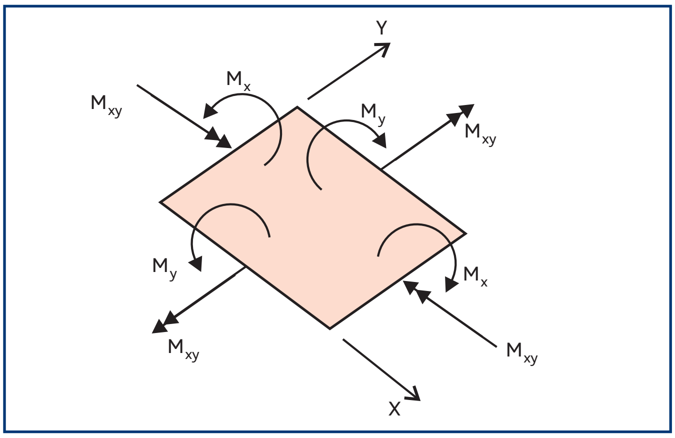
must be considered in the reinforcement design. M_{xy} does not act in the direction of the reinforcement and a method is required to allow for M_{xy} in the design. A popular method in the UK is known as Wood Armer moments, although it is not the only method used. Most software will calculate Wood Armer moments for the user. They have four components, top (hogging) moments in the x and y directions, $M_x(T)$ and $M_y(T)$, and bottom (sagging) moments in each direction, $M_x(B)$ and $M_y(B)$. The method is slightly conservative and these moments form an envelope of the worst-case design moments. It is possible to have both $M_x(T)$ and $M_x(B)$ moments at the same location in the slab (usually near the point of zero shear).

The four components can be used directly to calculate the required reinforcement for each of the four reinforcement layers in a flat slab.

Figure 12
Design bending moments compared with FE output



Design moment adjustment

Where high peak moments occur the concrete will crack and the reinforcement may yield if its the elastic limit is exceeded. The forces are then shed to the surrounding areas. Even if a slab were designed to resist this moment it is unlikely that it would actually achieve this capacity for the following reasons:

- The construction process often leads to construction stage overload.
- The reinforcement is unlikely to be placed at exactly the point of peak moment.

It is therefore necessary to acknowledge that some shedding of the peak moments to adjacent areas will occur due to the material properties of concrete, and not attempt to design against it. In fact a recent paper by Scott and Whittle[13] concluded that redistribution occurs even at the SLS because of the mismatch between the uniform flexural stiffnesses assumed and the variation in actual stiffness that occurs because of the variations in the reinforcement.

When using FE, especially for slabs with irregular geometry, it is not usually possible to carry out redistribution of the moments for the following reasons:

- It is not simple to determine where to distribute the hogging moment to.
- If the software is carrying out the design there is usually no method for changing the analysis output.

In the future, software that models the yielding of the reinforcement will automatically redistribute the moments and find an equilibrium solution.

Punching shear

Although an FE model will produce shear stresses, where the columns are modelled as pins they have no effective shear perimeter and the shear force is infinite. In this case the simplest way to check punching shear is to take the reactions from the model and carry out the checks in the normal way using the provisions in the codes of practice. This can be automated by using a spreadsheet for the design of reinforced concrete[11].

If the area of the column has been modelled, then realistic shear stresses can be obtained, but some engineering judgement may be required in using them because there will be peaks which may exceed the design limits in the codes.

Some software can undertake the punching shear checks and design of the reinforcement, and the user should ensure that openings within the shear perimeter are considered in the software.

Interpreting results

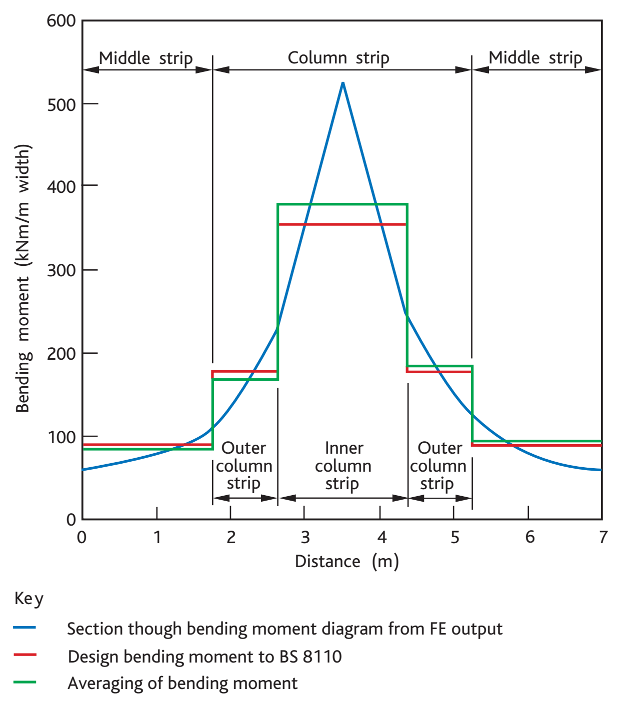
The results from an FE analysis will generally be in the form of contour plots of stresses and forces, although a 'section' through the contour plots (either bending moment or areas of steel) can usually be obtained. These will show very large peaks in bending moment at the supports. The temptation to provide reinforcement to resist this peak moment should be avoided. This potential error stems from a lack of understanding of the assumptions made in the modelling. The reinforcement in the concrete will yield at the support position and the moment will be distributed across a larger area; it is not therefore necessary to design to resist this peak moment. However, a method is required for distributing this peak moment across a larger area.

BS 8110 and Eurocode 2 deal with the peak in bending moment for flat slabs by averaging it over the column strip and middle strips (Cl.3.7.2.8, BS 8110 and Annex I, Eurocode 2), with the columns strip sub-divided into inner and outer areas. This method can be used for designing reinforcement using the results of an FE analysis. A section is taken across the bending moment diagram (i.e. in the y direction for moments in the x direction) at the face of the column (the blue line in Figure 13). The total bending moment is the area under the blue line (i.e. the integral), which can be apportioned according to rules given BS 8110 or Eurocode 2.

If the BS 8110 principles are adopted then the design moments would be as shown by the red line in Figure 13. Here three-quarters of the total moment is apportioned to the column strip (which is half the bay width) and of this two-thirds is apportioned to the inner column strip. The remaining column strip moments are assigned to the outer areas and the middle strip moment is distributed equally across the remaining bay width.

The rules in Eurocode 2, Annex I (Table I.1) allow more flexibility in apportioning the total moment for the bay width to the column and middle strips. However, Eurocode 2 is more rigid in terms of how much reinforcement should be applied to the inner column strip. Cl. 9.4.1(2) requires that half the total reinforcement area for the bay width is placed in a strip that extends to a quarter of the bay width and is centred over the support.

Figure 13
Design bending moments compared with FE output



Both BS 8110 (Cl. 3.7.2.6) and Eurocode 2 (Cl. 5.3.2.2 (3) & (4)) allow the design moment to be taken at the face of the support, indeed Eurocode 2 indicates this should be done. However, it may be prudent for the design moment at edge columns to be taken at the centre of the support. This is because of uncertainties in the modelling and because it is critical that the moment is transferred from the slab to the column in these locations, if this has been assumed in the design.

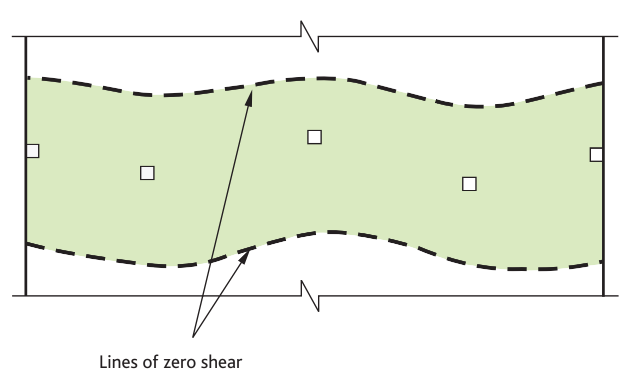
An alternative method is to simply average the bending moment over a width of slab. However, if designing to Eurocode 2 the requirements of Cl.9.4.1(2) should be adopted. The widths of these strips can be determined by the designer; an example is shown by the green line in Figure 13. Here the same strip widths as the BS 8110 method have been adopted to show how the results compare. This method has the advantage that it can be used for a slab with irregular geometry, because a fixed bay width is not required. It can also be used with area of steel results, removing the need to calculate the reinforcement areas by hand. It will be seen that both methods give a similar distribution of reinforcement when applied to the same strip widths.

An alternative way of determining design bay width is to use the method set out in Concrete Society report TR43[14]. This method has been developed for post-tensioned concrete design, assuming the analysis is at the serviceability limit state and for a homogeneous elastic plate. However, the principle that the bay width is taken as being the distance between the lines of 'zero shear' may still be applied (see Figure 14). This principle is particularly useful for unusual geometries where using the lines of zero shear give a good basis on which to determine the bay widths.

Whichever method is chosen, engineering judgement should be applied for unusual situations, making sure that there is sufficient reinforcement to resist the applied moment, without being overly-conservative.

A useful rule of thumb for verifying the results is that top reinforcement in the column strip will be in the order of twice the area of the bottom reinforcement (i.e. not the same as, or 4 times as much as, the bottom reinforcement).

Figure 14
Extract of shear diagram indicating lines of zero shear



Serviceability limit state design

The design of flat slab floors is usually governed by the serviceability requirements. Deflection is influenced by many factors, including the tensile and compressive strength of the concrete, the elastic modulus, shrinkage, creep, ambient conditions, restraint, loading, time, duration of loading, and cracking. With so many influences, and many which are difficult to accurately predict, the deflection calculation should be regarded as an estimate only. Concrete Society report *Deflections in concrete slabs and beams*[8] advises that the difference between calculated and actual deflections falls in the range +15% to –30%