AD 385
Questions and Answers on SCI P391

This Advisory Desk Note covers some Q & A on SCI publication P391 (Structural Robustness of Steel Framed Buildings, in accordance with Eurocodes and UK National Annexes, 2011).

Q1 Page 25, Section 3.3.3 Annex A.
If the specified cause of accidental action is less onerous than the requirements of BS EN 1991-1-7, for example a specified blast loading of 10 kN/m^2 (< 34 kN/m^2), should the design be based on the specified blast loading, or on the more onerous requirements of BS EN 1991-1-7?
A1. If there is a blast loading specified by the client, then the specified loading should be used. But, if there is also a requirement to satisfy Building Regulations and the key element approach is used, then the 34 kN/m^2 should be used.

Q2 Page 38, Section 5.2.2 Design rules and page 103, Example 1 – Class 1 building.
According to 5.2.2, for Class 1 buildings, roof beam-to-column connections need not be designed for a tie force of 75 kN if the steelwork only supports roof cladding that weighs not more than 0.7 kN/m^2 and carries only imposed loads and wind loads. However, Example 1 requires that roof beams subject to such loading should be capable of resisting the minimum level of horizontal tying i.e. 75 kN. Which is correct?
A2. The guidance given in Section 5.2.2 is correct. Example 1 is conservative in its approach.

Q3 Page 45, section 6.3.1 Chasing loads, paragraph 2.
This refers to a beam connected to a column web with an end plate connection. Presumably, the column web also needs to be checked if the connection is formed using a fin plate or web cleats?
A3. Yes, the guidance is also applicable to other connection types.

Q4 Page 49, Section 6.3.8 Beam arrangements.
In Table 6.1, for g_k = 4.0 kN/m^2 and q_k = 4.0 kN/m^2, T_3 is given as 270 kN. Should this be 135 kN? The calculation is based on the equation given in Figure 6.10, i.e
$T_3 = 0.4x(g_k + \Psi q_k)LB = 0.4 \times [4.0 + (0.5 \times 4.0)] \times 7.5 \times 7.5 = 135$ kN.
A4. Yes, in this case the value for T_3 should be 135 kN.

Q5 Page 60, Section 7.6.2 (f) Design strategy.
If the notional removal of any bracing element would result in the building being unstable, should that bracing element be designed as a key element regardless of the tying strategy?
A5. The member should only be considered as a key element if the tying requirements, or notional removal requirements, have not been satisfied.

Q6 Page 62/63, section 7.6.5 Combination of actions for notional removal.
It is stated that $\Psi_{1,1}$ is the factor for the frequent value of the variable action $Q_{k,i}$. Should this be $Q_{k,1}$?
A6. Yes, the factor should be $Q_{k,1}$

Q7 Page 121, Example 6 - Class 2b building – Transfer beam.
Horizontal ties. For an internal transfer beam, $T_i = 0.8(g_k + \Psi q_k)sL + 0.5V_c$ in which s and L are the spacing and the length of the transfer beam, respectively. In Example 6, s = 7.5m, and L = 12.5m. However, the tie force T_i is given as :
$T_i = 0.8(3.5 + 0.7 \times 6.0) \times 6.0 \times 7.5 + 0.5 \times 512 = 533$ kN. Is this correct?
A7. T_i should be calculated as: $T_i = 0.8(3.5 + 0.7 \times 6.0) \times 7.5 \times 12.0 + 0.5 \times 512 = 810$ kN

Q8 Page 126, Example 6 - Class 2b building – Transfer beam.
Under the heading "Resistance of beam slab connection", the upward push-out value of the shear stud is assumed to be 10 kN. Are these push-out values documented anywhere?
A8. Unfortunately, there is no such reference document for these values. An estimated value was used in the example.

Q9 Page 126, Example 6 - Class 2b building – Transfer beam.
Under the heading "Load on beam slab connection", the load on the beam-to-slab connection due to the accidental action is given by: $F_1 = 34 \times 7.5 = 255$ kN per m length. Why isn't F_1 given by:
$F_1 = [(2.25 \times \text{storey height})^2 \times 34]/9.0 = 306$ kN/m
i.e. using the same approach which was used earlier in Example 6?
A9. An alternative option would be to use the approach suggested in the question, but see answer to question 10 below.

Q10 Page 126, Example 6 - Class 2b building – Transfer beam.
In the case of "upwards accidental action", the slab becomes detached from the transfer beam; therefore the accidental load from the slab can be ignored. However, the beam still supports the column at mid-span. If we assume that under such accidental loading, the unrestrained secondary beams provide no lateral restraint to the transfer beam, we then have an unrestrained transfer beam subject to a destabilizing load. In which case, shouldn't the transfer beam be designed to resist a net sagging moment $M_{y,Ed}$ given by $M_{y,Ed} = 1539 - 190 = 1349$ kNm ? (Ignoring accidental loading applied upwards to the underside of secondary beams)
A10. Yes, it could be done that way. However, in the case of robustness and avoidance of disproportionate collapse the design codes cannot give rules for all situations so the engineer is required to develop a sensible, logical approach based on engineering principles. This does mean that there may be more than one way to approach particular situations.

Contact: **Andrew Way**
Tel: **01344 636525**
Email: **advisory@steel-sci.com**