The local effects of beam cranks and long notches

Richard Henderson of the SCI considers the effects of accommodating building services on the shear in beam webs.

Services integration is a necessary activity in the design of most structures. In comfort-cooled office buildings various approaches are adopted, depending on the design strategy that has been adopted for the building services. Cellular beams with frequent circular openings allow the passage of pipework and medium-sized circular ductwork. Lattice trusses provide the opportunity to introduce a rectangular opening at mid-span where the shear force in a uniformly loaded beam is small.

Ductwork from central air-handling plant is largest where it enters a floor, before branches are taken off. Such ducts may be routed next to columns where the bending moments in simply supported beams are lower and allow reductions in the beam depths to be made to allow ducts to pass underneath. The elevations of these beams vary depending on their manufacture: fabricated beams can have varying depths; rolled sections can be notched to produce an extended shallower section.

At the change in the depth of the beam, the bottom flange changes direction or position and the effects of the change on the internal forces in the beam must be dealt with somehow. Two possible approaches immediately present themselves:

- Maintain the continuity of the flange and crank it down to the deeper section;
- Continue the flange to the shallow section horizontally and overlap it with the flange of the deeper section.

The effect on the internal forces in the beam from these two options will be considered.

Cranked bottom flange

At the point where the bottom flange changes direction and the beam starts to get deeper, there is a vertical component of the flange force. At the deeper section where the flange becomes horizontal again, the vertical component is in the opposite direction. The forces on the flanges and web can be simply illustrated by assuming the flanges take all the bending and the web takes all the shear. The forces are shown in Figure 1, below.

It is instructive to substitute some realistic values and examine the shear stresses in the beam. Consider a non-composite primary beam of 9.0 m span supporting 9.0 m span secondary beams at third points. The ultimate load per square metre is 11.7 kPa; the reaction and shear force is 316 kN and the design bending moment is 948 kNm. A 600 mm deep beam is adopted in steel grade S355. The beam is reduced to 300 mm deep over a length of 1.2 m from one end to accommodate a major duct. The depth of the beam is increased to 600 mm over a transition length.

At 1.2 m into the span the bending moment is 40% of the maximum value; at 1.5m it is 50% and at 1.8m 60%. The values are 379 kNm, 474 kNm and 569 kNm respectively. Assuming h_1 = 285 mm and h_2 = 585 mm, the flange forces and shear forces in the web can be calculated. The depth is increased from 300 mm to 600 mm over a length of 600 mm ie a slope of 1 in 2. The results are presented in Table 1.

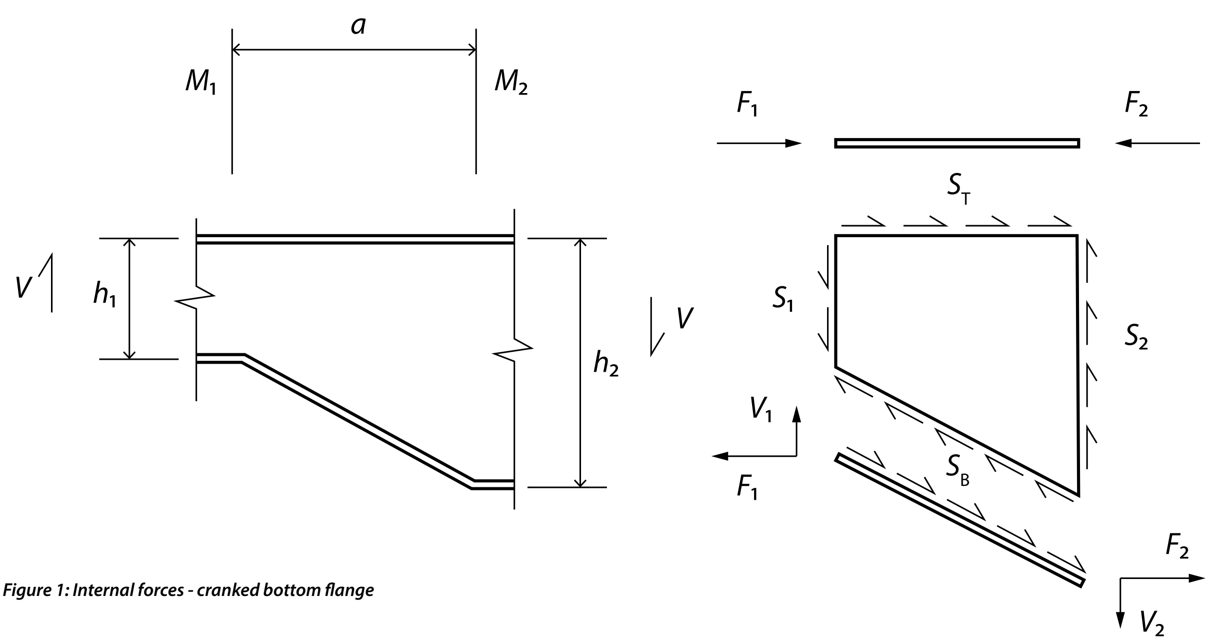

Figure 1: Internal forces - cranked bottom flange

Parameter		Section 1	Section 2
Spacing (mm)	$a = 600$		
Height (mm)		$h_1 = 285$	$h_2 = 585$
Moment (kNm)		$M_1 = 379$	$M_2 = 569$
Flange force (kN)		$F_1 = 1330$	$F_2 = 972$
Vertical component (kN)		$V_1 = 665$	$V_2 = 486$
Shear force on web (kN)		$S_1 = 349$	$S_2 = 170$
Average shear/mm (kN/mm)		$s_1 = 1.22$	$s_2 = 0.29$
Horizontal shear at top of web (kN)	$S_T = 358$		
Horizontal shear at bottom of web (kN)	$S_B = 400$		

Table 1: 1 in 2 slope

The vertical component of the flange forces results in an average shear force over the web height of 1.22 kN/mm at the shallower section and 0.29 kN/mm at the deeper section. These values compare with 0.54 kN/mm at the deeper section and 1.11 kN/mm at the support. For a 10 mm thick web (at the bottom end of the range for a 600 mm deep rolled section), the approximate average shear stresses on the web are between 29 MPa and 122 MPa respectively. The average shear stress in an unmodified beam would be 54 MPa.

Were the designer to choose to slope the bottom flange down at 45 degrees so the beam depth increases to 600 mm over a length of 300 mm, the values would be as shown in Table 2.

Parameter		Section 1	Section 2
Spacing (mm)	$a = 300$		
Height (mm)		$h_1 = 285$	$h_2 = 585$
Moment (kNm)		$M_1 = 379$	$M_2 = 474$
Flange force (kN)		$F_1 = 1330$	$F_2 = 810$
Vertical component (kN)		$V_1 = 1330$	$V_2 = 810$
Shear force on web (kN)		$S_1 = 1014$	$S_2 = 494$
Average shear/mm (kN/mm)		$s_1 = 3.56$	$s_2 = 0.85$
Horizontal shear at top of web (kN)	$S_T = 520$		
Horizontal shear at bottom of web (kN)	$S_B = 735$		

Table 2: 1 in 1 slope

The vertical component of the flange forces results in an average shear force over the web height of 3.56 kN/mm at the shallower section and 0.85 kN/mm at the deeper section. The approximate average shear stress at the shallower section on a 10 mm thick web is 356 MPa – well in excess of the limiting shear stress of $f_y/\sqrt{3} = 205$ MPa. Problems with shear resistance in rolled section beam webs are so rarely an issue that this overstress may catch out the unwary.

For both arrangements, the vertical component of the inclined flange force on the flange itself must also be considered. It acts across the full width of the flange, downward where the bottom flange turns downward. The web is incapable of sustaining the local tension and the force must therefore be transferred to the web through stiffeners. The flange will also tend to bend downward away from the web which acts as a central support. In the second case where the depth transition is more rapid, a stiffener must also be provided to carry the shear force across the trapezoidal web panel because the shear resistance of the web alone is insufficient. The actions at the point where the flange becomes horizontal are in the opposite sense.

Overlapping bottom flanges

If the bottom flange of the shallower portion of the beam is continued along the web to overlap with the bottom flange, the force in the flange sheds through the web into the top and bottom flanges. There is no vertical component of the flange force. The forces are shown in Figure 2.

The length a of the overlap can be chosen to suit the size of weld between the flange and web and the stress in the web panels. The web panel between the overlapping bottom flanges sustains a horizontal shear stress resulting from the flange force from the shallower section transferring across to the flanges of the deeper section. The forces on the different parts of the beam are shown in Table 3.

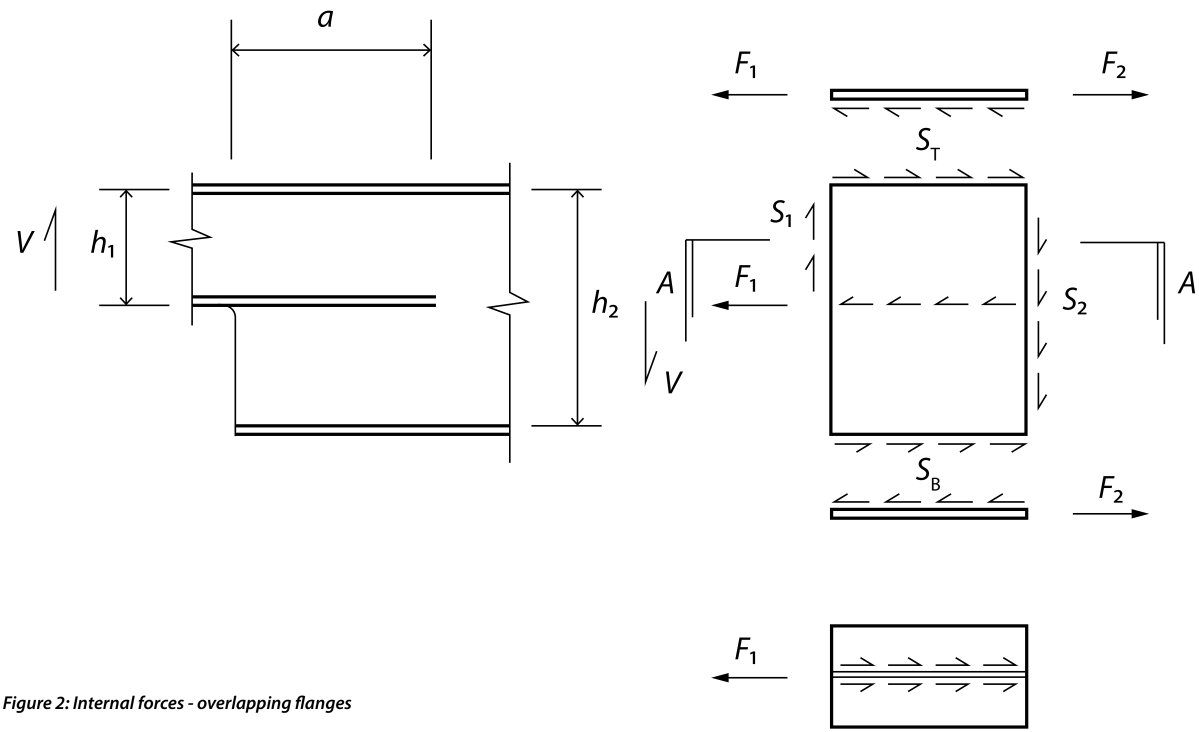

Figure 2: Internal forces - overlapping flanges

Parameter		**Section 1**	**Section 2**
Spacing (mm)	$a = 500$		
Height (mm)		$h_1 = 285$	$h_2 = 585$
Moment (kNm)		$M_1 = 379$	$M_2 = 537$
Flange force (kN)		$F_1 = 1330$	$F_2 = 918$
Shear force on web (kN)		$S_1 = 316$	$S_2 = 316$
Average shear/mm (kN/mm)		$s_1 = 1.11$	$s_2 = 0.54$
Horizontal shear at top of web (kN)	$S_T = 412$		
Horizontal shear at bottom of web (kN)	$S_B = 918$		

Table 3: Overlapping flanges

In the simple model for the force distribution on the section which is being adopted for this illustration, the web does not experience any direct stress. In the bottom panel, the average horizontal shear force/mm is 1.84 kN/mm. On a 10 mm thick web, the shear stress is 184 MPa ie less than $355/\sqrt{3} = 205$ MPa.

Both these approaches can potentially result in arrangements where the shear resistance of the web is exceeded. In the case of the cranks in the flange, the vertical components of the flange force require stiffeners to carry the flange bending about the web and a further stiffener may be required to carry the shear across the trapezoidal panel if the depth transition is too abrupt. If the bottom flanges are overlapped, there is no vertical component of flange force which is therefore conceptually simpler and the length of overlap merely needs to be made long enough to reduce the average shear over this length sufficiently.