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Important notice

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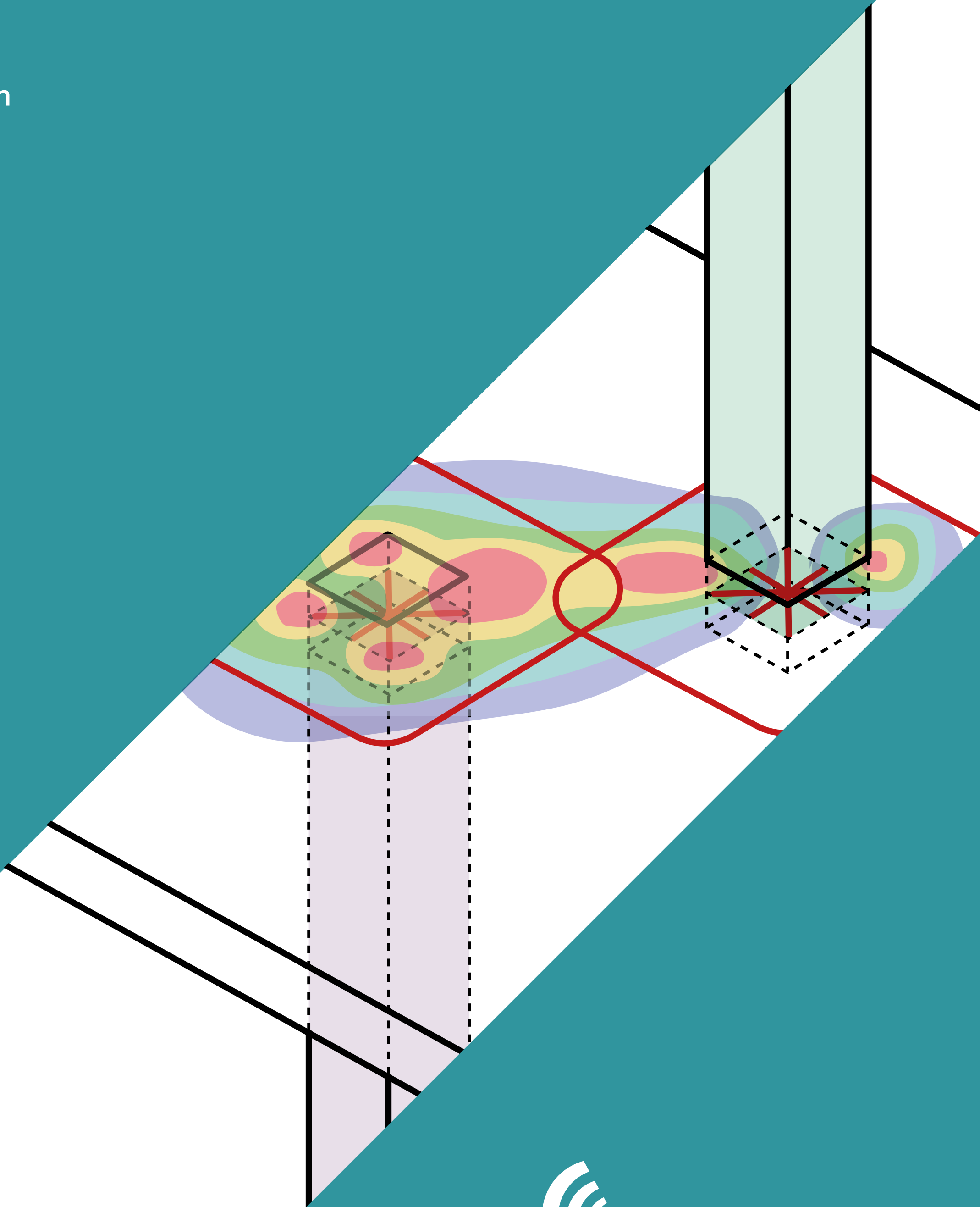
Free guidance

Design of transfer slabs

First edition



IStructE Guide

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Quality Information

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1 Introduction

Transfer slabs are an increasingly common structural typology in the UK, but at present there is no industry-wide guidance covering their design. This paper provides high-level guidance to engineers who are faced with the challenge of designing a transfer slab.

An introduction to the key considerations of transfer slab concept design is first provided, followed by more detailed guidance on appropriate finite element modelling, flexural design and shear design.

The attention of readers is particularly directed to the shear assessment guidance in Section 4, as this differs significantly from the shear assessment of typical reinforced concrete flat slabs.

This guidance has been produced by a cross-industry group in response to a gap in advice in the application of standards in the design subject. In such circumstances engineers have adopted different methods. This guidance is aimed at ensuring that the design of new transfer slabs is consistent and robust, in accordance with current knowledge. It is not a statutory document. It is acknowledged that there will be transfer slabs that were designed before the publication of this guidance which may not comply with all of the recommendations in this guide. Non-compliance with this guidance does not necessarily lead to the conclusion that the design of an existing transfer slab is structurally inadequate or statutorily non-compliant. The technical and legal issues concerning assessment of the adequacy of existing concrete structures is a specialist topic which is not covered by this guide. Members should consider the need to seek specialist advice if required to assess the adequacy of any existing structures designed using different methods than those outlined in this guide.

1.1 What is a transfer slab?

The term transfer slab refers to any slab (or section of slab) where the load from a planted column that sits on top of the slab is transferred to a supporting column beneath through the slab directly, without the provision of a transfer beam

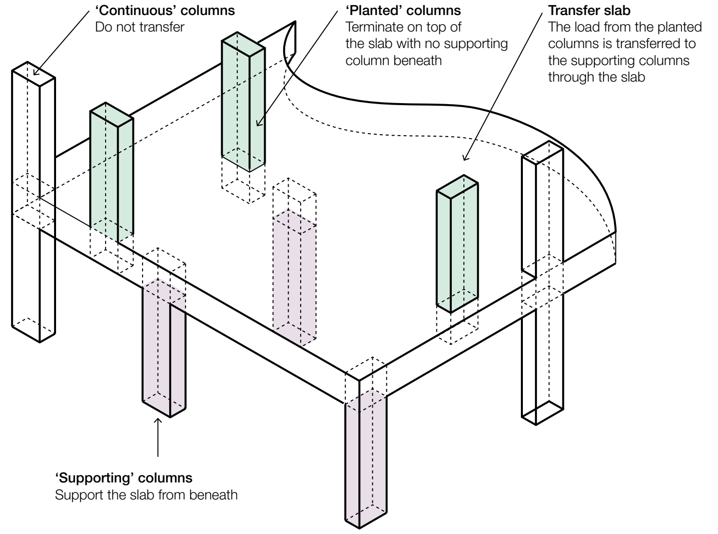


Figure 1: Transfer slab terminology

Transfer slabs are now commonly used in reinforced concrete (RC) buildings throughout the UK to facilitate the transition from one column grid in the lower section of a building to another grid higher up the building. Common locations where transfer slabs occur include:

- **As Ground Floor podium slabs:** separating an undercroft (where column spacing is defined by a car parking arrangement) from the main building above (where greater freedom of column positioning is desired).

- **At 1st or 2nd floor in a mixed-use tower block:** separating the Ground Floor retail space (with a wider column grid) from the residential apartments above (with typically a smaller grid).

- **At high level to facilitate a step back in the facade:** to allow the edge columns to step back from the façade line, creating a roof garden space around the perimeter of the roof.

This paper addresses the transfer of column elements only (ie vertical elements with an aspect ratio less than 4:1). It does not provide guidance on the design of transfer walls (vertical elements with an aspect ratio greater than 4:1) that may attract significant lateral load. It is recommended that the shear walls in a building are continuous over the full height of the building from foundations to roof wherever possible, without horizontal transfers.

1.2 Do you really need a transfer slab?

The first question that an engineer should ask when faced with the design of a transfer slab is whether the use of a transfer slab can be avoided.

The appeal of transfer slabs to an architect or client is that they provide almost total freedom to change the column grid above and below the slab. This architectural freedom, however, has significant structural implications. Most notably:

- **Structural inefficiency:** Transfer slabs need to be much thicker than regular flat slabs and require large diameter reinforcing bars to achieve minimum reinforcement provisions. The area where this deep slab is working efficiently is usually only between planted and supporting columns. The remainder of the slab is often underutilised, adding unnecessary weight and embodied carbon.

- **Significant load path uncertainty:** Irrespective of whether the assumed load path in the building is based on a whole-building finite element (FE) model or a tributary area load rundown, the inclusion of a transfer slab introduces significant uncertainty over the most appropriate load path for the design. The true load path in the building will depend on the construction and propping sequence of the building, which the designer is unlikely to know during detailed design.
  When a transfer slab is present, the engineer should therefore consider multiple methods of load rundown and compare the results before making a judgement on which is most appropriate.

- **No industry guidance:** In the absence of industry guidance, designing a transfer slab requires the engineer to have great confidence in their understanding of short- and long-term concrete behaviour, design against disproportionate collapse and the behaviour of concrete in shear.

Considering the above, it is desirable structurally to rationalise the grids above and below the slab and to provide a series of discrete transfer beams, rather than using a transfer slab.

Wherever possible, the engineer should be actively involved in the early design development of the building and should make a clear case for the rationalisation of column grids and the avoidance of transfer slabs. If the engineer is only engaged once the building concept and room layouts have been determined by the architect and client, their ability to influence the column grid will be limited (Figure 2).

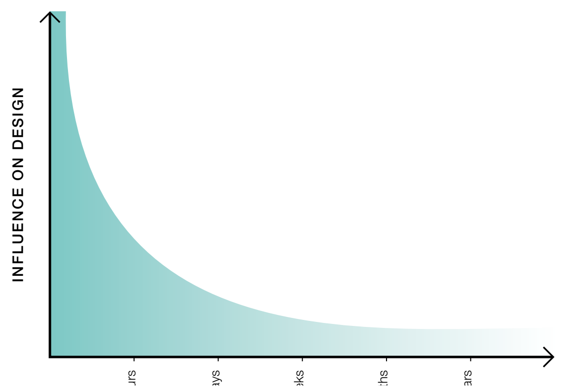


Figure 2: Ability to influence design versus how long the project has been running (reproduced from IStructE 'Conceptual Design of Buildings')

1.3 Limitations of this paper

- As with all aspects of RC design in the UK, transfer slabs should be designed and detailed in accordance with the provisions of BS EN 1992-1-1 and wider industry guidance. This paper highlights specific areas relating to the design of transfer slabs that may not be obvious to an engineer approaching the topic for the first time; it does not provide a comprehensive step-by-step guide or replace the need for a wider appreciation of RC design best practice.

- The guidance presented in this paper is limited to buildings of 15 storeys or fewer. Buildings taller than 15 storeys are categorized in BS EN 1991-1-7 as Consequence Class 3 for robustness design. These buildings require a systematic disproportionate collapse risk assessment, considering any foreseeable hazard that might lead to the building being significantly damaged and ensuring that the building is suitably robust to withstand these hazards.

  The robustness design method presented in Section 2.2 of this paper would not in itself be sufficient to justify that a transfer slab is suitably robust for use in a Class 3 building. Additionally, if a transfer slab is required to support more than 15 storeys then it is likely to be so deep that design using normal beam theory assumptions is not valid (see point below).

- When designing a deep slab, such as a transfer slab, engineers must consider whether normal beam theory still applies in the location considered (ie whether plane sections remain plane). In Section 4.2, the guidance provided for shear assessment is divided into regions where normal beam theory is typically applicable (often referred to as Bernoulli or B-regions), and regions where plane sections do not remain plane (so called disturbed or D-regions), based on the offset between the planted and supporting columns.

  However, engineers should be aware that for thick transfer slabs (typically where the slab thickness is greater than 1/10th of the span), the effects of shear deformation are likely to be significant **across the whole slab**, and therefore special consideration is required.

  For example, thick slab elements should be modelled and analysed based on 'Reissner-Mindlin plate theory', rather than the more commonly used 'Kirchhoff plate theory' (most FE packages will include the option of modelling 2D elements as Reissner-Mindlin plates). Detailed guidance on the use of thick shell elements is not provided in this paper.

- The guidance in this paper is for the design of new transfer slabs only. The assessment of transfer slabs in existing buildings presents wider challenges that are not covered here.

- This paper is only applicable in areas of low seismicity, such as the UK. In areas where consideration of seismic loading is required, there are likely to be design requirements which restrict the use of transfer structures and may prohibit the use of a transfer slab altogether.

2 Concept design of transfer slabs

2.1 Estimating required transfer slab thickness

The first challenge facing the engineer is estimating how thick the transfer slab should be. For transfer structures, the use of a simple span-to-depth ratio is not considered appropriate. The following criteria impact on the required thickness of a transfer slab, and should each be considered before settling on a final dimension:

- **The shear resistance of the slab in regions between planted and supporting columns:** This is the crucial driver of transfer slab thickness. If the slab is too thin, it will not be possible to achieve adequate shear resistance in the slab, no matter how much flexural and shear reinforcement is provided. The concentration of shear stress that occurs between planted and supporting columns is illustrated in Figure 3.

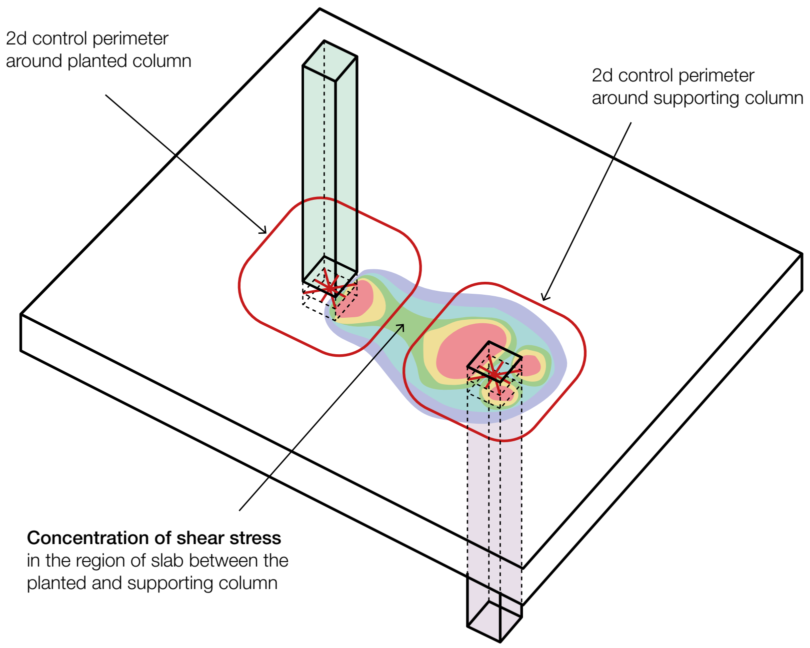


Figure 3: Concentration of shear stress in the slab around closely offset transfer columns

- **The required compression anchorage of column reinforcement:** If the planted and supporting columns rely on compression reinforcement for their axial capacity, then the effective depth of the slab will need to be at least the required compression anchorage length of the column reinforcement. Engineers are reminded that only the projected straight length of the bar is to be counted towards the compression anchorage length (Clause 8.4.1(3) of BS EN 1992-1-1).

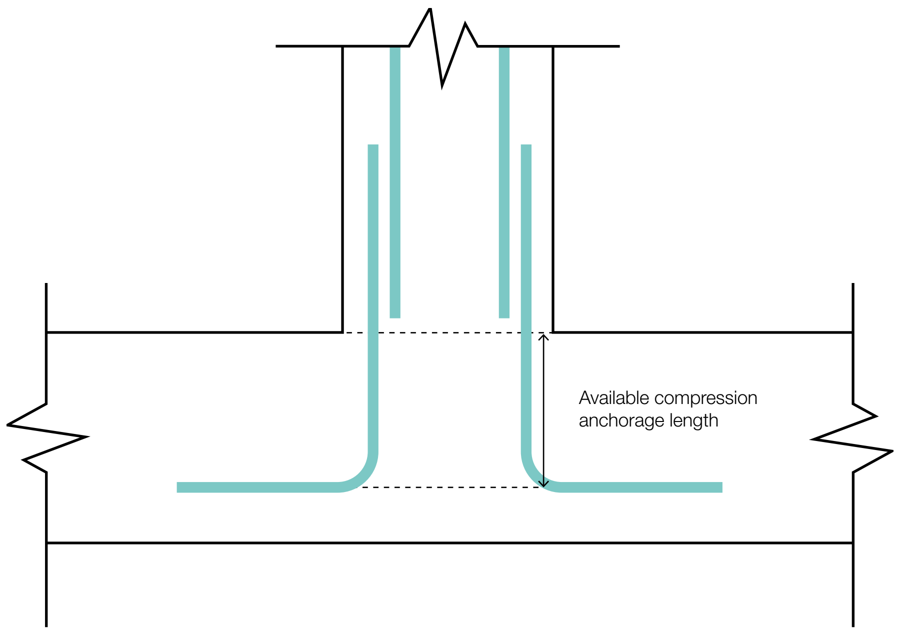


Figure 4: Section through a transfer slab showing planted column compression reinforcement

- **The required slab stiffness to achieve the desired building load path:** If the slab is too shallow (and hence flexible) then it will significantly affect the load path in the building. Some planted columns may experience very little load compared to the values given by a tributary area estimate, whilst other columns that are continuous through the slab may end up significantly overloaded.

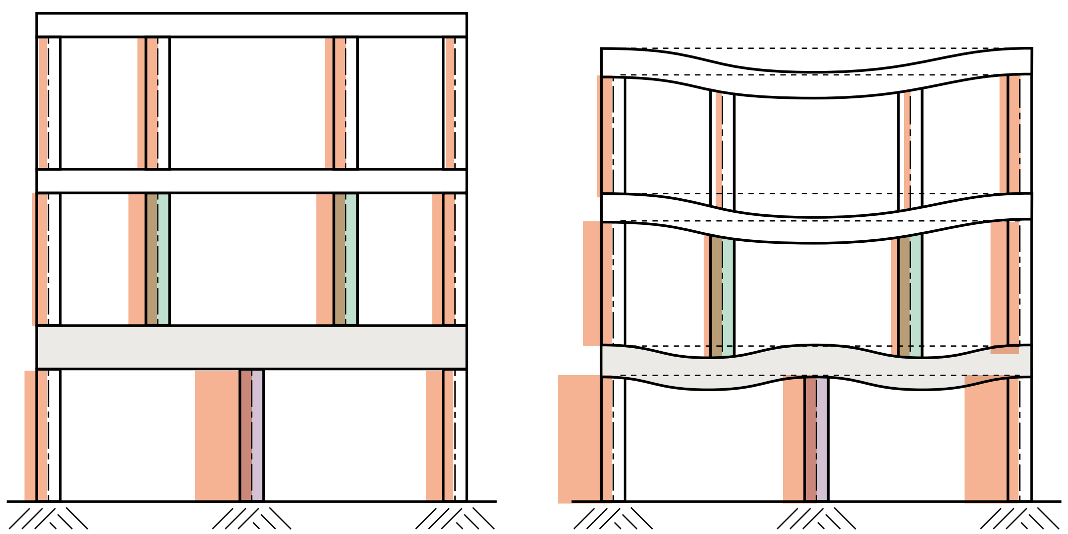


Figure 5: Impact of transfer slab stiffness on building load paths

2.1.1 Initial estimate of slab depth

When developing the concept design for the building, the minimum effective depth of the transfer slab can be estimated as:

$$d_{transfer} = \begin{cases} \mathbf{n_{storeys} \times 100 \geq 250} & (1 \leq n_{storeys} \leq 10) \\ \mathbf{250 + (n_{storeys} \times 75)} & (n_{storeys} > 10) \end{cases}$$

Where:
- $d_{transfer}$ = required effective depth of transfer slab (in mm)
- $n_{storeys}$ = number of storeys supported by the transfer slab (including the roof slab, but not including the transfer slab itself)

The required overall slab depth, $h_{transfer}$ can then be estimated as $h_{transfer} = d_{transfer} + 75$mm.

This method is valid for transfer slabs where there are no close offsets. In this context, a close offset is defined as a location where the planted column has a clear offset from the supporting column of less than 0.2L (where L is the typical bay width of the supporting columns).

2.1.2 Required depth for slabs with close offsets

If there are transfer locations where the planted column has a clear offset from the supporting column of less than 0.2L, then the initial slab thickness estimate method above is not appropriate.

In close offset locations, most of the axial load in the planted column will transfer directly to the closest supporting column. This gives rise to a peak shear stress in the zone of slab between the planted and supporting columns, as shown previously in Figure 3. A check needs to be completed to ensure that the slab in this zone is deep enough to resist the anticipated shear stress.

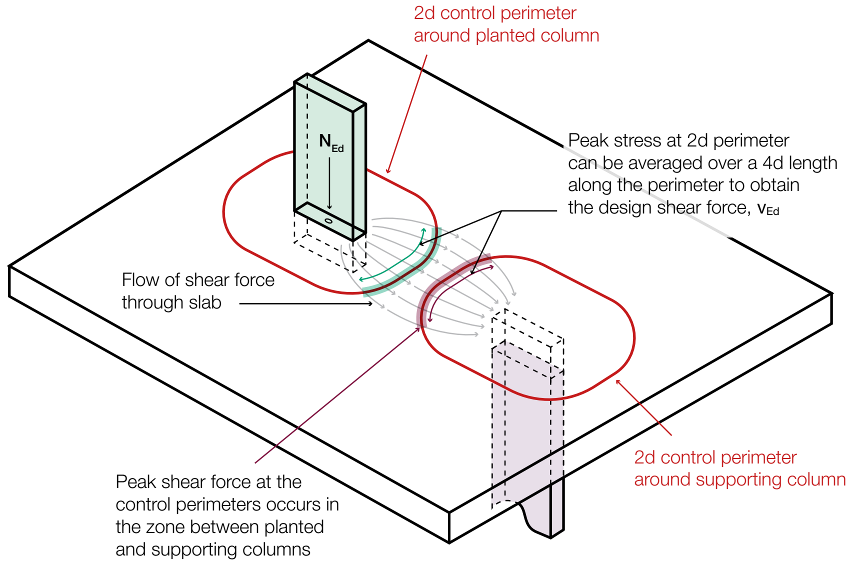


Figure 6: Shear force transfer between closely offset columns

The minimum effective depth of slab required to resist this shear concentration can be estimated for initial design by the expression:

$$d_{transfer} = 150 + (0.25 \times N_{Ed}) \geq 250 \qquad (N_{Ed} \leq 5000\text{kN})$$

$$850 + (0.1 \times N_{Ed}) \qquad (N_{Ed} > 5000\text{kN})$$

Where:
N_{Ed} = ULS axial load in the planted column (expressed in kN).

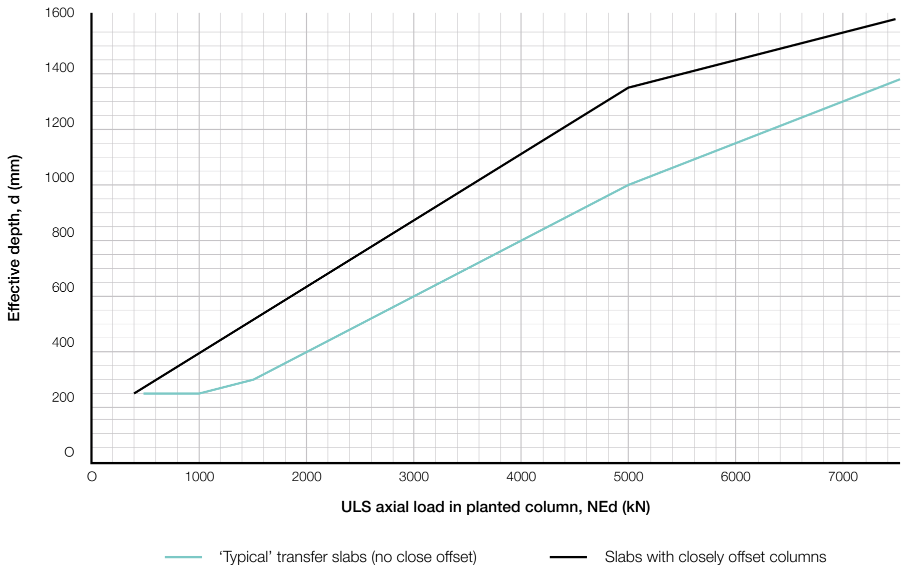


Figure 7: Comparison of minimum effective depth required for typical transfer slabs and for transfer slabs with closely offset columns

(NB: The green line in the graph above was plotted based on an assumed 'ULS load per floor' in the planted columns of 500kN).

This is an initial estimate only: *The methods presented in Sections 2.1.1 and 2.1.2 should be thought of as a starting point only. The final slab depth required will need to be determined by detailed calculation. Both methods consider only the strength of the slab; in some cases the thickness of the slab will be governed by stiffness considerations.*

2.2 Robustness

2.2.1 Eurocode requirements

BS EN 1992-1-1 9.10.2.5 (3) states: 'Where a column or wall is supported at its lowest level by an element other than a foundation (e.g. beam or flat slab) accidental loss of this element should be considered in the design and a suitable alternative load path should be provided.'

In other words, whenever there are transfer structures in a building, the engineer must ensure that the transferring element and its supporting structure are suitably robust to prevent disproportionate collapse. Transfer slabs clearly meet this criterion, and as such their robustness should be a key consideration from concept through to detailed design. It is important that the engineer is aware of this requirement in the early stages of the project as it may significantly affect the required slab geometry (or in some circumstances may make the use of a transfer slab unviable).

The recently published second edition of the IStructE 'Practical guide to structural robustness and disproportionate collapse in buildings' provides detailed guidance on design for robustness, and it is recommended that designers review this guidance in detail before proceeding with the design.

In brief, there are two options available for justifying that a transfer system is suitably robust:

1. **Consider notional removal of the transferring element (beam or slab)**

2. **Treat the transferring element (beam or slab) as a key element and design it to resist a specified accidental load**

Of these two, treating the transfer slab as a key element is probably the more practical for transfer slabs (as the notional removal option will inevitably give rise to questions such as: 'What extent of slab should be removed?' and 'What to do in locations where several planted columns are close together in the same bay?').

It is therefore recommended that engineers follow the key element approach (applying an area load of 34 kN/m^2). The accidental pressure load of +/-34 kN/m^2 can either be applied over the entire slab area (if practical), or Section 5.11 of the IStructE robustness guide suggests that applying this 34 kN/m^2 over a 6m x 6m area may be appropriate for slabs.

However, be aware that this is a relaxation of requirements and will need to be justified on a case-by-case basis considering the specific slab geometry. If the 6m x 6m option is pursued, then a suitable range of load patterns (and orientations) across the slab must be considered to ensure that the worst-case accidental loading has been reviewed.

The columns that support the transfer slab should also be treated as key elements and should be designed to resist an accidental pressure load of +/-34 kN/m^2 acting on their largest face.

2.2.2 Simple detailing rule to improve slab robustness

The UK National Annex to EC2 (2023), which is being developed at the time of writing, is likely to recommend that a minimum of 2 perimeters of punching shear reinforcement are provided around all columns in suspended slabs to improve slab ductility and robustness.

It is recommended that this simple detailing rule is applied to the design of transfer slabs, even if the slab in question is being designed to EC2 (2004). At all column locations (supporting, planted, and continuous) in transfer slabs, **a minimum of 2 perimeters of punching shear reinforcement should be provided**. These perimeters must be detailed in accordance with **EC2 (2004) Cl. 9.4.3**.

3 FE modelling of buildings containing transfer slabs

3.1 Determining transfer loads for design

There are several published documents providing excellent guidance on the correct and safe use of linear elastic FE analysis for RC design. As a starting point, engineers are referred to **Concrete Society Technical Report 64 'Guide to the Design and Construction of Reinforced Concrete Flat Slabs**'. This paper will not aim to reproduce the information presented in existing guides. Rather, designers are reminded of the following key considerations which are particularly relevant to transfer slab design:

- **Be aware of the limitations of the software that you are using:** Most FE analysis packages derive the load path through the structure based on the relative stiffness of each element in the structure. In an FE model of an RC structure, the stiffness of each element is calculated based on the assumption that concrete is a linear elastic material (ie that it is homogeneous, and that stiffness remains constant irrespective of the direction or magnitude of load applied).

  This is not an accurate representation of concrete behaviour. The behaviour of concrete in tension after cracking will be different from concrete in compression, will often be different in each direction, and the stiffness of elements subjected to long-term loading will be different from the stiffness of elements that experience only transient loading.

  Designers should always be aware of this fundamental limitation at the heart of their modelling when building and interpreting an FE model.

- **Consider construction sequence modelling:** If a 3D building geometry is modelled in FE software and then the calculation is run, the model will assume that the concrete self-weight and all applied loading appears simultaneously once the building is fully constructed. In the real building, the self-weight of the concrete will have been applied storey-by-storey as the building is constructed, with stresses associated with the self-weight of each floor locked into the supporting floors below.

  For tall buildings, and buildings with transfer structures, the effect of construction stage loading can be very significant. Most FE packages have functionality to model construction stage loading, and engineers are encouraged to make use of this functionality when modelling buildings with transfer slabs. At RIBA Stage 3, when most FE modelling and member sizing takes place, the engineer is unlikely to know the actual construction sequence of the building, but they should be able to make a reasonable approximation based on experience and consulting with other experienced engineers.

  However, it is important to remember that construction stage modelling will not capture complex behaviours such as propping and de-propping sequences, and as such the predicted load path is still just a bounding analysis, not the *true* behaviour of the building.

- **FE mesh density used for 2D elements should follow the rate of change of stress:** This means that the mesh provided around column heads, wall tips and wall corners will need to be more refined than the mesh at the midspan of the slab. For transfer slabs, the zone between the planted and supporting column will require a greater mesh density. Most FE software packages have an 'Automatic Mesh Refinement' function that will refine the mesh locally based on the stress gradient across the slab.

The practical take-away from the above points is that, where transfer slabs are present in an FE model, great care needs to be taken. It is recommended that the load path is assessed in multiple ways (e.g. tributary area method, wished in place FE model, construction stage FE model), to gain a detailed understanding of how the building frame is behaving and how reliable the results from the model are.

It is also recommended that the relative stiffness of the transfer slab is varied (consider say 0.25x, 0.5x and 1.0x elastic stiffness of the concrete section) to understand how sensitive the load path in the building is to long-term movement of the transfer slab. See also Section 6.3 on the importance of considering the long-term behaviour of the slab and the impact that this may have on the building load path.

Multiple transfer levels compounds load path uncertainty:
If there are multiple transfer levels in the building, the load path uncertainty is compounded, and even greater caution is required.

3.2 Detailed assessment of transfer slab behaviour

To eliminate some of the uncertainty associated with the behaviour of whole-building FE models, it is recommended that the detailed design of transfer slabs is undertaken with reference to an isolated floorplate model, ie an FE model of the transfer slab in isolation, with only the vertical supporting structure directly above and below the slab modelled.

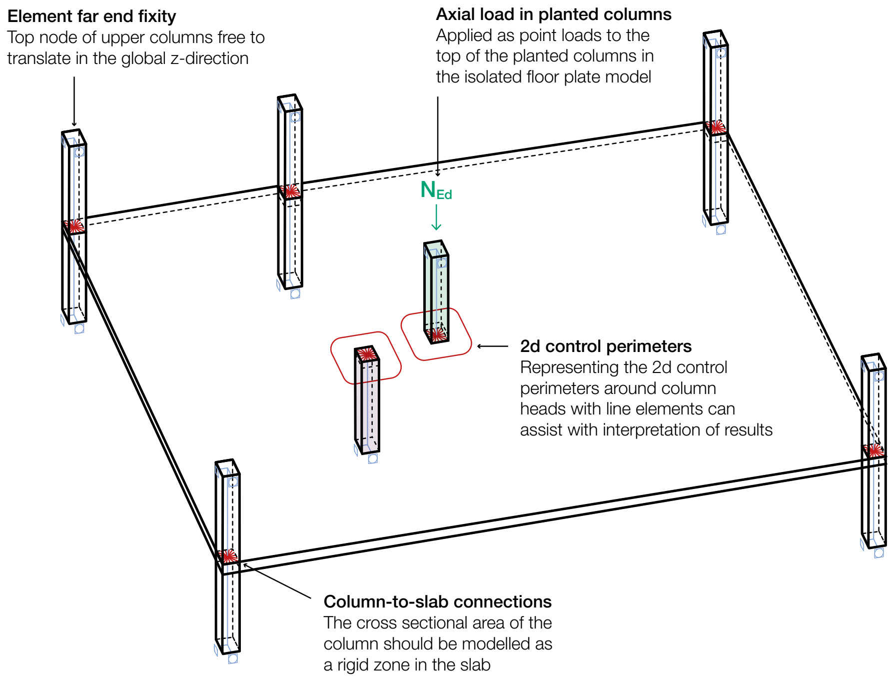


Figure 8: Example of an isolated floorplate model of a simple transfer slab

The design axial load in the planted columns (determined following a detailed consideration of possible load paths) should be applied as point loads to the top of the planted columns in the isolated floor plate model.

When setting up this model, careful consideration should be given to the modelling of the connection between the slab and the columns, and how accurately this will represent the moment and shear transfer behaviour. The most accurate way to represent the shear transfer between columns, and also ensure that the moment transfer between columns and slabs is accurately accounted for, is to model the cross-sectional area of all planted and supporting columns as rigid zones in the slab.

3.3 Use of Finite Element Analysis for shear assessment

All of the shear assessment methods presented rely on interrogation of a linear elastic finite element analysis (FEA) model. As such, ensuring that the FE models are set up correctly and reviewed in detail by a competent engineer is essential.

The following considerations are particularly pertinent when using an isolated FE model to determine the shear stresses in the slab.

3.3.1 Modelling of openings in the slab

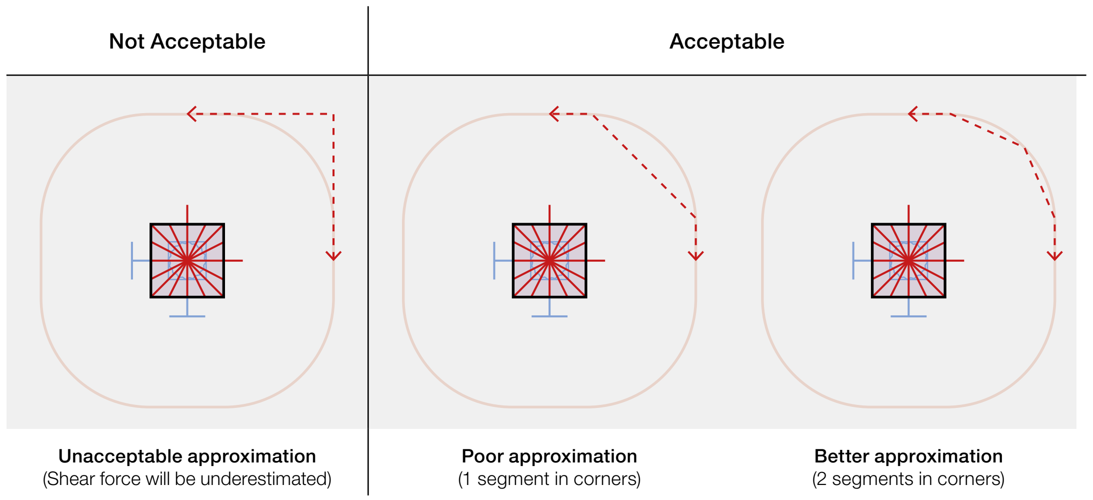
The level of detail required in the FE model is a matter of judgement for the engineer. However, for transfer slabs, one important area where detailed modelling is required is openings in slabs. **All openings within 6d of a column location should be modelled**, even small openings that might typically be ignored. This is crucial, as openings adjacent to columns can have a significant effect on the shear flow through the slab and hence the peak shear stress values obtained from the model.

3.3.2 Extracting shear forces from the slab

It is often not possible in FE packages to extract the forces in a slab along a curved section line. As such, the section lines used to extract the shear forces from the slab may need to be straight lines that approximately follow the control perimeter.

NB: The phrase ‘analysis section’ is used in this guide as a generic term referring to any element or tool that is used to extract forces from a 2D shell in the FE model. Most software packages will have this functionality but may use different terms for the 1d post-processing elements (sometimes they are line elements that ‘cut’ through the slab in a given location, and in other packages they may be strips that integrate the force in the slab over a defined width).

When setting up the analysis sections, they should always sit inside the control perimeter that is being assessed. Analysis sections that extend beyond the 2d perimeter are likely to result in underestimation of the design (average) shear force. The more segments the analysis section is broken into around the control perimeter corners, the better the approximation.



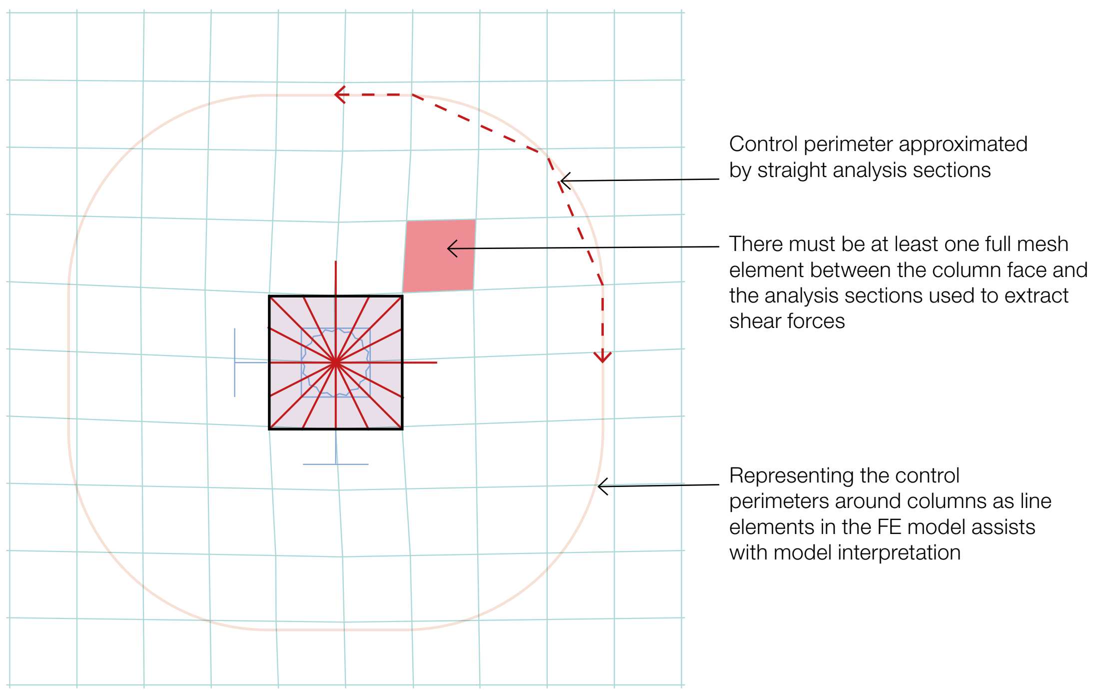
3.3.3 Refinement of mesh around column heads

As noted in Section 3.1, the FE mesh density used for 2D elements should follow the rate of change of stress in the slab. As there are significant stress concentrations in the slab around both planted and supporting columns in a transfer slab, it therefore follows that the mesh around the column heads will need to be suitably refined.

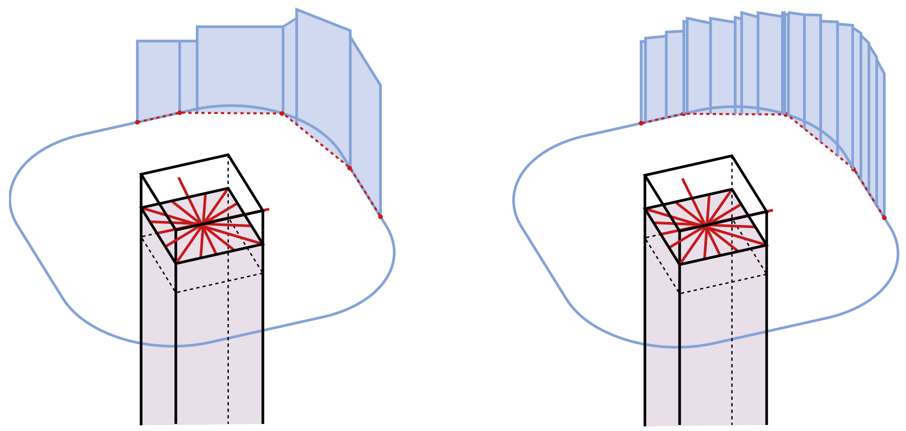
The mesh only needs to be fine enough for desired use:
Having a mesh that is very fine can lead to convergence issues within the FE model and will also make the model very slow and difficult to use. The aim here is to produce a mesh which is sufficiently fine for the intended use, not to make it as fine as possible.

There are two relatively simple checks to determine whether the mesh around column heads is suitably refined for the shear force assessment:

Check 1: There must be at least 1 full mesh element between the column face and the analysis section used to extract shear force values



Check 2: When the peak principal shear force values are extracted from the slab (without any averaging or post-processing), they should appear as a relatively smooth plot without large steps



Poor quality shear force plot
(Indicating an insufficiently refined mesh)

Good quality shear force plot
(Indicating that mesh is sufficiently refined)

This check is a little more subjective in nature; each engineer will have a different interpretation of what an acceptable shear quality plot looks like. One way of undertaking this second check in a more objective manner would be:

- Choose a continuous column location which is remote from any planted columns (ie where the shear behaviour is not dominated by the point load from planted columns)
- At this simple column location, calculate the design shear stress at the 2d control perimeter by interrogation of the FE model (using the method presented in Section 4.2.1)
- Compare this design shear stress value with the value obtained using the β-factor method in EC2 (2004)
- If the mesh is suitably refined, the results obtained from these two methods should be similar (within 10%).

4 Assessment of shear in transfer slabs

In this section, a standard methodology for the assessment of shear in transfer slabs is presented. The proposed methodology applies to all column locations in transfer slabs, both support columns beneath the slab and planted columns which are supported by the slab. The methods and examples presented assume that there is one planted column per bay. The methods are more generally applicable in cases where there are multiple planted columns in close proximity (using the theory of superposition), but they can become difficult to apply and greater care should be taken.

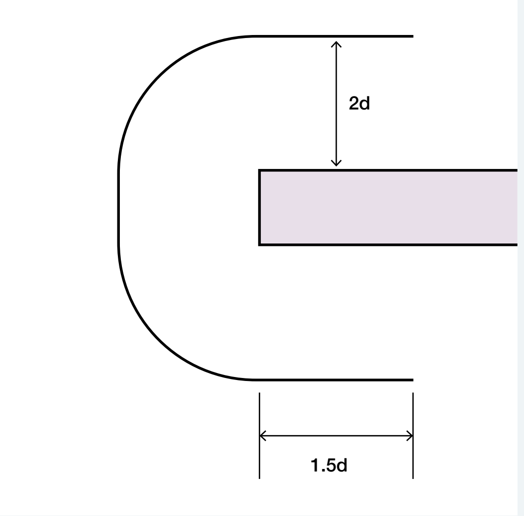
Within BS EN 1992-1-1: 2004 (EC2 (2004)), a standard method of assessing punching shear in flat slabs is presented. This is not appropriate for the design of transfer slabs in shear for the reasons covered in the next section. Its use for transfer slab design would result in an underestimation of shear effects, and hence a potentially unconservative design.

The shortcomings of the standard EC2 (2004) β-factor assessment method when applied to transfer slabs is first demonstrated by way of an example, and an alternative analysis method is then described in detail.

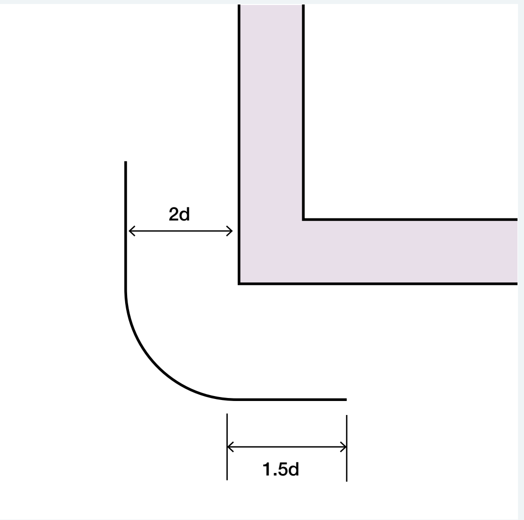
4.1 Limitations of EC2 (2004) punching shear method

When assessing shear in flat slabs, an assumption is made that the shear force within the main span of the slab is sufficiently low that it does not need to be explicitly checked. As such, shear force in flat slabs is typically only interrogated adjacent to supports and in other regions of the slab where a concentrated load is acting. Where the slab is subjected to a point load or supported by a discrete support (typically the regions of slab surrounding column heads), there will be a concentration of punching shear force, and the slab needs to be assessed for its ability to resist a local punching shear failure.

Shear around wall tips and corners: *Other areas where punching shear concentrations occur in flat slabs include around wall tips and wall corners. The punching resistance of slabs in these locations should also be assessed. Unfortunately, EC2 (2004) does not provide explicit guidance on the assessment of punching shear around wall tips and corners. In these locations, the method presented in Section 4.2.1 may be used, with the controlled perimeters around the loaded areas defined as shown below.*



Control perimeter definition around wall tip



Control perimeter definition around wall corner

Within EC2 (2004), a method for assessing punching shear at column support locations is presented (BS EN 1992-1-1 (6.4)).

First, a control perimeter is defined. The control perimeter is an imaginary line that encircles the column; EC2 (2004) places this line at 2d from the face of the column. The punching shear calculation then looks at the shear force crossing through the control perimeter and compares it to a calculated shear resistance for the slab.

It is important to be aware that the punching shear assessment method presented in EC2 (2004) is empirical: it was developed based on the results of destructive testing of hundreds of concrete samples. An example of this empirical nature is the definition of the control perimeter itself. In EC2 (2004), the control perimeter is placed at 2d from the column face so that the unreinforced shear capacity equation for punching shear is the same as the equation for the unreinforced shear capacity in a beam; there is no physical reason why it should be placed in that location. Because the punching shear equations are empirical, it is important to understand how the equations were developed and the limitations of their use.

One key assumption that is made in the EC2 (2004) punching shear equations is how the peak shear that acts around the control perimeter is assessed. The EC2 (2004) method estimates the peak concentration of shear force at the 2d control perimeter (known as the u_1 perimeter) by introducing a β-factor. The implicit assumption that is made when applying this β-factor method is that slab loading is predominantly uniform, with any variation in the distribution of shear stress around the u_1 perimeter arising due to the eccentricity of this uniform loading relative to the centroid of the column control perimeter. In slabs where loading is predominantly uniform, eccentricity of loading will be reflected as bending in the support column. It is therefore possible to estimate the design shear force at the control perimeter indirectly by considering bending in the column – shear forces in the slab are never assessed directly.

Punching shear is a brittle failure mechanism: *In a punching shear scenario, the slab may fail before significant redistribution of forces occurs. As such, the designer cannot assume that the shear force transferred into the column will be evenly distributed around the control perimeter – the capacity of the slab needs to be compared to the peak shear that acts around the control perimeter.*

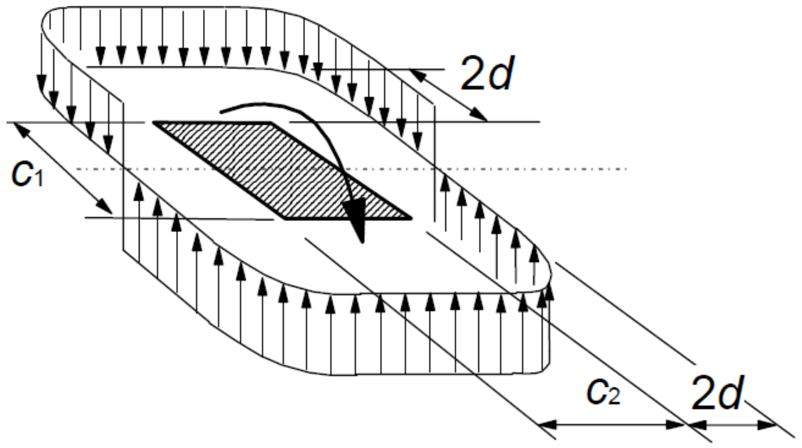


Figure 9: Assumed plastic shear distribution due to unbalanced moments at slab-to-internal column connection (reproduced from EC2 (2004))

This assumption is acceptable for a typical flat slab where the predominant loading is uniformly distributed. It is not appropriate for transfer slabs: when the slab is subject to heavy point loads, the peak shear force at the control perimeter will not be directly linked to moment transferred to the columns.

To illustrate this, a simple transfer example is considered. A punching assessment will be undertaken in a location where there is a closely offset planted column subject to relatively modest axial loading.

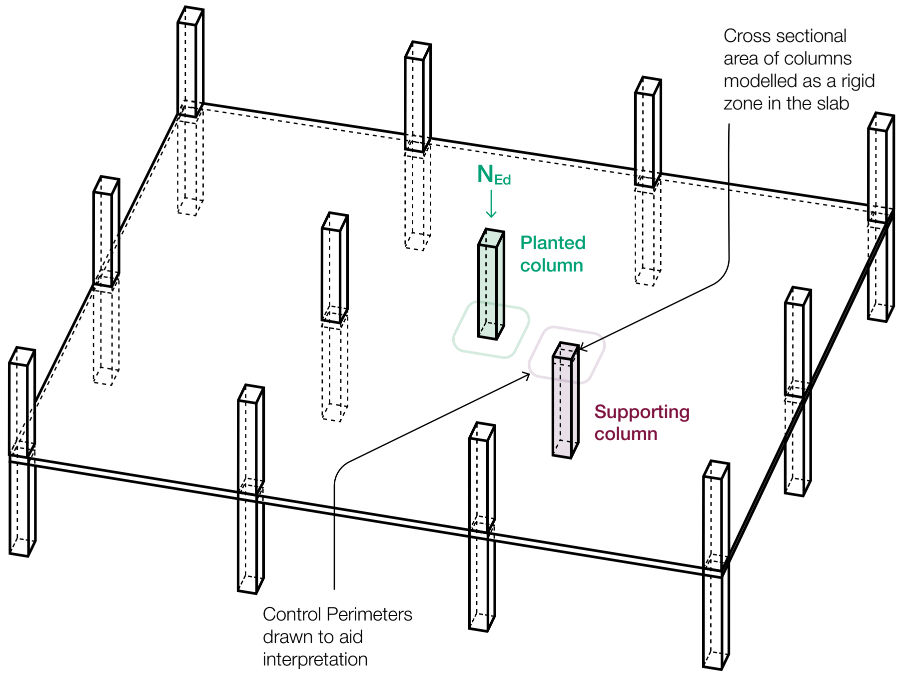


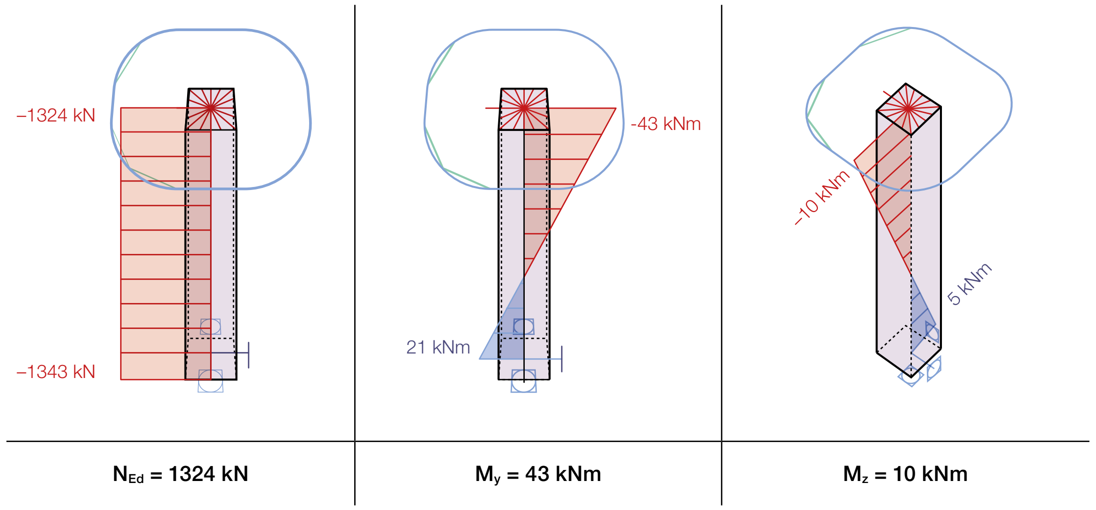
Figure 10: Simple test model - one column transfer location

Geometric information:

- Column dimensions (all columns) – 400 mm x 400 mm
- Slab effective depth, d_{eff} = 300 mm
- Clear offset between the faces of supporting and planted columns (measured along the line connecting centroids of the columns), S = 1600 mm (approx. 5.3d)
- ULS point load applied at planted column location, N_{Ed} = 750 kN (determined following a comprehensive assessment of the building load path, as described in Section 3.1). Live load reduction may be considered, as appropriate, when determining this planted column load.

4.1.1 Design shear stress, v_{Ed}, given by the EC2 (2004) β-factor method

Design forces in the supporting column (ULS forces resulting from the combined effect of the planted column load, slab self-weight and other uniformly distributed loading on the slab):



Equation	Value	Value
β -factor		1.04
Control perimeter (u_1)	$4\pi d_{eff} + 2(c_y + c_z)$	5370mm
Shear force around perimeter (if evenly distributed) (V_u)	N_{Ed}/u_1	246.6 N/mm
Design shear force with β -factor applied ($V_{Ed.EC2}$)	$\beta.V_u$	256 N/mm
Design shear stress ($v_{Ed.EC2}$)	**$V_{Ed,EC2}/d$**	**0.85 N/mm²**

4.1.2 Design shear stress, v_{Ed}, based on interrogation of the FE model

Considering the **principal shear force** acting at the 2d control perimeter of the supporting column closest to the planted column:

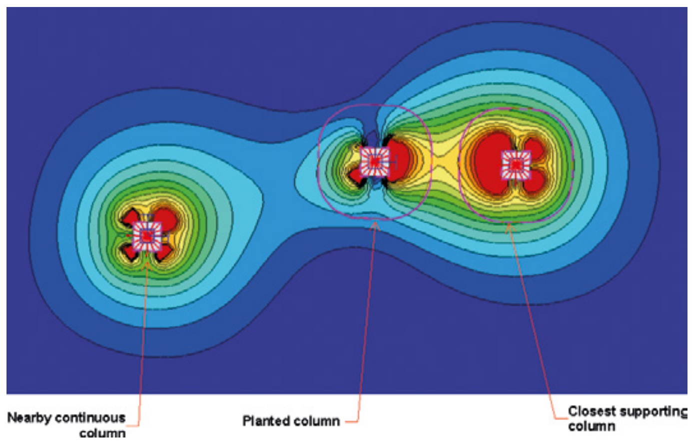
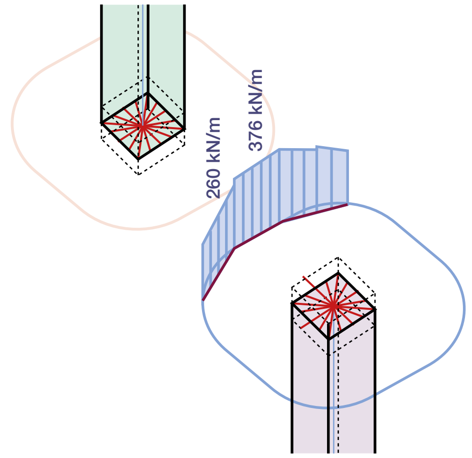


Figure 11: Principal shear force plot around the planted column

Take care in D-regions: *When the distance between the planted and supporting columns is small (a face-to-face clear offset of less than 1.5d) the zone between the columns is likely to be a D-region. In such regions, interrogation of 2D FEA plots can still be useful as an indication of the overall magnitude of forces, but be aware that 2D analysis does not capture the action of any direct strut that may form between the two columns. Detailed analysis of shear transfer in D-regions must always consider an appropriate strut-and-tie mechanism rather than relying solely on the output from a 2D FE model.*

Using analysis sections defined along the line of the control perimeter, the peak shear force at the control perimeter can be extracted (for definition of the term 'analysis section' as used in this paper, see Section 3.3.2):



Peak principal shear force at 2d perimeter of the supporting column, $V_{Ed,peak}$ = 376 kN/m

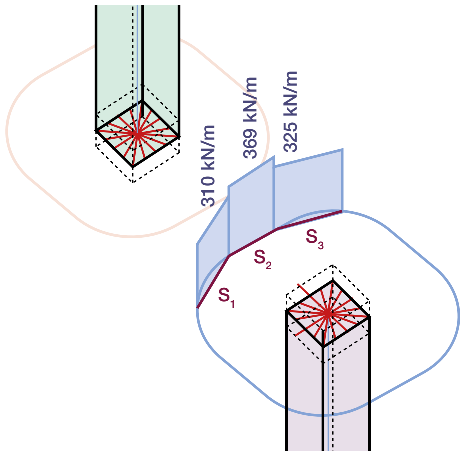
Allowing for lateral shear redistribution, the peak principal shear force is averaged around the length of the perimeter to determine the design shear force, $V_{Ed,design.}$

Permissible averaging length around the control perimeter:
The maximum averaging length permitted is **the least of**:

- 2d to either side of the peak (for a total averaging length of 4d) = 4 x 300 = 1200mm
- $0.125u_1$ to either side of the peak (for a total averaging length of $0.25u_1$) = 5370/4 = 1342mm

Hence, the limit of 4d governs in this instance. Total averaging length = 1200mm.

Averaging around the control perimeter: *The limiting length of 4d is taken from BS EN 1992-1-1:2023 (8.2.1(6)), where it applies specifically to linear (beam) shear assessment. Whilst it is more widely applicable to all shear assessments, when undertaking punching shear assessments it is sensible to limit the maximum averaging length to one quadrant of the control perimeter (ie $0.25.u_1$). This is to prevent an unrealistic degree of lateral shear distribution being accounted for when the cross section of the column is small relative to the slab thickness.*



	Analysis section length (mm)	Average shear force in section (N/mm)
Section S_1	400	310
Section S_2	400	369
Section S_3	400	325
Overall 4d section	**1200**	**335**

Hence, the shear force at the 2d control perimeter averaged over 4d,
$V_{Ed,design}$ = 335 N/mm

And therefore, the design shear stress from FE analysis,
$v_{Ed,FEA} = V_{Ed,design}/d = 335/300 =$ **1.12 N/mm²**

4.1.3 Comparison of results and conclusion

- Design shear from EC2 (2004) $v_{Ed,EC2}$ = 0.85 N/mm²
- Design shear from FE model $v_{Ed,FEA}$ = 1.12 N/mm².

In this example, the EC2 (2004) β-factor method does not provide an accurate assessment of the slab behaviour, and indeed it **underestimates** the shear forces in the slab, giving an **unsafe** result.

This is by no means an unusual or extreme example. Where columns are very closely offset, or where blade columns are transferred, the discrepancy between EC2(2004) values and the design shear obtained by the FE model is likely to be even more significant.

The reason for this disparity is that the distribution of shear forces around the column heads in transfer slabs is not driven predominantly by column bending, but rather depends on the geometry of the slab and the distance between planted and supporting columns (ie the transfer slab is behaving in a beam-like manner with respect to shear).

For all columns in transfer slabs, both above and below the slab, the standard EC2 (2004) β-factor method should therefore not be used to assess punching shear. Instead, the alternative methodology presented in this paper should be followed.

The appropriate method of assessment will vary depending on the spacing of columns above and below the slab in the area considered. Detailed guidance for various column geometries is given in Section 4.2.

All the methods proposed make use of Finite Element Analysis models, and guidance on the appropriate set-up and use of these models is provided in Section 3.

4.2 Transfer slabs – Shear design methods

The appropriate method for interrogating shear in the slab around a transfer column depends on the offset between the planted column and the supporting column. Transferred column cases can be divided into 4 categories for shear assessment.

In the table below:
- S = face-to-face offset between planted and supporting columns (measured along a line between the centroid of the columns)
- d = slab effective depth

Design Case		Description of design case
1	**S > 4d**	2d control perimeters around planted and supporting columns do not overlap. The planted and supporting columns can be assessed as separate cases. The shear capacity of the slab in the direct transfer zone must be assessed using both a punching shear model and a linear (beam) shear model to ensure that the most onerous design case is considered.
2	**1.5d ≤ S ≤ 4d**	2d control perimeters around planted and supporting columns overlap. In order to assess the planted and supporting columns as separate cases, new control perimeters need to be defined which are closer to the column face and do not overlap. Once the new inner control perimeters (u_{inner}) are defined and a modified punching shear capacity at these perimeters ($v_{Rd,inner}$) is calculated, the method is similar to Design Case 1.
3	**0 ≤ S < 1.5d**	The region of slab between the planted and supported columns will experience significant shear deformation and hence is a D-region. Perimeter assessment methods are not appropriate: The shear transfer must be assessed by considering a strut-and-tie mechanism.
4	**S < 0d**	The plan footprint of planted and supporting columns overlap. The predominant force transfer method will be via a direct compression strut in the overlapping zone. **This case is beyond the scope of this guidance.**

For all of the design cases above, the shear stress at the 2d control perimeter outside of the direct transfer zone (ie at the ‘back’ of the columns) must also be assessed.

The recommended assessment methods for Design Cases 1-3 are described in more detail in the following sections.

4.2.1 Design Case 1: S ≥ 4d

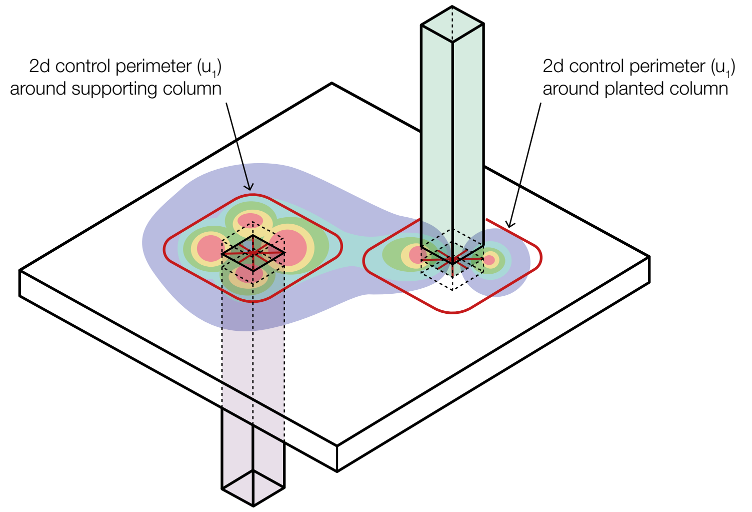


Figure 12: Shear force distribution in slab (S ≥ 4d)

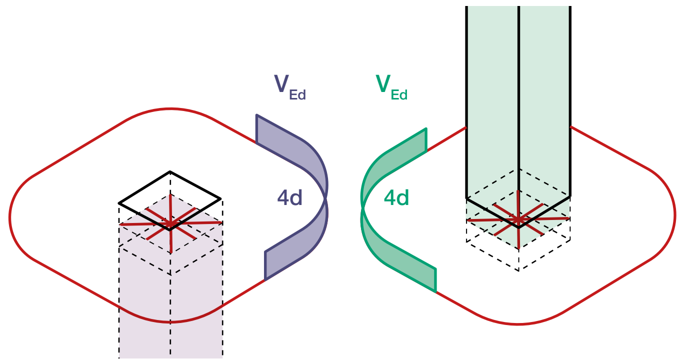


Figure 13: Punching shear analysis model (Step 1)

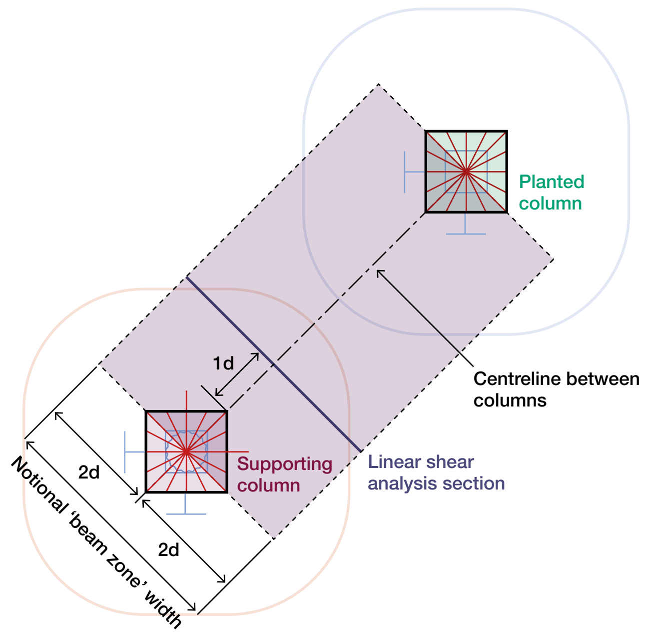


Figure 14: Linear shear analysis model (Step 2)

Ensure all ULS forces are included: *The shear force plot shows the effect of the full ULS loading of the slab (planted column point load, slab self-weight and any other uniformly distributed loads acting on the slab). All forces acting in the ULS condition need to be considered.*

ASSESSMENT METHOD

At this spacing, the 2d control perimeters around the supporting and transferred columns do not overlap. Punching shear around the two columns can be assessed independently of one another.

For both the planted and supporting columns separately:

Step 1: Check shear capacity in direct transfer zone based on punching shear model

1. Using an FE model of the transfer slab in isolation, find the peak shear force at the 2d control perimeter.

2 Determine the maximum **permissible averaging length** around the control perimeter. The maximum averaging length permitted is the least of:
- 2d to either side of the peak (ie a total length of 4d for internal columns)
- $0.125.u_1$ to either side of the peak (ie a total length of $0.25.u_1$ for internal columns)

NB: The limiting length of 4d is taken from BS EN 1992-1-1:2023 Cl.8.2.1(6), where it applies specifically to linear (beam) shear assessment. Whilst it is more widely applicable to all shear assessments, when undertaking a punching shear assessment it is sensible to limit the maximum averaging length to one quadrant of the control perimeter (ie $0.25.u_1$). This is to prevent an unrealistic degree of lateral shear distribution being accounted for when the cross section of the column is small relative to the slab effective depth.

3. Calculate the design punching shear force per metre, $V_{Ed,design}$, by averaging the principal shear force around the control perimeter over the permissible averaging width calculated above (centred on the location of peak shear force).

   (NB: For edge and corner columns, the total averaging length used is likely to be less than for internal columns. E.g., if the peak shear at the perimeter occurs at the slab edge, then the total averaging length permitted will be 2d or $0.125.u_1$.)

4. Calculate the design shear stress in the slab: $v_{Ed} = V_{Ed,design}/d$.

5. Calculate the shear capacity of the slab, $v_{Rd,c}$, in accordance with **EC2 (2004) (6.4.4)**, based on the flexural reinforcement that is to be provided. (NB: for a supporting column, the relevant flexural reinforcement is the top (hogging) reinforcement. For a planted column, it is the bottom (sagging) reinforcement because the column is trying to 'punch downwards'.)

6. Check that the punching shear capacity of the slab is greater than the design shear stress at the 2d control perimeter. If not, then either: increase the flexural reinforcement provided, locally increase the slab thickness, or provide punching shear reinforcement (Step 3). (NB: Regardless of the findings of this check, a minimum of 2 perimeters of punching shear reinforcement, detailed in accordance with **EC2 (2004) Cl. 9.4.3**, should be provided around both the supporting and planted columns.)

Step 2: Check shear capacity in direct transfer zone based on linear shear model

7. Consider a straight line between the centroid of the planted column and the centroid of the supporting column. The beam zone should extend a width of **2d** to either side of this centreline. For internal columns, this gives an overall beam zone width of **4d**. (NB: Notional beam width is based on **EC2 (2023) Cl. 8.2.1(6)**. When considering transfers parallel to a slab edge, the width of the beam zone should be reduced accordingly.)

8. The design linear shear force, $V_{Ed,linear}$, in the beam zone is found by placing an analysis section across the width of the beam zone at a distance of 1d from the face of the supporting column. $V_{Ed,linear}$ is then found by averaging the principal shear force over the width of this analysis section.

9. Calculate the unreinforced shear capacity of the beam zone, $V_{Rd.c}$, in accordance with **EC2 (2004) Cl.6.2.2**, based on the flexural reinforcement that is to be provided.

10. Check that the shear capacity of the slab in the beam zone is greater than the design shear force. If not, then either: increase the flexural reinforcement provided, locally increase slab thickness, or provide shear reinforcement (Step 3)
(NB: Regardless of the findings of this check, a minimum of 2 perimeters of punching shear reinforcement, detailed in accordance with **EC2 (2004) Cl. 9.4.3**, should be provided around both the supporting and planted columns.)

Step 3: Design and detailing of shear reinforcement in the direct transfer zone (if required)

11. The calculated reinforced slab capacity depends on whether the punching shear check (Step 1) or the linear shear check (Step 2) is critical.

12. If the punching shear check (Step 1) found that shear reinforcement is required, then the reinforced punching shear capacity, $v_{Rd,cs}$, should be calculated based on the punching shear capacity equations in **EC2 (2004) Cl.6.4.5**.

13. If the linear shear check (Step 2) found that shear reinforcement is required, then the reinforced shear capacity of the slab, $V_{Rd,s}$, should be calculated based on the linear (beam) shear equations in **EC2 (2004) Cl. 6.2.3**.

14. If both checks in Step 1 and Step 2 found that shear reinforcement is required, then adequate shear reinforcement should be provided to satisfy both the punching shear and linear shear capacity equations.

15. Regardless of whether designing to punching or linear shear requirements, the reinforced shear capacity of the slab should be limited to twice the unreinforced shear capacity ($2.v_{Rd,c}$). If the design shear stress exceeds the unreinforced shear capacity of the slab by more than a factor of 2, then redesign is required (either thickening the slab or providing additional flexural reinforcement).

16. The shear reinforcement that is provided in the direct transfer zone (shaded in grey in Figure 15) should be evenly distributed across the width of the direct transfer zone and should extend in equally spaced rows between the face of the supporting column and the face of the planted column.

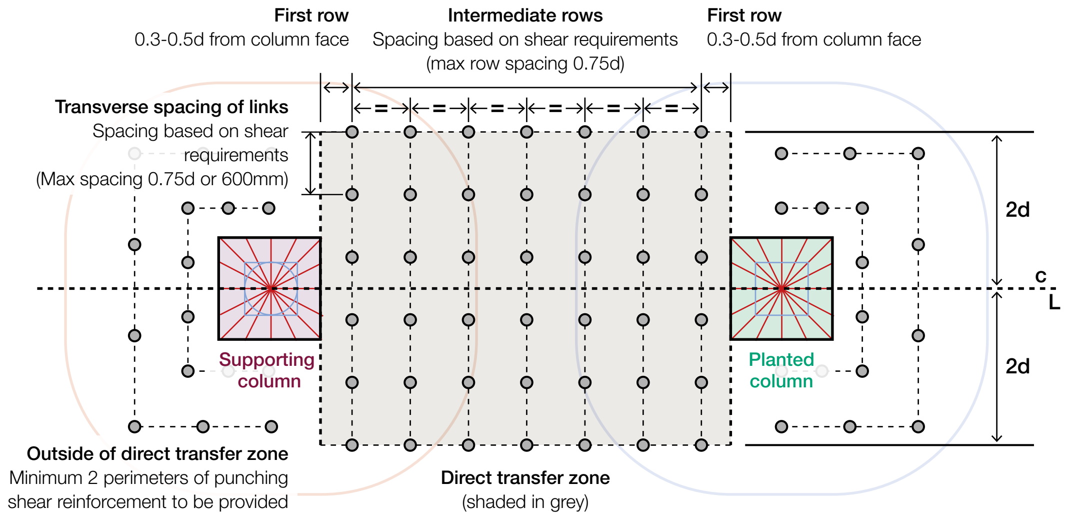


Figure 15: Plan view of typical transfer location showing shear reinforcement detailing

Step 4: Is punching shear reinforcement required outside of the direct transfer zone?

17. The requirement for punching shear reinforcement outside of the linear shear zone must also be checked. As per the guidance given in Section 2.2.2 (Detailing for Robustness), a minimum of two perimeters of punching shear reinforcement, detailed in accordance with **EC2 (2004) Cl. 9.4.3**, must be provided outside of the direct transfer zone. This check is to confirm whether any additional shear reinforcement, beyond the 2 perimeters of minimum reinforcement, is required.

18. The easiest way to check this in the FE model is to modify the 2D principal shear results plot so that the shading threshold is set to the $v_{Rd,c}$ value. Then, if the shaded region of slab extends beyond the 2d perimeter outside of the direct transfer zone, punching shear reinforcement is required outside of the direct transfer zone (Figure 16). A detailed assessment should then be undertaken to confirm whether the 2 perimeters of minimum shear reinforcement are sufficient.

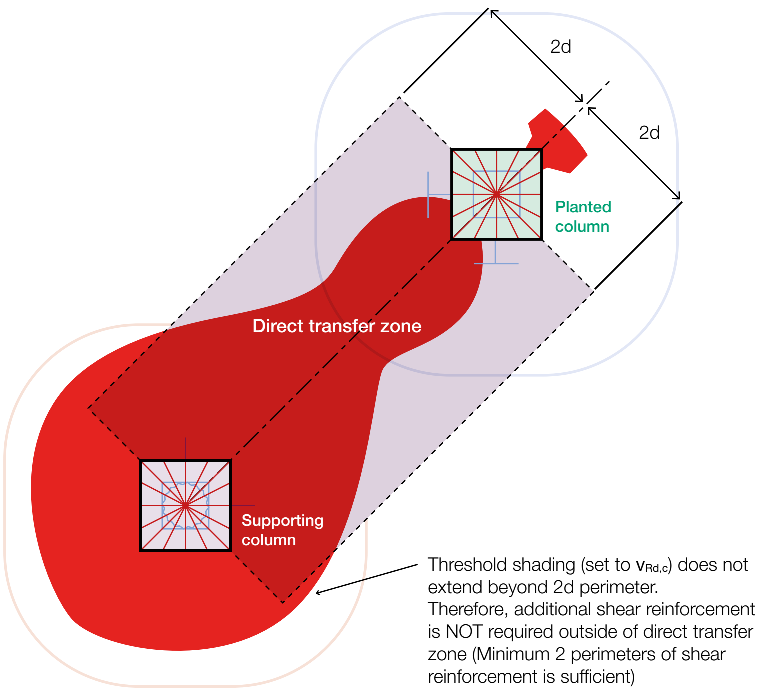


Figure 16: Plan view showing check for reinforcement requirement outside of the direct transfer zone

Step 5: Check limiting shear capacity, $V_{Rd,max}$, at the column face

19. Calculate an 'effective beta factor' by comparing the design shear stress, v_{Ed}, calculated in Step 1(4), with the value if stress were evenly distributed around the 2d perimeter: $\beta_{eff} = v_{Ed}/[N_{Ed}/(u_1 \times d)]$.

20. Calculate the design shear force acting at the column face, $V_{Ed,face} = \beta_{eff} \times N_{Ed}$, and check this against the limiting shear capacity, $V_{Rd,max}$ (NA to BS EN 1992-1-1 (6.4.5(3)).

4.2.2 Design Case 2: 1.5d ≤ S < 4d

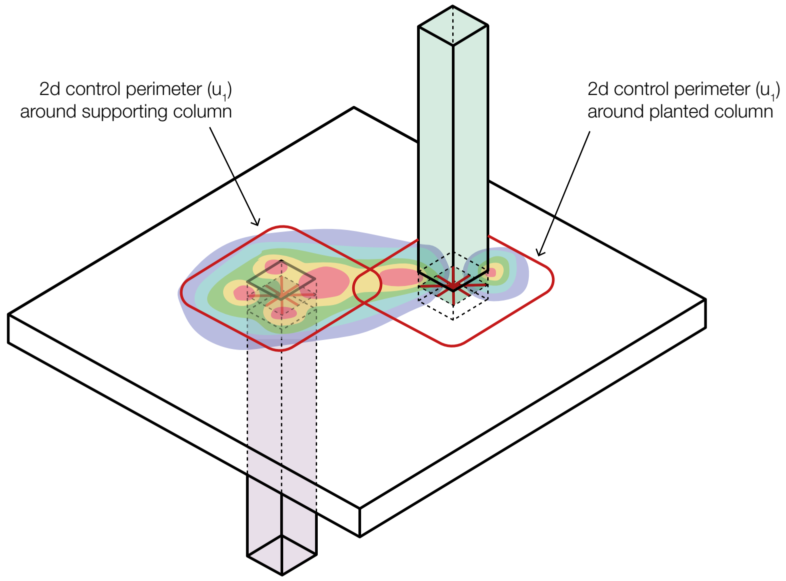


Figure 17: Shear force distribution in slab (1.5d ≤ S < 4d)

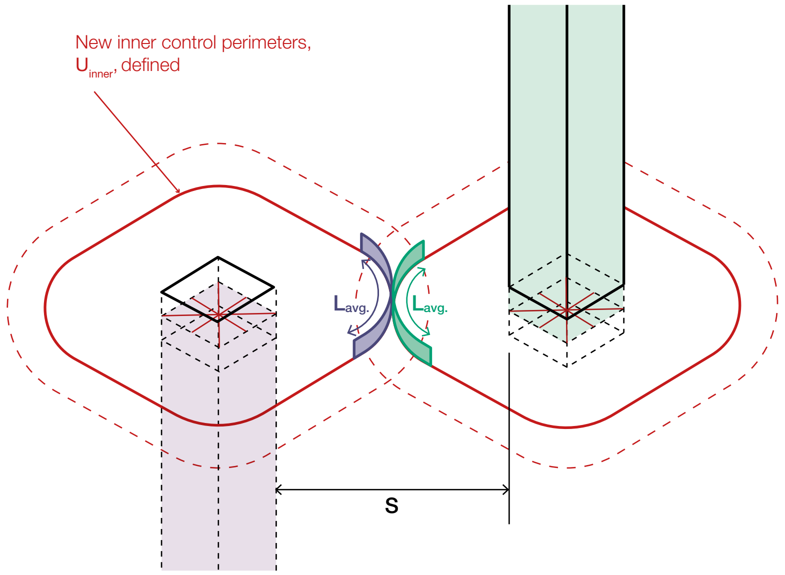


Figure 18: Punching shear analysis model (Step 1)

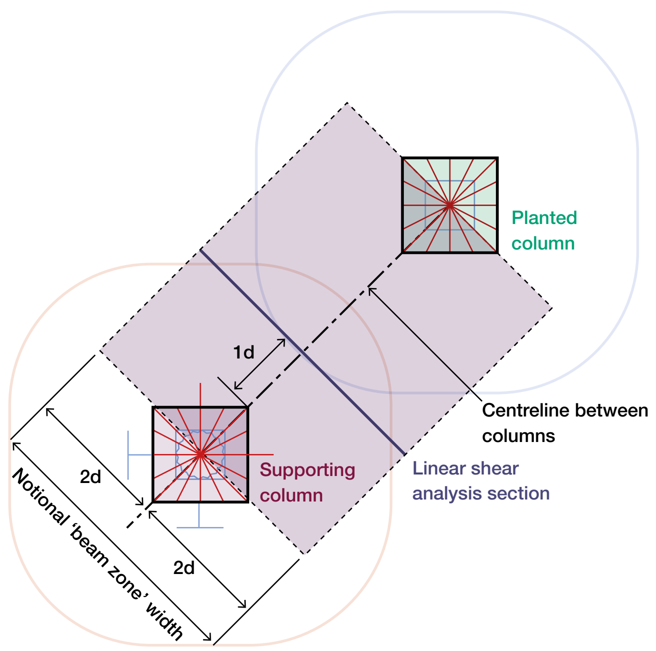


Figure 19: Linear shear analysis model (Step 2)

Ensure all ULS forces are included: *The shear force plot shows the effect of the full ULS loading of the slab (planted column ULS load, slab self-weight and any other uniformly distributed loads acting on the slab). All forces acting in the ULS condition need to be considered.*

ASSESSMENT METHOD

The 2d control perimeters around the supporting and transferred columns overlap. In order to assess the planted and supporting columns as separate cases, new control perimeters (u_{inner}) need to be defined which are closer to the column face and do not overlap. Once the new inner control perimeters are defined and a modified punching shear capacity at these perimeters ($v_{Rd,inner}$) is calculated, the method is then the same as for Design Case 1.

Step 1: Check shear capacity in direct transfer zone based on punching shear model

1. First define the inner control perimeters. The u_{inner} perimeters should be offset S/2 from the face of each column (where S is the column face-to-face spacing), such that the perimeters touch at the midpoint between columns.

2. Using an FE model of the transfer slab in isolation, find the peak shear force acting at the **inner control perimeter, u_{inner}** (Figure 18). This peak is likely to occur at, or close to, where the two inner control perimeters touch.

3. Determine the 'design shear force' per metre, $V_{Ed,inner}$, at the inner perimeter by averaging around the control perimeter to either side of the peak. The maximum total averaging length that can be used depends on the column spacing:

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Transfer Slab Design

Column spacing, S	Total averaging length (L_{avg}) permitted is the least of:
2d ≤ S < 4d	S (S/2 either side of peak) or $0.25u_{inner}$ ($0.125u_{inner}$ either side)
S < 2d	2d (1d either side of peak) or $0.25u_{inner}$ ($0.125u_{inner}$ either side)

The design shear stress at the inner control perimeter is then given by $v_{Ed,inner} = V_{Ed,inner}/d$.

4. Calculate the EC2 shear capacities of the slabs at the 2d control perimeter, $v_{Rd.c}$, in accordance with BS EN 1992-1-1(2004) (6.4.4), based on the flexural reinforcement that is to be provided.

5. Calculate the shear capacity at the inner control perimeter, $v_{Rd,c,inner}$, by modifying the standard EC2 shear capacity: $v_{Rd,c,inner} = v_{Rd,c} \times (u_1/u_{inner})$. (**NB: This is NOT a shear capacity enhancement –** The equation above simply accounts for the fact that the punching shear stress increases in proportion to the ratio u_1/u_{inner} as the diameter of the control perimeter reduces. The overall shear capacity of the slab remains the same as the standard EC2(2004) calculation.)

6. Check that the punching shear capacity of the slab is greater than the design shear force at the inner control perimeter. If not, then either: increase the flexural reinforcement provided, provide punching shear reinforcement, or increase the slab thickness, as required.

Step 2: Check shear capacity in direct transfer zone based on linear shear model

Linear shear assessment method is as per Step 2 of Design Case 1, with additional reference to Figure 19.

When the column face-to-face spacing, S, is less than 2d, the contribution of the planted column point load (N_{Ed}) to the design linear shear force in the slab ($V_{Ed,linear}$) may be reduced by a factor of **(S/2d)** (in accordance with BS EN 1992-1-1(2023) Cl. 8.2.2 (9)). In practice, this is relatively simple to achieve: the full design shear force passing through the 4d 'beam zone' ($V_{Ed,linear}$) is found in accordance with Design Case 1 Step 2, and then this value is multiplied by (**S/2d**) to find the 'effective shear force'.

Step 3: Design and detailing of shear reinforcement in the direct transfer zone (if required)

Design and detailing of shear reinforcement in the direct transfer zone is as per Step 2 of Design Case 1.

When calculation the reinforced punching shear capacity of the slab at the inner control perimeter ($v_{Rd,cs,inner}$), $v_{Rd,c}$ should be substituted with $v_{Rd,c,inner}$ and u_1 should be substituted with u_{inner}.

ie $v_{Rd,cs,inner} = 0.75 v_{Rd,c,inner} + 1.5 \left(\frac{d}{s_r}\right) A_{sw} f_{ywd,ef} \, [1/(u_{inner} d)] \sin\alpha \leq k_{max} v_{Rd,c,inner}$

Step 4: Is punching shear reinforcement required outside of the beam zone?

As per Step 4 of Design Case 1.

Step 5: Check limiting shear capacity, $V_{Rd,max}$, at the column face

1. Calculate an 'effective beta factor' by comparing the design shear stress at the inner perimeter, $v_{Ed,inner}$, with the value if stress were evenly distributed around the inner perimeter: $\beta_{eff} = v_{Ed,inner}/[N_{Ed}/(u_{inner} \times d)]$.

2. Calculate the design shear force acting at the column face, $V_{Ed,face} = \beta_{eff} \times N_{Ed}$, and check this against the limiting shear capacity, $V_{Rd,max}$ (NA to BS EN 1992-1-1 (6.4.5(3)).

4.2.3 Design Case 3: $0 \leq S < 1.5d$

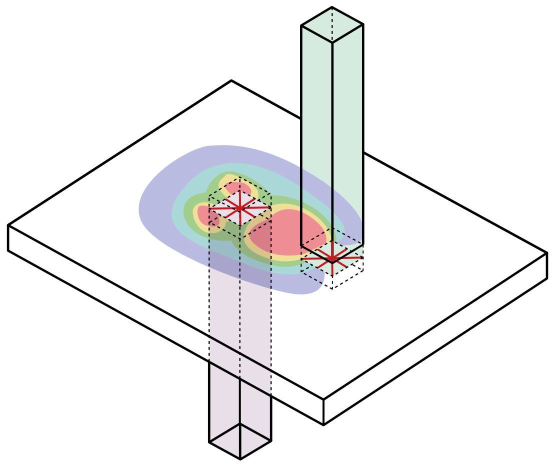


Figure 20: Shear force distribution in slab

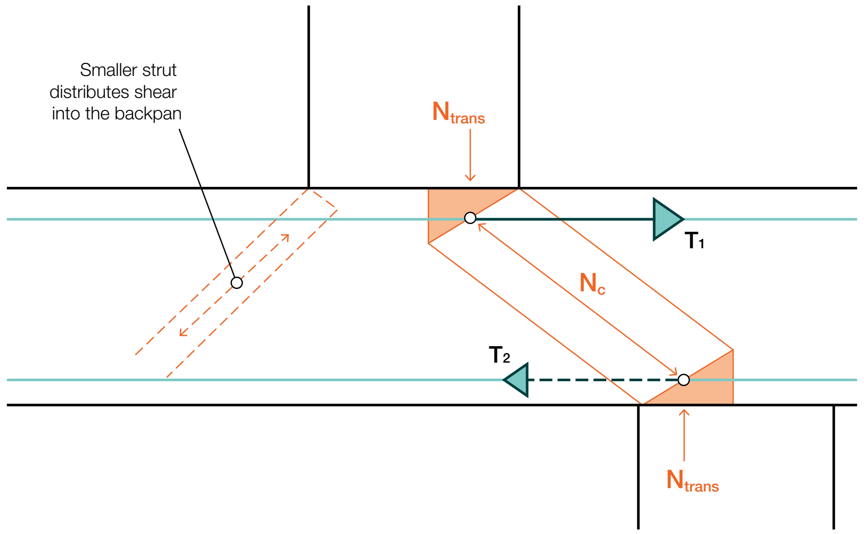


Figure 21: Suggested analysis model

ASSESSMENT METHOD

At this reduced offset, the region of slab between the planted and supported columns will experience significant shear deformation and hence is a D-region. Perimeter assessment methods are not appropriate: the shear transfer must be assessed using a strut-and-tie model.

The appropriate strut-and-tie model for a given transfer location will depend on the specific geometry of the location.

Detailed guidance on designing concrete members using strut-and-tie models can be found in the Concrete Centre guide 'Strut-and-tie Models – How to design concrete members using strut-and-tie models in accordance with Eurocode 2'. In addition to the direct strut action, field shear arising from self-weight and other slab loading will need to be considered.

NB: In the diagram shown, the top and bottom reinforcement provided to resist T1 and T2 should be fully anchored behind the node. The T1 tie will be required to resist the hogging moment arising from uniform loading of the slab in addition to the tensile force generated by the strut-and-tie mechanism. The engineer must ensure that sufficient top steel is provided in the transfer zone to resist the combined flexure and strut-and-tie behaviour.

5 Flexural design

The design of flexural reinforcement in transfer slabs should follow the same principles as design of normal flat slab flexural reinforcement.

Due to their complex geometry and heavy loading, the design of flexural reinforcement in transfer slabs will necessarily be based on the results of FE analysis. Design using a more traditional hand design method such as the equivalent frame analysis method presented in EC2 Annex I is likely to be impractical due to the highly irregular column grids encountered.

Detailed guidance on the use of FE analysis to design flexural reinforcement in flat slabs is provided in Concrete Society Technical Report 64 (Section 4.6.5 and Appendix A.5). These examples include advice on the appropriate use of averaging strips at locations of point loads (planted columns) and point supports (supporting columns) to produce sensible and buildable reinforcement layouts.

5.1 Reinforcement detailing

The detailing of flexural reinforcement is essential to the performance of transfer slabs in both flexure and shear.

Flexural reinforcement in transfer locations

Care needs to be taken with the detailing of the top bars over supporting column heads and the bottom bars beneath planted columns. Figure 22 shows some pointers for the detailing of top reinforcement over column heads at supporting column locations. The same considerations apply to the bottom reinforcement beneath planted columns.

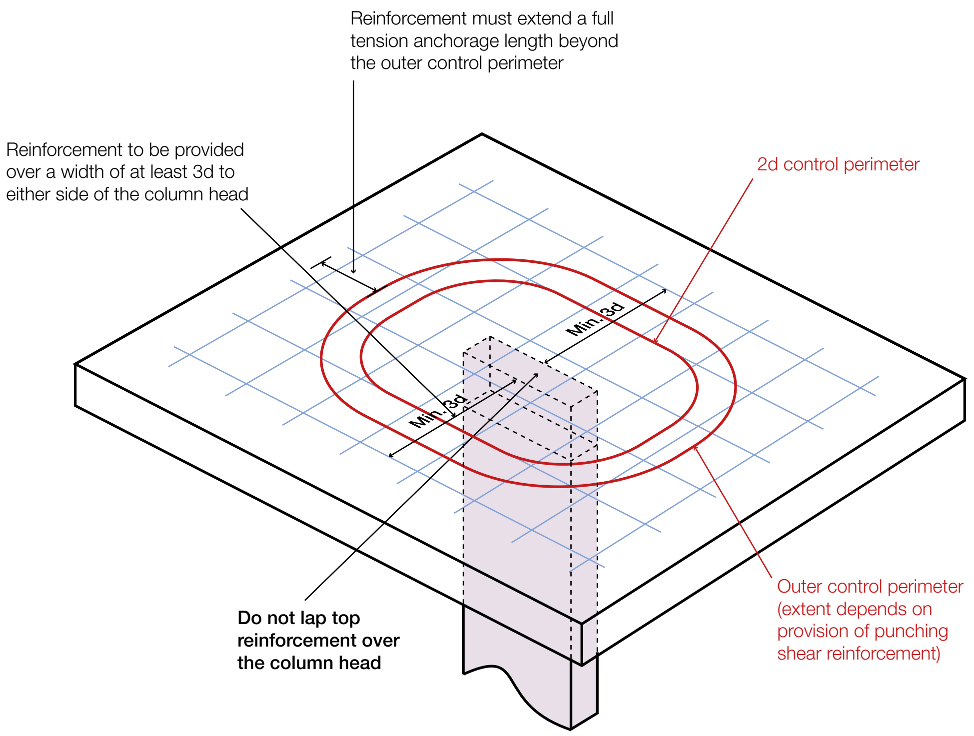


Figure 22: Top reinforcement detailing considerations over supporting columns

If the planted column is offset from the supporting column by a clear distance of less than 1.5d, then the region of slab between the columns will be subjected to significant shear deformation, and as such is categorised as a D-region (ie normal bending theory does not apply). These transfer locations should be designed by consideration of a strut-and-tie mechanism (STM), and reinforcement should be detailed accordingly to allow the assumed STM to develop.

Anchorage of large diameter bars into core walls

Careful consideration must be given to the interface between deep transfer slabs and core walls. If the slab requires large diameter bars to achieve minimum flexural reinforcement, then the wall thickness may be governed by the depth required to anchor these slab bars into the wall (complying with both the minimum bend radius of the bars and avoiding over-congestion of reinforcement in the wall at the slab-to-core interface).

6 Other design considerations specific to transfer slabs

Each of the topics listed below could be a paper or guide in their own right. There is not space to provide specific or detailed guidance for each here; this list is a signpost towards important items for further consideration and research.

6.1 Bending moments in supporting columns

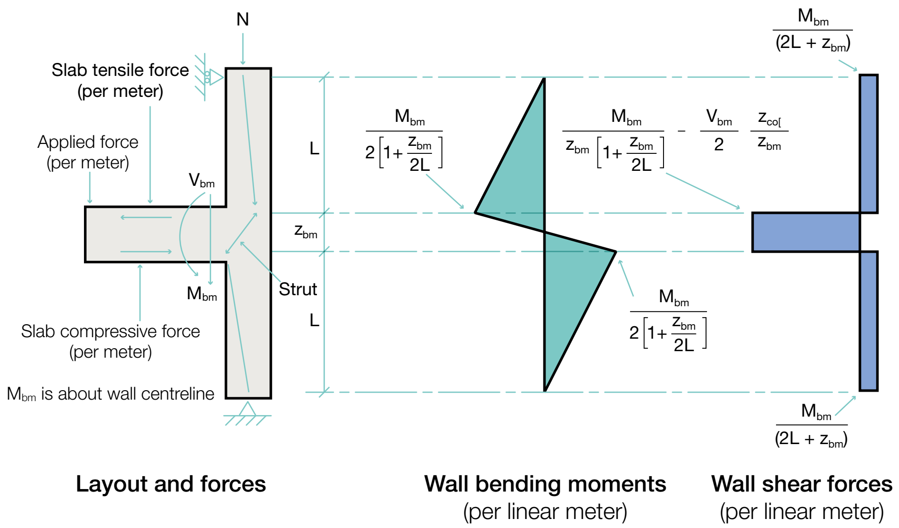
The bending moments induced in the supporting columns by first order frame effects may be large (particularly in edge and corner columns). The ability of supporting columns to resist these bending moments should be verified at an early stage in design, as this may be a governing consideration.

The moment transferred to edge and corner columns should be limited in accordance with BS EN 1992-1-1(2004) (I.1.2 (5)). If this limiting moment is exceeded, then the geometry of the slab-column interface should be changed.

6.2 Interface between deep slab and support walls

It is important to consider the impact that the deep slab will have on the design of supporting walls. Modelling the wall-to-slab interface as a hinged connection in an FE model is unlikely to capture the interface behaviour in sufficient detail to enable the walls to be designed correctly.

Where the deep slab connects to the wall, it will induce bending in the minor axis of the wall and will also cause a localised concentration of shear force (Figure 23). These effects need to be accounted for when designing the wall.



(Adapted from Scott, R.H., Feltham, I., Whittle, R.T., 'Reinforced concrete beam-column connections and BS8110', The Structural Engineer, 72/4. 15 February 1994)

Figure 23: Indicative design forces at the wall-to-deep slab interface

6.3 Movement, shrinkage and long-term deflection

Effect of stiffness and long-term movement on load paths

If a transfer slab is used, then the engineer needs to be more aware of how the structure will be built and the impact that this will have on the load path in the building.

Understanding the construction sequencing and propping/de-propping timings is essential. The time between casting the slab and it being loaded (ie props being struck) will affect the long-term deflection of the slab and this will, in turn, have a significant impact on the load path in the building. As discussed in Section 3, it is recommended that designers undertake a load path sensitivity analysis in their whole-building FE model by varying the relative stiffness of the transfer slab.

The engineer should work closely with the temporary works designer (TWD) to develop the sequence of propping and de-propping for the slab and understand how these timings will affect the long-term behaviour of the transfer slab.

If the design of the slab is based on achieving a minimum shear strength before planted column loads are applied (e.g. that the slab has achieved 28-day strength ($f_{ck,28}$) through its full thickness before props are struck and forces are released into the system) then this needs to be clearly noted on construction drawings and the engineer needs to ensure that the contractor is aware and plans their construction sequence accordingly.

Thermal gradients and shrinkage forces

In deep slabs, a high thermal gradient can develop through the depth of the slab as it cures, leading to significant locked-in stresses in the slab. An appropriate concrete specification and pour sequence should be chosen to reduce the locked-in stresses that develop.

As a separate phenomenon: in deep slabs, the long-term shrinkage forces across the slab as a whole may be significant. The engineer must consider how the slab is restrained and where high shrinkage forces will develop. It is likely that large tension forces will develop in areas of slab between cores. These forces may be so large that it is impractical to design the core walls to resist them. The engineer may then be forced to accept a certain imposed deformation in the core walls and plan for how this is managed (e.g. ensuring that the client and contractor are aware that 45-degree cracks may form in the lower core walls, and providing a specification for how these cracks can be remediated).

This imposed deformation may also impose an inclination on the columns that support the transfer slab; the impact of this inclination should be considered in the design of the columns.

Engineers are referred to the following guidance:

- **Concrete Society Technical Report 67 '*Movement, restraint and cracking in concrete structures*'** for a general overview of the subject.
- **CIRIA Guide C766 '*Control of cracking caused by restrained deformation*'** for detailed guidance and worked examples.

6.4 Sustainability

Most structural engineering companies now have ambitious environmental commitments focussed on driving down embodied carbon in their designs. To this end, we should as an industry be pushing our clients to avoid the use of transfer slabs wherever possible. They are heavy and structurally inefficient and drive up the embodied carbon of the development.

If a transfer slab is used on a development, consider whether the use of high percentage GGBS/PFA mixes is appropriate. An advantage of using low-carbon mixes is that they tend to cure more slowly, which will help to reduce locked-in stresses in deep slabs. However, the corresponding disadvantage is that they tend to develop strength more slowly, so longer propping times and other construction constraints will require careful consideration.

Please refer to the recent guidance paper 'The efficient use of GGBS in reducing global emissions', which IStructE members can download for free from the IStructE website, for further information on use of GGBS and other cement substitutes.

6.5 Health and Safety Considerations

As a designer working in the UK, structural engineers have a legal obligation under the CDM Regulations (2015) to foresee and mitigate risks incurred as a result of their design decisions. As such, engineers need to be conscious of the following specific health and safety risks incurred by including a transfer slab in their design:

- **Design of reinforcement chairs:** The chairs that support the top mat of reinforcement require careful consideration for deep slabs. Lattice-style wire chairs are typically only available in standard depths of up to 400mm. Therefore, if the transfer slab required is deeper than around 500mm, the chairs will need to be specially designed and fabricated. The design of these chairs would typically be undertaken by the TWD, but the engineer should be aware of the additional complexity (and hence risk) incurred in the design and construction of deep slabs.
- **Large volume continuous pours:** Deep slabs over a large area will require long continuous pours due to the volume of concrete. If this requires pouring to continue through the night, then the risk of injury to workers is increased and special management procedures will be required. Opting for a rational arrangement of transfer beams would avoid the need for a large volume pour and the associated risks.

- **Use of large diameter bars:** When detailing the slab, try to avoid specifying 32mm and 40mm diameter bars. These large diameter bars would need to be positioned using a crane and may need to be connected using couplers, adding complexity to the site logistics and reinforcement detailing. For deep slabs with high shear forces, it may be difficult to achieve a high enough reinforcement percentage without resorting to large diameter bars.

- **Construction loading:** If the contractor wishes to use the transfer slab as a working platform to construct the rest of the building, then the temporary loading from construction activities (traffic and stockpiling) must be considered.

- **Complexity of propping and construction:** As described in Section 6.3, the long-term performance of transfer slabs is highly dependent on the propping sequence during construction, and the engineer must work closely with the contractor's TWD to ensure that the propping sequence is both safe during construction and also achieves the required long-term performance of the structure.

  Consider:

  - **Propping to support the wet load of the transfer slab:** It is unlikely to be economical to design any slabs below the transfer slab to support the full self-weight of the wet concrete in the transfer slab. As such, propping will likely need to be continued down to foundations, and these props may in turn need to be supported by temporary foundations at their base.

  - **Back-propping of slabs above the transfer slab:** If the design intent is to prevent loading onto the transfer slab until it has achieved its full (28-day) design strength, then either construction of the frame will need to pause for a month after casting the transfer slab, or substantial back-propping will be required beneath planted columns. This back-propping should be carried down to foundation level, and again these props may require temporary foundations.

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