nb \cdot F_{t,Rd})/(m+n)$	(2•2556.6 + 34.19•2•141.1)/(27.35 + 34.19)	239.9 kN	
$F_{T,Rd}$ =min($F_{T,Rd}$ mode1,2,3)	min(482.3, 239.9, 282.2)	239.9 kN	

Step 1C Plastic distribution Limit

$T_p < d/1.9 \cdot \sqrt{(f_{ub}/f_{yp})}$	15.0 < 20/1.9•√(800.0/275)	15.0 <= 18.0	Plastic
$F_{t\ 1,Rd} < 1.9\ F_{t,rd}$	267.9 < 1.9•141.1		Plastic
Classification	Plastic Deformation occurs.	Use Plastic distribution	

Potential Tension Capacity

Sigma $F_{ti,Rd}$	267.9 + 239.9 kN	507.8 kN	

Step 2: Compression Zone

Beam Compression

Beam Compression Zone	Flange and Web in Compression Utilising 20% OverStressing		
Total Area Flange and Web	133.2•7.8 + 5.7•99.0	16.0 cm²	
$F_{c,fb,Rd}$	16.0•275•1.20	529.1 kN	

Potential Compression Capacity

$F_{c,Rd}$min	529.1	529.1 kN	OK

Step 4: Moment Capacity

$F_{c,Rd} =F_{c,fc,Rd}$		529.1 kN	
$F_{t,Rd}$ Total $< F_{c,Rd} - N_{Ed}$	No reduction in bolt forces required		
Final Bolt Forces and Moment Capacities			
Bolt row 2 $M_{c,Rd}2 =F_{t2,Rd} \cdot h_2$	239.9•148.7	35.7 kN.m	
Bolt row 1 $M_{c,Rd}1 =F_{t1,Rd} \cdot h_1$	267.9•270.7	72.5 kN.m	
$M_{c,Rd}$		108.2 kN.m	
$M_{mod,Ed} =M - N_{Ed} \cdot h_n$	38.0 - 16.9•97.0	36.4 kN.m	OK
Tension Bolts	Only the first 1 rows are required to resist the applied moment The remaining rows shall be considered to be part of the shear zone.		
$M_{c,Rd}'$ for 1 rows	72.5	72.5 kN.m	
$F_{ri,Ed}$ for 1 rows	267.9	267.9 kN	
$F_{ri,Ed}$ design=$F_{ri,Ed} \cdot M_{Ed}/M_{c,Rd}'$	267.9•36.4/72.5	134.3 kN	
Final Web Compression Zone Height			
	Reducing Compression zone for applied moments.		
F red=$F_{c,beam}$-$F_{ri,Ed}$ design	529.1 - 134.3	394.7 kN	
h red=F red/t/p_y/1.2	394.7/5.7/275/1.2	209.9 mm	
h=max(0, h old-h red)	max(0,99.0 - 209.9)	0.0 mm	OK

Step 5: Shear Bolts

$F_{v,Rd}=\alpha_v \cdot f_{ub} \cdot A /\gamma_{M2}$	0.6 • 800 • 245.0 / 1.25	94.1 kN	
Bearing $F_{b,Rd,End}$-End Plate, End	e1=65, e2=40, k_1=2.5,α_b=0.98, d=20, t=15, f_u=410	242.3 kN	
Bearing $F_{b,Rd}$-End Plate, Inner	p1=110, e2=40, k_1=2.5,α_b=1.00, d=20, t=15, f_u=410	246.0 kN	
$F_{v,Rd,Sh}$=Min(bearing, shear)	Min(246.0, 94.1)	94.1 kN	
$F_{v,Rd,T}$=min($F_{b,Rd}$,0.28•shr)	Min(246.0, 26.3)	26.3 kN	
$F_{v,Rd,TEnd}$=min($F_{b,Rd,End}$,0.28•shr)	Min(242.3, 26.3)	26.3 kN	
Shear=Ns•$F_{v,Rd,Sh}$	4•94.1	376.3 kN	
Tension= (Nt-1)•$F_{v,Rd,T}$+ $F_{v,Rd,TEnd}$	0•26.3 + 2•26.3	52.7 kN	

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$F_{v,Rd}$ Total=Shear+Tension	376.3 + 52.7	429 kN	OK

Step 6A: Beam Tension Stiffeners

Stiffener at Bolt Row 1

$A_{sn}=b_{sn}•t_s$	(60 - 0)•8	468 mm²	
$L_{wt}=L_u+P+L_l$	60.6 + 45 + 60.6	166.1 mm	
$F_{t,wc,Rd}=L_{wt}•t_{wc}•f_y/\gamma_{M0}$	166.1•5.7•275/1000/1.00	260.4 kN	
$F_{ri}=F_{r1,Rd}+F_{r2,Rd}$	267.9 + 239.9	507.8 kN	
$F_{s,Ed}$ Web=(Fri -$F_{t,wc,Rd}$)/2	(507.8 - 260.4)/2	123.7 kN	
$F_1=F_{r1,Rd}$ •$m_1/(m_1+m_{2L})$	267.9•27.4/(27.4 + 66.0)	78.5 kN	
$F_2=F_{r2,Rd}$ •$m_1/(m_1+m_{2U})$	239.9•27.4/(27.4 + 38.3)	99.9 kN	
$F_{s,Ed}$ Flange=(F_1+F_2)/2	(78.5 + 99.9)/2	89.2 kN	
$F_{s,Ed,max}$=max($F_{s,Ed,Web}$, $F_{s,Ed,Flng}$)	max(123.7, 89.2)	123.7 kN	
A_{sn}req =$F_{s,Ed}•\gamma_{M0}/f_{y,s}$	(123.7•1000•1.00)/275	449.9 mm²	OK
Ten weld leg >= S_{tk}•0.71	6 >= 8•0.71	6 >= 5.6	OK

Steps 7&8: Welds

Beam $f_{vw,d}=f_u/(\sqrt{3}•\beta_w)/\gamma_{M2}$	410.0 /√3 / 0.85 / 1.25	222.8 N/mm²	

Flange Tension Weld

$Ft_{,flng}$ =min(B•T•P_y, F_{r1})	Min(133.2•7.8•275, 134.3)	134.3 kN	
$F_{vw,Rd}$ =2•K•0.7•t_s•L•$f_{vw,d}$	2•1.225•0.7•6•(133.2 - 2•0.7•6)•223	288.8 kN	OK

Flange Compression Weld

Direct Bearing assumed. No check required

Web Welds in Tension Zone

Web Weld OK if >= 0.71•T	6 >= 0.71•5.7	>= 4.1 mm	OK

Web Welds in Shear Zone

Lws=D-(T_t+T_b)-r_t-r_b-Lwt	336.7 - 16.1 - 7.6 - 7.6 - 105	200.7 mm	
F_{wCap}=2•0.7•ts•Lws•$f_{vw,d}$	2•0.7•6•200.7•223	379.3 kN	OK

Haunch Welds

F_h,Ed=min($M_h/(h_c-t_{fb})$, $b_b•t_{fb}•f_y$)	Min(38.4/(203.2 - 7.8), 133.2•7.8•275)	196.7 kN	
Haunch/Beam Flange area ratio,α	133.2•7.8/(133.2•7.8 + 133.2•7.8)	0.50	
F_{hnch}=F_h•min(0.5,α)	196.7•min(0.5, 0.50)	98.3 kN	
Fh_{ten}=(Fr1)/Cos(θ)	(267.9) / Cos(-9.4)	271.6 kN	

Haunch End Weld

t=min(S_{hw}, t_f) •Cos((90-ThetaH1)/2)	min(8, 7.8)•Cos((90-27.6)/2)	6.7 mm	
K=√(3/(1+2•Cos((90-ThetaH1)/2)²))	√(3/(1+2•Cos((90-27.6)/2)²)	1.10	
End_{Cap}=K•t(B-2•w)•$f_{vw,d}$	1.104•6.7•(133.2 - 2•0.7•8)•223	199.9 kN	
End_{Cap}>= F_{hnch}	199.9 >= 98.3	98.3 kN	OK

Haunch Web Weld

Web force F_{wh}=Fh_{ten}-F_{hnch}	271.6 - 98.3	173.3 kN	
L_w =(H_l-D_c/2-T_{ep})/Cos(Theta)	(279 - 0.0/2 - 15)/Cos(-9.4)	268.0 mm	
L_w=L_w -t_w-(T+t_{w1})/Sin(Theta1)	268.0 - 6 -(7.8 + 6)/Sin(27.6)= 268.0-6-29.8	232.2 mm	
Web_{Cap}=2•0.7•t(L_w-2•t)•$f_{vw,d}$	2•0.7•6(232.2 - 2•0.7•6)•223	418.7 kN	
Web_{Cap}>= F_{wh}	418.7 >= 173.3	173.3 kN	OK

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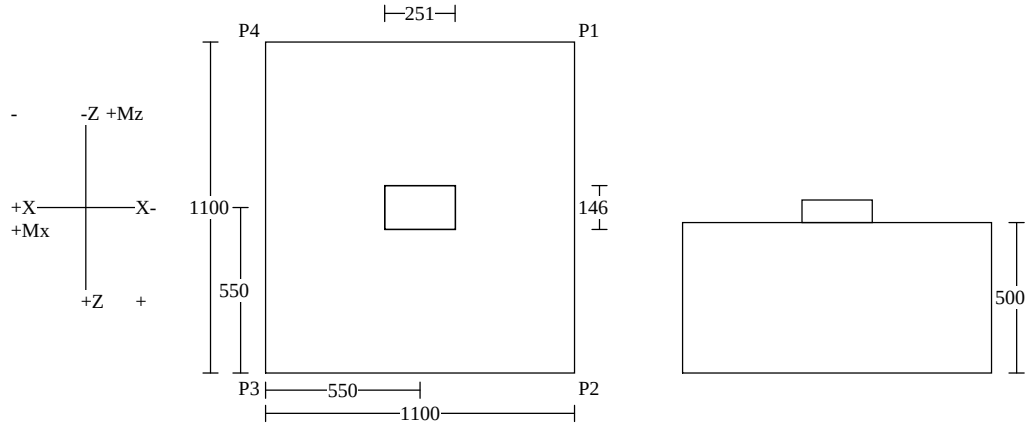
Pad @ Node 1 : (Grid A1)



Basic Properties

Design to	EC 2: 2004 - Using UK values
Fy, Fcu, Covers T, B, S	460 N/mm², 30 N/mm², 50 mm, 50 mm, 50 mm
Gross: Area, Area1, Z zz, Z xx	1.21, 0.037, 0.222, 0.222
Conc Den, Surcharge, LFsrv , LFult, SWP	23.4, 1.5, 1.0, 1.0, 100

Mass Concrete Pad Design

x-x projections	425, 425	500 mm	OK
z-z projections	477, 477	500 mm	OK

Critical Serviceability : 82 : W3 Wind on Gable

Fpad = Den•d•Area•LF	23.4 x 0.5 x 1.21 x 1.00	14.2 kN	
Fsur = Sur•(Area-Area1)•LF	1.5 x (1.21 - 0.037) x 1.00	1.8 kN	
Fcol = F	-8.8 +	-8.8 kN	
Fres = F + Fpad + Fsur	-8.8 + 14.2 + 1.8	7.1 kN	
Horizontal Shear Ignored			

Pressure

Pmax = F res / Area	7.1 / 1.210	5.8 kN/m²	OK

FOS Up-lift

FOS = F stat / F up	(14.2 + 1.8)/ 8.8	1.80 > 1.5	OK

Uplift Design

Uplift detected.	-11.8 kN Uplift in Case 27 : 1 Dead + 1.5 Side Wind P3		
	Design Top Steel to resist (SelfWeight + Surcharge) x 1.4		
Load W = (d x den + sur) x 1.4	(500x23.4 + 1.5) x 1.4	18.48 kN/m²	

Z-Z Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 30)	1.50 N/mm²	
Z zz = B x D² / 6	1100 x 500² / 6	0.05m³	
Mu zz = Z x Fcu ten	0.05 x 1.50 x 1000	68.75 kN.m	

X-X Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 30)	1.50 N/mm²	
Z xx = B x D² / 6	1100 x 500² / 6	0.05m³	
Mu xx = Z x Fcu ten	0.05 x 1.50 x 1000	68.75 kN.m	

Moments at Column Face

M zz = Fn(W,Wd,Lmax)	18.48, 1100, 424	1.83 kN.m	OK
M xx = Fn(W,Wd,Lmax)	18.48, 1100, 477	2.31 kN.m	OK

Shear at Column Face

v zz = Fn(W,Lmax,d)	18.48, 424, 450	0.02 N/mm²	OK
v xx = Fn(W,Lmax,d)	18.48, 477, 450	0.02 N/mm²	OK

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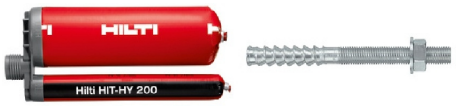
Specifier's comments:

1 Input data

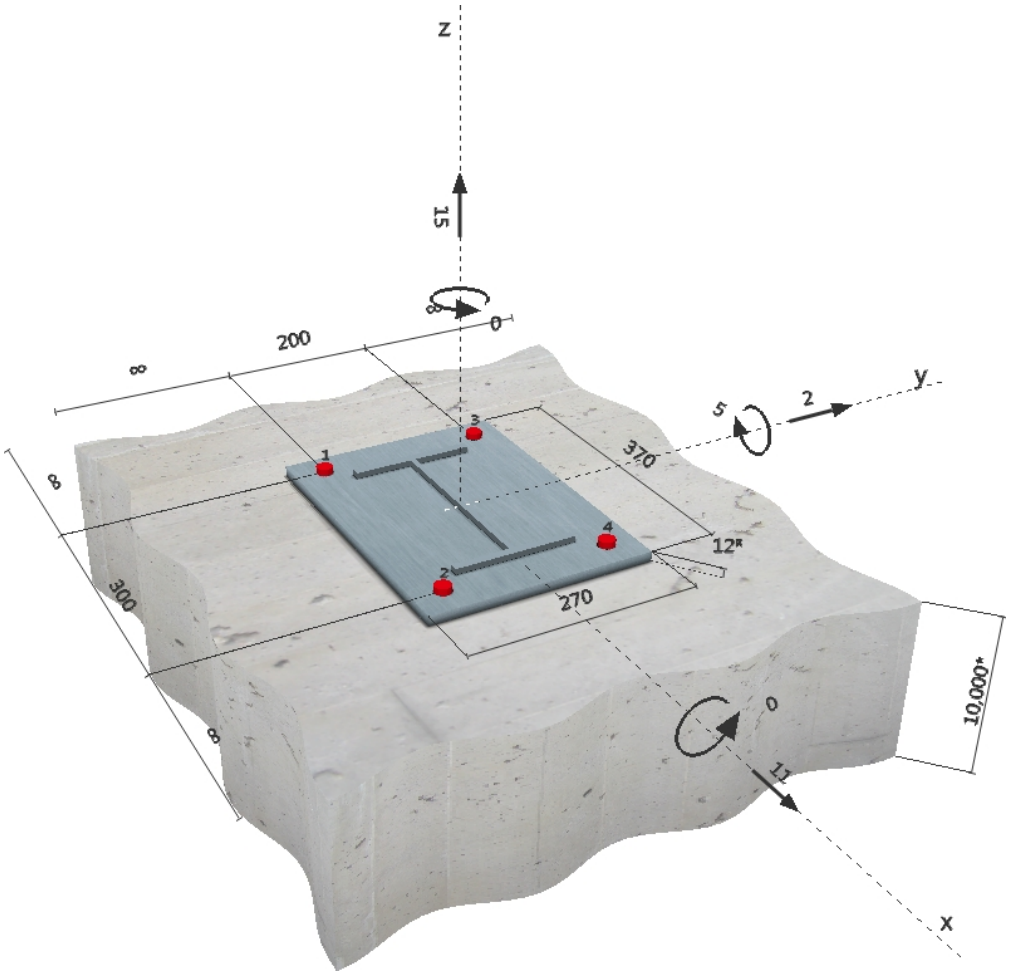


SAFE-SET

Anchor type and diameter:	**HIT-HY 200-A + HIT-Z M20**
Return period (service life in years):	50
Effective embedment depth:	$h_{ef,act}$ = 100 mm ($h_{ef,limit}$ = - mm)
Material:	DIN EN ISO 4042
Evaluation Service Report:	ETA 12/0006
Issued I Valid:	4/11/2019 | -
Proof:	Design method ETAG (No. 001 Annex C/2010)
Stand-off installation:	e_b = 0 mm (no stand-off); t = 12 mm
Anchor plate:	l_x x l_y x t = 370 mm x 270 mm x 12 mm; (Recommended plate thickness: not calculated
Profile:	Advance UKB, 254 x 146 x 37; (L x W x T x FT) = 256 mm x 146 mm x 6 mm x 11 mm
Base material:	cracked concrete, C25/30, $f_{c,cube}$ = 30.00 N/mm^2; h = 10,000 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



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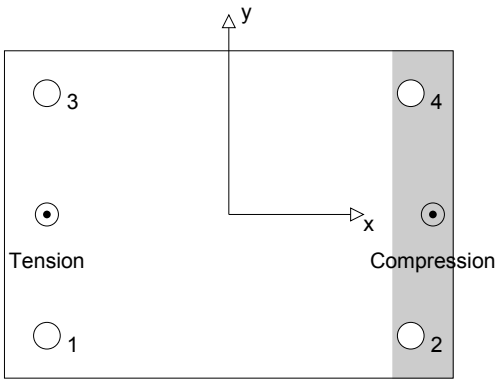
1.1 Load combination

Case	Description	Forces [kN] / Moments [kNm]	Seismic	Fire	Max. Util. [%]
1	**Combination 1**	**V_x = 11.000; V_y = 2.000; N = 15.000; M_x = 0.000; M_y = 5.000; M_z = 0.000;**	**no**	**no**	**54**
2	Combination 2	V_x = 17.000; V_y = 0.000; N = -40.200; M_x = 0.000; M_y = 0.000; M_z = 0.000;	no	no	10

2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [kN]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11.820	2.795	2.750	0.500
2	0.000	2.795	2.750	0.500
3	11.820	2.795	2.750	0.500
4	0.000	2.795	2.750	0.500

max. concrete compressive strain: 0.04 [‰]
max. concrete compressive stress: 1.28 [N/mm²]
resulting tension force in (x/y)=(-150/0): 23.640 [kN]
resulting compression force in (x/y)=(168/0): 8.640 [kN]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load (ETAG, Annex C, Section 5.2.2)

	Load [kN]	Capacity [kN]	Utilization β_N [%]	Status
Steel Strength*	11.820	97.333	13	OK
Pullout Strength*	11.820	90.000	14	OK
Concrete Breakout Strength**	23.640	43.818	54	OK
Splitting failure**	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{Rk,s}$ [kN]	$\gamma_{M,s}$	$N_{Rd,s}$ [kN]	N_{Sd} [kN]
146.000	1.500	97.333	11.820

3.2 Pullout Strength

$N_{Rk,p}$ [kN]	ψ_c	$\gamma_{M,p}$	$N_{Rd,p}$ [kN]	N_{Sd} [kN]
135.000	1.000	1.500	90.000	11.820

3.3 Concrete Breakout Strength

$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]
150,000	90,000	150	300

$e_{c1,N}$ [mm]	$\psi_{ec1,N}$	$e_{c2,N}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0	1.000	0	1.000	1.000	1.000

k_1	$N^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$N_{Rd,c}$ [kN]	N_{Sd} [kN]
7.200	39.436	1.500	43.818	23.640

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4 Shear load (ETAG, Annex C, Section 5.2.3)

	Load [kN]	Capacity [kN]	Utilization β_V [%]	Status
Steel Strength (without lever arm)*	2.795	58.400	5	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	11.180	175.271	7	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength (without lever arm)

$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V_{Rd,s}$ [kN]	V_{Sd} [kN]
73.000	1.250	58.400	2.795

4.2 Pryout Strength

$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k-factor
300,000	90,000	150	300	2.000

$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0	1.000	0	1.000	1.000	1.000

$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V_{Rd,cp}$ [kN]	V_{Sd} [kN]
39.436	1.500	175.271	11.180

5 Combined tension and shear loads (ETAG, Annex C, Section 5.2.4)

Steel failure

β_N	β_V	α	Utilization $\beta_{N,V}$ [%]	Status
0.540	0.064	1.500	42	OK

$\beta_N^\alpha + \beta_V^\alpha \leq 1.0$

6 Displacements (highest loaded anchor)

Short term loading:

N_{Sk} = 8.756 [kN] $\quad \delta_N$ = 0.139 [mm]

V_{Sk} = 2.070 [kN] $\quad \delta_V$ = 0.083 [mm]

δ_{NV} = 0.162 [mm]

Long term loading:

N_{Sk} = 8.756 [kN] $\quad \delta_N$ = 0.293 [mm]

V_{Sk} = 2.070 [kN] $\quad \delta_V$ = 0.124 [mm]

δ_{NV} = 0.318 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the anchor plate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

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7 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with ETAG 001, Annex C(2010)Section 7! The software considers that the grout is installed under the anchor plate without creating air voids and before application of the loads.
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 4.1 of ETAG 001, Annex C! For larger diameters of the clearance hole see Chapter 1.1. of ETAG 001, Annex C!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The characteristic bond resistances depend on the return period (service life in years): 50

Fastening meets the design criteria!

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8 Installation data

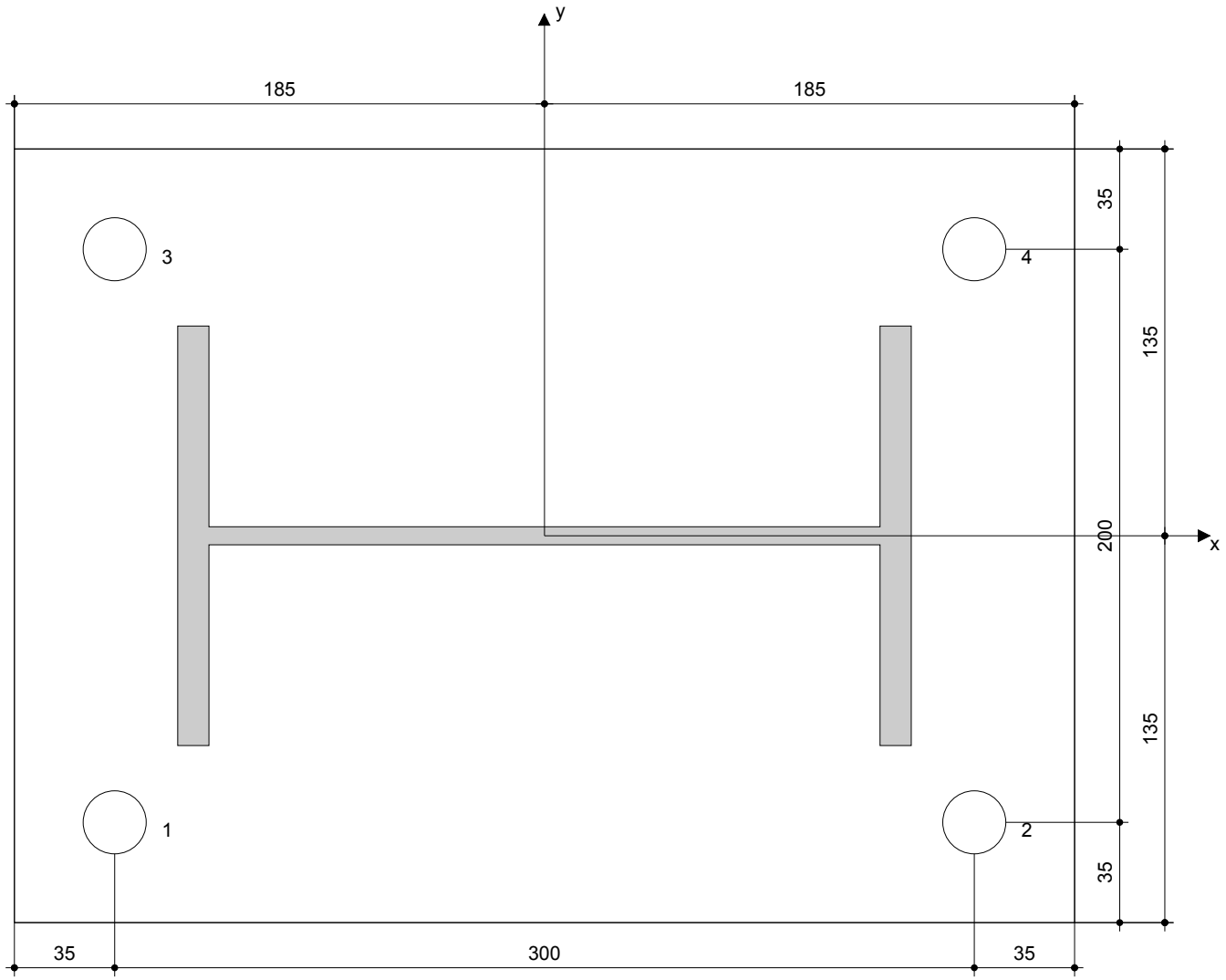
Anchor plate, steel: -
Profile: Advance UKB, 254 x 146 x 37; (L x W x T x FT) = 256 mm x 146 mm x 6 mm x 11 mm
Hole diameter in the fixture (pre-setting) : d_f = 22 mm
Hole diameter in the fixture (through fastening) : d_f = 24 mm
Plate thickness (input): 12 mm
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HIT-HY 200-A + HIT-Z M20
Installation torque: 0.150 kNm

Hole diameter in the base material: 22 mm
Hole depth in the base material: 156 mm
Minimum thickness of the base material: 200 mm

8.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer	• No accessory required	• Dispenser including cassette and mixer
• Properly sized drill bit		• Torque wrench



Coordinates Anchor [mm]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-150	-100	-	-	-	-
2	150	-100	-	-	-	-
3	-150	100	-	-	-	-
4	150	100	-	-	-	-

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9 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

ASP Consulting (Melbourne) Ltd York House, Smisby Road, Ashby de la Zouch, Leicestershire, LE65 2UG Tel:- +44(0)1530 561802	Project Hilton Scout Headquarter Extension				Job Ref. 6190	
	Section East Extension Steel beam supporting floor				Sheet no./rev. 50	
	Calc. by GB	Date 16/04/2020	Chk'd by HD	Date 16/04/2020	App'd by	Date

Beam to carry east extension floor - Length = 4.0 m Total Length

Carries:

1) Floor

Summary of Applied Loads:

Floor Dead	0.665	kN/m^2	Imposed	3	kN/m^2

UDLs

Floor	Floor span	5.300	m
	Loaded Width	2.650	m

UDLs:

Dead Floor	**1.762**	kN/m	Imposed Floor	**7.950**	kN/m

Loads to beam

Total UDLs (uf)

Dead load	**1.76**	**kN/m**	*Imposed load*	**7.95**	**kN/m**

Beam reactions

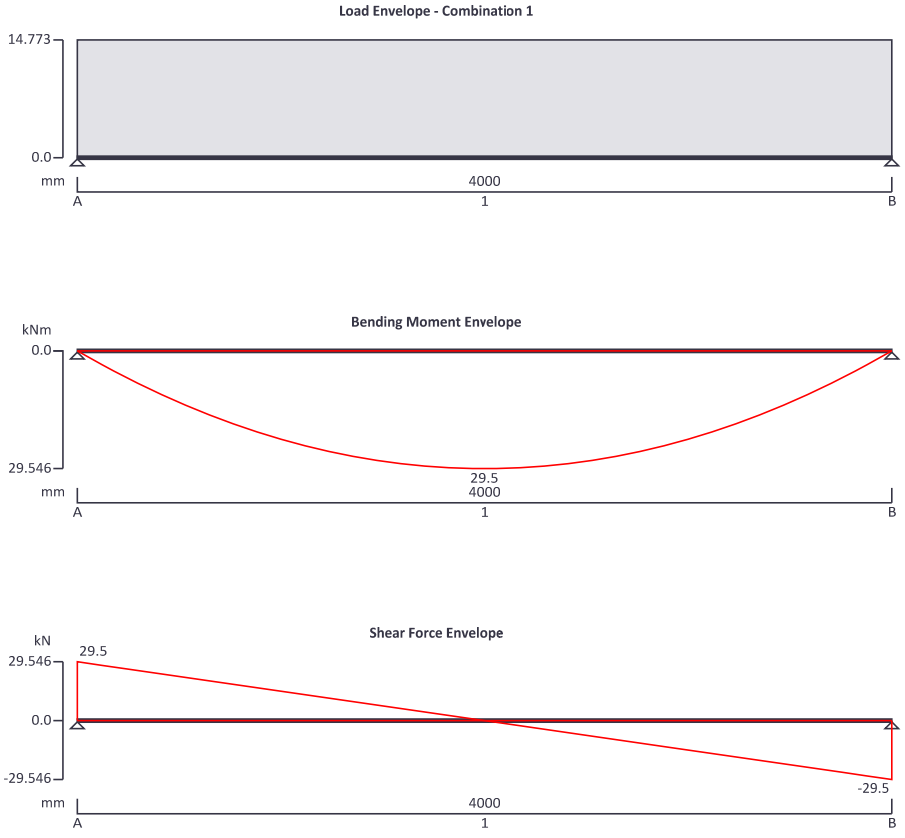
Dead Reaction	4.10	kN
Imposed Reaction	16.00	kN
Max Reaction	29.54	kN

ADOPT UB 203x133x30

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Variable full UDL 8 kN/m
	Permanent full UDL 1.76 kN/m
	Permanent self weight of beam × 1

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **29.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **29.5** kN	V_{min} = **-29.5** kN
Deflection	δ_{max} = **5.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **29.5** kN	R_{A_min} = **29.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **4.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **16** kN	
Maximum reaction at support B	R_{B_max} = **29.5** kN	R_{B_min} = **29.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **4.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **16** kN	

Section details

Section type: **UKB 203x133x30 (Tata Steel Advance)**

Steel grade: **S275**

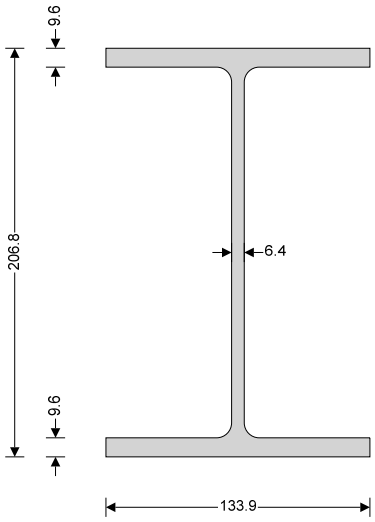
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element: $t = \max(t_f, t_w)$ = **9.6** mm

Nominal yield strength: f_y = **275** N/mm^2

Nominal ultimate tensile strength: f_u = **410** N/mm^2

Modulus of elasticity: E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.400** + 2 × h
	$K_{LT.B}$ = **1.400** + 2 × h

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d =$ **172.4** mm

$c / t_w = 29.1 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 =$ **56.2** mm

$c / t_f = 6.3 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f =$ **187.6** mm

Shear area factor — $\eta =$ **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **29.5** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1458** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} =$ **231.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **29.5** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} =$

$W_{pl.y} \times f_y / \gamma_{M0} =$ **86.5** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c =$ **1**

$C_1 = 1 / k_c^2 =$ **1**

Destabilised load condition factor — $D =$ **1.2**

Curvature factor — $g = \sqrt{1 - (I_z / I_y)} =$ **0.931**

Poissons ratio — $\nu =$ **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2

Unrestrained length — $L = 1.4 \times L_{s1} + 2 \times h =$ **6014** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **51.6** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **1.553**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} =$ **0.34**

Correction factor for rolled sections — $\beta =$ **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **1.601**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.405**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **1.000**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.405**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **35** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **11.1** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **5.512** mm

PASS - Maximum deflection does not exceed deflection limit

EAST EXTENSION TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

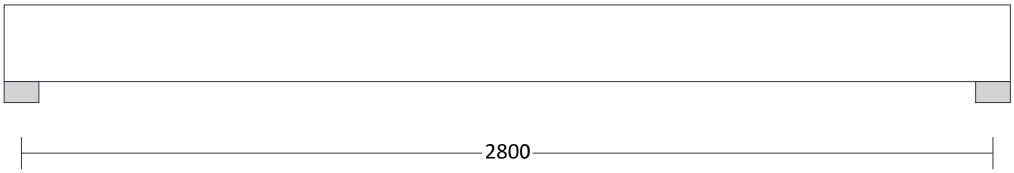
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.00

Joist details

Description — 60 x 220 C24 timber joists

Joist spacing — s_{Joist} = **400** mm



Forces input on Joist

Vertical permanent load on joist — F_{G_Joist}= **0.45** kN/m^2

Vertical imposed load on joist — F_{Q_Joist}= **5.00** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist — $p_G = F_{G\_Joist} \times s_{Joist}$= **0.18** kN/m

Vertical imposed load on joist — $p_Q = F_{Q\_Joist} \times s_{Joist}$= **2.00** kN/m

ANALYSIS

Tedds calculation version 1.0.28

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.18 kN/m at 0 m to 2.8 m
Member	Imposed	UDL	GlobalZ	2 kN/m at 0 m to 2.8 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection

1.00G + 1.00Q (Service) - Total deflection

Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.29568	
2	0	0	-0.29568	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.19913	
2	0	0	-0.19913	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	9.3
1.00G + 1.00Q (Service)	0	6.2

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

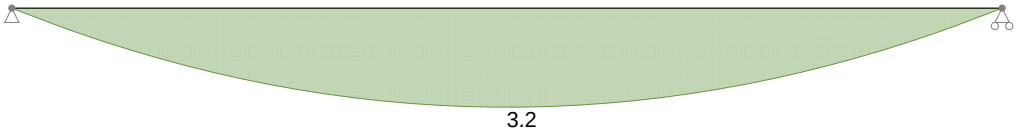
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.8	1	0	-4.6	0
		2	0	-4.6	0

Load combination: 1.00G + 1.00Q (Service)

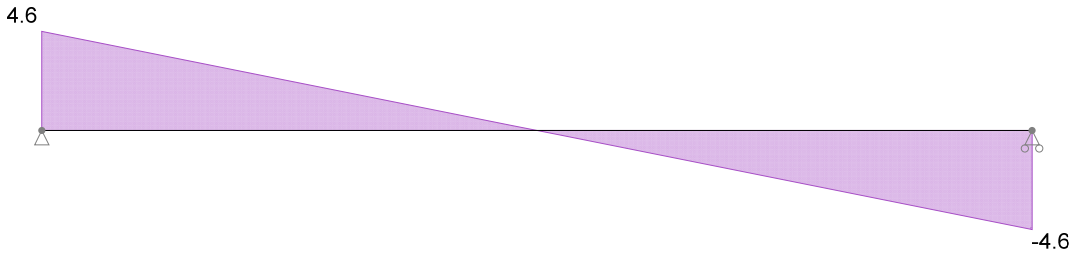
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.8	1	0	-3.1	0
		2	0	-3.1	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	4.6 (max abs)		0 (min)	
	1.4	0		3.2 (max)	
	2.8	-4.6		0 (min)	

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

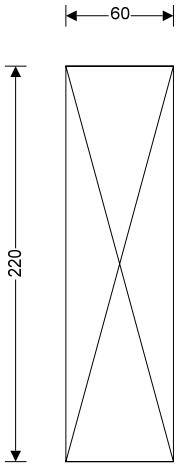
Timber section details

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

Breadth of sections b = **60** mm

Depth of sections h = **220** mm

Timber strength class - EN 338:2016 Table 1 **C24**



60x220 timber section
Cross-sectional area, A, 13200 mm^2
Section modulus, W_y, 484000 mm^3
Section modulus, W_z, 132000 mm^3
Second moment of area, I_y, 53240000 mm^4
Second moment of area, I_z, 3960000 mm^4
Radius of gyration, i_y, 63.5 mm
Radius of gyration, i_z, 17.3 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.7**
Deformation factor - Table 3.2	k_{def} = **0.6**
Bending stress re-distribution factor - cl.6.1.6(2)	k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2)	k_{cr} = **0.67**
System strength factor - cl.6.6	k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d}$ = **4.626** kN
Effective contact length	$L_{b,ef} = L_b$ = **100** mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.771** N/mm^2
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.481** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.521**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **4.626** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.785** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.331**

PASS - Design shear strength exceeds design shear stress

Check design 1400 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **3.238** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **6.69** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **14.215** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.471**

PASS - Design bending strength exceeds design bending stress

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d}$ = **4.626** kN
Effective contact length	$L_{b,ef} = L_b$ = **100** mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.771** N/mm^2
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.481** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.521**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **4.626** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.785** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.331**

PASS - Design shear strength exceeds design shear stress

Consider Combination 2 - 1.00G + 1.00Q (Service)

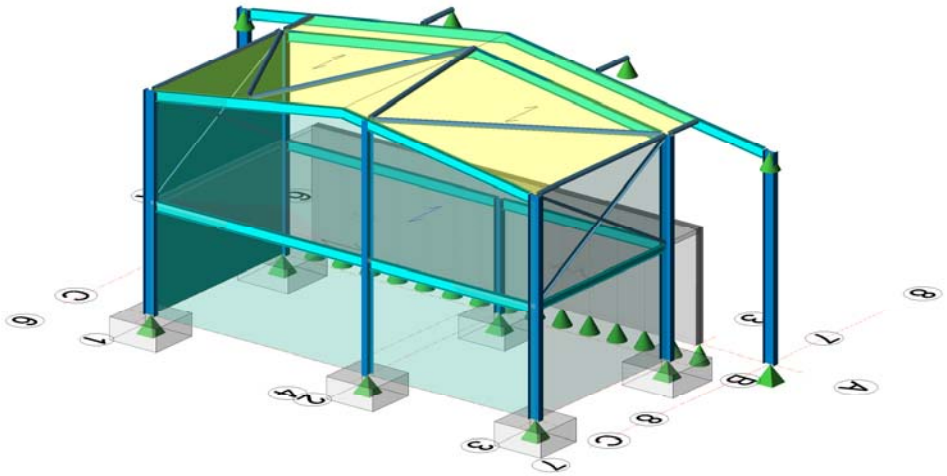
Check design 1400 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	δ_y = **3.3** mm
Quasi-permanent variable load factor	ψ_2 = **0.3**
Final deflection with creep	$\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **5.3** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **11.2** mm
	$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.475**

PASS - Allowable deflection exceeds final deflection

Contents



3D Model

Loadings

Dead Load

Floor – 0.65 kN/m^2
Roof – 0.20 kN/m^2

Imposed

Floor - 3kN/m^2
Roof – 0.60 kN/m^2

Combinations Summary

1 STR_1-1.35G+1.5Q+1.5RQ

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	350.6
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	511.3
Total Load on Structure	0.0	0.0	511.3
Total Reaction	0.0	0.0	511.3

2 $STR_{6.1}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6

Load Type	Force [kN]		
	X	Y	Z
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.5	0.0
Decomposable Loads	8.8	10.8	347.7
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	8.8	13.3	508.4
Total Load on Structure	8.8	13.3	508.4
Total Reaction	-8.8	-13.3	508.4

3 $STR_{6.2}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.5	0.0
Decomposable Loads	8.8	9.7	340.8
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	8.8	12.2	501.5
Total Load on Structure	8.8	12.2	501.5
Total Reaction	-8.8	-12.2	501.5

4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.5	0.0
Decomposable Loads	18.6	10.8	338.2
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	18.6	13.3	498.9
Total Load on Structure	18.6	13.3	498.9
Total Reaction	-18.6	-13.3	498.9

5 $STR_{6.4}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.5	0.0
Decomposable Loads	18.6	9.7	331.3
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	18.6	12.2	492.1
Total Load on Structure	18.6	12.2	492.1
Total Reaction	-18.6	-12.2	492.1

6 $STR_{6.5}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.5	0.0
Decomposable Loads	13.8	7.8	336.1
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	13.8	10.3	496.8
Total Load on Structure	13.8	10.3	496.8
Total Reaction	-13.8	-10.3	496.8

7 $STR_{6.6}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.5	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-14.3	0.0	355.3
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-16.8	0.0	516.1
Total Load on Structure	-16.8	0.0	516.1
Total Reaction	16.8	0.0	516.1

8 $STR_{6.7}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.5	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-14.3	0.0	341.7
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-16.8	0.0	502.4
Total Load on Structure	-16.8	0.0	502.4
Total Reaction	16.8	0.0	502.4

9 $STR_{6.8}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.5	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-8.0	0.0	349.3
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-10.5	0.0	510.0
Total Load on Structure	-10.5	0.0	510.0
Total Reaction	10.5	0.0	510.0

10 $STR_{6.9}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.5	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-8.0	0.0	335.6
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-10.5	0.0	496.3
Total Load on Structure	-10.5	0.0	496.3
Total Reaction	10.5	0.0	496.3

11 $STR_{6.10}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.5	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-9.7	0.0	338.9
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-12.2	0.0	499.7
Total Load on Structure	-12.2	0.0	499.7
Total Reaction	12.2	0.0	499.7

12 $STR_{6.11}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.5	0.0
Decomposable Loads	7.0	-8.5	348.3
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	7.0	-11.0	509.0
Total Load on Structure	7.0	-11.0	509.0
Total Reaction	-7.0	11.0	509.0

13 $STR_{6.12}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.5	0.0
Decomposable Loads	7.0	-7.6	342.9
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	7.0	-10.1	503.6
Total Load on Structure	7.0	-10.1	503.6
Total Reaction	-7.0	10.1	503.6

14 $STR_{6.13}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.5	0.0
Decomposable Loads	14.7	-8.5	340.8
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	14.7	-11.0	501.5
Total Load on Structure	14.7	-11.0	501.5
Total Reaction	-14.7	11.0	501.5

15 $STR_{6.14}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.5	0.0
Decomposable Loads	14.7	-7.6	335.4
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	14.7	-10.1	496.1
Total Load on Structure	14.7	-10.1	496.1
Total Reaction	-14.7	10.1	496.1

16 $STR_{6.15}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.5	0.0
Decomposable Loads	10.9	-6.2	339.2
1 Way Decomp Results	0.0	0.0	350.6
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	10.9	-8.7	499.9
Total Load on Structure	10.9	-8.7	499.9
Total Reaction	-10.9	8.7	499.9

17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.1	0.0
Decomposable Loads	17.7	21.6	270.1
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	17.7	23.7	430.8
Total Load on Structure	17.7	23.7	430.8
Total Reaction	-17.7	-23.7	430.8

18 $STR_{8.2}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.1	0.0
Decomposable Loads	17.7	19.3	256.4
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	17.7	21.5	417.1
Total Load on Structure	17.7	21.5	417.1
Total Reaction	-17.7	-21.5	417.1

19 $STR_{8.3}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.1	0.0
Decomposable Loads	37.3	21.6	251.2
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	37.3	23.7	411.9
Total Load on Structure	37.3	23.7	411.9
Total Reaction	-37.3	-23.7	411.9

20 $STR_{8.4}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.1	0.0
Decomposable Loads	37.3	19.3	237.5
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	37.3	21.5	398.2
Total Load on Structure	37.3	21.5	398.2
Total Reaction	-37.3	-21.5	398.2

21 $STR_{8.5}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	2.1	0.0
Decomposable Loads	27.6	15.6	247.0
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	27.6	17.8	407.7
Total Load on Structure	27.6	17.8	407.7
Total Reaction	-27.6	-17.8	407.7

22 $STR_{8.6}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.1	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-28.5	0.0	285.5
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-30.6	0.0	446.2
Total Load on Structure	-30.6	0.0	446.2
Total Reaction	30.6	0.0	446.2

23 $STR_{8.7}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.1	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-28.5	0.0	258.1
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-30.6	0.0	418.9
Total Load on Structure	-30.6	0.0	418.9
Total Reaction	30.6	0.0	418.9

24 $STR_{8.8}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.1	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-15.9	0.0	273.3
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-18.0	0.0	434.0
Total Load on Structure	-18.0	0.0	434.0
Total Reaction	18.0	0.0	434.0

25 $STR_{8.9}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.1	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-15.9	0.0	246.0
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-18.0	0.0	406.7
Total Load on Structure	-18.0	0.0	406.7
Total Reaction	18.0	0.0	406.7

26 $STR_{8.10}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	-2.1	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-19.4	0.0	252.7
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-21.5	0.0	413.4
Total Load on Structure	-21.5	0.0	413.4
Total Reaction	21.5	0.0	413.4

27 $STR_{8.11}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.1	0.0
Decomposable Loads	13.9	-17.0	271.4
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	13.9	-19.1	432.1
Total Load on Structure	13.9	-19.1	432.1
Total Reaction	-13.9	19.1	432.1

28 $STR_{8.12}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.1	0.0
Decomposable Loads	13.9	-15.2	260.6
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	13.9	-17.4	421.3
Total Load on Structure	13.9	-17.4	421.3
Total Reaction	-13.9	17.4	421.3

29 $STR_{8.13}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.1	0.0
Decomposable Loads	29.3	-17.0	256.4
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	29.3	-19.1	417.2
Total Load on Structure	29.3	-19.1	417.2
Total Reaction	-29.3	19.1	417.2

30 $STR_{8.14}$-1.35G+1.5ψ$_0$Q+1.5ψ$_0$S+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.1	0.0
Decomposable Loads	29.3	-15.2	245.7
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	29.3	-17.4	406.4
Total Load on Structure	29.3	-17.4	406.4
Total Reaction	-29.3	17.4	406.4

31 $STR_{8.15}$-1.35G+1.5ψ$_0$Q+1.5ψ$_0$S+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	154.1
Nodal Loads	0.0	0.0	6.6
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-2.1	0.0
Decomposable Loads	21.7	-12.3	253.1
1 Way Decomp Results	0.0	0.0	276.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	21.7	-14.4	413.9
Total Load on Structure	21.7	-14.4	413.9
Total Reaction	-21.7	14.4	413.9

32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.9	0.0
Decomposable Loads	17.7	21.6	69.6
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	17.7	22.5	188.6
Total Load on Structure	17.7	22.5	188.6
Total Reaction	-17.7	-22.5	188.6

33 $STR_{9.2}$-G+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.9	0.0
Decomposable Loads	17.7	19.3	55.9
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	17.7	20.3	175.0
Total Load on Structure	17.7	20.3	175.0
Total Reaction	-17.7	-20.3	175.0

34 $STR_{9.3}$-G+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.9	0.0
Decomposable Loads	37.3	21.6	50.7
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	37.3	22.5	169.7
Total Load on Structure	37.3	22.5	169.7
Total Reaction	-37.3	-22.5	169.7

35 $STR_{9.4}$-G+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.9	0.0
Decomposable Loads	37.3	19.3	37.0
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	37.3	20.3	156.0
Total Load on Structure	37.3	20.3	156.0
Total Reaction	-37.3	-20.3	156.0

36 $STR_{9.5}$-G+1.5W+EHF_{Dir2+}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.9	0.0
Decomposable Loads	27.6	15.6	46.5
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	27.6	16.6	165.5
Total Load on Structure	27.6	16.6	165.5
Total Reaction	-27.6	-16.6	165.5

37 $STR_{9.6}$-G+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	-0.9	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-28.5	0.0	85.0
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-29.4	0.0	204.0
Total Load on Structure	-29.4	0.0	204.0
Total Reaction	29.4	0.0	204.0

38 $STR_{9.7}$-G+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	-0.9	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-28.5	0.0	57.6
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-29.4	0.0	176.7
Total Load on Structure	-29.4	0.0	176.7
Total Reaction	29.4	0.0	176.7

39 $STR_{9.8}$-G+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	-0.9	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-15.9	0.0	72.8
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-16.8	0.0	191.8
Total Load on Structure	-16.8	0.0	191.8
Total Reaction	16.8	0.0	191.8

40 $STR_{9.9}$-G+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	-0.9	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-15.9	0.0	45.5
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-16.8	0.0	164.5
Total Load on Structure	-16.8	0.0	164.5
Total Reaction	16.8	0.0	164.5

41 $STR_{9.10}$-G+1.5W+EHF_{Dir1-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	-0.9	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-19.4	0.0	52.2
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-20.3	0.0	171.2
Total Load on Structure	-20.3	0.0	171.2
Total Reaction	20.3	0.0	171.2

42 $STR_{9.11}$-G+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-0.9	0.0
Decomposable Loads	13.9	-17.0	70.8
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	13.9	-17.9	189.9
Total Load on Structure	13.9	-17.9	189.9
Total Reaction	-13.9	17.9	189.9

43 $STR_{9.12}$-G+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-0.9	0.0
Decomposable Loads	13.9	-15.2	60.1
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	13.9	-16.2	179.1
Total Load on Structure	13.9	-16.2	179.1
Total Reaction	-13.9	16.2	179.1

44 $STR_{9.13}$-G+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-0.9	0.0
Decomposable Loads	29.3	-17.0	55.9
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	29.3	-17.9	175.0
Total Load on Structure	29.3	-17.9	175.0
Total Reaction	-29.3	17.9	175.0

45 $STR_{9.14}$-G+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-0.9	0.0
Decomposable Loads	29.3	-15.2	45.2
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	29.3	-16.2	164.2
Total Load on Structure	29.3	-16.2	164.2
Total Reaction	-29.3	16.2	164.2

46 $STR_{9.15}$-G+1.5W+EHF_{Dir2-}

Load Type	Force [kN]		
	X	Y	Z
Member Loads	0.0	0.0	114.1
Nodal Loads	0.0	0.0	4.9
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	-0.9	0.0
Decomposable Loads	21.7	-12.3	52.6
1 Way Decomp Results	0.0	0.0	75.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	21.7	-13.2	171.7
Total Load on Structure	21.7	-13.2	171.7
Total Reaction	-21.7	13.2	171.7

Envelopes

Name
1 Envelope

1 Envelope

Combination Title
46 $STR_{9.15}$-G+1.5W+EHF_{Dir2-}
45 $STR_{9.14}$-G+1.5W+EHF_{Dir2-}
44 $STR_{9.13}$-G+1.5W+EHF_{Dir2-}
43 $STR_{9.12}$-G+1.5W+EHF_{Dir2-}
42 $STR_{9.11}$-G+1.5W+EHF_{Dir2-}
41 $STR_{9.10}$-G+1.5W+EHF_{Dir1-}
40 $STR_{9.9}$-G+1.5W+EHF_{Dir1-}
39 $STR_{9.8}$-G+1.5W+EHF_{Dir1-}
38 $STR_{9.7}$-G+1.5W+EHF_{Dir1-}
37 $STR_{9.6}$-G+1.5W+EHF_{Dir1-}
36 $STR_{9.5}$-G+1.5W+EHF_{Dir2+}
35 $STR_{9.4}$-G+1.5W+EHF_{Dir2+}
34 $STR_{9.3}$-G+1.5W+EHF_{Dir2+}
33 $STR_{9.2}$-G+1.5W+EHF_{Dir2+}
32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}
31 $STR_{8.15}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}
30 $STR_{8.14}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}
29 $STR_{8.13}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}
28 $STR_{8.12}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}
27 $STR_{8.11}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2-}
26 $STR_{8.10}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}
25 $STR_{8.9}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}
24 $STR_{8.8}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}
23 $STR_{8.7}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}
22 $STR_{8.6}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir1-}
21 $STR_{8.5}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
20 $STR_{8.4}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
19 $STR_{8.3}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
18 $STR_{8.2}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
16 $STR_{6.15}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}
15 $STR_{6.14}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}
14 $STR_{6.13}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}
13 $STR_{6.12}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}
12 $STR_{6.11}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2-}

Combination Title
11 $STR_{6.10}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}
10 $STR_{6.9}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}
9 $STR_{6.8}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}
8 $STR_{6.7}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}
7 $STR_{6.6}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir1-}
6 $STR_{6.5}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
5 $STR_{6.4}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
3 $STR_{6.2}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
2 $STR_{6.1}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
1 STR_1-1.35G+1.5Q+1.5RQ

Wind Data

General

Method
BS EN 1991-1-4:2005 UK NA
Wind speed and ground roughness databases derived from data, © BRE Ltd.
Altitude databases derived from data, © NASA 2006

Site Details

Grid Reference	SK247302	
Altitude	63	m
Air Density	1.226	kg/m^3
Site Ground Level	0.000	m
Orientation of North relative to TSD axes	270.0000	°
Tall neighbouring buildings not considered		
Orography considered		
Shelter effect from obstructions is not included		
Basic wind speed, $v_{b,map}$	21.8	m/s
Probability Factor, c_{prob}	1.000	
Seasonal Factor, c_{season}	1.000	
Default Height for Internal Pressure, z_i	5.600	m

Intermediate Factors

270° North

Direction Factor, c_{dir}	0.780	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.059	
Orography factor, c_o	1.007	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	24.8	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.059	
Orography factor, c_o	1.007	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	25.4	m/s

300°

Direction Factor, c_{dir}	0.820	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m

Altitude factor, c_{alt}	1.060	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	26.0	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.060	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	26.6	m/s

330°

Direction Factor, c_{dir}	0.910	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.056	
Orography factor, c_o	1.006	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	28.8	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.056	
Orography factor, c_o	1.006	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	29.5	m/s

0° West

Direction Factor, c_{dir}	0.990	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.054	
Orography factor, c_o	1.010	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	31.4	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.054	
Orography factor, c_o	1.010	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	32.1	m/s

30°

Direction Factor, c_{dir}	1.000	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.053	
Orography factor, c_o	1.009	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	31.7	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.053	
Orography factor, c_o	1.009	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	32.4	m/s

60°

Direction Factor, c_{dir}	0.930	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.002	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	29.3	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.002	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	29.9	m/s

90° South

Direction Factor, c_{dir}	0.850	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.004	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	26.8	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.004	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	27.4	m/s

120°

Direction Factor, c_{dir}	0.800	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.010	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	25.3	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.052	
Orography factor, c_o	1.010	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	25.9	m/s

150°

Direction Factor, c_{dir}	0.730	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.055	
Orography factor, c_o	1.006	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	23.1	m/s
Height above ground	5.600	m

Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.055	
Orography factor, c_o	1.006	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	23.6	m/s

180° East

Direction Factor, c_{dir}	0.740	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.061	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	23.5	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.061	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	24.0	m/s

210°

Direction Factor, c_{dir}	0.730	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.060	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	23.1	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.060	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	23.7	m/s

240°

Direction Factor, c_{dir}	0.730	
Height above ground	4.850	m
Effective Height, z - h_{dis}	4.850	m
Altitude factor, c_{alt}	1.063	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.888	
Peak wind velocity, v_p	23.2	m/s
Height above ground	5.600	m
Effective Height, z - h_{dis}	5.600	m
Altitude factor, c_{alt}	1.063	
Orography factor, c_o	1.000	
Exposure factor, c_e	1.973	
Peak wind velocity, v_p	23.7	m/s

Building Directions

90°

Height above ground	4.850	m

Peak wind velocity, v_p	29.3	m/s
Peak velocity pressure, q_p	0.526	kN/m^2
Height above ground	5.600	m
Peak wind velocity, v_p	29.9	m/s
Peak velocity pressure, q_p	0.550	kN/m^2

180°

Height above ground	4.850	m
Peak wind velocity, v_p	23.5	m/s
Peak velocity pressure, q_p	0.338	kN/m^2
Height above ground	5.600	m
Peak wind velocity, v_p	24.0	m/s
Peak velocity pressure, q_p	0.353	kN/m^2

270°

Height above ground	4.850	m
Peak wind velocity, v_p	26.0	m/s
Peak velocity pressure, q_p	0.414	kN/m^2
Height above ground	5.600	m
Peak wind velocity, v_p	26.6	m/s
Peak velocity pressure, q_p	0.433	kN/m^2

Generated Loadcases

40 Wind 270,-Cpe, All

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m^2]	Area [m^2]	Applied Load [kN]
WI 1	E	-0.131	20	-2.6
WI 2	A	-0.515	5	-2.6
	B	-0.344	22	-7.4
	C	-0.215	21	-4.5
WI 3	D	0.258	20	5.1

Average Wall Loads

Windward	0.258	kN/m^2
Leeward	-0.131	kN/m^2
Left	0.000	kN/m^2
Right	-0.305	kN/m^2

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m^2]	Area [m^2]	Applied Load [kN]
RI 1	F180	-1.068	1	-0.7
	F180	-1.068	1	-0.7
	G180	-0.454	1	-0.6
	H180	-0.362	21	-7.5
RI 2	F0	-0.642	1	-0.4
	F0	-0.642	1	-0.4
	G0	-0.440	1	-0.6
	H0	-0.220	21	-4.6

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	3.5	-2.0	2.999	-5.292

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. 1 (1)	6.7	-3.8	2.999	-5.292
St. 2 (2)	3.9	-2.0	2.720	-5.352
St. 3 (3)	0.3	-0.4	3.935	-3.189
Total	**14.5**	**-8.2**	-	-

Total Loads

X 14.5 kN
Y -8.2 kN
Z -15.2 kN

39 Wind 270,Cpi 0.2,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	E	-0.250	20	-4.9
WI 2	A	-0.636	5	-3.2
	B	-0.453	22	-9.8
	C	-0.315	21	-6.6
WI 3	D	0.235	20	4.6

Average Wall Loads

Windward 0.235 kN/m²
Leeward -0.250 kN/m²
Left 0.000 kN/m²
Right -0.411 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F180	-1.225	1	-0.8
	F180	-1.225	1	-0.8
	G180	-0.570	1	-0.7
	H180	-0.473	21	-9.8
RI 2	F0	-0.771	1	-0.5
	F0	-0.771	1	-0.5
	G0	-0.556	1	-0.7
	H0	-0.321	21	-6.7

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	4.7	-2.5	2.762	-5.262
St. 1 (1)	9.1	-4.8	2.762	-5.262
St. 2 (2)	5.3	-2.5	2.472	-5.316
St. 3 (3)	0.5	-0.4	3.656	-3.767
Total	**19.6**	**-10.2**	-	-

Total Loads

X 19.6 kN
Y -10.2 kN
Z -20.2 kN

38 Wind 270,Cpi 0.2,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	E	-0.250	20	-4.9
WI 2	A	-0.636	5	-3.2
	B	-0.453	22	-9.8
	C	-0.315	21	-6.6
WI 3	D	0.235	20	4.6

Average Wall Loads

Windward	0.235	kN/m²
Leeward	-0.250	kN/m²
Left	0.000	kN/m²
Right	-0.411	kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F180	-1.225	1	-0.8
	F180	-1.225	1	-0.8
	G180	-0.570	1	-0.7
	H180	-0.473	21	-9.8
RI 2	F0	-0.047	1	0.0
	F0	-0.047	1	0.0
	G0	-0.047	1	-0.1
	H0	-0.047	21	-1.0

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	4.7	-2.5	2.762	-5.262
St. 1 (1)	9.1	-4.8	2.762	-5.262
St. 2 (2)	5.3	-3.1	2.962	-5.035
St. 3 (3)	0.5	-1.0	3.943	-1.864
Total	**19.6**	**-11.3**	-	-

Total Loads

X	19.6	kN
Y	-11.3	kN
Z	-13.0	kN

37 Wind 270,Cpi -0.3,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	E	-0.034	20	-0.7
WI 2	A	-0.420	5	-2.1
	B	-0.236	22	-5.1
	C	-0.099	21	-2.1
WI 3	D	0.452	20	8.9

Average Wall Loads

Windward	0.452	kN/m²
Leeward	-0.034	kN/m²
Left	0.000	kN/m²
Right	-0.195	kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F180	-1.009	1	-0.7
	F180	-1.009	1	-0.7
	G180	-0.354	1	-0.5
	H180	-0.256	21	-5.3
RI 2	F0	-0.554	1	-0.4
	F0	-0.554	1	-0.4
	G0	-0.340	1	-0.4
	H0	-0.105	21	-2.2

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	2.2	-2.5	4.529	-4.097
St. 1 (1)	4.3	-4.8	4.529	-4.097
St. 2 (2)	2.5	-2.5	4.305	-4.375
St. 3 (3)	0.2	-0.4	4.057	-1.949
Total	**9.3**	**-10.2**	-	-

Total Loads

X	9.3	kN
Y	-10.2	kN
Z	-10.3	kN

36 Wind 270,Cpi -0.3,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	E	-0.034	20	-0.7
WI 2	A	-0.420	5	-2.1
	B	-0.236	22	-5.1
	C	-0.099	21	-2.1
WI 3	D	0.452	20	8.9

Average Wall Loads

Windward	0.452	kN/m²
Leeward	-0.034	kN/m²
Left	0.000	kN/m²
Right	-0.195	kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F180	-1.009	1	-0.7
	F180	-1.009	1	-0.7
	G180	-0.354	1	-0.5

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
	H180	-0.256	21	-5.3
RI 2	F0	0.170	1	0.1
	F0	0.170	1	0.1
	G0	0.170	1	0.2
	H0	0.170	21	3.5

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	2.2	-2.5	4.529	-4.097
St. 1 (1)	4.3	-4.8	4.529	-4.097
St. 2 (2)	2.5	-3.1	4.526	-3.635
St. 3 (3)	0.2	-1.0	3.488	-0.768
Total	**9.3**	**-11.3**	-	-

Total Loads

X 9.3 kN
Y -11.3 kN
Z -3.1 kN

35 Wind 180,-Cpe, All

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	A	-0.390	9	-3.4
	B	-0.260	11	-2.8
WI 2	D	0.272	48	12.9
WI 3	A	-0.390	9	-3.4
	B	-0.260	11	-2.8

Average Wall Loads

Windward 0.272 kN/m²
Leeward 0.000 kN/m²
Left -0.319 kN/m²
Right -0.319 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	FL	-0.640	2	-1.3
	FU	-0.807	2	-1.7
	H	-0.253	17	-4.2
	I	-0.253	2	-0.6
RI 2	FU	-0.807	2	-1.7
	FL	-0.640	2	-1.3
	H	-0.253	17	-4.2
	I	-0.253	2	-0.6

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	-3.1	0.0	0.000	-4.550
St. 1 (1)	-6.0	0.0	0.000	-4.550
St. 2 (2)	-3.5	0.0	0.000	-4.550

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. 3 (3)	-0.3	0.0	0.000	-4.550
Total	**-12.9**	**0.0**	-	-

Total Loads

X -12.9 kN
Y 0.0 kN
Z -15.5 kN

34 Wind 180,Cpi 0.2,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	A	-0.491	9	-4.3
	B	-0.351	11	-3.8
WI 2	D	0.223	48	10.6
WI 3	A	-0.491	9	-4.3
	B	-0.351	11	-3.8

Average Wall Loads

Windward 0.223 kN/m²
Leeward 0.000 kN/m²
Left -0.414 kN/m²
Right -0.414 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	FL	-0.762	2	-1.6
	FU	-0.942	2	-2.0
	H	-0.344	17	-5.8
	I	-0.344	2	-0.8
RI 2	FU	-0.942	2	-2.0
	FL	-0.762	2	-1.6
	H	-0.344	17	-5.8
	I	-0.344	2	-0.8

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	-2.5	0.0	0.000	-4.550
St. 1 (1)	-4.9	0.0	0.000	-4.550
St. 2 (2)	-2.9	0.0	0.000	-4.550
St. 3 (3)	-0.3	0.0	0.000	-4.550
Total	**-10.6**	**0.0**	-	-

Total Loads

X -10.6 kN
Y 0.0 kN
Z -20.0 kN

33 Wind 180,Cpi 0.2,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	A	-0.491	9	-4.3
	B	-0.351	11	-3.8
WI 2	D	0.223	48	10.6
WI 3	A	-0.491	9	-4.3
	B	-0.351	11	-3.8

Average Wall Loads

Windward	0.223	kN/m²
Leeward	0.000	kN/m²
Left	-0.414	kN/m²
Right	-0.414	kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	FL	-0.039	2	-0.1
	FU	-0.039	2	-0.1
	H	-0.039	17	-0.6
	I	-0.039	2	-0.1
RI 2	FU	-0.039	2	-0.1
	FL	-0.039	2	-0.1
	H	-0.039	17	-0.6
	I	-0.039	2	-0.1

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	-2.5	0.0	0.000	-4.550
St. 1 (1)	-4.9	0.0	0.000	-4.550
St. 2 (2)	-2.9	0.0	0.000	-4.550
St. 3 (3)	-0.3	0.0	0.000	-4.550
Total	**-10.6**	**0.0**	-	-

Total Loads

X	-10.6	kN
Y	0.0	kN
Z	-1.8	kN

32 Wind 180,Cpi -0.3,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	A	-0.314	9	-2.8
	B	-0.174	11	-1.9
WI 2	D	0.400	48	19.0
WI 3	A	-0.314	9	-2.8
	B	-0.174	11	-1.9

Average Wall Loads

Windward	0.400	kN/m²
Leeward	0.000	kN/m²
Left	-0.237	kN/m²

Right -0.237 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	FL	-0.585	2	-1.2
	FU	-0.766	2	-1.6
	H	-0.167	17	-2.8
	I	-0.167	2	-0.4
RI 2	FU	-0.766	2	-1.6
	FL	-0.585	2	-1.2
	H	-0.167	17	-2.8
	I	-0.167	2	-0.4

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	-4.5	0.0	0.000	-4.550
St. 1 (1)	-8.8	0.0	0.000	-4.550
St. 2 (2)	-5.2	0.0	0.000	-4.550
St. 3 (3)	-0.5	0.0	0.000	-4.550
Total	**-19.0**	**0.0**	-	-

Total Loads

X -19.0 kN
Y 0.0 kN
Z -11.9 kN

31 Wind 180,Cpi -0.3,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	A	-0.314	9	-2.8
	B	-0.174	11	-1.9
WI 2	D	0.400	48	19.0
WI 3	A	-0.314	9	-2.8
	B	-0.174	11	-1.9

Average Wall Loads

Windward 0.400 kN/m²
Leeward 0.000 kN/m²
Left -0.237 kN/m²
Right -0.237 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	FL	0.138	2	0.3
	FU	0.138	2	0.3
	H	0.138	17	2.3
	I	0.138	2	0.3
RI 2	FU	0.138	2	0.3
	FL	0.138	2	0.3
	H	0.138	17	2.3
	I	0.138	2	0.3

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	-4.5	0.0	0.000	-4.550
St. 1 (1)	-8.8	0.0	0.000	-4.550
St. 2 (2)	-5.2	0.0	0.000	-4.550
St. 3 (3)	-0.5	0.0	0.000	-4.550
Total	**-19.0**	**0.0**	-	-

Total Loads

X -19.0 kN
Y 0.0 kN
Z 6.3 kN

30 Wind 90,-Cpe, All

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	D	0.327	20	6.4
WI 2	A	-0.654	5	-3.3
	B	-0.436	22	-9.4
	C	-0.273	21	-5.7
WI 3	E	-0.166	20	-3.3

Average Wall Loads

Windward 0.327 kN/m²
Leeward -0.166 kN/m²
Left -0.387 kN/m²
Right 0.000 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F0	-0.815	1	-0.5
	F0	-0.815	1	-0.5
	G0	-0.559	1	-0.7
	H0	-0.280	21	-5.8
RI 2	F180	-1.356	1	-0.9
	F180	-1.356	1	-0.9
	G180	-0.576	1	-0.7
	H180	-0.460	21	-9.5

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	4.4	2.5	-0.905	-1.596
St. 1 (1)	8.6	4.8	-0.905	-1.596
St. 2 (2)	5.0	2.5	-0.956	-1.880
St. 3 (3)	0.4	0.5	-0.516	-0.418
Total	**18.4**	**10.4**	-	-

Total Loads

X 18.4 kN
Y 10.4 kN
Z -19.3 kN

29 Wind 90,Cpi 0.2,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	D	0.299	20	5.9
WI 2	A	-0.808	5	-4.0
	B	-0.575	22	-12.4
	C	-0.401	21	-8.4
WI 3	E	-0.318	20	-6.2

Average Wall Loads

Windward	0.299	kN/m²
Leeward	-0.318	kN/m²
Left	-0.522	kN/m²
Right	0.000	kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F0	-0.979	1	-0.6
	F0	-0.979	1	-0.6
	G0	-0.706	1	-0.9
	H0	-0.408	21	-8.4
RI 2	F180	-1.556	1	-1.0
	F180	-1.556	1	-1.0
	G180	-0.724	1	-0.9
	H180	-0.600	21	-12.4

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	6.0	3.1	-0.983	-1.872
St. 1 (1)	11.5	6.1	-0.983	-1.872
St. 2 (2)	6.8	3.1	-1.007	-2.166
St. 3 (3)	0.6	0.6	-0.892	-0.919
Total	**24.8**	**12.9**	-	-

Total Loads

X	24.8	kN
Y	12.9	kN
Z	-25.7	kN

28 Wind 90,Cpi 0.2,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	D	0.299	20	5.9
WI 2	A	-0.808	5	-4.0
	B	-0.575	22	-12.4
	C	-0.401	21	-8.4
WI 3	E	-0.318	20	-6.2

Average Wall Loads

Windward	0.299	kN/m²
Leeward	-0.318	kN/m²
Left	-0.522	kN/m²

Right 0.000 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F0	-0.059	1	0.0
	F0	-0.059	1	0.0
	G0	-0.059	1	-0.1
	H0	-0.059	21	-1.2
RI 2	F180	-1.556	1	-1.0
	F180	-1.556	1	-1.0
	G180	-0.724	1	-0.9
	H180	-0.600	21	-12.4

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	6.0	3.1	-0.983	-1.872
St. 1 (1)	11.5	6.1	-0.983	-1.872
St. 2 (2)	6.8	4.0	-1.015	-1.726
St. 3 (3)	0.6	1.2	0.428	0.202
Total	**24.8**	**14.4**	-	-

Total Loads

X 24.8 kN
Y 14.4 kN
Z -16.5 kN

27 Wind 90,Cpi -0.3,-Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	D	0.574	20	11.3
WI 2	A	-0.533	5	-2.7
	B	-0.300	22	-6.5
	C	-0.126	21	-2.6
WI 3	E	-0.043	20	-0.8

Average Wall Loads

Windward 0.574 kN/m²
Leeward -0.043 kN/m²
Left -0.248 kN/m²
Right 0.000 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F0	-0.704	1	-0.5
	F0	-0.704	1	-0.5
	G0	-0.431	1	-0.6
	H0	-0.133	21	-2.8
RI 2	F180	-1.281	1	-0.8
	F180	-1.281	1	-0.8
	G180	-0.449	1	-0.6
	H180	-0.326	21	-6.7

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	2.8	3.1	0.002	0.002
St. 1 (1)	5.5	6.1	0.002	0.002
St. 2 (2)	3.2	3.1	-0.244	-0.248
St. 3 (3)	0.3	0.6	0.506	0.243
Total	**11.8**	**12.9**	-	-

Total Loads

X 11.8 kN
Y 12.9 kN
Z -13.0 kN

26 Wind 90,Cpi -0.3,+Cpe

Wall Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
WI 1	D	0.574	20	11.3
WI 2	A	-0.533	5	-2.7
	B	-0.300	22	-6.5
	C	-0.126	21	-2.6
WI 3	E	-0.043	20	-0.8

Average Wall Loads

Windward 0.574 kN/m²
Leeward -0.043 kN/m²
Left -0.248 kN/m²
Right 0.000 kN/m²

Roof Zone Loads

Reference	Zone	Nett Pressure [kN/m²]	Area [m²]	Applied Load [kN]
RI 1	F0	0.216	1	0.1
	F0	0.216	1	0.1
	G0	0.216	1	0.3
	H0	0.216	21	4.5
RI 2	F180	-1.281	1	-0.8
	F180	-1.281	1	-0.8
	G180	-0.449	1	-0.6
	H180	-0.326	21	-6.7

Lateral Loads by Level

Level	Total [kN]		Centre [m]	
	X	Y	X	Y
St. Base (Base)	2.8	3.1	0.002	0.002
St. 1 (1)	5.5	6.1	0.002	0.002
St. 2 (2)	3.2	4.0	0.083	0.067
St. 3 (3)	0.3	1.2	1.576	0.347
Total	**11.8**	**14.4**	-	-

Total Loads

X 11.8 kN
Y 14.4 kN
Z -3.9 kN

Action Codes

General Loading	BS EN 1991-1-1 + UK NA (2002)
Wind Loading	BS EN 1991-1-4 + UK NA (2005)
Snow Loading	BS EN 1991-1-3 + UK NA (2003)
Seismic Loading	BS EN 1998-1 + UK NA (2004)
Combinations	BS EN 1990 + UK NA (2002)

Resistance Codes

Steel Design	BS EN 1993-1-1 + UK NA (2005)
Concrete Design	BS EN 1992-1-1 + UK NA (2004)
Composite Design	BS EN 1994-1-1 + UK NA (2004)
Timber Design	BS EN 1995-1-1 + UK NA (2004)
Masonry Design	BS EN 1996-1-1 + UK NA (2005)
Foundation Design	BS EN 1997-1 + UK NA (2004)
Seismic Design and Detailing	BS EN 1998-1 + UK NA (2004)

Analysis

Element-Member

No.	Node 1	Node 2	Member	Construction	Material	Section	Grade
6	100	81	SBR 2.1/%18-2.1/A/#263	Steel brace	Steel	CHS 114.3x3.6	S275
8	88	17	SB 1/B/1-1/B/2	Non-composite beam	Steel	UB 203x133x30	S275
10	95	88	SB 1/B/1-1/B/2	Non-composite beam	Steel	UB 203x133x30	S275
12	61	19	SBR 2/A/1-2/B/1	Steel brace	Steel	CHS 114.3x3.6	S275
13	52	53	ELM Base/A/#70-1/A/#71	Bearing wall column	General	Wall Column Section	Bearing Wall
17	5	20	SBR IP 4/C/2-IP 4/B/3	Steel brace	Steel	CHS 114.3x3.6	S275
24	46	47	ELM Base/A/#64-1/A/#65	Bearing wall column	General	Wall Column Section	Bearing Wall
36	19	137	SBR IP 3/B/1-IP 3/C/#264	Steel brace	Steel	CHS 114.3x3.6	S275
38	12	89	SC 8/A	Non-composite column	Steel	UKB 254x146x31	S275
40	19	22	SBR 2/B/1-2/C/1	Steel brace	Steel	CHS 114.3x3.6	S275
41	93	86	SB 1/C/1-1/C/2	Non-composite beam	Steel	UB 203x133x30	S275
42	54	55	ELM Base/A/#72-1/A/#73	Bearing wall column	General	Wall Column Section	Bearing Wall
44	53	55	ELM 1/A/#71-1/A/#73	Bearing wall beam	General	Wall Beam Section	Bearing Wall
46	20	21	SBR 2/B/3-2/C/3	Steel brace	Steel	CHS 114.3x3.6	S275
47	158	81	SB IP 4/A/7-IP 4/A/4	Non-composite beam	Steel	UB 203x133x25	S275
48	34	35	ELM Base/A/#54-1/A/#55	Bearing wall column	General	Wall Column Section	Bearing Wall
51	39	41	ELM 1/A/#59-1/A/#61	Bearing wall beam	General	Wall Beam Section	Bearing Wall
56	69	22	SC C/1	Non-composite column	Steel	UKB 254x146x31	S275
57	8	148	SB 3/4/B-2/B/1	Non-composite beam	Steel	UB 203x133x25	S275
59	118	96	SB 1/B/2-1/B/3	Non-composite beam	Steel	UB 203x133x30	S275
66	37	39	ELM 1/A/#57-1/A/#59	Bearing wall beam	General	Wall Beam Section	Bearing Wall
77	45	47	ELM 1/A/4-1/A/#65	Bearing wall beam	General	Wall Beam Section	Bearing Wall
85	11	137	SB 3/C/4-2/C/1	Non-composite beam	Steel	UB 203x133x25	S275
86	43	45	ELM 1/A/#63-1/A/4	Bearing wall beam	General	Wall Beam Section	Bearing Wall
87	62	144	SC C/3	Non-composite column	Steel	UKB 254x146x31	S275
92	27	35	ELM 1/A/#53-1/A/#55	Bearing wall beam	General	Wall Beam Section	Bearing Wall
95	49	51	ELM 1/A/#67-1/A/#69	Bearing wall beam	General	Wall Beam Section	Bearing Wall
109	50	51	ELM Base/A/#68-1/A/#69	Bearing wall column	General	Wall Column Section	Bearing Wall
111	42	43	ELM Base/A/#62-1/A/#63	Bearing wall column	General	Wall Column Section	Bearing Wall
123	144	21	SC C/3	Non-composite column	Steel	UKB 254x146x31	S275
130	55	57	ELM 1/A/#73-1/A/#75	Bearing wall beam	General	Wall Beam Section	Bearing Wall
131	95	117	SB 1/B/1-1/C/1	Non-composite beam	Steel	UB 152x89x16	S275
133	5	8	SBR IP 4/C/2-3/B/4	Steel brace	Steel	CHS 114.3x3.6	S275

No.	Node 1	Node 2	Member	Construction	Material	Section	Grade
135	31	23	ELM Base/A/3-1/A/3	Bearing wall column	General	Wall Column Section	Bearing Wall
138	96	21	SBR 1/3/B-2/3/C	Steel brace	Steel	RSA 70x70x10	S275
146	81	119	SB IP 4/A/7-IP 4/A/4	Non-composite beam	Steel	UB 203x133x25	S275
149	41	43	ELM 1/A/#61-1/A/#63	Bearing wall beam	General	Wall Beam Section	Bearing Wall
150	23	27	ELM 1/A/3-1/A/#53	Bearing wall beam	General	Wall Beam Section	Bearing Wall
151	96	140	SB 1/B/3-1/C/3	Non-composite beam	Steel	UB 152x89x16	S275
156	89	158	SB IP 4/A/7-IP 4/A/4	Non-composite beam	Steel	UB 203x133x25	S275
158	25	27	ELM Base/A/#52-1/A/#53	Bearing wall column	General	Wall Column Section	Bearing Wall
159	6	17	SC B/2	Non-composite column	Steel	UC 152x152x23	S275
162	149	20	SC B/3	Non-composite column	Steel	UKB 254x146x31	S275
163	5	11	SB 2/C/3-3/C/4	Non-composite beam	Steel	UB 203x133x25	S275
173	96	149	SC B/3	Non-composite column	Steel	UKB 254x146x31	S275
175	137	22	SB 3/C/4-2/C/1	Non-composite beam	Steel	UB 203x133x25	S275
201	119	84	SB IP 3/A/4-IP 3/A/6	Non-composite beam	Steel	UB 203x133x25	S275
203	61	2	SB IP 3/A/4-IP 3/A/6	Non-composite beam	Steel	UB 203x133x25	S275
210	40	41	ELM Base/A/#60-1/A/#61	Bearing wall column	General	Wall Column Section	Bearing Wall
213	93	69	SC C/1	Non-composite column	Steel	UKB 254x146x31	S275
217	10	29	SC B/3	Non-composite column	Steel	UKB 254x146x31	S275
223	47	49	ELM 1/A/#65-1/A/#67	Bearing wall beam	General	Wall Beam Section	Bearing Wall
236	17	118	SB 1/B/2-1/B/3	Non-composite beam	Steel	UB 203x133x30	S275
241	35	37	ELM 1/A/#55-1/A/#57	Bearing wall beam	General	Wall Beam Section	Bearing Wall
249	85	84	SBR 2.1/%17-2.1/A/#262	Steel brace	Steel	CHS 114.3x3.6	S275
256	21	5	SB 2/C/3-3/C/4	Non-composite beam	Steel	UB 203x133x25	S275
269	78	93	SC C/1	Non-composite column	Steel	UKB 254x146x31	S275
315	86	92	SB 1/C/1-1/C/2	Non-composite beam	Steel	UB 203x133x30	S275
321	1	131	SC C/3	Non-composite column	Steel	UKB 254x146x31	S275
325	131	62	SC C/3	Non-composite column	Steel	UKB 254x146x31	S275
326	56	57	ELM Base/A/#74-1/A/#75	Bearing wall column	General	Wall Column Section	Bearing Wall
327	58	59	ELM Base/A/1-1/A/1	Bearing wall column	General	Wall Column Section	Bearing Wall
331	20	80	SB 2/B/3-3/4/B	Non-composite beam	Steel	UB 203x133x25	S275
335	68	2	SC A/6	Non-composite column	Steel	UKB 254x146x31	S275
336	62	20	SBR 1/3/C-2/3/B	Steel brace	Steel	RSA 70x70x10	S275

No.	Node 1	Node 2	Member	Construction	Material	Section	Grade
338	38	39	ELM Base/A/#58-1/A/#59	Bearing wall column	General	Wall Column Section	Bearing Wall
339	84	61	SB IP 3/A/4-IP 3/A/6	Non-composite beam	Steel	UB 203x133x25	S275
344	157	19	SC B/1	Non-composite column	Steel	UKB 254x146x31	S275
347	57	59	ELM 1/A/#75-1/A/1	Bearing wall beam	General	Wall Beam Section	Bearing Wall
363	4	78	SC C/1	Non-composite column	Steel	UKB 254x146x31	S275
365	93	19	SBR 1/1/C-2/1/B	Steel brace	Steel	RSA 70x70x10	S275
366	95	22	SBR 1/1/B-2/1/C	Steel brace	Steel	RSA 70x70x10	S275
369	87	5	SC C/2	Non-composite column	Steel	UKB 203x133x25	S275
374	92	87	SC C/2	Non-composite column	Steel	UKB 203x133x25	S275
379	140	62	SB 1/B/3-1/C/3	Non-composite beam	Steel	UB 152x89x16	S275
381	36	37	ELM Base/A/#56-1/A/#57	Bearing wall column	General	Wall Column Section	Bearing Wall
382	80	8	SB 2/B/3-3/4/B	Non-composite beam	Steel	UB 203x133x25	S275
384	44	45	ELM Base/A/4-1/A/4	Bearing wall column	General	Wall Column Section	Bearing Wall
386	92	124	SB 1/C/2-1/C/3	Non-composite beam	Steel	UB 203x133x30	S275
388	13	30	SC C/2	Non-composite column	Steel	UKB 203x133x25	S275
390	158	20	SBR 2/A/3-2/B/3	Steel brace	Steel	CHS 114.3x3.6	S275
394	154	12	SC 8/A	Non-composite column	Steel	UKB 254x146x31	S275
397	7	116	SC B/1	Non-composite column	Steel	UKB 254x146x31	S275
399	51	53	ELM 1/A/#69-1/A/#71	Bearing wall beam	General	Wall Beam Section	Bearing Wall
400	117	93	SB 1/B/1-1/C/1	Non-composite beam	Steel	UB 152x89x16	S275
401	95	157	SC B/1	Non-composite column	Steel	UKB 254x146x31	S275
402	29	96	SC B/3	Non-composite column	Steel	UKB 254x146x31	S275
403	48	49	ELM Base/A/#66-1/A/#67	Bearing wall column	General	Wall Column Section	Bearing Wall
410	148	19	SB 3/4/B-2/B/1	Non-composite beam	Steel	UB 203x133x25	S275
413	30	92	SC C/2	Non-composite column	Steel	UKB 203x133x25	S275
414	83	68	SC A/6	Non-composite column	Steel	UKB 254x146x31	S275
423	124	62	SB 1/C/2-1/C/3	Non-composite beam	Steel	UB 203x133x30	S275
426	16	6	SC B/2	Non-composite column	Steel	UC 152x152x23	S275
431	116	95	SC B/1	Non-composite column	Steel	UKB 254x146x31	S275
432	137	8	SBR FRM C/#215/#265-IP 3/B/4	Steel brace	Steel	CHS 114.3x3.6	S275

Sway

First-order linear

Ref.	Combination Dir 1	Stack Dir 1	$\alpha_{Dir\ 1}$	Combination Dir 2	Stack Dir 2	$\alpha_{Dir\ 2}$	Combination Dir 1/2	Twist	Status
SC A/6	1 STR_1-1.35G+1.5Q+1.5RQ	1	22873.626	1 STR_1-1.35G+1.5Q+1.5RQ	1	94.604	1 STR_1-1.35G+1.5Q+1.5RQ	1.000	✓ Pass
SC 8/A	1 STR_1-1.35G+1.5Q+1.5RQ	1	22873.626	1 STR_1-1.35G+1.5Q+1.5RQ	1	94.604	1 STR_1-1.35G+1.5Q+1.5RQ	1.000	✓ Pass
SC C/3	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.751	1 STR_1-1.35G+1.5Q+1.5RQ	1	11.439	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.016	✓ Pass
SC C/1	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.639	1 STR_1-1.35G+1.5Q+1.5RQ	1	11.439	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.017	✓ Pass
SC C/2	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.702	1 STR_1-1.35G+1.5Q+1.5RQ	1	11.439	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.000	✓ Pass
SC B/1	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.639	1 STR_1-1.35G+1.5Q+1.5RQ	1	10.628	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.012	✓ Pass
SC B/3	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.751	1 STR_1-1.35G+1.5Q+1.5RQ	1	10.628	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.012	✓ Pass
SC B/2	1 STR_1-1.35G+1.5Q+1.5RQ	1	7.702	1 STR_1-1.35G+1.5Q+1.5RQ	1	10.628	32 $STR_{9.1}$-G+1.5W+EHF_{Dir2+}	1.000	✓ Pass

Wind Drift

Reference	Combination	Stack	Deflection [mm]		Drift [mm]		Ratio		Status	
			Dir 1	Dir 2	Dir 1	Dir 2	Dir 1	Dir 2	Dir 1	Dir 2
SC C/1	19 $STR_{8.3}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W	1	13.8	7.3	13.8	7.3	180.826	342.546	✓ Pass	✓ Pass
SC B/2	20 $STR_{8.4}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W	1	12.4	4.8	12.4	4.8	201.330	518.167	✓ Pass	✓ Pass
SC C/2	20 $STR_{8.4}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W	1	12.4	5.9	12.4	5.9	201.330	425.458	✓ Pass	✓ Pass
SC B/1	19 $STR_{8.3}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W	1	13.8	6.2	13.8	6.2	180.826	404.509	✓ Pass	✓ Pass
SC C/3	20 $STR_{8.4}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W	1	11.4	5.9	11.4	5.9	219.551	425.458	✓ Pass	✓ Pass

Steelwork design

Beam Design

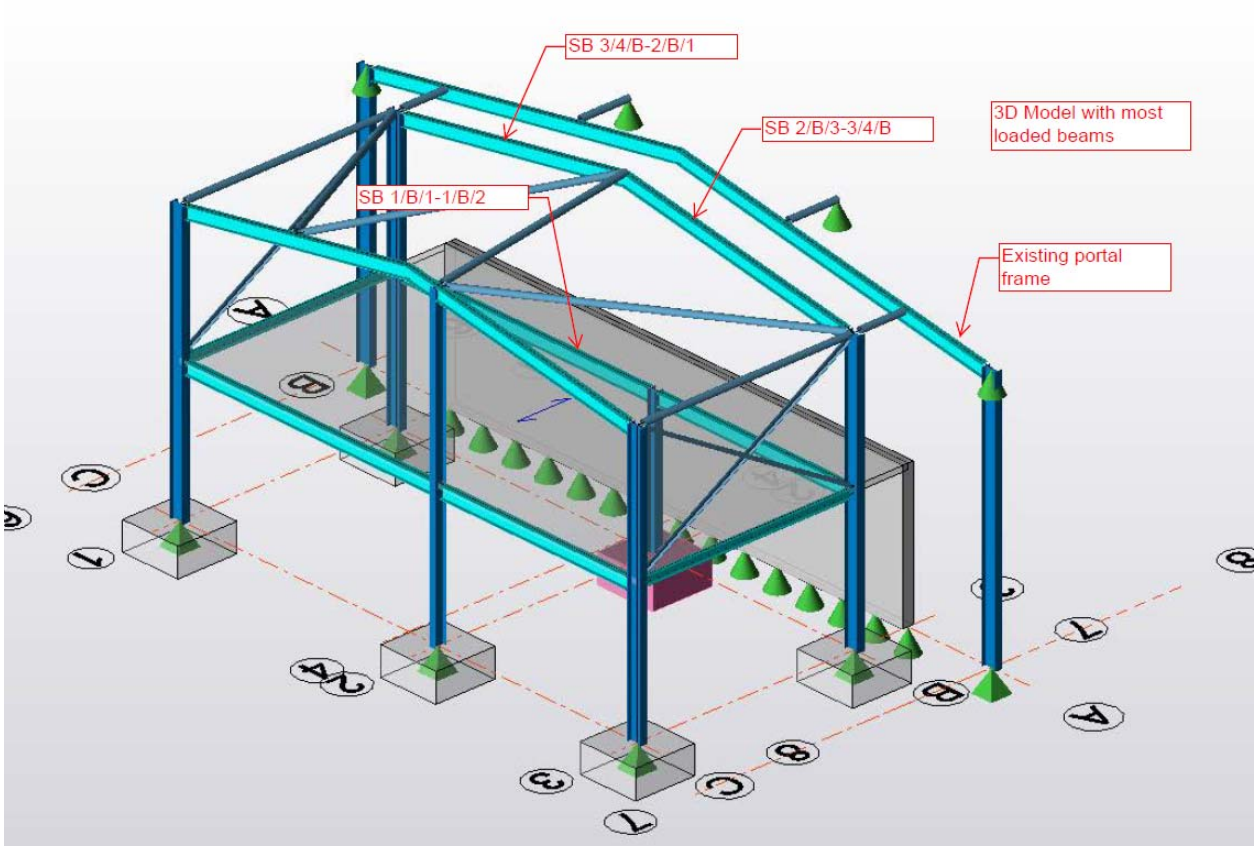
Beam Design Summary

Static

Member Reference	Group Ref.	Span	Section	Grade	Length [m]	No. Connectors	Utilization	Status
SB 3/4/B-2/B/1	SBR4	1	UB 203x133x25	S275	4.611		0.393	✓ Pass
SB 2/B/3-3/4/B	SBR5	1	UB 203x133x25	S275	4.611		0.421	✓ Pass
SB 2/C/3-3/C/4	SBR3	1	UB 203x133x25	S275	4.611		0.240	✓ Pass
SB 3/C/4-2/C/1	SBR4	1	UB 203x133x25	S275	4.611		0.233	✓ Pass
SB 1/B/1-1/C/1	SBR9	1	UB 152x89x16	S275	4.050		0.016	✓ Pass
SB 1/C/2-1/C/3	SBR7	1	UB 203x133x30	S275	3.970		0.303	✓ Pass
SB 1/C/1-1/C/2	SBR6	1	UB 203x133x30	S275	5.130		0.645	✓ Pass
SB 1/B/3-1/C/3	SBR9	1	UB 152x89x16	S275	4.050		0.016	✓ Pass

Member Reference	Group Ref.	Span	Section	Grade	Length [m]	No. Connectors	Utilization	Status
SB 1/B/1-1/B/2	SBR6	1	UB 203x133x30	S275	5.130		0.805	✔ Pass
SB 1/B/2-1/B/3	SBR7	1	UB 203x133x30	S275	3.970		0.378	✔ Pass
SB IP 3/A/4-IP 3/A/6	SBR13	1	UB 203x133x25	S275	6.304		0.548	✔ Pass
SB IP 4/A/7-IP 4/A/4	SBR13	1	UB 203x133x25	S275	6.304		0.448	✔ Pass

Most loaded beam design



SB 1/B/1-1/B/2

Restraints

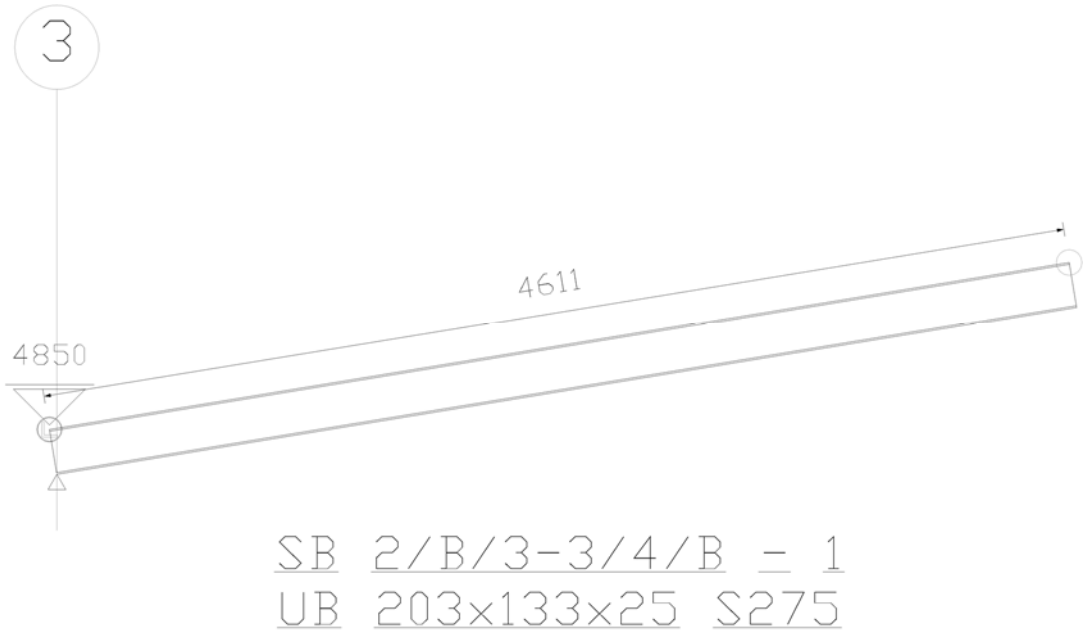
Source	Distance / Length [m]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0.000	●		●		●		●	
sub-beam	5.130	●	1.000		1.000		1.000		1.000
support	5.130	●		●		●		●	

Static

Summary UB 203x133x30(S275)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	1	38.5	231.4	kN	0.166	✓ Pass
Shear Minor	-	No	Forces	kN	-	Not required
Buckling Shear Web	-	29.313	66.558	-	-	✓ Pass
Moment Major	1	49.3	86.5	kNm	0.571	✓ Pass
Moment Minor	-	No	Forces	kNm	-	Not required
Axial	-	No	Forces	kN	-	Not required
Axial Bending Combined	-	No	Forces	-	-	Not required
Buckling Lateral Torsional	-	-	-	-	-	Not required
Buckling Compression	-	No	Forces	-	-	Not required
Buckling Combined	-	No	Forces	-	-	Not required
Torsion	-	No	Significant	Forces	-	Not required
Deflection Self weight	1	0.4	-	mm	-	-
Deflection Slab	-	No	Loads	mm	-	Not required
Deflection Dead	1	3.6	10.3	mm	0.354	✓ Pass
Deflection Imposed	1	11.5	14.3	mm	0.805	✓ Pass
Deflection Wind	-	No	Loads	mm	-	Not required
Deflection Total	1	15.5	25.7	mm	0.606	✓ Pass

SB 2/B/3-3/4/B



Restraints

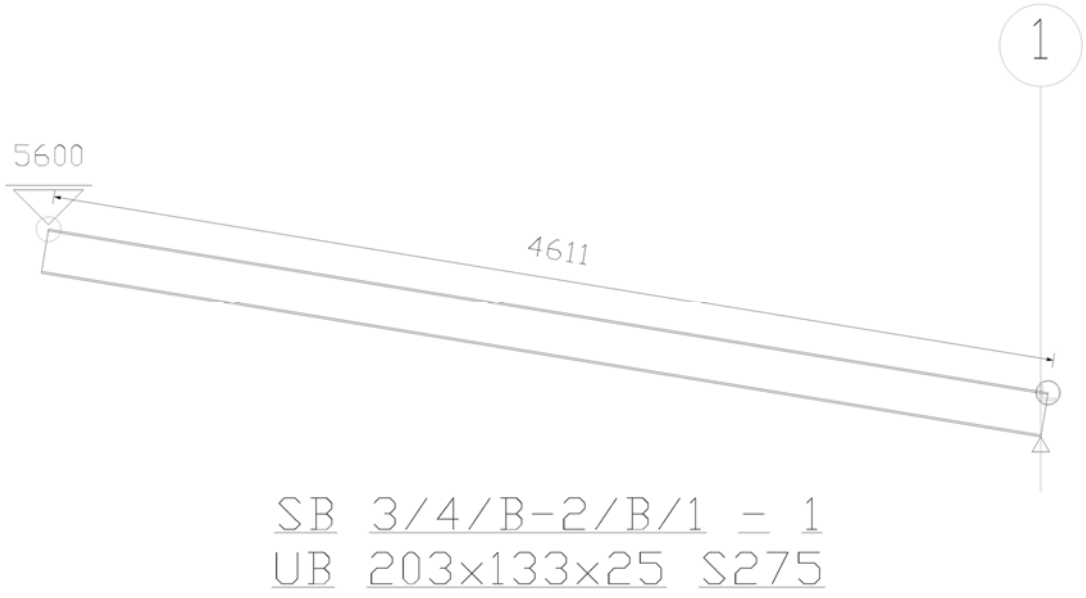
Source	Distance / Length [m]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0.000	●		●		●		●	
sub-beam	4.611	●	1.000		1.000		1.000		1.000
support	4.611	●		●		●		●	

Static

Summary UB 203x133x25(S275)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	7	15.4	203.5	kN	0.076	✓ Pass
Shear Minor	-	No	Forces	kN	-	Not required
Buckling Shear Web	-	32.912	66.558	-	-	✓ Pass
Moment Major	2	-23.7	70.9	kNm	0.334	✓ Pass
Moment Minor	-	No	Forces	kNm	-	Not required
Axial	2	15.2	879.1	kN	0.017	✓ Pass
Axial Bending Combined	2	-	-	-	0.334	✓ Pass
Buckling Lateral Torsional	2	-23.7	70.9	kNm	0.334	✓ Pass
Buckling Compression	2	15.2	241.5	kN	0.063	✓ Pass
Buckling Combined	2	-	-	-	0.421	✓ Pass
Deflection Self weight	1	0.1	-	mm	-	-
Deflection Slab	-	No	Loads	mm	-	Not required
Deflection Dead	1	0.3	9.2	mm	0.033	✓ Pass
Deflection Imposed	1	0.7	12.8	mm	0.057	✓ Pass
Deflection Wind	19	-2.4	23.1	mm	0.103	✓ Pass
Deflection Total	27	2.3	23.1	mm	0.099	✓ Pass

SB 3/4/B-2/B/1



Restraints

Source	Distance / Length [m]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0.000	●		●		●		●	
sub-beam	4.611	●	1.000		1.000		1.000		1.000
support	4.611	●		●		●		●	

Static

Summary UB 203x133x25(S275)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	7	-15.1	203.5	kN	0.074	✓ Pass
Shear Minor	-	No	Forces	kN	-	Not required
Buckling Shear Web	-	32.912	66.558	-	-	✓ Pass
Moment Major	12	-21.9	70.9	kNm	0.310	✓ Pass
Moment Minor	-	No	Forces	kNm	-	Not required
Axial	7	14.8	879.1	kN	0.017	✓ Pass
Axial Bending Combined	12	-	-	-	0.310	✓ Pass
Buckling Lateral Torsional	12	-21.9	70.9	kNm	0.310	✓ Pass
Buckling Compression	7	14.8	241.5	kN	0.061	✓ Pass
Buckling Combined	12	-	-	-	0.393	✓ Pass
Deflection Self weight	1	0.2	-	mm	-	-
Deflection Slab	-	No	Loads	mm	-	Not required
Deflection Dead	1	0.3	9.2	mm	0.037	✓ Pass
Deflection Imposed	1	0.8	12.8	mm	0.064	✓ Pass
Deflection Wind	17	2.1	23.1	mm	0.089	✓ Pass
Deflection Total	17	2.9	23.1	mm	0.124	✓ Pass

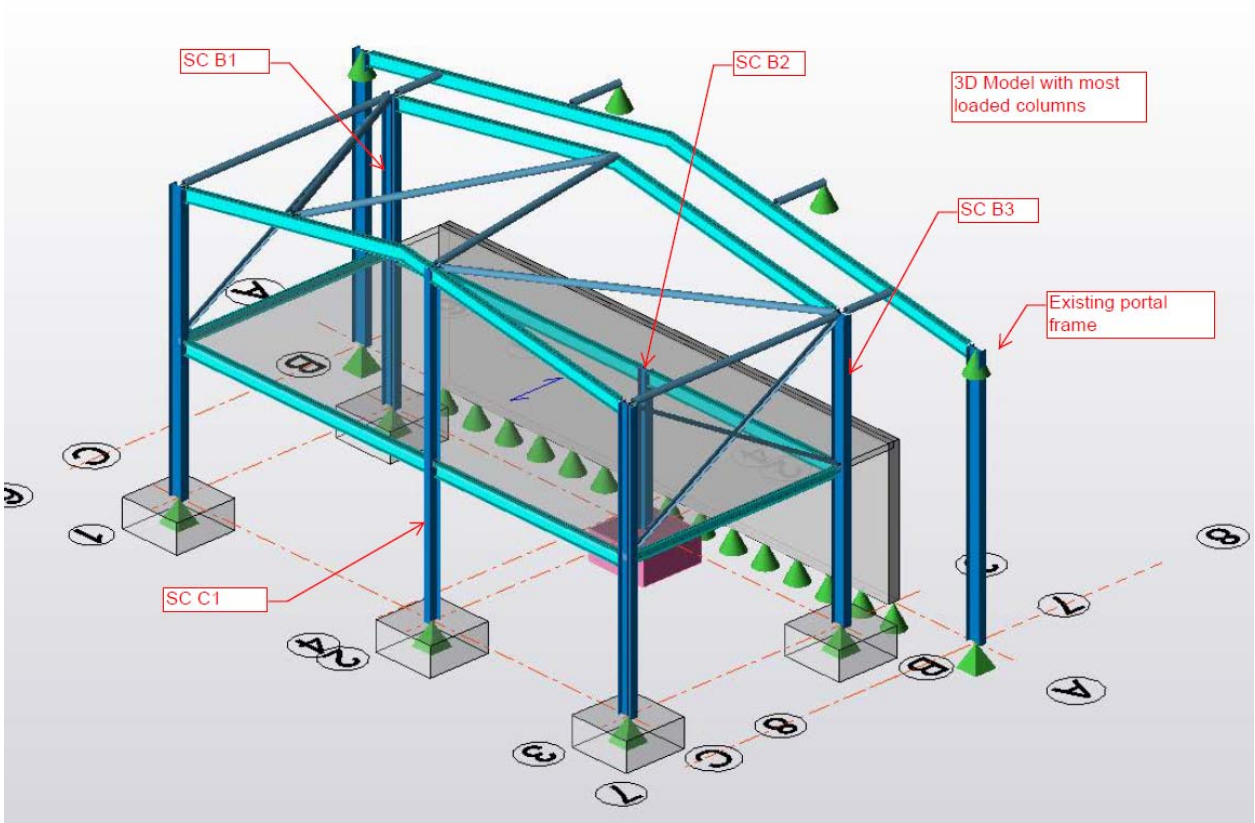
Column Design

Column Design Summary

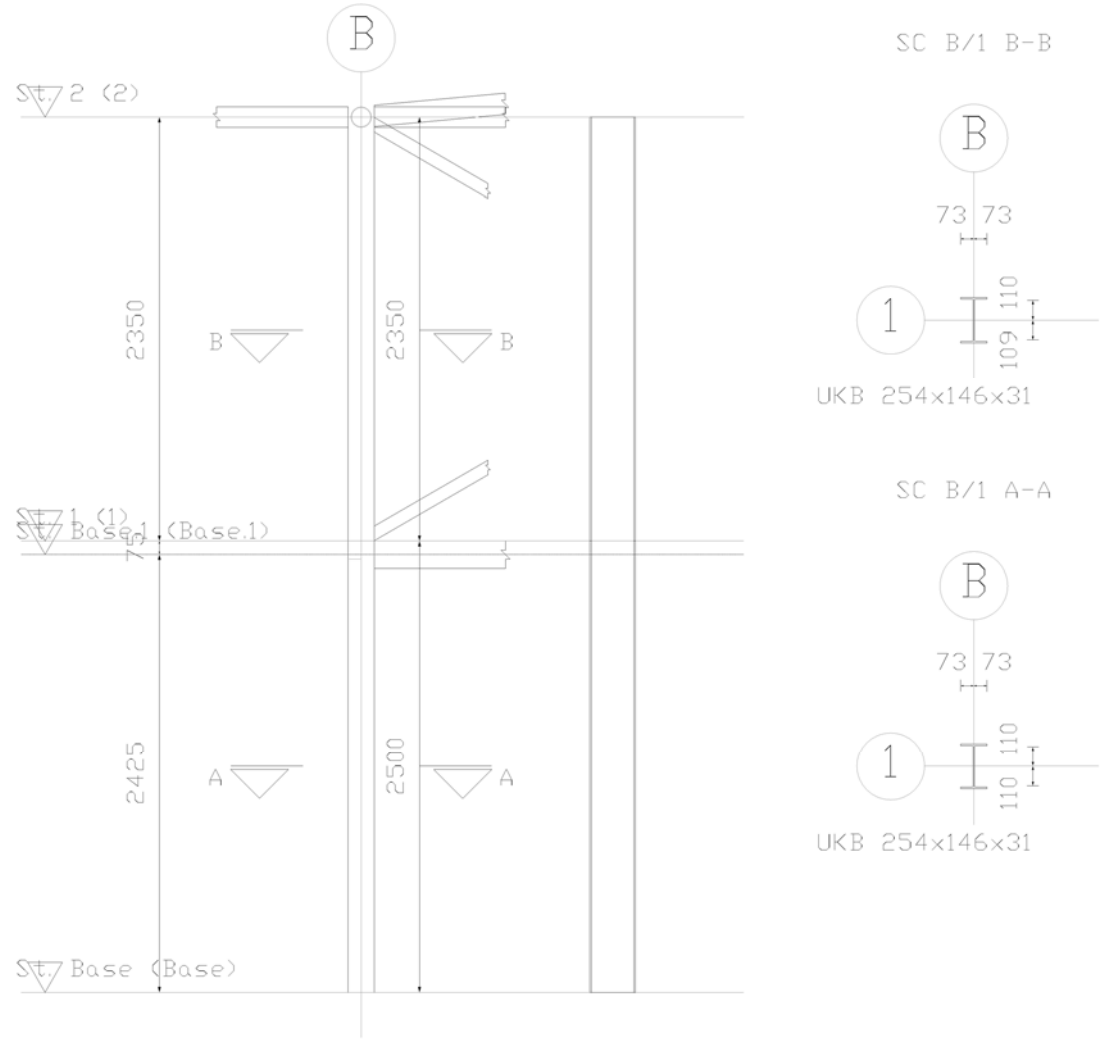
Static

Member Reference	Group Ref.	Stack	Section	Grade	Length [m]	Utilization	Status
SC C/3	SCR4	1	UKB 254x146x31	S275	2.500	0.233	Pass
SC C/3	SCR4	2	UKB 254x146x31	S275	2.350	0.233	Pass
SC C/1	SCR4	1	UKB 254x146x31	S275	2.500	0.239	Pass
SC C/1	SCR4	2	UKB 254x146x31	S275	2.350	0.239	Pass
SC B/1	SCR5	1	UKB 254x146x31	S275	2.500	0.311	Pass
SC B/1	SCR5	2	UKB 254x146x31	S275	2.350	0.311	Pass
SC B/3	SCR5	1	UKB 254x146x31	S275	2.500	0.320	Pass
SC B/3	SCR5	2	UKB 254x146x31	S275	2.350	0.320	Pass
SC C/2	SCR3	1	UKB 203x133x25	S275	2.500	0.158	Pass
SC C/2	SCR3	2	UKB 203x133x25	S275	3.004	0.056	Pass
SC B/2	SCR2	1	UC 152x152x23	S275	2.500	0.187	Pass
SC 8/A	SCR6	1	UKB 254x146x31	S275	4.575	0.143	Pass
SC A/6	SCR7	1	UKB 254x146x31	S275	4.575	0.144	Pass

Most loaded column design



SC B/1



SC B/1

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	4.850	Yes		Yes	
	sub-beam	2.350	No	1.000	No	1.000
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Strut Restraints

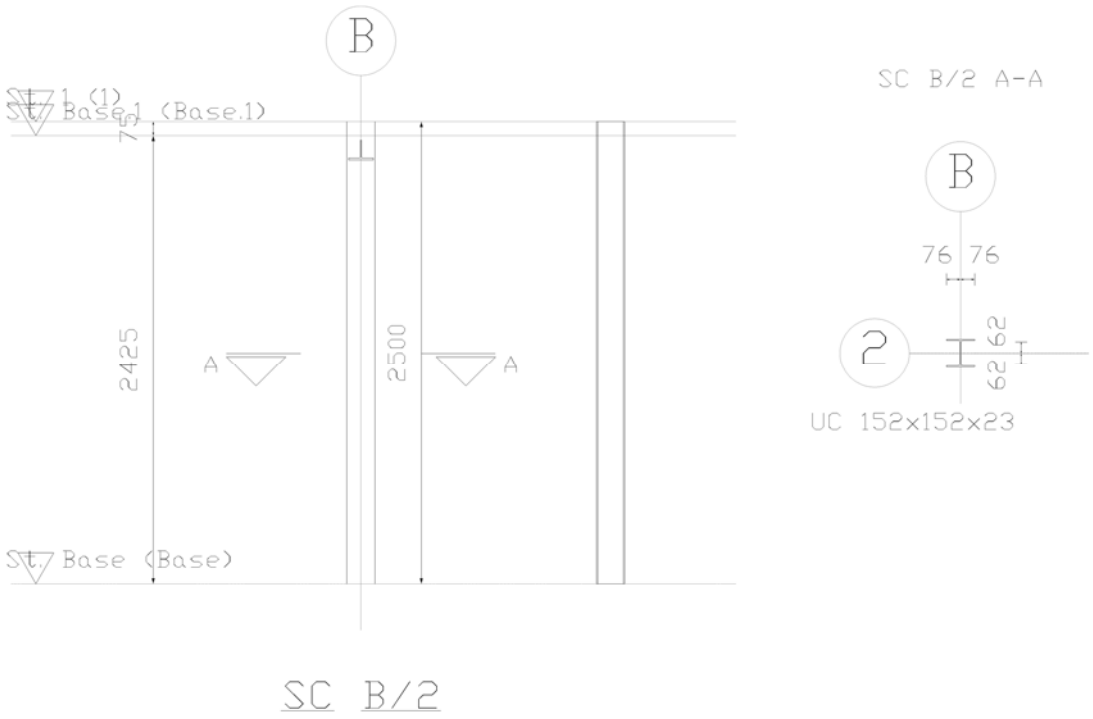
Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	4.850	Yes		Yes	
	sub-beam	2.350	No	1.000	No	1.000
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Static

Summary UKB 254x146x31(S275)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	7	-12.8	259.9	kN	0.049	✓ Pass
Shear Minor	35	3.3	370.0	kN	0.009	✓ Pass
Buckling Shear Web	-	39.03	66.56	-	-	✓ Pass
Moment Major	12	-21.9	108.1	kNm	0.203	✓ Pass
Moment Minor	19	6.0	25.9	kNm	0.234	✓ Pass
Axial	7	63.1	1091.1	kN	0.058	✓ Pass
Axial Bending Combined	19	-	-	-	0.240	✓ Pass
Buckling Lateral Torsional	12	-21.9	105.4	kNm	0.208	✓ Pass
Buckling Compression	7	63.1	751.1	kN	0.084	✓ Pass
Buckling Combined	22	-	-	-	0.311	✓ Pass

SC B/2



SC B/2

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Strut Restraints

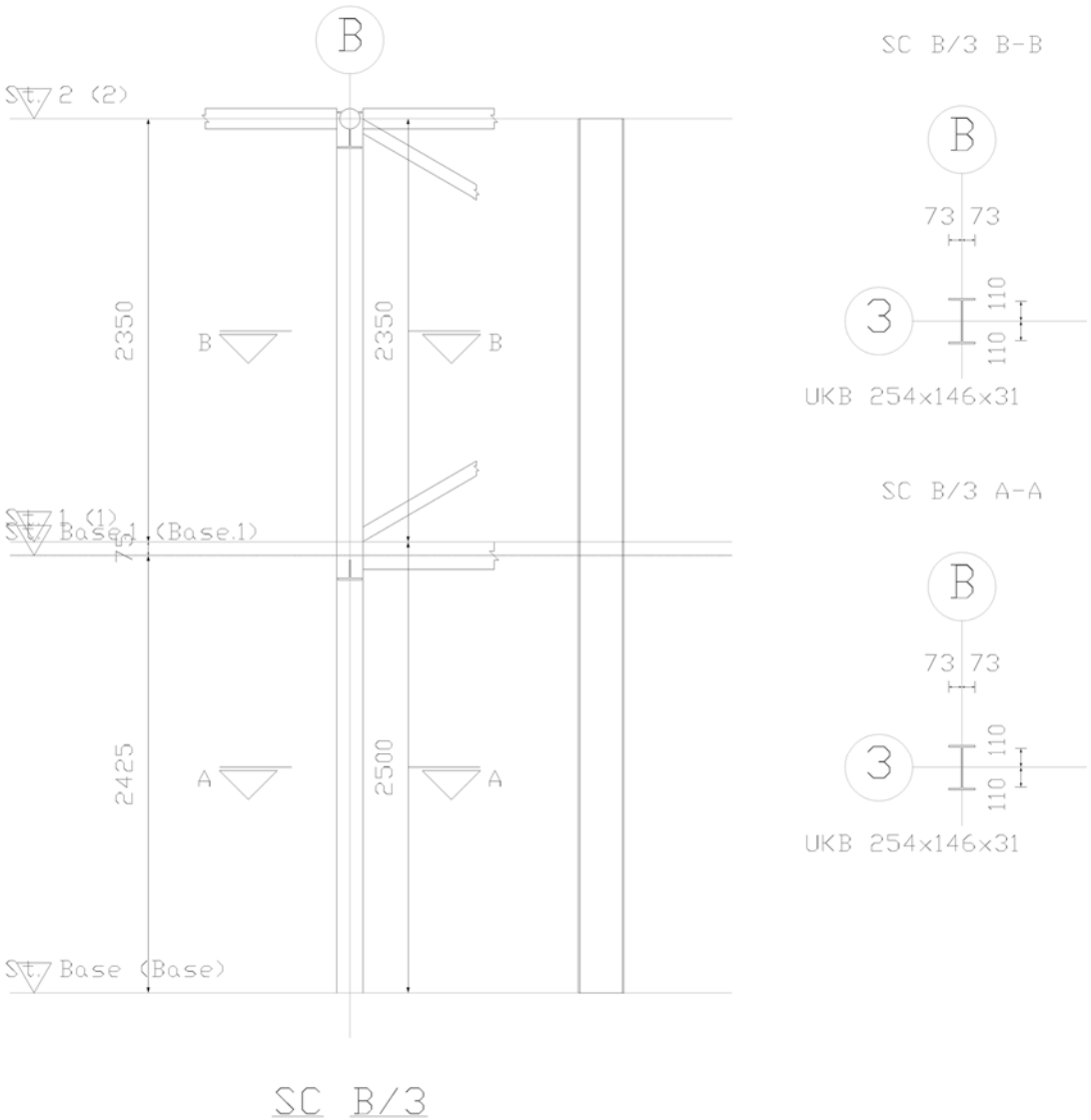
Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Static

Summary UC 152x152x23(S275)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 3	-	-	-	✓ Pass
Shear Major	32	0.6	158.4	kN	0.003	✓ Pass
Shear Minor	No	Significant	Forces	kN	-	Not required
Buckling Shear Web	-	23.93	66.56	-	-	✓ Pass
Moment Major	17	-1.7	45.1	kNm	0.038	✓ Pass
Moment Minor	19	-1.0	14.5	kNm	0.069	✓ Pass
Axial	1	69.0	804.2	kN	0.086	✓ Pass
Axial Bending Combined	19	-	-	-	0.172	✓ Pass
Buckling Lateral Torsional	17	-1.7	44.1	kNm	0.038	✓ Pass
Buckling Compression	1	69.0	543.3	kN	0.127	✓ Pass
Buckling Combined	4	-	-	-	0.187	✓ Pass

SC B/3



SC B/3

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	4.850	Yes		Yes	
	sub-beam	2.350	No	1.000	No	1.000
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Strut Restraints

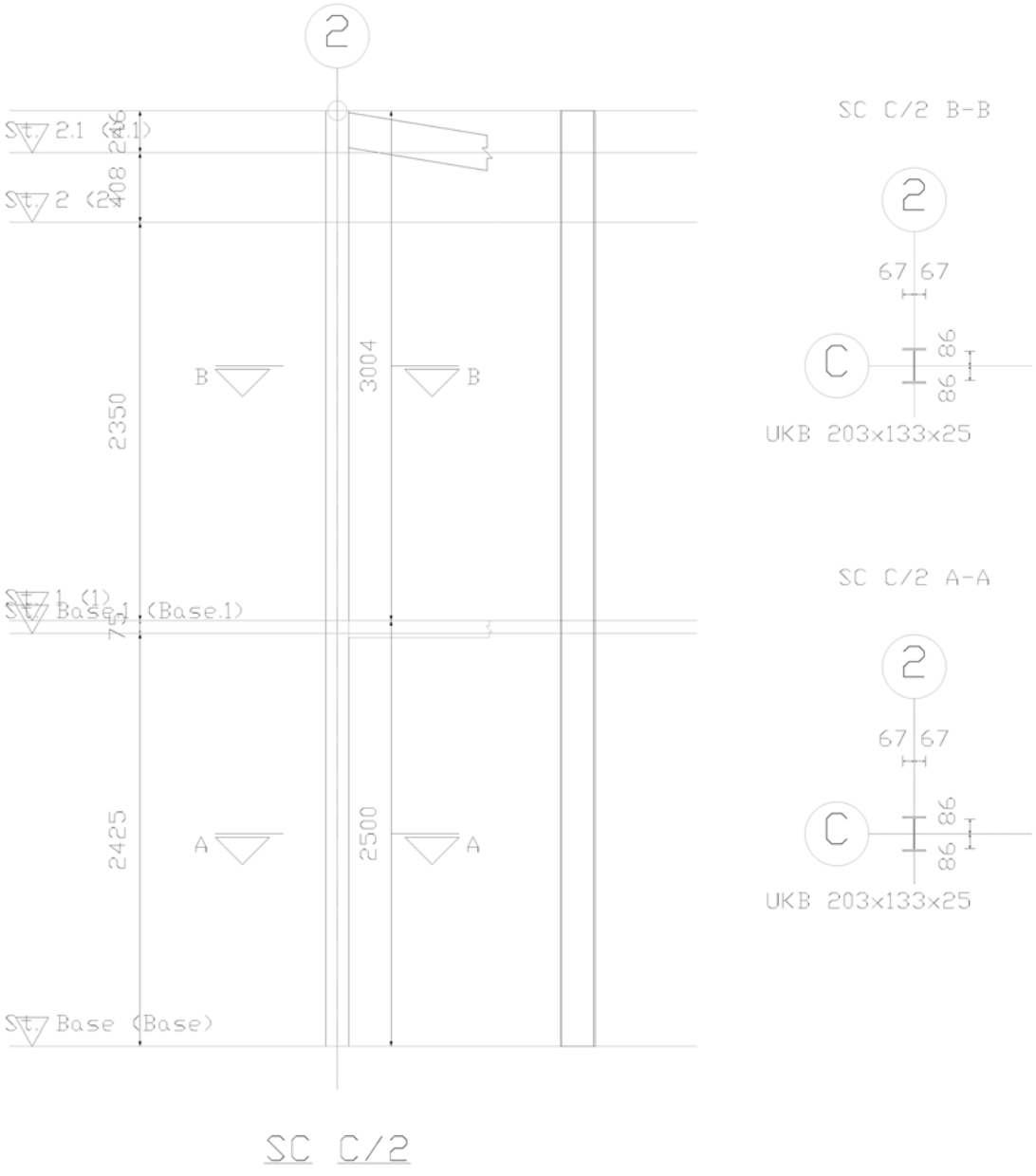
Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	4.850	Yes		Yes	
	sub-beam	2.350	No	1.000	No	1.000
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Static

Summary UKB 254x146x31(S275)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	7	13.7	259.9	kN	0.053	✓ Pass
Shear Minor	20	2.8	370.0	kN	0.007	✓ Pass
Buckling Shear Web	-	39.03	66.56	-	-	✓ Pass
Moment Major	2	23.7	108.1	kNm	0.219	✓ Pass
Moment Minor	20	5.1	25.9	kNm	0.198	✓ Pass
Axial	7	51.3	1091.1	kN	0.047	✓ Pass
Axial Bending Combined	19	-	-	-	0.199	✓ Pass
Buckling Lateral Torsional	2	23.7	105.3	kNm	0.225	✓ Pass
Buckling Compression	7	51.3	751.1	kN	0.068	✓ Pass
Buckling Combined	22	-	-	-	0.320	✓ Pass

SC C/2



SC C/2

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	5.504	Yes		Yes	
	sub-beam	3.004	No	1.000	No	1.000
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	5.504	Yes		Yes	
	sub-beam	3.004	No	1.000	No	1.000

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	2.500	Yes		Yes	
	sub-beam	2.500	No	1.000	No	1.000
1	floor	0.000	Yes		Yes	

Static

Summary UKB 203x133x25(S275)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	✓ Pass
Shear Major	No	Significant	Forces	kN	-	Not required
Shear Minor	No	Significant	Forces	kN	-	Not required
Buckling Shear Web	-	32.91	66.56	-	-	✓ Pass
Moment Major	No	Significant	Forces	kNm	-	Not required
Moment Minor	1	0.4	19.5	kNm	0.020	✓ Pass
Axial	7	81.4	879.1	kN	0.093	✓ Pass
Axial Bending Combined	7	-	-	-	0.020	✓ Pass
Buckling Lateral Torsional	No	Significant	Forces	kNm	-	Not required
Buckling Compression	7	81.4	565.2	kN	0.144	✓ Pass
Buckling Combined	7	-	-	-	0.158	✓ Pass

Brace Design

Brace Design Summary

Static

Member Reference	Group Ref.	Span	Section	Grade	Length [m]	Utilization	Status
SBR 2.1/%18-2.1/A/#263	SBrR5	1	CHS 114.3x3.6	S275	1.000	0.011	✓ Pass
SBR 2/A/3-2/B/3	SBrR5	1	CHS 114.3x3.6	S275	1.000	0.025	✓ Pass
SBR IP 4/C/2-IP 4/B/3	SBrR1	1	CHS 114.3x3.6	S275	5.709	0.033	✓ Pass
SBR IP 3/B/1-IP 3/C/#264	SBrR4	1	CHS 114.3x3.6	S275	4.660	0.013	✓ Pass
SBR 2/B/1-2/C/1	SBrR2	1	CHS 114.3x3.6	S275	4.050	0.015	✓ Pass
SBR 2/B/3-2/C/3	SBrR2	1	CHS 114.3x3.6	S275	4.050	0.013	✓ Pass
SBR 1/3/B-2/3/C	SBrR3	1	RSA 70x70x10	S275	4.682	0.082	✓ Pass
SBR 1/3/C-2/3/B	SBrR3	1	RSA 70x70x10	S275	4.682	0.084	✓ Pass
SBR 1/1/C-2/1/B	SBrR3	1	RSA 70x70x10	S275	4.682	0.119	✓ Pass
SBR 1/1/B-2/1/C	SBrR3	1	RSA 70x70x10	S275	4.682	0.084	✓ Pass
SBR FRM C/#215/#265-IP 3/B/4	SBrR4	1	CHS 114.3x3.6	S275	4.660	0.021	✓ Pass
SBR 2/A/1-2/B/1	SBrR5	1	CHS 114.3x3.6	S275	1.000	0.032	✓ Pass
SBR 2.1/%17-2.1/A/#262	SBrR5	1	CHS 114.3x3.6	S275	1.000	0.014	✓ Pass
SBR IP 4/C/2-3/B/4	SBrR2	1	CHS 114.3x3.6	S275	4.092	0.009	✓ Pass

Foundation Reactions

Envelopes, First-order linear, Service Factors

Supports

Support	Support rotation [°]	Column Ref.	Column rotation [°]	Envelope	Combination	Reactions					
						F_{vert} [kN	F_{major}	F_{minor}	M_{major} [kNm	M_{minor} [kNm	M_{tor} [kNm
SUP %17	0.0000			1 Envelope	Maximum	0.0	0.0	1.8	0.0	0.0	0.0
					Minimum	0.0	0.0	-3.0	0.0	0.0	0.0
SUP %18	0.0000			1 Envelope	Maximum	0.0	0.0	1.8	0.0	0.0	0.0
					Minimum	0.0	0.0	-2.4	0.0	0.0	0.0
SUP A/1	180.0000	ELM Base/A/1-1/A/1 (Wall Column Section)	180.0000	1 Envelope	Maximum	3.8	0.0	0.0	0.0	0.0	0.0
					Minimum	3.3	0.0	0.0	0.0	0.0	0.0

Support	Support rotation [°]	Column Ref.	Column rotation [°]	Envelope	Combination	Reactions					
						F_{vert} [kN	F_{major} [kN	F_{minor} [kN	M_{major} [kNm	M_{minor} [kNm	M_{tor} [kNm
SUP A/3	180.0000	ELM Base/A/3-1/A/3 (Wall Column Section)	180.0000	1 Envelope	Maximum	3.8	0.0	0.0	0.0	0.0	0.0
					Minimum	3.3	0.0	0.0	0.0	0.0	0.0
SUP A/4	180.0000	ELM Base/A/4-1/A/4 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/6	0.0000	SC A/6 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	5.2	1.7	0.0	0.1	0.0	0.0
					Minimum	2.3	0.2	0.0	-0.8	0.0	0.0
SUP A/6	0.0000	SC A/6 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	0.0	0.0	2.6	0.0	0.0	0.0
					Minimum	0.0	0.0	-3.8	0.0	0.0	0.0
SUP A/7	0.0000	SC 8/A (UKB 254x146x31)	0.0000	1 Envelope	Maximum	0.0	0.0	2.6	0.0	0.0	0.0
					Minimum	0.0	0.0	-3.0	0.0	0.0	0.0
SUP A/7	0.0000	SC 8/A (UKB 254x146x31)	0.0000	1 Envelope	Maximum	5.2	-0.3	0.0	0.8	0.0	0.0
					Minimum	2.3	-1.7	0.0	0.0	0.0	0.0
SUP A/#76	180.0000	ELM Base/A/#52-1/A/#53 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#77	180.0000	ELM Base/A/#54-1/A/#55 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#78	180.0000	ELM Base/A/#56-1/A/#57 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#79	180.0000	ELM Base/A/#58-1/A/#59 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#80	180.0000	ELM Base/A/#60-1/A/#61 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#81	180.0000	ELM Base/A/#62-1/A/#63 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#82	180.0000	ELM Base/A/#64-1/A/#65 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#83	180.0000	ELM Base/A/#66-1/A/#67 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#84	180.0000	ELM Base/A/#68-1/A/#69 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#85	180.0000	ELM Base/A/#70-1/A/#71 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#86	180.0000	ELM Base/A/#72-1/A/#73 (Wall Column Section)	180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0
					Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP A/#87	180.0000		180.0000	1 Envelope	Maximum	7.7	0.0	0.0	0.0	0.0	0.0

Support	Support rotation [°]	Column Ref.	Column rotation [°]	Envelope	Combination	Reactions					
						F_{vert} [kN	F_{major} [kN	F_{minor} [kN	M_{major} [kNm	M_{minor} [kNm	M_{tor} [kNm
		ELM Base/A/#74-1/A/#75 (Wall			Minimum	6.7	0.0	0.0	0.0	0.0	0.0
SUP B/1	0.0000	SC B/1 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	43.9	2.6	1.5	6.5	1.2	0.0
					Minimum	9.3	-5.9	-2.1	-4.5	-1.7	0.0
SUP B/2	0.0000	SC B/2 (UC 152x152x23)	0.0000	1 Envelope	Maximum	47.3	0.3	0.2	1.0	0.5	0.0
					Minimum	12.8	-0.4	-0.2	-0.7	-0.6	0.0
SUP B/3	0.0000	SC B/3 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	35.7	5.2	1.5	6.0	1.2	0.0
					Minimum	6.5	-3.6	-1.8	-4.9	-1.4	0.0
SUP C/1	0.0000	SC C/1 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	36.7	2.8	2.5	4.1	0.7	0.0
					Minimum	14.3	-4.6	-3.9	-3.1	-0.9	0.0
SUP C/2	90.0000	SC C/2 (UKB 203x133x25)	90.0000	1 Envelope	Maximum	56.6	2.8	0.0	0.0	0.0	0.0
					Minimum	17.6	-2.3	0.0	0.0	0.0	0.0
SUP C/3	0.0000	SC C/3 (UKB 254x146x31)	0.0000	1 Envelope	Maximum	25.7	3.4	2.2	4.0	0.6	0.0
					Minimum	9.0	-3.2	-2.7	-3.1	-0.8	0.0

Concrete

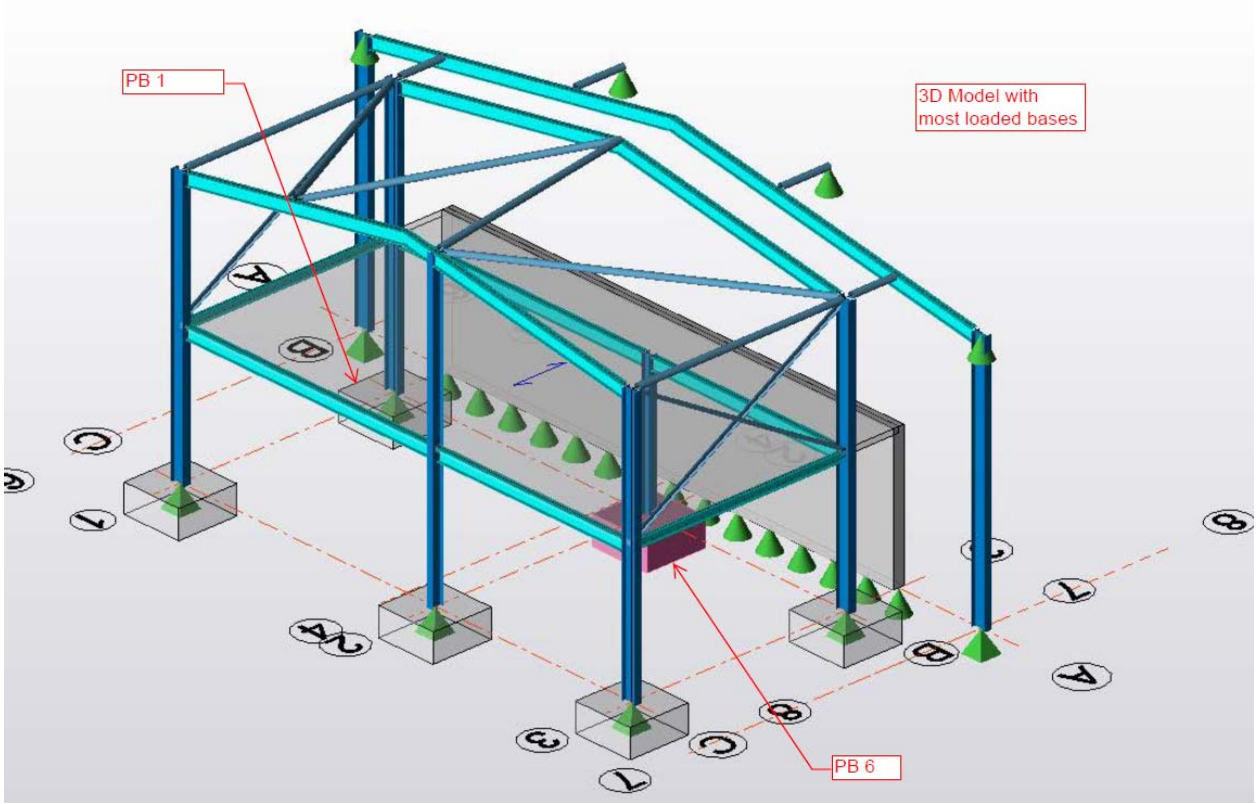
Pad Base Design

Pad Base Design Summary

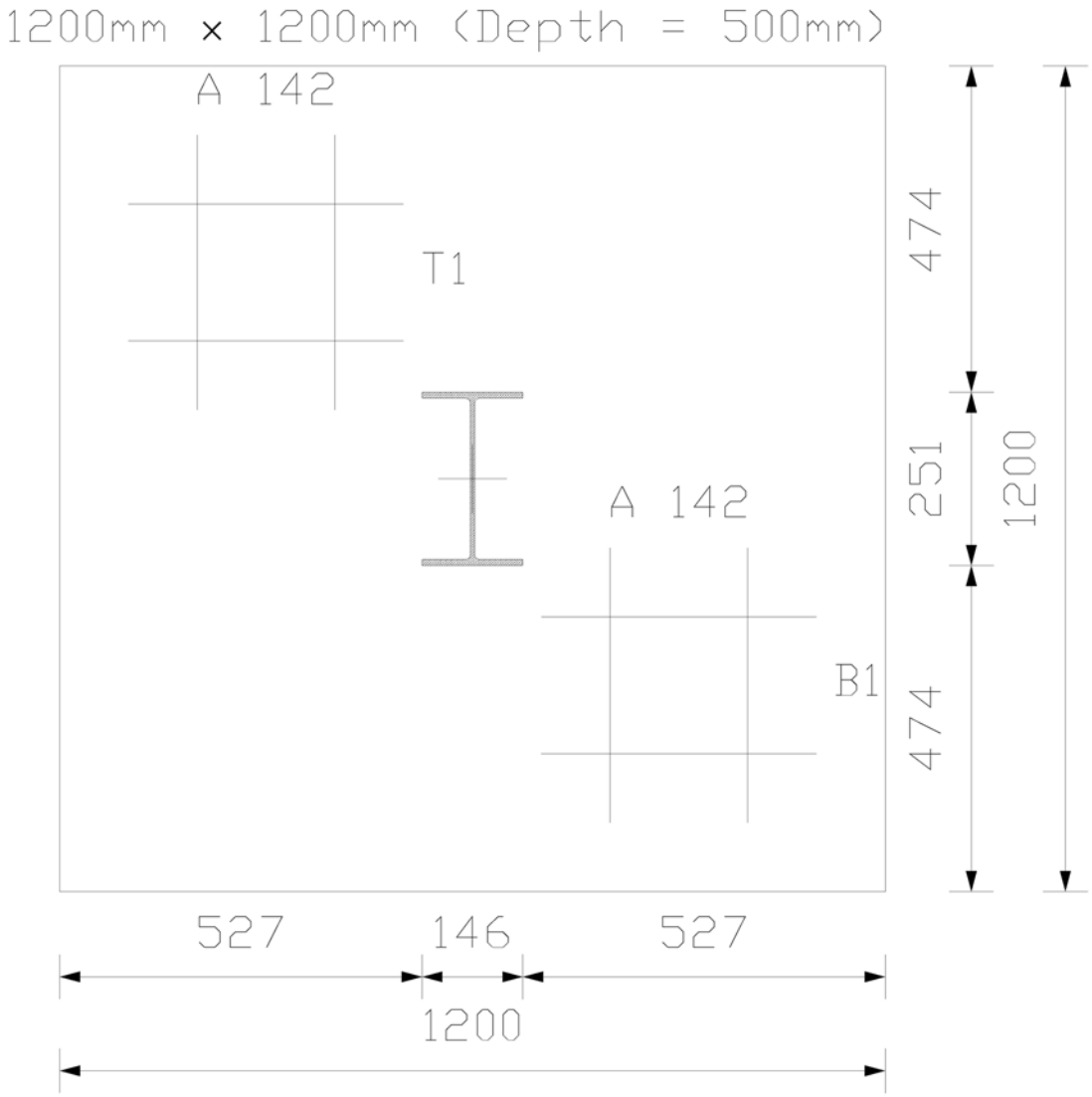
Static

Member Reference	Grade	Depth [mm]	Utilization	Status
PB 1	C32/40	500.0	0.935	✓ Pass
PB 2	C32/40	500.0	0.801	✓ Pass
PB 3	C32/40	500.0	0.880	✓ Pass
PB 4	C32/40	500.0	0.685	✓ Pass
PB 5	C32/40	500.0	0.822	✓ Pass
PB 6	C32/40	500.0	0.888	✓ Pass

Most loaded Pad Base design



PB 1



PB 1

Static

Applied Loads Summary

Analysis	3D Building Analysis
Combination	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
$F_{z,sup}$	49.0 kN
$F_{x,sup}$	-1.6 kN
$F_{y,sup}$	-9.7 kN
$M_{x,sup}$	11.4 kNm
$M_{y,sup}$	-1.5 kNm
	Added Loads
F_{swt}	17.7 kN
F_{soil}	11.2 kN

$F_{sur,G}$	0.0 kN
$F_{sur,Q}$	2.1 kN

Foundation Details

Foundation Type	Pad Base
Concrete Class	C32/40
Size	1200mm × 1200mm
Overall Depth	500.0 mm
Top Cover	40.0 mm
Bottom Cover	40.0 mm
Side Cover	40.0 mm
Depth From Surface	400.0 mm

Bearing Capacity

Summary

Size	1200.0 × 1200.0
Analysis	3D Building Analysis
Combination	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
f_{dz}	93.5 kN/m^2
n_f	100.0 kN/m^2
Ratio	**0.935**

Bending Capacity

Summary

Direction	X-Bot	Y-Bot	X-Top	Y-Top
Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis
Combination	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
m_{Ed}	11.061 kNm/m	13.653 kNm/m	3.292 kNm/m	4.063 kNm/m
d	451.0 mm	457.0 mm	451.0 mm	457.0 mm
K / K'	**0.008**	**0.010**	**0.002**	**0.003**
z	428.5 mm	434.2 mm	428.5 mm	434.2 mm
$A_{s,reqd}$	59 mm^2/m	72 mm^2/m	18 mm^2/m	22 mm^2/m
$A_{s,prov}$	142 mm^2/m	142 mm^2/m	142 mm^2/m	142 mm^2/m
Ratio	**0.418**	**0.509**	**0.124**	**0.152**
Reinforcement	A 142	A 142	A 142	A 142

Shear Capacity

Summary

Direction	X	Y
Analysis	3D Building Analysis	3D Building Analysis
Combination	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}
v_{Ed}	0.015 N/mm^2	0.005 N/mm^2
$v_{Rd,max}$	5.581 N/mm^2	5.581 N/mm^2
v_{Rd}	0.424 N/mm^2	0.426 N/mm^2
Ratio	**0.035**	**0.012**

Punching Shear

Summary

Analysis	Combination	Perimeter	u_0 / u_a [mm]	v_{Ed} [N/mm^2]	v_{Rd} [N/mm^2]	Ratio	Status
3D Building Analysis	17 $STR_{8.1}$-1.35G+1.5ψ_0Q+1.5ψ_0S+1.5W+EHF_{Dir2+}	Loaded	795.0	0.282	5.581	0.051	✔ Pass

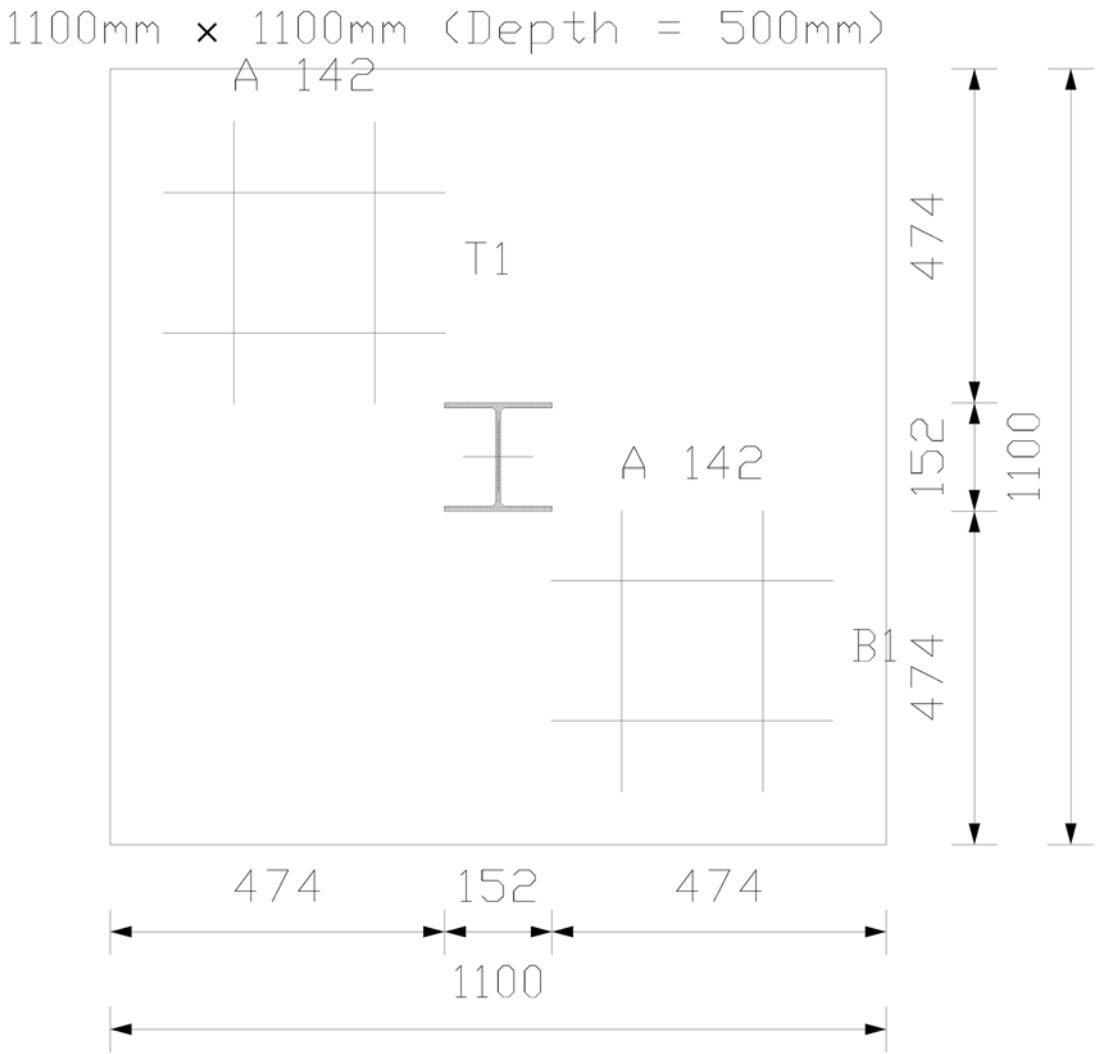
Analysis	Combination	Perimeter	u_0 / u_a [mm]	v_{Ed} [N/mm²]	v_{Rd} [N/mm²]	Ratio	Status
3D Building Analysis	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}	At 1d	2111.9	0.092	0.850	0.108	✓ Pass

Uplift

Summary

Analysis	3D Building Analysis
Combination	1 STR_1-1.35G+1.5Q+1.5RQ
$F_{z,sup}$	0.0 kN
$F_{G,stb}$	0.0 kN
Ratio	**0.000**

PB 6



PB 6

Static

Applied Loads Summary

Analysis	3D Building Analysis
Combination	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
$F_{z,sup}$	69.0 kN
$F_{x,sup}$	0.1 kN
$F_{y,sup}$	-0.2 kN
$M_{x,sup}$	1.0 kNm
$M_{y,sup}$	-0.5 kNm
	Added Loads
F_{swt}	14.8 kN
F_{soil}	9.5 kN
$F_{sur,G}$	0.0 kN
$F_{sur,Q}$	1.8 kN

Foundation Details

Foundation Type	Pad Base
Concrete Class	C32/40
Size	1100mm × 1100mm
Overall Depth	500.0 mm
Top Cover	40.0 mm
Bottom Cover	40.0 mm
Side Cover	40.0 mm
Depth From Surface	400.0 mm

Bearing Capacity

Summary

Size	1100.0 × 1100.0
Analysis	3D Building Analysis
Combination	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
f_{dz}	88.8 kN/m^2
n_f	100.0 kN/m^2
Ratio	**0.888**

Bending Capacity

Summary

Direction	X-Bot	Y-Bot	X-Top	Y-Top
Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis
Combination	2 $STR_{6.1}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0	34 $STR_{9.3}$-G+1.5W+EHF_{Dir2+}	34 $STR_{9.3}$-G+1.5W+EHF_{Dir2+}
m_{Ed}	6.805 kNm/m	6.571 kNm/m	2.257 kNm/m	2.258 kNm/m
d	451.0 mm	457.0 mm	451.0 mm	457.0 mm
K / K'	**0.005**	**0.005**	**0.002**	**0.002**
z	428.5 mm	434.2 mm	428.5 mm	434.2 mm
$A_{s,reqd}$	37 mm^2/m	35 mm^2/m	12 mm^2/m	12 mm^2/m
$A_{s,prov}$	142 mm^2/m	142 mm^2/m	142 mm^2/m	142 mm^2/m
Ratio	**0.257**	**0.245**	**0.085**	**0.084**
Reinforcement	A 142	A 142	A 142	A 142

Shear Capacity

Summary

Direction	X	Y
Analysis	3D Building Analysis	3D Building Analysis
Combination	4 $STR_{6.3}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}	2 $STR_{6.1}$-1.35G+1.5Q+1.5ψ_0S+1.5ψ_0W+EHF_{Dir2+}
v_{Ed}	0.002 N/mm^2	0.003 N/mm^2

$v_{Rd,max}$	5.581 N/mm^2	5.581 N/mm^2
v_{Rd}	0.424 N/mm^2	0.426 N/mm^2
Ratio	**0.005**	**0.007**

Punching Shear

Summary

Analysis	Combination	Perimeter	u_0 / u_a [mm]	v_{Ed} [N/mm^2]	v_{Rd} [N/mm^2]	Ratio	Status
3D Building Analysis	1 STR_1-1.35G+1.5Q+1.5RQ	Loaded	609.2	0.281	5.581	0.050	✓ Pass
3D Building Analysis	1 STR_1-1.35G+1.5Q+1.5RQ	At 0.25d	1322.3	0.118	3.399	0.035	✓ Pass

Uplift

Summary

Analysis	3D Building Analysis
Combination	1 STR_1-1.35G+1.5Q+1.5RQ
$F_{z,sup}$	0.0 kN
$F_{G,stb}$	0.0 kN
Ratio	**0.000**

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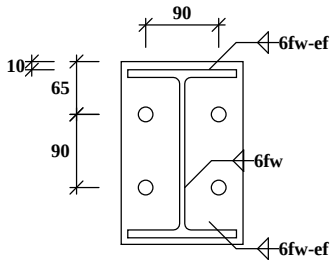
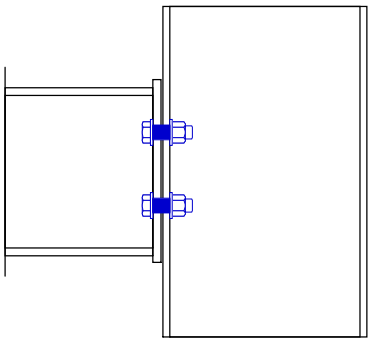
Sheet	**: / 114**
Made by	**: GB**
Date	**: 17 April 2020 / Ver. 2018.06**
Checked	**: HD**
Approved	**:**

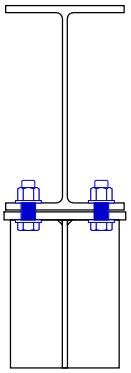
Beam to portal frame column flange



Beam 1: 203x133 UB 30 [S275]
Plate: 10 x 150 x 225 mm dp. (3 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Column: 254x146 UB 31 [S 275]
End-Plates S 275



Beam to Column Flexible End-Plate
Beam to Column Flexible End-plate Connection to EC 3 (UK NAD)
Basic Data

User Defined Applied Forces at Interface

Shear Forces	Flange 1 = 40.0 kN, Flange 2 = 30.0 kN
Tie Forces	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Flange 1-203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Column -254x146UB31 [28]	D=251.4, B=146.1, T=8.6, t=6.0, r=7.6, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 275	All weld grades provided to suit minimum connected steel grade

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Summary of Results (Unity Ratios)

Checks for Flange Beam on Side 1

Check 4 Supported Beam Web Shear	231.4 >= 40.0kN	0.17	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.70, 0.710.61	1.00	OK
Check 8 Bearing	192.9, 524.8, 451.3 >= 40.0kN	0.21	OK
Check 10 Shear	293.6, 331.3 >= 20.0kN	0.07	OK
Check 11-14 Tie Forces	348.3, 265.3, 411.1, 244.8, 238.1, 447.5, 533.8 >= 75.0kN	0.31	OK

Checks for Flange Beam on Side 1 as Full Depth End-Plate

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 206.8 = 124.1 mm	225 mm	OK
10 >= t_p<= 12	10.0 mm		OK
90 <= p_3 <= 140	90 mm		OK
t_p<= d/1.9•√(f_{ub}/ f.y)	16/1.9 •√(800 / 275)	14 mm	OK

Check 4: Supported Beam Web Shear

A_v=A-t_b•B_b-t_t•B_t+(t_w+2•r)•T	3821 - 133.9•9.6 - 133.9•9.6 + (6.4+2•7.6)•9.6	1457 mm²	
$V_{pl_{Rd}}$=Av•f_{ybw} / (√3•γ_{M0})	1457 • 275 / (√3 • 1.0)	231.4 kN	OK

Check 2: Supported Beam Welds

a=0.4 t_{wb1}	0.4 • 6.4	2.6 mm	
a_1=a•V_{Ed}/ $V_{pl\cdot Rd}$•1.27	2.6 • 40.0 / 231.4 • 1.27 (SCI AD370)	0.6 mm	
S_{min}=max(3, min(a, a_1)) / 0.7		4.3 mm	OK

Check 8a: Bolt Shear

$F_{v,Rd}$=α_v•f_{ub}•A /γ_{M2}	0.6 • 800 • 157.0 / 1.25	60.3 kN	
V_{Rd1}=0.8•n•$F_{v,Rd}$•cols	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

P_1, e_1, P_2, e_2,α_b, k_1	90.0, 65.0, 90.0, 30.0, 1.000, 2.500		
$F_{b,Rd}$=k_1•α_b•f_{up}•d•t_p/γ_{M2}	2.5 • 1.000 • 410 • 16 • 10.0 / 1.25	131.2 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
F_{Rd}=n•$f_{b,Rd}$•rows	2 • 131.2 • 2	524.8 kN	OK

Check 8c: Supporting Column Flange Bearing

α_b=Min(p_1/d_0/3-¼, f_{ub}/ f_u, 1)	Min(90/18/3-¼, 800/410, 1) » Min(1.417, 1.951, 1)	1.000	
$k_{1\cdot inn}$=min(2.5, 1.4•p_2/d_0-1.7)	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}$=k_1•α_b•$f_{u,2}$ •d•t_p/γ_{M2}	2.5 • 1.000 • 410 • 16.0 • 8.6 / 1.25	112.8 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
F_{Rd}=n•$f_{b,Rd}$•rows	2 • 112.8 • 2	451.3 kN	OK

Check 10: Supporting Member - Local Resistance

A_v, A_{Vnet}=Fn(e_t, e_b, p_1, n, dia, T)	fn(80, 45, 90, 2, 18.0, 8.6)	1849, 1539 mm²	
V_{Rd}=A_v•F_{yp}/(√3•γ_{M0})	1849 • 275/(√3 • 1.0)	293.6 kN	OK
V_{Rd}=$A_{vnet\cdot F_{up}}$/(√3•γ_{M2})	1539 • 410/(√3 • 1.1)	331.3 kN	OK

Tie Forces

Applied Tie Force	75.0 kN	

Check 11: End-Plate in Bending

L_{eff}=$L_{eff\cdot t}$+(n-1)•p+$L_{e^{\alpha}\cdot t}$	103.1 + (2-1) • 90 + 99.6	292.7 mm	
M_{pl1}=0.25•L_{eff}•t^2•$f_{u\cdot p}$/γ_{Mu}	0.25 • 292.7 • 10² • 410 / 1.1	2.73 kN.m	
$F_{Rd\cdot u\cdot 1}$=(8•n-2•e_w)• m_{pl1} / (2•m •n-e_w•(m+n))	(8 • 28.1 - 2 • 6.5) • 3 / (2 • 37.0 • 28.1 - 6.5 • (37.0 + 28.1))	348.3 kN	
$F_{Rd\cdot u\cdot 2}$= (2•M_{pl2}+n•Σ(k_2•f_{ub}•A /γ_{mu}))/(m+n)	(2•2.73 + 30.00•4•0.90•800•157/1.1)/ (37.0 + 30.0)	265.3 kN	
$F_{rd\cdot u\cdot 3}$=Σ•k_2•F_{ub}•A_s/γ_{Mu}	4 • 0.90•800 • 157/1.1	411.1 kN	
F_{Rd}=min($F_{rd\cdot u\cdot 1}$, $F_{rd\cdot u\cdot 2}$, $F_{rd\cdot u\cdot 3}$)	Min (348.34, 265.28, 411.05)	265.3 kN	OK
L_{eff}=$L_{eff\cdot t}$+(n-1)•p+$L_{eff\cdot t}$	89.4 + (2-1) • 90 + 89.4	268.7 mm	
$M_{pl1,col}$=0.25•L_{eff}•t_f^2•$f_{u.c}$ /γ_{Mu}	0.25 • 268.7 • 8.6² • 410 / 1.1	1.85 kN.m	

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$F_{Rd.u.1,col} = (8 \cdot n - 2 \cdot e_w) \cdot m_{pl1}$	(8 • 28.1 - 2 • 6.5) • 1.85		
$/ (2 \cdot m \cdot n - e_w \cdot (m+n))$	/ (2 • 35.9 • 28.1 - 6.5 • (35.9 + 28.1))	244.8 kN	
$F_{Rd.u.2,col} =$			
$(2 \cdot M_{pl2} + \Sigma(k_2 \cdot f_{ub} \cdot A \cdot n / \gamma_{mu})) / (m+n)$	(2•1.85 + 4•0.90•800•157•28.05/1.1)/ (35.9 + 28.1)	238.1 kN	
$F_{Rd,col} = \min(F_{rd.u.1,col}, F_{rd.u.2,col})$	Min (244.81, 238.15)	238.1 kN	OK

Check 12: Beam Web Tension

$F_{Rd} = t_{bw} \cdot (D - T_t - T_b) + F_{ubw} / \gamma_{Mu}$	6.4 • (206.8 - 9.6 - 9.6) • 410 / 1.1	447.5 kN	OK

Check 13: Welds

Web	Covered by Check 2	n/a	OK
Flange $F_{vw\text{-}d} = f_u / \sqrt{3} / \beta_w / \gamma_{Mu}$	410.0 /√3 / 0.85 / 1.1	253.2 N/mm²	
$F_{rd\text{-}flange} = 2 \cdot F_{vw\text{-}d} \cdot a \cdot (b_{b1} - 2 \cdot a)$	2 • 253.2 • 4.2 • (133.9 - 2 • 4.2)	266.9 kN	OK

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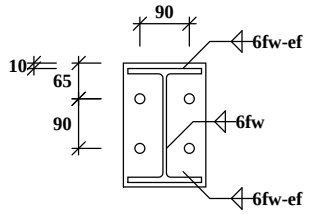
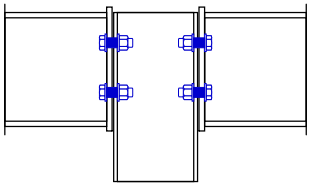
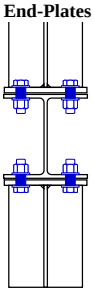
Beams to internal column flange

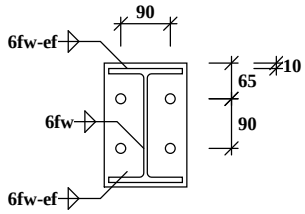


Beam 1: 203x133 UB 30 [S275]
Plate: 10 x 150 x 225 mm dp. (3 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Column: 152x152 UC 23 [S 275]
End-Plates S 275





Beam 2: 203x133 UB 30 [S275]
Plate: 10 x 150 x 225 mm dp. (3 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.

Beam to Column Flexible End-Plate
Beam to Column Flexible End-plate Connection to EC 3 (UK NAD)
Basic Data

User Defined Applied Forces at Interface

Shear Forces	Flange 1 = 40.0 kN, Flange 2 = 30.0 kN
Tie Forces	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Flange 1-203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Flange 2-203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Column -152x152UC23 [28]	D=152.4, B=152.2, T=6.8, t=5.8, r=7.6, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 275	All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Checks for Flange Beams 1 & 2

Check 4 Supported Beam Web Shear	231.4 >= 40.0kN	0.17	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.70, 0.710.61	1.00	OK
Check 8 Bearing	192.9, 524.8, 356.9 >= 40.0kN	0.21	OK
Check 10 Shear	205.1, 225.4 >= 20.0kN	0.10	OK
Check 11-14 Tie Forces	352.2, 266.6, 411.1, 153.7, 225.5, 447.5, 533.8 >= 75.0kN	0.49	OK

Checks for Flange Beams 1 & 2 as Full Depth End-Plate

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 206.8 = 124.1 mm	225 mm	OK
$10 >= t_p <= 12$	10.0 mm		OK
$90 <= p_3 <= 140$	90 mm		OK
$t_p <= d/1.9 \bullet \sqrt{(f_{ub}/ f.y)}$	$16/1.9 \bullet \sqrt{(800 / 275)}$	14 mm	OK

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Check 4: Supported Beam Web Shear

$A_v=A-t_b•B_b-t_t•B_t+(t_w+2•r)•T$	3821 - 133.9•9.6 - 133.9•9.6 + (6.4+2•7.6)•9.6	1457 mm²	
$V_{pl_{Rd}}=Av•f_{ybw} / (\sqrt{3}•\gamma_{M0})$	1457 • 275 / (√3 • 1.0)	231.4 kN	OK

Check 2: Supported Beam Welds

$a=0.4\ t_{wb1}$	0.4 • 6.4	2.6 mm	
$a_1=a•V_{Ed}/ V_{pl\cdot Rd}•1.27$	2.6 • 40.0 / 231.4 • 1.27 (SCI AD370)	0.6 mm	
$S_{min}=max(3, min(a, a_1)) / 0.7$		4.3 mm	OK

Check 8a: Bolt Shear

$F_{v,Rd}=\alpha_v•f_{ub}•A /\gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd1}=0.8•n•F_{v,Rd}•cols$	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

$P_1,\ e_1,\ P_2,\ e_2,\alpha_b,\ k_1$	90.0, 65.0, 90.0, 30.0, 1.000, 2.500		
$F_{b,Rd}=k_1•\alpha_b•f_{up}•d•t_p/\gamma_{M2}$	2.5 • 1.000 • 410 • 16 • 10.0 / 1.25	131.2 kN	
$F_{b,Rd}<= 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 131.2 • 2	524.8 kN	OK

Check 8c: Supporting Column Flange Bearing

$\alpha_b=Min(p_1/d_0/3-¼,\ f_{ub}/ f_u,\ 1)$	Min(90/18/3-¼, 800/410, 1) » Min(1.417, 1.951, 1)	1.000	
$k_{1\cdot inn}=min(2.5, 1.4•p_2/d_0-1.7)$	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}=k_1•\alpha_b•f_{u,2}•d•t_p/\gamma_{M2}$	2.5 • 1.000 • 410 • 16.0 • 6.8 / 1.25	89.2 kN	
$F_{b,Rd}<= 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 89.2 • 2	356.9 kN	OK

Check 10: Supporting Member - Local Resistance

$A_v,\ A_{Vnet}=Fn(e_t, e_b, p_1, n, dia, T)$	fn(55, 45, 90, 2, 18.0, 6.8)	1292, 1047 mm²	
$V_{Rd}=A_v•F_{yp}/(\sqrt{3}•\gamma_{M0})$	1292 • 275/(√3 • 1.0)	205.1 kN	OK
$V_{Rd}=A_{vnet•F_{up}}/(\sqrt{3}•\gamma_{M2})$	1047 • 410/(√3 • 1.1)	225.4 kN	OK

Tie Forces

Applied Tie Force	75.0 kN	

Check 11: End-Plate in Bending

$L_{eff}=L_{eff\cdot t}+(n-1)•p+L_{e\cdot t}$	105.6 + (2-1) • 90 + 101.9	297.6 mm	
$M_{pl1}=0.25•L_{eff}•t^2•f_{u\cdot p}/\gamma_{Mu}$	0.25 • 297.6 • 10² • 410 / 1.1	2.77 kN.m	
$F_{Rd\cdot u\cdot 1}=(8•n-2•e_w)•\ m_{pl1}$	(8 • 30.0 - 2 • 6.5) • 3		
$/ (2•m\ •n-e_w•(m+n))$	/ (2 • 37.0 • 30.0 - 6.5 • (37.0 + 30.0))	352.2 kN	
$F_{Rd\cdot u\cdot 2}=$			
$(2•M_{pl2}+n•\Sigma(k_2•f_{ub}•A /\gamma_{mu}))/(m+n)$	(2•2.77 + 30.00•4•0.90•800•157/1.1)/ (37.0 + 30.0)	266.6 kN	
$F_{rd\cdot u\cdot 3}=\Sigma•k_2•F_{ub}•A_s/\gamma_{Mu}$	4 • 0.90•800 • 157/1.1	411.1 kN	
$F_{Rd}=min(F_{rd\cdot u\cdot 1}, F_{rd\cdot u\cdot 2}, F_{rd\cdot u\cdot 3})$	Min (352.24, 266.64, 411.05)	266.6 kN	OK
$L_{eff}=L_{eff\cdot t}+(n-1)•p+L_{eff\cdot t}$	91.5 + (2-1) • 90 + 91.5	273.0 mm	
$M_{pl1,col}=0.25•L_{eff}•t_f^2•f_{u.c} /\gamma_{Mu}$	0.25 • 273.0 • 6.8² • 410 / 1.1	1.18 kN.m	
$F_{Rd.u.1,col} =(8•n-2•e_w)•\ m_{pl1}$	(8 • 31.1 - 2 • 6.5) • 1.18		
$/ (2•m\ •n-e_w•(m+n))$	/ (2 • 36.0 • 31.1 - 6.5 • (36.0 + 31.1))	153.7 kN	
$F_{Rd.u.2,col} =$			
$(2•M_{pl2}+\Sigma(k_2•f_{ub}•A•n /\gamma_{mu}))/(m+n)$	(2•1.18 + 4•0.90•800•157•31.10/1.1)/ (36.0 + 31.1)	225.5 kN	
$F_{Rd,col} =min(F_{rd.u.1,col}, F_{rd.u.2,col})$	Min (153.71, 225.51)	153.7 kN	OK

Check 12: Beam Web Tension

$F_{Rd}=t_{bw•}(D-T_t-T_b)+F_{ubw} /\gamma_{Mu}$	6.4 • (206.8 - 9.6 - 9.6) • 410 / 1.1	447.5 kN	OK

Check 13: Welds

Web	Covered by Check 2	n/a	OK
Flange $F_{vw\cdot d}=f_u /\sqrt{3}/ \beta_w/\gamma_{Mu}$	410.0 /√3 / 0.85 / 1.1	253.2 N/mm²	
$F_{rd\cdot flange}=2•F_{vw\cdot d}•a•(b_{b1}-2•a)$	2 • 253.2 • 4.2 • (133.9 - 2 • 4.2)	266.9 kN	OK

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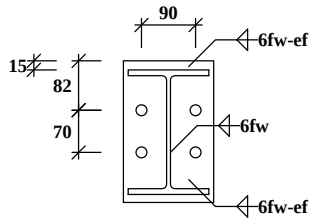
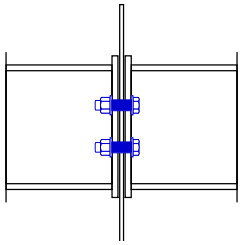
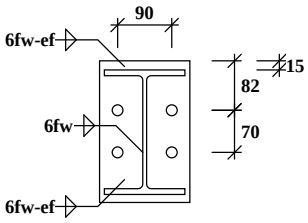
Beam to gable column web



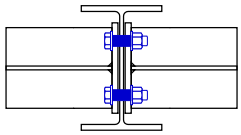
Beam 1: 203x133 UB 30 [S275]
Plate: 10 x 150 x 235 mm dp. (3 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Column: 203x133 UB 30 [S275]
Plates S 275



Beam 2: 203x133 UB 30 [S275]
Plate: 10 x 150 x 235 mm dp. (3 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Beam to Column Flexible End-Plate
Beam to Column Flexible End-plate Connection to EC 3 (UK NAD)
Basic Data

User Defined Applied Forces at Interface

Shear Forces	Web 1 = 31.1 kN, Web 2 = 24.0 kN
Tie Forces	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Web 1-203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Web 2-203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Column -203x133UB30 [28]	D=206.8, B=133.9, T=9.6, t=6.4, r=7.6, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 275	All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Checks for Web Beams 1 & 2

Check 4 Supported Beam Web Shear	231.4 >= 31.1kN	0.13	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.70, 0.710.61	1.00	OK
Check 8 Bearing	192.9, 524.8, 335.9 >= 31.1kN	0.16	OK
Check 10 Shear	198.1, 219.0 >= 27.6kN	0.14	OK

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Check 10 Combined Bearing	84.0 >= 13.8kN	0.16	OK
Check 14 Tie Forces Col Web	82.0 >= 7.1kN	0.09	OK
Check 11-13 Tie Forces	318.2, 258.6, 411.1, 447.5, 533.8 >= 75.0kN	0.29	OK

Checks for Web Beams 1 & 2 as Full Depth End-Plate

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 206.8 = 124.1 mm	235 mm	OK
10 >= t_p<= 12	10.0 mm		OK
90 <= p_3 <= 140	90 mm		OK
t_p<= d/1.9•√(f_{ub}/ f.y)	16/1.9 •√(800 / 275)	14 mm	OK

Check 4: Supported Beam Web Shear

A_v=A-t_b•B_b-t_t•B_t+(t_w+2•r)•T	3821 - 133.9•9.6 - 133.9•9.6 + (6.4+2•7.6)•9.6	1457 mm²	
$V_{pl_{Rd}}$=Av•f_{ybw} / (√3•γ_{M0})	1457 • 275 / (√3 • 1.0)	231.4 kN	OK

Check 2: Supported Beam Welds

a=0.4 t_{wb1}	0.4 • 6.4	2.6 mm	
a_1=a•V_{Ed}/ $V_{pl.Rd}$•1.27	2.6 • 31.1 / 231.4 • 1.27 (SCI AD370)	0.4 mm	
S_{min}=max(3, min(a, a_1)) / 0.7		4.3 mm	OK

Check 8a: Bolt Shear

$F_{v,Rd}$=α_v•f_{ub}•A /γ_{M2}	0.6 • 800 • 157.0 / 1.25	60.3 kN	
V_{Rd1}=0.8•n•$F_{v,Rd}$•cols	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

P_1, e_1, P_2, e_2,α_b, k_1	70.0, 82.0, 90.0, 30.0, 1.000, 2.500		
$F_{b,Rd}$=k_1•α_b•f_{up}•d•t_p/γ_{M2}	2.5 • 1.000 • 410 • 16 • 10.0 / 1.25	131.2 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
F_{Rd}=n•$f_{b,Rd}$•rows	2 • 131.2 • 2	524.8 kN	OK

Check 8c: Supporting Column Web Bearing

α_b=Min(p_1/d_0/3-¼, f_{ub}/ f_u, 1)	Min(70/18/3-¼, 800/410, 1) » Min(1.046, 1.951, 1)	1.000	
$k_{1.inn}$=min(2.5, 1.4•p_2/d_0-1.7)	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}$=k_1•α_b•$f_{u,2}$ •d•t_p/γ_{M2}	2.5 • 1.000 • 410 • 16.0 • 6.4 / 1.25	84.0 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
F_{Rd}=n•$f_{b,Rd}$•rows	2 • 84.0 • 2	335.9 kN	OK

Check 10: Supporting Member - Local Resistance

V_{ed}=(V_{ed1}+V_{ed2})/2	(31 + 24)/2 (per shear plane)	27.6 kN	
A_v, A_{Vnet}=Fn(e_t, e_b, p_1, n, dia, T)	fn(80, 45, 70, 2, 18.0, 6.4)	1248, 1018 mm²	
V_{Rd}=A_v•F_{yp}/(√3•γ_{M0})	1248 • 275/(√3 • 1.0)	198.1 kN	OK
V_{Rd}=$A_{vnet•F_{up}}$/(√3•γ_{M2})	1018 • 410/(√3 • 1.1)	219.0 kN	OK

Check 10: Combined Load Bearing on supporting web

F_{ed}=V_{ed1}/n_1+V_{ed2}/n_2 <= $F_{b,Rd}$	31.1/4 + 24.0/4	(per bolt) 13.8 kN	OK

Tie Forces

Applied Tie Force	75.0 kN		

Check 11: End-Plate in Bending

L_{eff}=$L_{eff.t}$+(n-1)•p+$L_{e^{\alpha}.t}$	99.9 + (2-1) • 70 + 99.0	268.8 mm	
M_{pl1}=0.25•L_{eff}•t^2•$f_{u.p}$/γ_{Mu}	0.25 • 268.8 • 10² • 410 / 1.1	2.51 kN.m	
$F_{Rd.u.1}$=(8•n-2•e_w)• m_{pl1} / (2•m •n-e_w•(m+n))	(8 • 30.0 - 2 • 6.5) • 3 / (2 • 37.0 • 30.0 - 6.5 • (37.0 + 30.0))	318.2 kN	
$F_{Rd.u.2}$= (2•M_{pl2}+n•Σ(k_2•f_{ub}•A /γ_{mu}))/(m+n)	(2•2.51 + 30.00•4•0.90•800•157/1.1)/ (37.0 + 30.0)	258.6 kN	
$F_{rd.u.3}$=Σ•k_2•F_{ub}•A_s/γ_{Mu}	4 • 0.90•800 • 157/1.1	411.1 kN	
F_{Rd}=min($F_{rd.u.1}$, $F_{rd.u.2}$, $F_{rd.u.3}$)	Min (318.20, 258.65, 411.05)	258.6 kN	OK

Check 12: Beam Web Tension

F_{Rd}=t_{bw}•(D-T_t-T_b)+F_{ubw} /γ_{Mu}	6.4 • (206.8 - 9.6 - 9.6) • 410 / 1.1	447.5 kN	OK

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Check 13: Welds

Web	Covered by Check 2	n/a	OK
Flange $F_{vw\cdot d}=f_u/\sqrt{3}/\beta_w/\gamma_{Mu}$	410.0 /√3 / 0.85 / 1.1	253.2 N/mm²	
$F_{rd\cdot flange}=2\cdot F_{vw\cdot d}\cdot a\cdot(b_{b1}-2\cdot a)$	2 • 253.2 • 4.2 • (133.9 - 2 • 4.2)	266.9 kN	OK

Check 14: Supporting Web

Web Tie Force Applied	7.1 kN		
$8\cdot M_{pl,rd,u}\cdot(\eta+1.5\cdot\sqrt{(1-\beta_1)}\cdot\sqrt{(1-\gamma_1)})$	8 • 3816.73 • (0.30 + 1.5•√(1 - 0.52)•√(1 - 0.10))		
$/(1-\beta_1)$	/ (1 - 0.52)	81.96 kN	OK

WEST EXTENSION TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

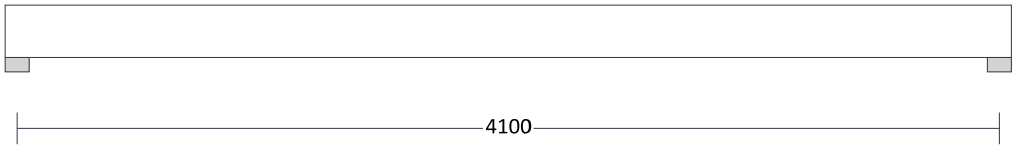
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.00

Joist details

Description — 60 x 220 C24 timber joists

Joist spacing — s_{Joist} = **400** mm



Forces input on Joist

Vertical permanent load on joist — F_{G_Joist}= **0.45** kN/m^2

Vertical imposed load on joist — F_{Q_Joist}= **3.00** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist — $p_G = F_{G\_Joist} \times s_{Joist}$= **0.18** kN/m

Vertical imposed load on joist — $p_Q = F_{Q\_Joist} \times s_{Joist}$= **1.20** kN/m

ANALYSIS

Tedds calculation version 1.0.28

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

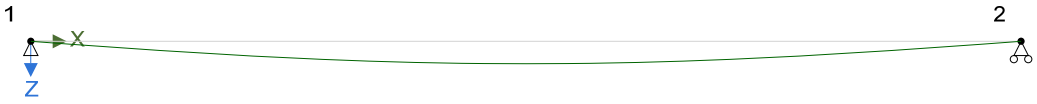
Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.18 kN/m at 0 m to 4.1 m
Member	Imposed	UDL	GlobalZ	1.2 kN/m at 0 m to 4.1 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection

1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.59117	
2	0	0	-0.59117	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.40044	
2	0	0	-0.40044	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	8.6
1.00G + 1.00Q (Service)	0	5.8

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

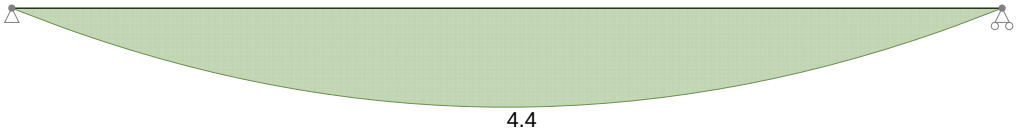
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.1	1	0	-4.3	0
		2	0	-4.3	0

Load combination: 1.00G + 1.00Q (Service)

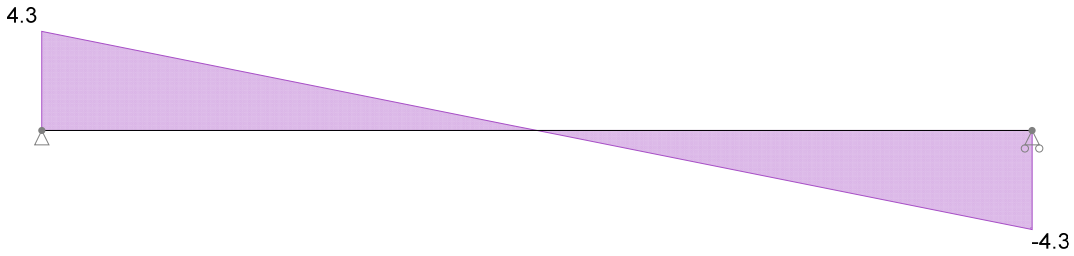
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.1	1	0	-2.9	0
		2	0	-2.9	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	4.3 (max abs)		0 (min)	
	2.05	0		4.4 (max)	
	4.1	-4.3		0 (min)	

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

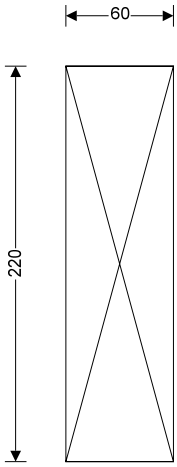
Timber section details

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

Breadth of sections b = **60** mm

Depth of sections h = **220** mm

Timber strength class - EN 338:2016 Table 1 **C24**



60x220 timber section
Cross-sectional area, A, 13200 mm^2
Section modulus, W_y, 484000 mm^3
Section modulus, W_z, 132000 mm^3
Second moment of area, I_y, 53240000 mm^4
Second moment of area, I_z, 3960000 mm^4
Radius of gyration, i_y, 63.5 mm
Radius of gyration, i_z, 17.3 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.7**
Deformation factor - Table 3.2	k_{def} = **0.6**
Bending stress re-distribution factor - cl.6.1.6(2)	k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2)	k_{cr} = **0.67**
System strength factor - cl.6.6	k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d}$ = **4.314** kN
Effective contact length	$L_{b,ef} = L_b$ = **100** mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.719** N/mm^2
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.481** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.486**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **4.314** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.732** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.309**

PASS - Design shear strength exceeds design shear stress

Check design 2050 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **4.421** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **9.135** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **14.215** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.643**

PASS - Design bending strength exceeds design bending stress

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 2050 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	δ_y = **9.3** mm
Quasi-permanent variable load factor	ψ_2 = **0.3**
Final deflection with creep	$\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **15** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **16.4** mm
	$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.912**

PASS - Allowable deflection exceeds final deflection