Column strengthening

Re-use and adaptation of existing buildings may make column strengthening more common in the future. In this article, Richard Henderson of the SCI considers some of the issues and gives an example of a strengthening design.

1 Possible scenarios

Building refurbishment is likely to be carried out when a building is empty, particularly if the works are substantial and involve changing the internal arrangement of the structure and adding floors for plant rooms or other uses. This means that column loads during refurbishment are reduced significantly below their original design loads.

Other changes of use in multi-tenant buildings may involve the necessity for strengthening due to a change of use, but the presence of other tenants in occupation may limit the scope of work that is possible.

Columns can be strengthened by adding supplementary plates to provide additional area and enhance the other section properties, such that the strengthened section is capable of carrying the additional loads. The additional material can be welded or bolted to the original section. The form of the connection limits where the plates can be attached and affects the efficiency of the strengthening. Attachment by bolting can be done through the flanges of a UC section column, or through the web, but the most efficient strengthening arrangement for an open section element is by welding plates to the toes of the flanges to form a box section.

2 Design issues

The column supports the load present during the refurbishment and will be subject to an average compressive stress and a bending stress due to the compressive load multiplied by the amplified initial bow. Any strengthening plates provided to increase its resistance will be unloaded when they are fixed to the column. In the final state, the existing amplified bow will be further increased due to the additional load and the strengthened section will be subject to an average stress and a bending stress as already described. The original section will be subject to the sum of the stress present at the time of refurbishment and the stress due to the additional load.

3 Example

3.1 Requirement for strengthening

Consider a 305 UC 137 in grade S355 material supporting four storeys of 1092 kN each, with a system length of 4.0 m. According to the Blue Book, the compression resistance $N_{bz,Rd}$ = 4500 kN. the existing design load is 4368 kN and it is desired to add an additional load equivalent to another floor, making the final load equal to 5460 kN. The permanent load on the existing floors during refurbishment is 2.7 kN/m² and the column supports 108 m² of floor. A construction live load of 0.5 kN/m² on one floor is also present, so that the factored column load during refurbishment (and prior to adding the extra storey) is 1656 kN. (Note: a construction live load of 0.75 kN/m² is required by the loading code).

The relevant properties of the column are shown in the table.

Table 3.1 Column properties

Property	Units	Symbol	Value
height	mm	h	320.5
width	mm	b	309.2
flange thickness	mm	t_f	21.7
Area	cm²	A	174
minor axis second moment of area	cm⁴	I_z	10700
minor axis section modulus	cm³	W_z	692

3.2 Construction stage

The initial bow in the column is given by:

$$e_0 = \alpha(\bar{\lambda} - 0.2)\frac{W_z}{A}$$

where α is the imperfection factor for the relevant buckling curve and $\bar{\lambda}$ is the non-dimensional slenderness for flexural buckling. From EC3-1-1 Tables 6.1 and 6.2, buckling curve c applies and the value of α is 0.49. The elastic critical load is:

$$N_{cr} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 1.07 \times 10^{-4}}{16} = 13861\text{kN}$$

The non-dimensional slenderness is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{17400 \times 345 \times 10^{-3}}{13861}} = 0.658$$

Substituting values, e_0 = 8.93 mm

The maximum stress in the column during construction can be calculated using the axial load and the bending moment due to the amplified initial bow:

$$f_c = \frac{N_c}{A} + \frac{N_c e_0}{W_z} \frac{1}{\left(1 - {N_c}/{N_{cr}}\right)}$$

N_c is the design load during the construction stage.

$$f = \frac{1656 \times 10^3}{17400} + \frac{1656 \times 10^3 \times 8.93}{692 \times 10^3} \times \frac{1}{\left(1 - {1656}/{13861}\right)}$$

$$f = 95.2 + 21.4 \times 1.4 = 119.6\text{MPa}$$

This value is the maximum stress in the flange tips.

The original average design stress in the column is:

$$f = \frac{4368 \times 10^3}{17400} = 251.0\text{MPa}$$

3.3 Permanent stage

The additional load to be carried by the strengthened column is 5460 − 1656 = 3804kN. Assuming the stress in the strengthened column is 250MPa, the new area is:

$$\frac{5460 \times 10^3}{250} - 17400 = 4440\text{mm}^2$$

Consider plates welded to the flange toes to box out the section: the distance between the centrelines of the flanges is close to 300 mm. The limiting slenderness for class 3 internal compression elements is:

$$\frac{c}{t} \leq 42\varepsilon = 34$$

The limiting thickness for class 3 is therefore 8.8 mm: use 10 mm plates.

The additional area is 6000 mm² and the area of the strengthened column is 234 cm² – See Figure 1.

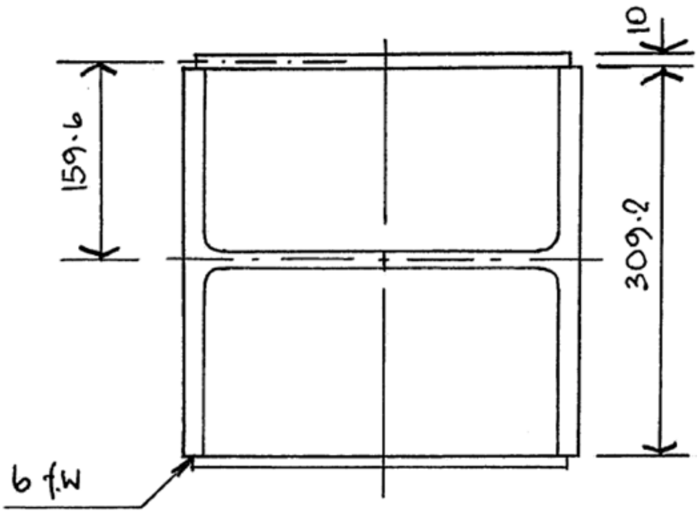

Figure 1: Cross section through strengthened column

The additional plates form a closed section with high torsional stiffness so by inspection, no check of torsional buckling is necessary.

The revised section properties of the column are:

$$I_z = 10700 \times 10^4 + 2 \times 3000 \times 159.6^2 = 2.598 \times 10^8 \text{ mm}^4$$

$$W_z = \frac{2.598 \times 10^8 \times 2}{329.2} = 1578000\text{mm}^3$$

The bow in the column at construction is the initial bow multiplied by the amplifier already calculated i.e. 8.93 × 1.14 = 10.2mm

The stress in the column due to the new load can be calculated as before, using the Euler load for the strengthened column. This is:

$$N_{cr} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 2.598 \times 10^{-4}}{16} = 33654\text{kN}$$

The maximum stress in the column at the extreme fibre is:

$$f = \frac{3804 \times 10^3}{23400} + \frac{3804 \times 10^3 \times 10.2}{1578 \times 10^3} \times \frac{1}{\left(1 - {3804}/{33654}\right)}$$

$$f = 162.6 + 24.6 \times 1.13 = 190.4\text{MPa}$$

The maximum stress in the original column section is the stress at the construction stage plus the stress on the revised section from the additional load. The magnitude of this stress is approximately

$$f_{total} = 119.4 + 190.6 = 310\text{MPa} < 345\text{MPa}$$

The strengthening is satisfactory.

Based on the average axial stress, the load in the new plates is in proportion to their area:

$$\frac{A_{plts}}{A_{tot}} = \frac{6000}{23400} = 0.256$$

The load in each plate is therefore 12.8% of the additional load in the strengthened column i.e about 488 kN per plate. If the plates are site fillet welded between the top surface of the concrete slab and the underside of the beam above, connecting to the column web, the original column section in the ceiling zone must be able to carry the new load so a cross-section check is required. The cross-section resistance is 6000 kN so the column is satisfactory.

Welds are required to get the load into the strengthening plates and out again and it is appropriate to achieve this over a short length. Check the weld size required to develop the plate load over a length equal to the plate width i.e. about 300 mm. The design force per mm is therefore 488 / (2 × 300) = 0.813kN/mm Using the Blue Book, a 5 mm fillet weld would be adequate. Using a common weld size, provide a 6 mm fillet weld over 270 mm on each side at the top and bottom of each plate. See Figure 2.

Once the force has been transferred into the strengthening plates, a connection between the plates and the column is required to transfer longitudinal shear into the plates due to the change in bending moment along the column and to prevent them from buckling under the axial load. Intermittent fillet welds could be used down the length of the column to achieve this.

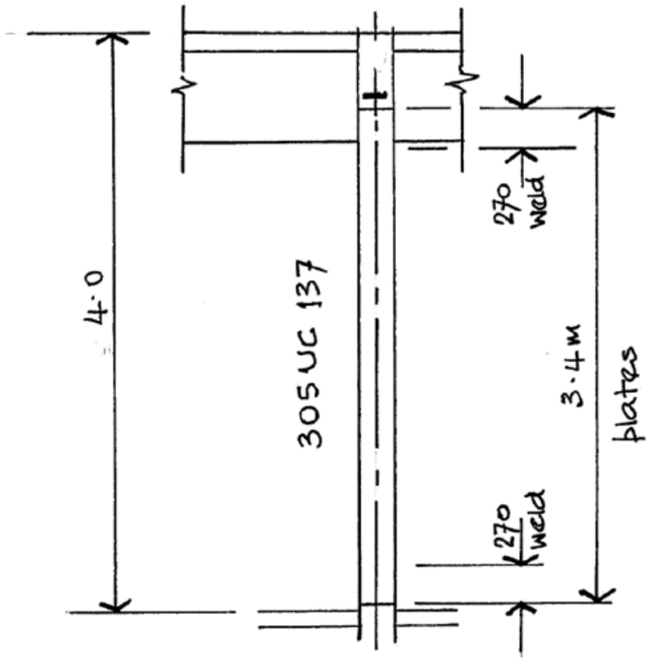

Figure 2 Elevation on strenthened column

3.4 Discussion

The arrangement of plates to strengthen the column was chosen to maximise the effect of the additional material. By boxing the column, the minor axis bending was increased significantly with plates of modest thickness. Alternatives such as adding plates to the flanges or webs would have been much less effective. For example, to achieve the same second moment of area, adding 300 mm wide plates to the flanges would have required 34 mm thick plates. Adding plates to the web would have required even thicker plates. Plating the flanges would have allowed the connection to have been achieved by bolting but this option does not seem to have a net benefit. ■