# Structural Guidance Note

## 2008 SGN 01

# Modelling of concrete core stability systems: procedural guidance

SSN Insight

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# Contents

# 1 Introduction

This Note gives guidance on the modelling of concrete core stability systems in building structures. The idea for this Note originated from the Structural Surgery presentation on Modelling Cores first given in autumn 2007, which can be found in the SSN Presentations Library.

The Note is written such that the general principles of modelling are addressed first followed by different methods of analysis being presented in order of increasing complexity. This is followed by sections on the choice of material properties for the analysis and the typical checks that are required in the design process. The note concludes with a brief summary and list of references.

Whilst written from a UK Building Engineering perspective, this Note is not code specific nor aimed at a particular class of building structure. The principles presented are generally applicable to any given location though further checks may be required depending on circumstance, e.g. seismicity. Although specific software packages are not considered, Oasys's GSA and AdSec are used to illustrate given concepts as required.

Although seismic design is not normally part of standard UK design practice, brief consideration is given to this topic as an introduction to this specialist area.

The designer should read this Note in the context of the particular design problem being tackled and modify or add to the modelling, analysis, and design advice given in this Note as appropriate. Advice from those who have designed similar structures should be sought to benefit from their experience. Similarly, this Note is not intended to be prescriptive with respect to defining an approach; it should be considered as setting out a range of methods considered suitable for carrying out the analysis and design of concrete core stability systems in the absence of other guidance.

Various internal Arup documents are referred to within this Note. Hyperlinks are provided to these in Section 8 References where electronic versions are available.

# 2 General principles

In this section, a number of general principles will be outlined that apply to different modelling approaches.

## 2.1 The purpose of modelling

The purpose of modelling a structural stability system is to acquire a sufficient amount of data to allow the design calculations to be carried out. In most cases, the simplest possible model to achieve this goal should be used. In the context of this note, 'simple' relates to the constructability of the model, understandability of what the output from the analysis is communicating, ease of use of the output in the design calculations, and the ease with which the model and design may be checked.

Typically there will be a compromise between these factors but consideration should be given to the final point, checking. Making a model easy to self-check, and straightforward for an independent checker to understand will assist in both 'Getting It Right' and reducing the amount of time taken to sign-off a particular set of calculations. Guidance Note RGN01 – 'Project Reviews' should be referred to for further information.

The above implies that the method of analysis and design, and thus the selection of modelling technique, be planned before constructing a computer model.

1997 SGN 01, Good Practice Guide to Calculations, should also be referred to with respect to the calculation process.

## 2.2 Lessons from previous projects

Timely reviews at all stages of a project are vital. Such reviews may focus on the project as a whole or on specific aspects. This gives the designer an opportunity to benefit from the experience of others and can highlight successful approaches or particular problems that may have occurred on other projects.

Ideally, the lessons learnt on a given project should be recorded for future reference and dissemination if appropriate. A good example of this, and one that is recommended to those wishing for an overview of the approach taken on a particular project, is Andy Pye's 'Lessons from the design of the GLA building RC core' document, available on the SSN.

## 2.3 Analysis and design software

Although initial conceptual design will most likely be developed through simple hand calculations, at some stage computer analysis and design packages will typically be used in the examination of a structure's stability system.

It should be emphasised that there is seldom any need to develop a 2D (or even 3D) finite element model in the early stages of the design process. In many instances, well thought-out 1D element models may be used from an early scheme stage through to detailed design. Thus the maxim that ‘simple is best’ should apply wherever possible in using and implementing the guidance given in this document. In modelling terms, Fig 1 presents possible choices with respect to an increasing level of complexity.

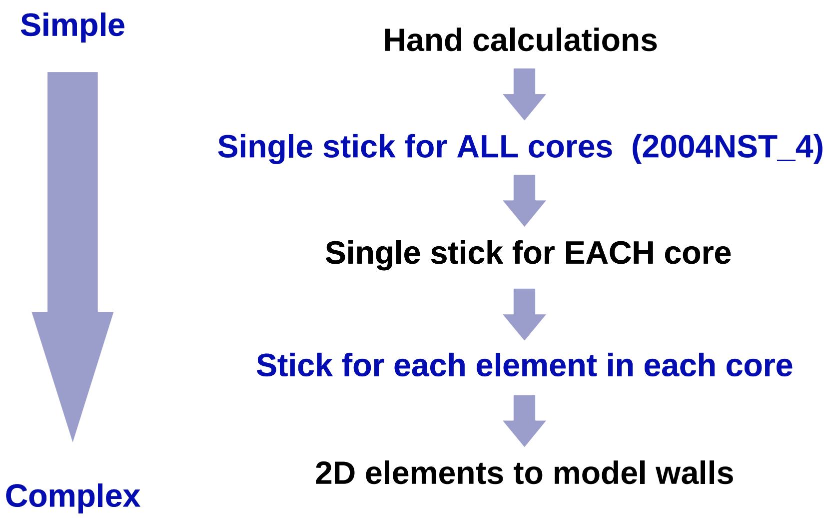


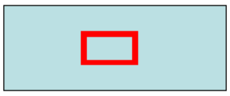

Fig 1. Modelling choices.

The ‘simple is best’ maxim is based on the reality that concrete is not an elastic material; building a complex elastic analysis model is likely to result in limited improvements to the accuracy of the analysis.

## 2.4 Distribution of stability elements

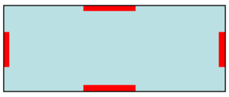
It is not possible to consider all potential arrangements of stability elements within this Note. However, the following simple guidance is given as a starting point to highlight the potential strengths and weaknesses of a number of different structural arrangements:

**Offset core**: Good lateral stiffness in each direction though torsionally poor due to the lever arm between the centre of mass of the floor and the centre of stiffness of the core.



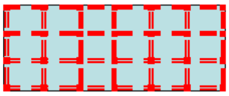
**Central core**: Good lateral stiffness in each direction. Although the centre of mass and centre of stiffness coincide, this arrangement may be susceptible to torsion due to non-symmetrical lateral loading of the 'wings' on either side of the core.

**Symmetrical cores**: Good lateral stiffness in each direction and torsionally stiff. The stiff points created at each end of the slab may be of concern in some situations, e.g. post-tensioned construction, thermal loading, shrinkage, etc.



**Symmetrical walls**: Good lateral stiffness in each direction and torsionally stiff. Minimises the stress in the floorplate due to shortening.

**Singly symmetric walls**: Good lateral stiffness in each direction and torsionally stiff. However, torsional stiffness relies primarily on the end walls and has less redundancy than the symmetrical wall case above.



**Moment frame**: Stability is provided by frame action.

## 2.5 Loading and load combinations

Various actions should be considered by the designer and these are briefly discussed in this section. However, given the variability in magnitude and application, the following should be treated as indicative by the designer.

### 2.5.1 Static dead and live loads

These are the normal loads that the gravity design will be based on, i.e. the self-weight of the structure plus the weight of finishes, and the live loading based on use.

### 2.5.2 Wind

The effect of wind loading will be one of the main checks to be carried out. The loading imposed by wind may be extremely complex depending on structure shape and specialist advice should be sought as appropriate.

However in carrying out early concept and scheme stage calculations, an assessment of wind load can often be made by considering a simplified structural envelope to which wind load is applied.

Clearly, wind loading should be evaluated from various directions and where stability walls are not orientated in an orthogonal manner, it is important to check sufficient wind orientations to determine the maximum force to be resisted by any given wall.

With respect to the application of wind load, the line of action for a given storey will be normal to the centre of area of the loaded surface, e.g. the façade of a building. Load cases that consider torsion induced by wind and the combination of winds from different directions are typically codified and should be considered as appropriate.

### 2.5.3 Notional horizontal loads

The concept of a notional horizontal load (NHL) features in many codes of practice and is generally used to ensure that a minimum lateral load is considered such that considerations of robustness are met. However, there is neither a consistent definition nor calculation method of NHL between codes of practice.

With respect to the application of a NHL, the line of action for a given storey will pass through the centre of mass of the floorplate.

### 2.5.4 Notional loading due to construction tolerances

Some codes of practice, e.g. EN1992, require that the unfavourable effects of geometric imperfections be taken into account in addition to other loading effects.

Irrespective of whether this is a codified requirement in a given location, the designer should consider whether the inclusion of such loading would be appropriate given the local construction conditions expected on-site and susceptibility to second order effects.

### 2.5.5 Loads transferred into cores

Depending on the connection between the core and other parts of the structure, the forces and moments transferred to a core should be accounted for.

In addition to the more usual actions such as wind, an increasingly common feature in Arup designed buildings is the use of inclined walls or columns. Irrespective of whether these are modelled explicitly, the lateral forces they transfer through the floorplate and into the stability system must be considered. The engineer should account for these in the analysis, design, and importantly the detailing of all members required to transmit the load, e.g. interface region between column and slab, in-plane resistance of the slab, shear stress in the core walls, bending induced in the core walls, etc. Such stresses should be superimposed (for a linear analysis) on the stresses from other appropriate load cases to determine the critical design cases.

### 2.5.6 Effects specific to the method of construction

This may cover a wide range of project specific issues such as:

- Post-tensioned construction – although not strictly a destabilising effect, post-tensioning may have the effect of pulling in the stability elements, possibly reducing their stiffness due to cracking.
- Outriggers – where stability is provided in part by the use of outriggers, care should be taken in understanding the load sharing between the various structural elements. It is likely that in such instances the point at which the outriggers are connected to the main structural frame, and begin to resist the applied loads, will see a significant change in the imposed stresses on the structure and these should be investigated during the analysis and modelling stages and designed for as appropriate.

### 2.5.7 Eccentricity of applied load

Irrespective of the codified requirement for the consideration of eccentric loading, the designer should be satisfied that the structure will be stable under the action of eccentrically applied load, e.g. wind. As has been discussed, certain arrangements of stability elements may lead to a structure that is potentially sensitive to torsion and sensitivity studies may be required in order to investigate this.

### 2.5.8 Load combinations

The effects of adverse and beneficial loading should be accounted for, as should the various permutations of load, see Section 5.1. As such, the designer must select the appropriate load cases in order to ensure that the structure satisfies the limit states being considered.

Load combinations and their associated adverse and beneficial load factors will typically be specified in the appropriate codes of practice. However, it is important to confirm that these adequately cover the specific design situation and further load cases may need to be specified.

When considering the action of wind, it is important to consider the angle at which such loads are applied with respect to the orientation of the stability elements to ensure that the worst case for any given element is reported. In the UK, PD6688-1-4:2009 defines a simplified approach to account for skewed wind loads by applying a net pressure coefficient of 2.2 divided equally between the wind loads that are orthogonal to the building's faces.

Note that PD6688-1-4 also suggests that torsional effects may be accounted for by displacing the point of application of the wind load horizontally by 10% of the face width.

### 2.5.9 Application of load

Lateral load may be applied as distributed or point loading to represent the actual load distribution or simplified into an equivalent point load and point moment. In the latter case, lateral loading due to the weight of a floorplate in a notional horizontal load case, say, would be applied at the centre of mass of the floorplate, whilst loading due to wind would be applied at the centre of wind area for a given story lift.

With respect to the application of wind load, some codes of practice require a nominal eccentricity to be applied as noted above. If the centre of wind is defined by a single node that is rigidly linked to the stability system, this may be represented by a point moment. There is no clear method of adding this eccentricity manually if the wind load is applied to a vertical surface in the analysis model and thought should be given to this in planning and selecting a method of modelling.

## 2.6 Foundations

To retain conciseness and due to the complexity of the subject, foundations and their interaction with the underlying strata are not considered in this Note. Similarly, the stiffness of foundations, which may be of equal or greater importance than their strength, is not addressed.

Advice should be sought from Arup Geotechnics at the earliest opportunity to assess the particular ground conditions in the context of the planned construction and a methodology developed to address the modelling and design of the foundation system. This should be reviewed during the various technical design reviews during the project.

## 2.7 Restraints at ground level

Often, a stability core or wall penetrates through a number of basement levels to its foundation. In such cases, the core is often connected to a perimeter capping beam or retaining wall by way of the structural slab.

The way in which this restraint is modelled at this interface has a significant effect on the analysis results. If the restraint is considered rigid then there will be a large shear reversal at the point of the restraint. In reality, the restraint provided by the slab and perimeter support is not rigid. Thus, for the model to be more representative of the physical condition, such restraints should be modelled as springs. As with many aspects of modelling, there is no prescriptive guide as to what spring stiffness to assign. However, as a first pass, the axial spring stiffnesses may be estimated as $k_{axial} = EA/L$, where 'E' is the Young's modulus of the slab, 'A' the average effective cross sectional area of slab that links the stability element to the perimeter restraint, and 'L' the distance from the stability element to the perimeter restraint.

## 2.8 Rigid body motion and the use of rigid links

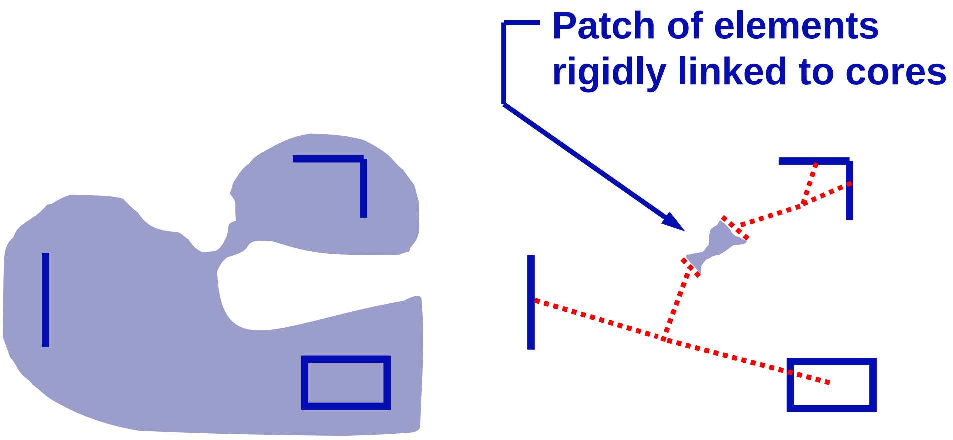
Often a floorplate will be modelled as a rigid diaphragm in an analysis, whether carried out by hand methods or by computer analysis, where some form of rigid constraint system is used in order to simplify the analysis process. In choosing to represent a floorplate in this manner, the engineer is making the assumption of rigid body motion of the floorplate as a whole.

Two of the principal questions to be addressed by the engineer in adopting this approach are whether the floor will act as a rigid diaphragm and whether the members have adequate resistance to the stresses assumed in diaphragm action.

***N.B.*** when modelling a rigid diaphragm in the context of a floor plate, this should be modelled as rigid in its plane but not rigid out-of-plane.

### 2.8.1 Does the floor act as a diaphragm?

Consider the floorplate and shear wall system shown in Fig 2:




Floorplate modelled using shells    Floorplate modelled predominately by rigid links

Fig 2. Floorplate diaphragm.

It is highly unlikely that in such a case the floorplate will act as a rigid diaphragm with the narrow 'neck' of the floorplate being relatively flexible compared to the large mass of the floorplates connected to it.

In such situations, the designer may adopt one of the following methods:

1. The floorplate may be modelled explicitly using shell elements to get an accurate representation of the behaviour of the floor system. The disadvantage of such an approach is that the model size, and thus solution time, of the analysis increases significantly.
2. The areas of floorplate on either side of the narrow region may be assumed as rigid bodies and the walls within them rigidly linked. The narrow region may then be modelled either by a 1D stick element with its properties set to represent the narrow region or explicitly by creating a patch of shell elements.

In reality, the geometry is unlikely to be as contrived as in the above example and thus a degree of engineering judgement is required to estimate the stiffness of the floorplate to justify the assumption of rigid body motion.

Determining whether the assumption of rigid body motion is valid is further complicated by the introduction of penetrations within the floorplate; this requires engineering judgement to resolve. Similarly, where large penetrations exist adjacent to core walls, the engineer should question whether the stresses can be transmitted through the floorplate into the wall to determine whether it is valid to use a rigid link in such situations or whether an alternative load path should be modelled.

Fig 3 illustrates this point. The wall on the right-hand side may be rigidly linked but the left-hand wall should not be connected in this way unless the penetration is significantly shorter than the length of the wall itself or if load transfer through end-bearing on the wall is assumed and an accompanying check is made on the stresses in this region.

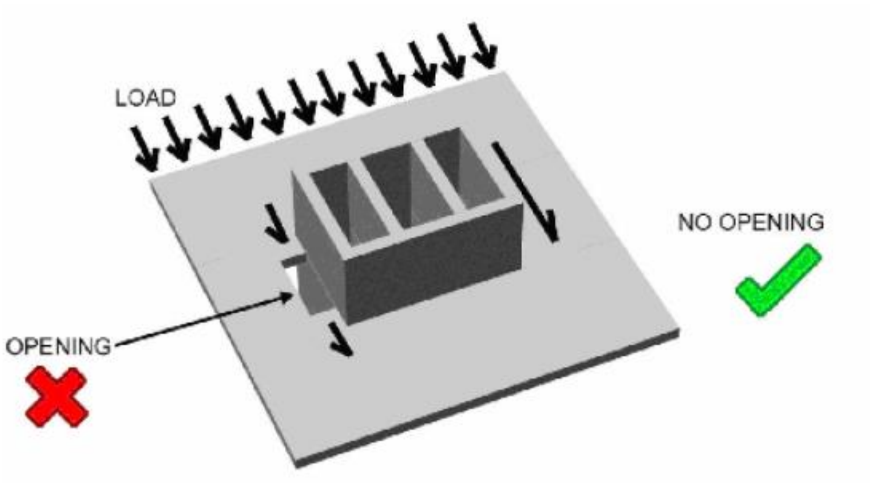



Fig 3. Transmission of diaphragm forces to stability walls.

### 2.8.2 What forces are being transmitted through the rigid diaphragm?

A rigid diaphragm, whether modelled by rigid constraints or explicitly by finite elements is required to transmit the lateral stresses to the various stability elements that are being linked. Irrespective of the form of analysis model, the adequacy of both the floorplate and the connection between floorplate and wall must be checked in order to ensure that the sections are not overstressed.

If the analysis is carried out in a software package, it is important to plot the stresses in the diaphragm and at the connection to the vertical members and then check that they have adequate resistance. Note that in the current version of GSA, the forces in rigid links may be displayed.

It is particularly important to check this transmission of stress when the design of the vertical structure and the design of the floorplates are split between different designers to ensure that the design assumptions made are compatible. By example, regions where the diaphragm is in tension may influence the punching shear capacity and/or the confined strength of the concrete within plan area of the columns (refer 2014 NST 05).

## 2.9 Shear centre, warping stiffness, and shear stiffness

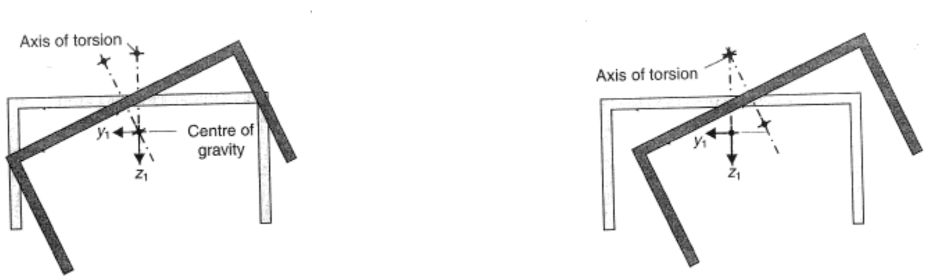
The accurate modelling of the shear centre, sectional warping stiffness, and shear stiffness, is important in modelling the overall behaviour of a section and stability system as a whole. The following summarises these effects and for a detailed treatment of this subject, refer to Timoshenko's 'Theory of Elastic Stability', Section 5. The first two of these topics were addressed in the Notes on Structures 2003 NST 03 and 2004 NST 04 and these may be referred to for further information. Notes on Structures 2006 NST 08 gives details of the 'Advanced section properties spreadsheet', available on the SSN, which may be used in determining the properties of sections that comprise point nodes and line elements.

Details of the analysis models used in this section to illustrate section behaviour are given in Appendix A.

### 2.9.1 Shear centre

The shear centre, or centre of stiffness, is the point about which a section rotates. Thus, a lateral load applied at the shear centre will cause only lateral movement with no rotation whilst a twisting moment applied at the shear centre will cause only rotation with no lateral movement.

In some software packages, including GSA, the shear centre is set at the same position as the centre of gravity when a 1D stick element is used to define a given section shape. For sections where this is not the case, this simplification is incorrect and results in a section that is overly stiff in the presence of lateral loads; this is illustrated in Fig 4.




Rotation about the centre of gravity
**WRONG**

Rotation about the centre of stiffness
**CORRECT**

Fig 4. Shear centre.

Although incorrect, the effect on the accuracy of the analysis will depend on core shape, position, number of cores in the system, and the load resisted by each core. Thus, it is not possible to generalise as to whether this effect can be safely ignored and this should be judged on a case-by-case basis.

However, it is recommended that other than in cases where a section's centre of mass is coincident with the section's centre of stiffness, a single 1D stick element should not be used to represent a core other than in the case of initial scheme design.

***N.B.*** Fig 4 and the issues raised in this section highlight the need to understand the assumptions made within a piece of analysis software in order to make informed decisions on modelling and the interpretation of results.

### 2.9.2 Warping stiffness

Warping can be defined as out-of-plane distortion such that plane sections no longer remain plane.

Although some sections may warp about their axis, e.g. an I-section under pure torsion, in the case of most cores, warping will result in the lateral movement of the shear centre as the section twists. In many software packages, including GSA, 1D elements have no warping stiffness in their formulation and thus report large rotations. An example of this is shown in Fig 5.

Three C-shaped cores are modelled in GSA; from left to right they use: 2D finite elements, 3 x 1D stick elements, and 1 x 1D stick element.

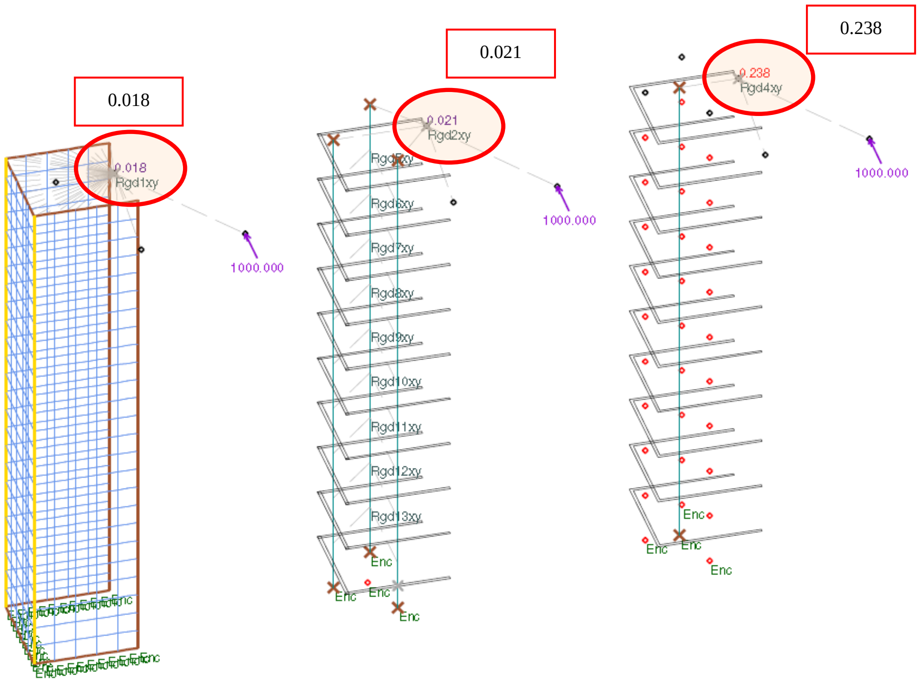



Fig 5. Modelling convention and warping stiffness.

The rotation about the vertical axis is circled in each case with the value annotated for clarity. As can be seen, the model that represents the core by a single stick reports significantly larger rotations than the other two models. The reason for this is that in GSA, a 1D stick element has no warping stiffness function. By modelling each wall of the core with a stick and rigidly constraining the walls in the xy-plane at each floor level, the rotations of the multiple 1D stick element model approach those of the 2D element model though the rotation is still overestimated when compared to the 2D element model. Calculating the rotation at the top of the section by hand gives θ = 0.0181 radians, i.e. approximately the same as reported by the 2D element model.

To better illustrate the effect of warping Fig 6 shows the plan view of the deflected shape of the C-section. The resultant deflection of the mid-point of the wall over its height is shown. Note the upward movement of the mid-point at the top of the wall relative to the bottom illustrating the movement of the shear centre due to warping.

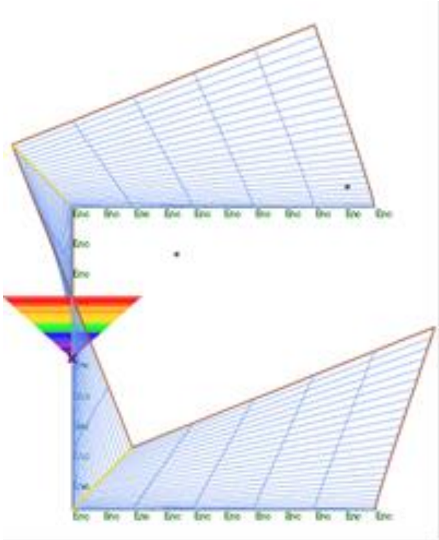


Fig 6. Warping.

***N.B.*** it is sometimes suggested that the centre of stiffness may be determined by applying two orthogonal lateral loads to a model and using the resisting moment about the vertical axis to determine the offset from the point of load application to the centre of stiffness. This is not correct as it assumes the position of the centre of stiffness remains unchanged, i.e. warping is neglected.

If the same process is repeated for a box section, Fig 7, the 4 x 1D stick element model on the left hand side does not perform well when compared against the results of the 2D element model. A hand calculation for rotation at the top of the section gives θ = 0.003 radians, i.e. the same as reported by the 2D element model.

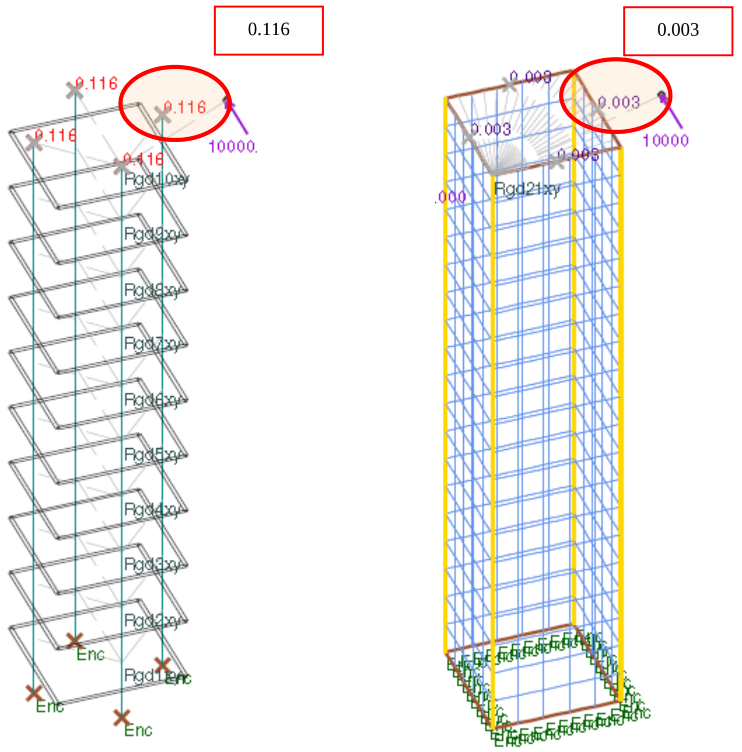



Fig 7. Behaviour of a closed section.

This discrepancy, and that highlighted for the C-shaped core, is due to the way in which the shear stiffness is accounted for and this is discussed in the next section.

For the purposes of concept or early scheme design, it may be acceptable to neglect warping stiffness as this will likely result in a conservative design. This then allows simple hand calculations and stick models to be used in these cases.

### 2.9.3 Shear stiffness

A vertical stick element is not coupled to other parts of a given wall system. This lack of shear stiffness leads to rotations being reported that exceed those determined from a 2D element model or from hand calculations for the same geometry.

In order to model the shear stiffness of a section of wall, the concept of horizontal stiff arms may be adopted by the designer as one method of addressing this issue. The stiff-arms are typically placed at story level and span from the centreline of the wall to its edge and for a general wall arrangement, the stiff-arm concept is shown in Fig 8.

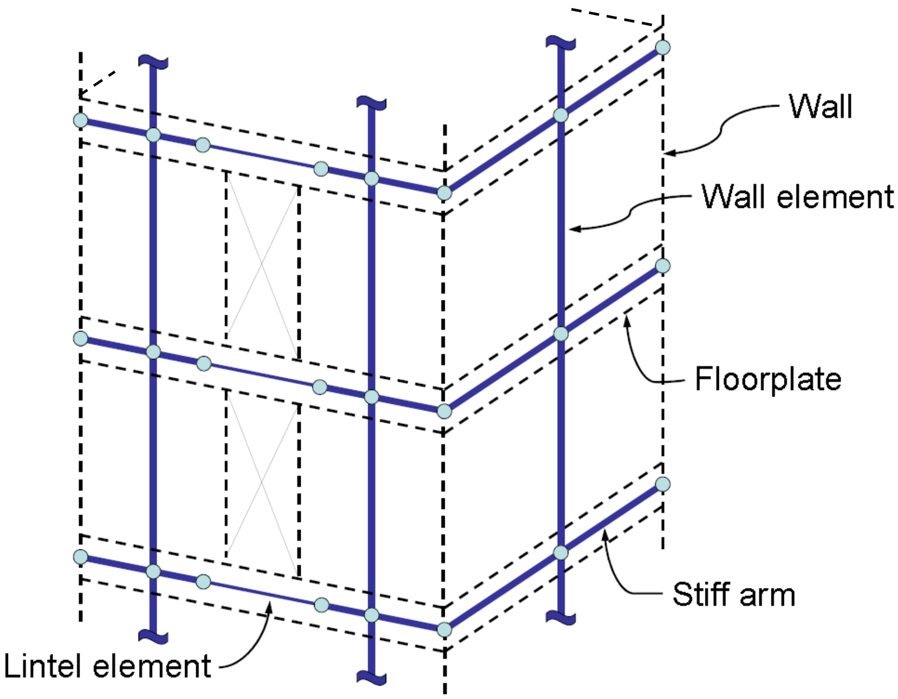



Fig 8. Stiff-arms.

Note that where penetrations are offset between levels, or do not occur at every level, care should be taken in modelling the transition zone. One option would be to use additional vertical elements to avoid imposing large loads on the lintel elements. These should be rigidly linked, and at the next floor level, re-combined into a single stick to represent the centre of the wall.

When this is applied to the box section example, the rotations shown in Fig 9 are reported. From left to right the models shown are 4 x 1D elements with no stiff arms, 4 x 1D elements with stiff arms at each story level and stiff arms at story and half-story level respectively, and a 2D element model.

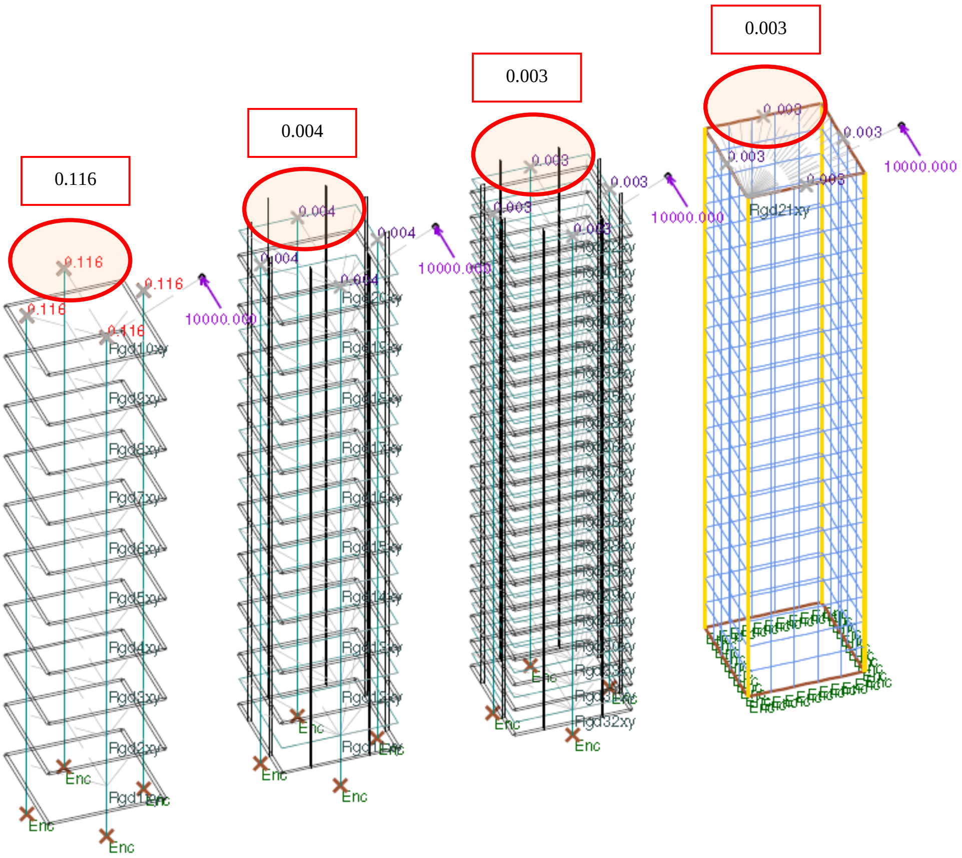



Fig 9. Rotation due to lateral load.

The introduction of stiff arms allows the overall shear stiffness to be modelled. As shown in Fig 9, this results in rotations that are approximately the same as reported by the 2D element model and the hand calculation. The introduction of mid-story stiff arms will typically only be required in low to mid-rise models to allow a better approximation of the shear stiffness. Sensitivity studies may be carried out to determine if this is necessary.

Stiff arms are in effect virtual elements introduced to ensure that plane sections remain plane for horizontal sections of each panel. Thus, it is suggested that the following properties are adopted in the definition of these elements:

- $I_{yy}$ modified by a factor of 1E+03%
- $I_{zz}$ modified by a factor of 1% (i.e. no resistance to out-of-plane bending)
- $K_z$ , the shear area factor, should be set such that shear deformation is ignored, i.e. effectively an infinite shear stiffness. If the engineer is confident that the effects of shear lag are small, this may be represented by setting $K_z = 0$ in GSA.
- J modified by a factor of 1% (i.e. reducing the torsional stiffness, GJ, to prevent the model from becoming over-stiff) – note that it is equally viable to torsionally release the ends of the arms to achieve the same effect
- the cross section of the arm should be as per the storey height

- the material properties used should be the same as those of the vertical elements representing the wall panel
- the mass of the stiff arm should be set to zero if gravity load cases are used in the analysis.

Stiff arms with the above section properties will result in plane sections remaining almost plane. In reality, there will be some distortion of wide flanges and not all of their width will be effective. This effect, known as shear lag, is automatically included in 2D element models. It can be approximately modelled in 1D element models by doubling the shear stiffness of the vertical sticks and setting the shear area factor $K_z$ of the stiff arms to 2, thus assigning half of the shear flexibility to the vertical elements and half to the horizontal elements.

The engineer may wish to carry out a sensitivity analysis to examine the effect of changing the above modifiers. Care should be taken if modifiers are set to very high or low values to avoid numerical problems in the analysis engine of a given software package.

An example of the use of stiff arms is given in Section 3.5 – Single stick to model each element in each core.

With respect to the suggested modification factors, when carrying out a buckling analysis, reducing the value of J may result in spurious results being reported for the stiff arms. In addition, when modelling long lengths of wall, it may be appropriate to limit the width of the stick element to ~15 × wall thickness. For typical cases, this should ensure that modifications made to the element properties do not affect the results of the analysis.

The above limit is derived by considering that the torsional buckling of a beam element will occur when the axial stress equals $GJ/(I_{yy}+I_{zz})$, at which load the element will twist about its axis. For an element width b and depth t, the critical stress will be $G.bt^3/3 / (bt^3/12 + b^3t/12)$, which approximates to $4Gt^2/b^2$. For $G = 14/2.4 = 5.8$GPa and $b/t = 15$, the critical stress will be about 100MPa, which would normally be at least 10 times the average stress in the stick.

## 2.10 Penetrations in walls

Most stability walls are penetrated to some extent. In 2D element models, these penetrations may be modelled explicitly, but in 1D element models they are approximated either by reducing the stiffness of the wall as a whole or by modelling the lintel beams around the penetration. The cases of 1D and 2D element models will be examined in this section.

Although slabs can be assumed to link walls, it is important not to overestimate their stiffness in terms of coupling the walls, nor to try and include large amounts of reinforcement which although may result in the ‘correct’ numerical answer, in practice may result in an unconstructable detail.

***N.B.*** it should be remembered that if a stability model is constructed at an early stage in a project, account should be taken of the likelihood of the introduction of additional penetrations and the change in size of those penetrations currently part of the structure resulting in a reduction of stiffness.

### 2.10.1 1D element models

For relatively simple stability systems, simple hand calculations may be used to analyse linked or coupled shear walls. 1997 SGN 08, dating from November 1983, and CIRIA Report 102 – 'Design of shear wall buildings' give guidance on carrying out such calculations. Note that the concept of a pair of walls being coupled or linked applies equally to simple frame modelling where walls that are coupled act together as a composite element linked by the cross beams; conversely, in a linked system, although connected by linking beams, the walls act as separate entities due to the slenderness of the linking member. Whether a pair of walls are coupled and to what extent is a function of the geometry with guidance being given in 1997 SGN 08. Note that although a floorplate may act as a couple, its relative flexibility makes this an inefficient link and thus would typically be neglected.

Where a wall is coupled, the effectiveness of the stiffness of the coupling beam will dictate the effectiveness with which the walls act as a single stiff element.

As noted in Section 11.14 of 2005 SGN 01 'Structural Modelling Manual', modelling such beams as a single element spanning between wall centrelines will underestimate the system stiffness whilst assuming full fixity at the face of the wall will overestimate the stiffness.

Full fixity is typically assumed to occur at h/2 from the face of the wall where 'h' is the depth of the coupling beam as shown in Fig 10. However, this approach is conservative where the depth of the lintel is large relative to the width of wall it frames into, e.g. where $b_{wall}$ /6 < h. Therefore, in the absence of parametric studies being carried out for a given case, it is suggested that a nominal limit of 200mm on beam embedment is appropriate.

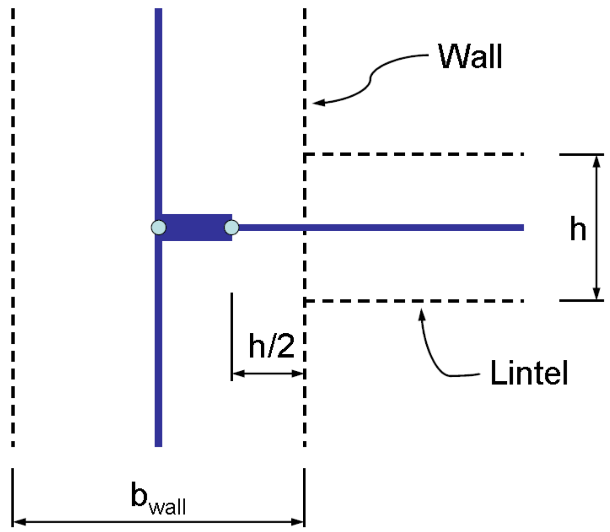



Fig 10. Model for joint rigidity.

In many cases, it will not be practicable to define explicitly the geometry of the coupling beams as described above. Thus, to model the stiffness of the coupling member, the following three options are suggested:

1. element offsets, if available in a given software package, may be used to locate the ends of the coupling beam
2. connect the coupling beam between wall centrelines and transform the section properties
3. connect the coupling beam between the faces of the walls and transform the section properties

In #2 and #3, the shear area and bending inertia of the coupling beams are transformed to account for the length of the element considered flexible, the effective length, Le = l + min(400mm ; h), and the actual length of the element in the model, L, Fig 11. Thus, the shear area should be factored by (L/Le) and the bending inertia by $(L/Le)^3$.

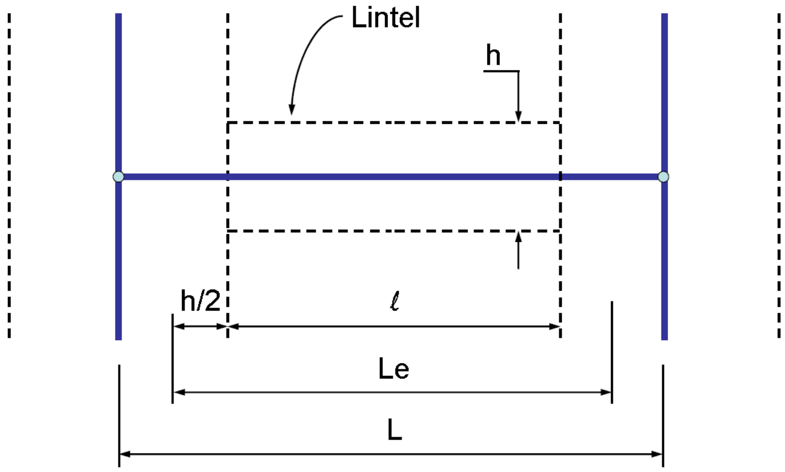



Fig 11. Coupling member.

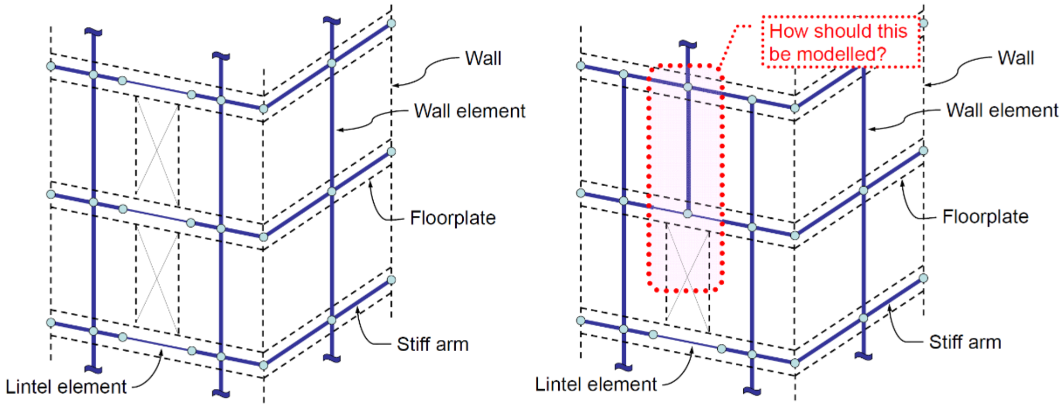
Note that the dimension L is the length of the lintel element, which may not be the distance between the centrelines of the walls.

In the above, the uncracked stiffness is being modified. Further adjustment would be required to consider the effects of cracking in the section.

The coupling beams are not considered as deep, i.e. span < 3 times the overall section depth as defined in EN1992. If such sections are classified as deep beams, a single stick element will not perform well as the assumption that plane sections remain plane no longer holds true and consideration should be given as to whether such zones should be modelled using 2D elements.

### 2.10.2 Penetrations under a wall element

The typical modelling convention and the specific situation of penetration occurring under a wall element is shown in Fig 12(a) and Fig 12(b) respectively. This section considers the options for modelling the latter situation.




Typical model

Wall element above penetration

Fig 12(a – b). Penetrations in wall.

Two methods are suggested for dealing with such situations:

1. deep beam analogy
2. strut-and-tie model.

## Deep beam analogy

The deep beam analogy, Fig 13, is the recommended approach.

As in the design of a deep beam, a band of tension reinforcement should be provided over the penetration with 100% of the mid-span reinforcement being anchored at each end.

For the purposes of determining the amount of reinforcement required (which is in addition to that required for shear), the lever arm of the deep beam should be taken as 60% of its effective span, i.e. 0.6Le.

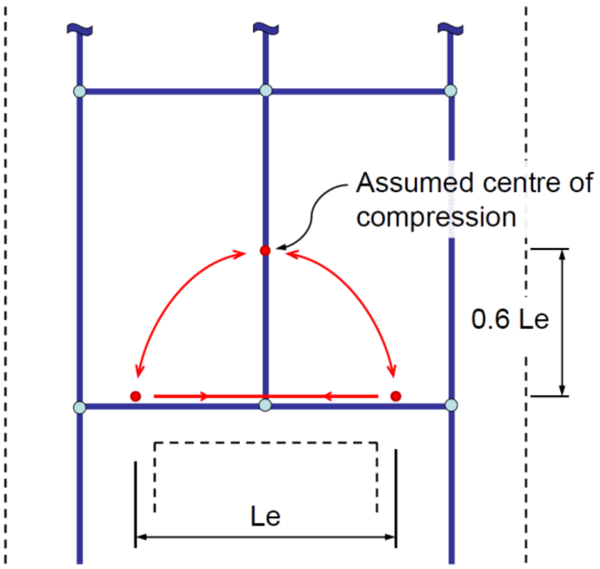



Fig 13. Deep beam analogy.

### Strut-and tie model

In the strut-and-tie approach, stick elements are used to explicitly model the struts, Fig 14.

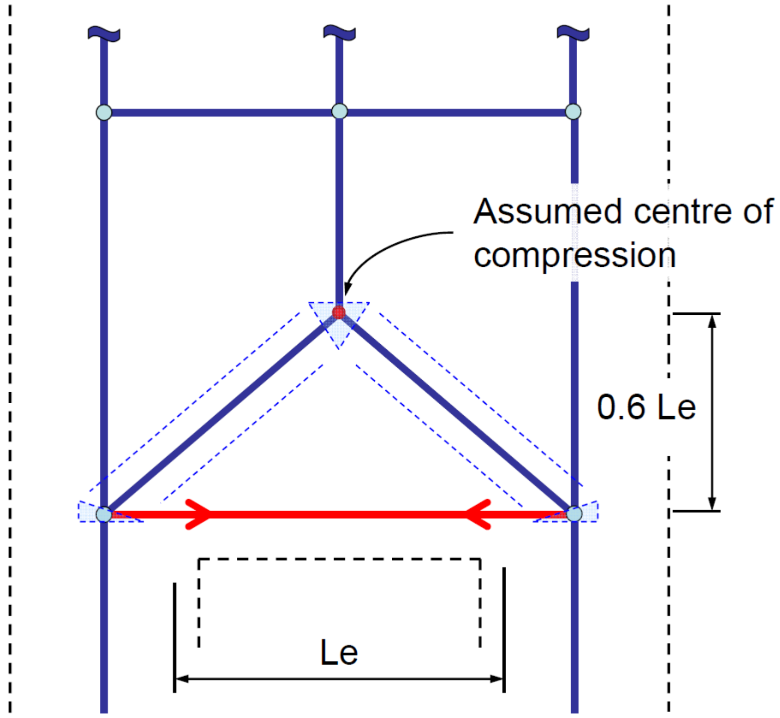



Fig 14. Strut-and-tie analogy.

The apex of the triangle is set such that it gives approximately the correct lever arm between the centres of compression and tension.

With respect to the properties of the struts, a starting point would be to define them as having half the area of the vertical strut loading them. The tie element should have the axial stiffness of the reinforcement required.

## 2.10.3 2D element models

If 2D elements are used to model the shear walls, it is important to have sufficient elements in the coupling beams in order to model accurately the stress flow between beam and wall. It is suggested that a minimum of four elements through the depth and six elements across the span are used. Attention should also be paid to maintaining a good element aspect ratio, which may in turn lead to significantly more elements being required in the model.

Note that unless automatic post-processing of the stress field is available, the stresses must still be integrated over the section depth in order to carry out the section design.

If it is felt necessary to model the cracked behaviour of the coupling beams in such models, the following two options are presented:

- Reduce the stiffness of all elements in the coupling beam – this is perhaps the simplest option though it is likely to underestimate the stiffness, and hence coupling effect, of the beam.
- Alter the section stiffness on an element-by-element basis – although notionally more accurate, the time and effort required to alter the stiffness of all coupling beam elements may prove impractical without the aid of an automatic processor. If this approach is adopted, the stress state used to evaluate stiffness should be considered in that an envelope of maximum, and most likely non-coexistent stresses, may result in an overestimate of the reduction in stiffness.

Note that in some instances it is unconservative to underestimate the stiffness of the core such as when the effects of temperature or shrinkage are being considered.

### 2.10.4 Modelling wall returns adjacent to penetrations

Whether 1D or 2D elements are used, the designer has to decide whether to model short returns such as those that may occur in a lift shaft core between the penetration for the lift door edge and the corner of the core wall. In the context of a typical building structure, ‘short’ may be considered as less than twice the wall thickness.

Possible approaches to this are as follows:

- ignore short returns – the simplest option and will result in a conservative design
- model the return as part of the sidewall – i.e. in the stick model approach, define the sidewall as an L-section. For short returns, the effect of the discrepancy between the position of the centre of mass and shear centre of the section will be negligible
- model the return explicitly – whether modelling using 1D or 2D elements, the return wall may be modelled using the guidance given within this Note.

Whatever method is adopted in considering short returns, the designer should ensure that the design assumptions are followed through into the detailing of the wall itself, the design and detailing of adjacent structure, and that a clear load path is present for the chosen solution.

Although in modelling terms, the simplest option is to ignore short returns, in terms of the physical reality, and as noted elsewhere, wherever possible openings should not be placed adjacent to the corner of a wall, as shown below, as the lack of a return wall reduces the stiffness of the section, Fig 15. It also complicates the detailing between lintel beams and the end wall.

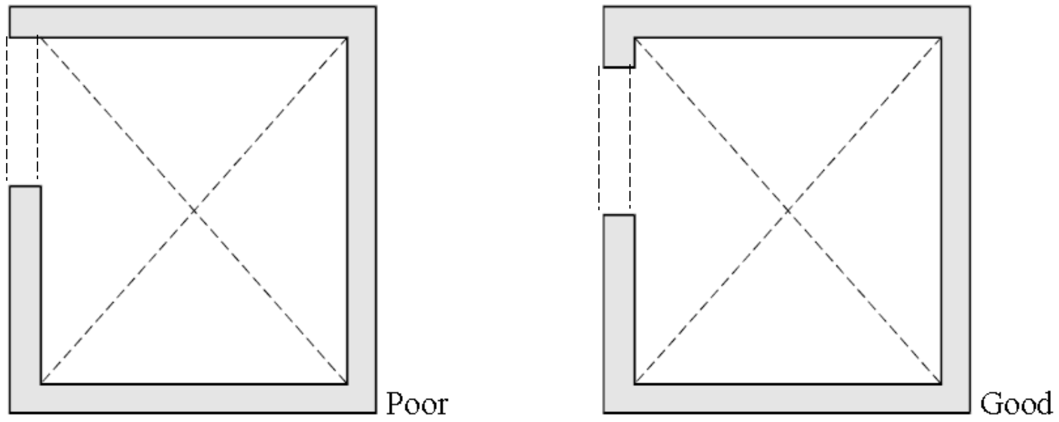



Fig 15. Return walls in cores.

## 2.11 Temporary Conditions

Depending on the form of construction and the overall project programme, the cores and walls may be constructed in advance of the surrounding structure. In such cases, consideration should be given to the lack of restraint in this temporary condition as an additional design case.

# 3 Modelling options

## 3.1 Introduction

In order of increasing complexity, and in addition to hand calculations, the options in Fig 16 are considered for modelling cores.

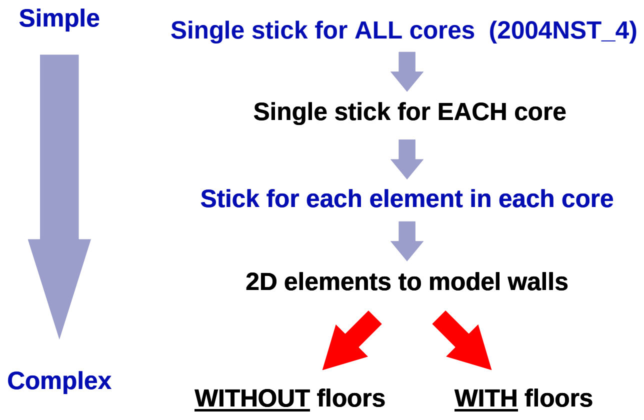



Fig 16. Modelling options.

## 3.2 Hand calculations

It is often possible to reduce complex systems to simple models that may be solved by straightforward hand calculations or with simple computer models. Although in many cases a more detailed model may be required for the final design, the starting point for any analysis should be a set of simple calculations to allow comparison with the more complex analysis models that may follow. This process allows the expected behaviour as predicted by simple models to be compared against a more complex analysis to confirm that the behaviour, e.g. whether penetrated walls are coupled or linked, load paths, etc., and the magnitudes of forces, deflections, rotations, etc., are as expected otherwise unexpected differences between models should be investigated. Significant differences in results between modelling approaches should always be explained by the engineer before proceeding further with the analysis or design.

Examples of simplifications that may be made in deriving section properties for a single stick are given in Appendix B. It is important to note that the assumptions made in deriving a set of simplified set of properties must be confirmed by the engineer, e.g. wall stiffness relative to that of the lintels.

## 3.3 Single stick to model all cores

As noted in Section 3.2, complex systems may be reduced to relatively simple models. Given that the layout, geometrical, and mechanical properties of a series of stability elements are known, the system may be represented by a single stick whose properties are explicitly defined.

This process is detailed in 2004 NST 04 and offers a quick way to investigate the behaviour of a stability system formed of individual elements or cores. However, other than for simple individual wall or core shapes, it will prove difficult to use this method to carry out design calculations beyond those that would be expected in the early stages of a scheme design.

An approximation of the reduction in system stiffness due to penetrations in walls and cracked section properties should be made by the engineer where these effects are deemed significant. However, a simple percentage reduction in overall stiffness may be appropriate dependant on the overall accuracy of the model and applied loads.

## 3.4 Single stick to model each core

In this form of modelling, a single stick is used to model each core within a building. Given the limitations of single stick models to model rotation as described previously, this form of modelling tends to be limited to scheme design calculations or for stability systems consisting of single wall elements. However, it may be suitable for carrying out detailed design of sections of relatively simple geometry if deflection is not critical.

The advantage of this form of model is that it is relatively quick to construct the analysis model and it is straightforward to interpret the results. As in the case of a single stick being used to represent the entire system, Section 3.3, an approximation of the reduction in wall stiffness about both axes due to penetrations, and due to the effects of cracking, must be made by the engineer.

## 3.5 Single stick to model each element in each core

A common method of modelling a stability system is to represent each part of a wall panel by a stick element. An example of this is shown in Fig 17 for a single storey of a building core:

Each wall panel is represented by a vertical 2-noded beam element at its centroid. These panels are connected to other panels by a system of horizontal 2-noded beams, or stiff arms, as described in Section 2.9.3, between wall centre and edges to ensure that plane sections remain plane for horizontal sections of each panel.

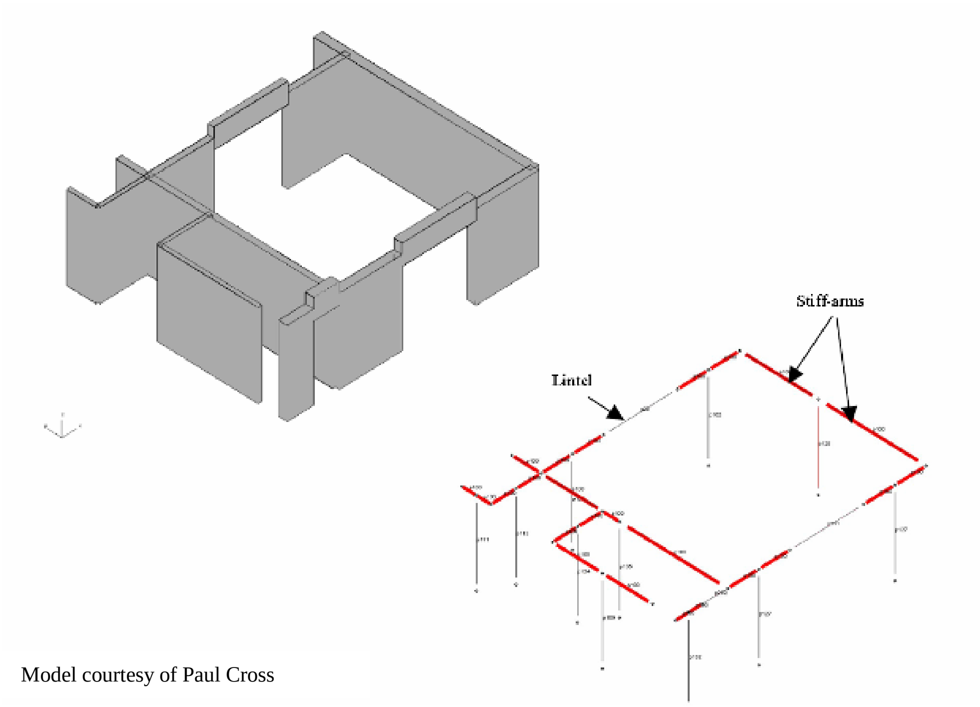



Fig 17. Stick model with rigid arms.

For the case of penetrations through a wall, flexible beams representing the lintels span between the wall edges, i.e. the ends of the horizontal beams. Local deformation at the junction of the walls and the lintel beams is simulated by reducing the bending and shear stiffness of the lintels as previously described.

The floorplate is assumed to be a horizontal rigid diaphragm at each floor level (but see Section 2.8) and represented by rigid constraints in the global xy-plane. In addition to linking the wall elements, the centre of mass and/or centre of façade area may also linked on a given floor plane to allow lateral loads to be applied and allows the mass of the floor to be modelled for notional loading and dynamic analysis.

## 3.6 2D finite element models

With the increasing ease of modelling using 2D elements in various software packages, the engineer now has the option to model the stability system, in addition to the overall structure, in a single whole building model. In many cases, for an experienced engineer, creating such a model will be simpler and quicker than the stick modelling approach described previously. However, this ease of modelling requires a good understanding of assumptions made in the finite element method, understanding of structural behaviour, the ability to interpret the output data both before and after post-processing, and design experience to know what needs to be modelled. These requirements are vital in creating models that retain simplicity whilst delivering the required design data. Often unnecessarily complex models are created before thought is given to what data is required and as mentioned elsewhere, planning the analysis will ‘hopefully’ reduce the likelihood of over-modelling.

The issues described in the following sections are typically associated with a concrete floorplate solution as opposed to a composite deck. In the latter case, the designer should consider the form of the load transfer at the interface between their particular floor system and core walls.

***N.B.*** by definition, plane stress elements resist in-plane stresses only; plate elements resist out-of-plane stresses only; shell elements resist both in-plane and out-of-plane stresses; the use of the prefix 1D, 2D, and 3D, refers to the element type and not the number of dimensions the model is constructed in.

### 3.6.1 Floorplate modelling

As with 1D element models, the floorplate may either be represented by rigid constraints or modelled using 2D elements. Issues surrounding the former case have been discussed in previous sections thus instances where the floorplate is explicitly modelled will be examined here.

Floorplates may be modelled using plate or shell elements though in the case of a stability analysis, it is likely that shell elements would be used in order to allow the in-plane effects of lateral loading to be examined.

If the sole purpose of explicitly modelling the floor is to model the floor diaphragm action without the intention of subsequently using the model for floorplate design, it is common to increase the size of the elements to reduce the model size. In such cases, refinement of the floorplate mesh will be required close to the stability walls in order to ensure an even transfer of stress. Note that it is not correct to leave nodes unconnected in such circumstances (or indeed to leave nodes unconnected generally).

Issues surrounding the way in which column-slab connections are modelled are discussed in 2006 NST 01.

### 3.6.2 Out-of-plane bending in walls

If shell elements are used to model a wall, moments induced by out-of-plane bending will be reported by the analysis. These moments cannot be ignored on the assumption that other walls in a perpendicular direction will resist the load and they should be designed for this moment.

One option open to the designer is to use elements to model the walls that only resist in-plane stresses. In effect, the designer is controlling the load path by forcing those walls that are not perpendicular to the line of action of the resultant force to resist the lateral load. The validity of such an approach should be considered and validated by the designer, as out-of-plane bending will still be present irrespective of the results reported by the analysis model. One possible means of justifying this assumption would be to consider whether there is sufficient ductility in the system to allow this assumption to be made. If this approach were adopted, it is important that the assumption is followed through to the detailing of such joints and the specification of reinforcement with sufficient ductility (2007 NST 06). Where proprietary systems are used, e.g. prefabricated strips of pullout bars, the ductility of such systems should be investigated.

However, if concrete failure is governed by out-of-plane bending, for example due to high axial loads, it cannot be ignored.

Note that if the wall is modelled with plane stress elements and the floorplate is modelled using either plates or shells, there will not be rotational compatibility at the wall/slab junction, i.e. the slab will be simply supported on the wall. However, it is important to remember that there will be a concentration of stress at the slab – wall junction and that the variation of shear stress along this junction is likely to be non-uniform, particularly at corners. Thus, the designer should justify modelling assumptions with particular attention being paid to appropriate reinforcement detailing to account for the stresses that will exist in these areas.

***N.B.*** this is an example of the complexity of interface zones, and should act as a reminder to the designer that the assumptions made in one part of the analysis model must be carried through the design as a whole, e.g. in the above, the assumptions made in the slab design should be compatible with those of the wall design. Given that design responsibility is often split within a design team, it is important that such assumptions be stated in the calculation plan and communicated to the design team on a regular basis.

## 3.7 Inclusion of secondary elements in the stability model

There are two main reasons for including secondary elements, e.g. columns or façade structure, in the stability model: to allow the stability model to be used in the load takedown or to gain some benefit from their stiffness when assessing stability.

If such elements are to be used only for the purpose of a load rundown, they should be released within the model such that they do not contribute to the stability of the structure. If these elements are assumed to contribute to the stiffness of the stability model, the stresses that they develop must be accounted for, and added to, the stresses generated in the non-stability related design cases.

In the latter case, in addition to designing for the stresses a member experiences, any construction sequence related assumptions should be recorded in the calculation plan and must be communicated to the contractor. Failure on the part of a design team in this respect was highlighted in 2005 NST 03.

# 4 Concrete section properties

When considering the accuracy of a given modelling approach, it should be remembered that the values used in defining the material properties of the concrete section, e.g. Young's modulus, E, may have a significant effect on the results obtained and this should be remembered when evaluating the accuracy of the results.

Section stiffness, whether flexural (EI), shear (GA), axial (EA), or torsional (GJ), is a function of the section modulus and the effective section properties. In this context, 'effective' relates to that part of the section actively resisting the applied stresses, e.g. in a cracked concrete section, only part of the section will contribute to the resistance.

Factors affecting these values are briefly considered below and further guidance may be obtained from Concrete Society Technical Report No. 67 – 'Movement, restraint and cracking in concrete structures.'

## 4.1 Section stiffness

The value of E, and hence G, will be related to the proportion of the applied load that can be considered to be permanent for a given load combination or load type. In the case of wind load for example, the designer may take the view that the effect of wind is transient and adopt a short-term set of analysis properties. When considering gravity loading, some proportion of that will be permanent resulting in a reduction to the section modulus.

Thus in some instances, two (or more) sets of material properties may be required in an analysis model to allow the effects of the various actions, and their durations, to be considered.

## 4.2 Creep and shrinkage

Creep and shrinkage may be determined in accordance with the relevant code of practice. The effects of this may then be incorporated into the analysis, e.g. using AdSec to account for these effects.

## 4.3 Cracking

Cracking will reduce the stiffness of a section. Unfortunately, it is difficult to determine with any degree of certainty the exact level of cracking and in many instances the 'best' solution is to carry out upper and lower bound analyses for a given case.

An initial assessment may be made as to the reduction in stiffness by plotting the moment-stiffness curve for the section in question for a given set of co-existent stresses, e.g. using AdSec, and reducing the section stiffness as appropriate. However, it should be remembered that this is an iterative process in that after the stiffnesses are altered, the load is likely to redistribute requiring the stiffnesses to be altered again. This process is repeated until a converged, or stable, solution is reached. Alternatively, a blanket reduction in stiffness may be applied, possibly with coupling or link beams being assumed to be more cracked than the walls.

***N.B.*** ULS reinforcement design is not carried out using an iterative cracked analysis approach.

# 5 Design checks

General comments on checking are made followed by consideration of the principal design checks to be carried out that fall within the scope of global or local checks and these are described in this section.

It should also be remembered that the stiffness provided by the stability system is likely to reduce with height, e.g. as walls are either removed or reduced in thickness. Thus, it may be necessary to carry out design checks at various levels of a structure based on the structural layout.

## 5.1 Identification of the critical load case

Whether global or local effects are being checked, it is clearly important to identify the critical load case(s) to be considered.

Designing for the worst case from an envelope of results can often lead to an overly conservative design, as the maximum values may not co-exist.

It is sometimes suggested that to overcome this, the permutations resulting in the worst action from a particular effect be identified, and the associated utilisation for that permutation be calculated.

Fig 18 shows that this approach may lead to the worst-case permutation being missed; it illustrates a case where:

$M_1 > M_2 > M_3$ and $F_3 > F_2 > F_1$

Only checking the maximum values and their co-existing actions would lead to the combination of ($F_2$, $M_2$) being neglected which in this instance is the critical case.

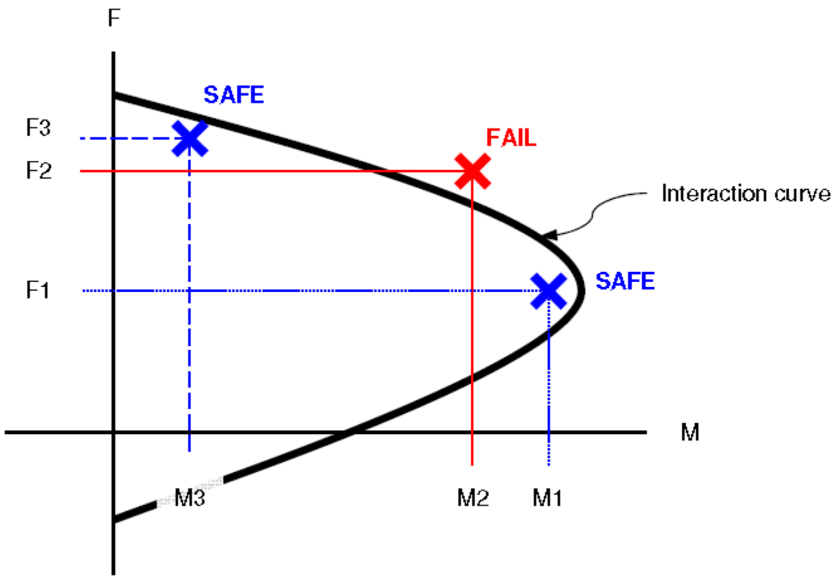



Fig 18. Determination of the critical load case.

Thus, a number of permutations, beyond those with a given maximum or minimum value should be examined to confirm that the worst case is identified and designed for as appropriate. Similarly, the adverse effects of actions should also be considered when identifying the appropriate design cases, e.g. the reduced shear capacity in the presence of axial tension.

Note that for a given section, software such as AdSec may be used to plot the design permutations on N–M or $M_1$–$M_2$ interaction curves.

## 5.2 Identification of the critical section

Once the critical load case(s) has been determined, the critical elements to be checked need to be identified. However, in a complex concrete core, it is often difficult to identify what section to check. If a core comprises a series of walls, e.g. an enclosed box section, then the behaviour of the section at the ultimate limit state would be non-linear and the design would need to consider the box section acting as a single unit rather than a set of discrete members. Therefore, it is important to identify which elements are likely to be acting as composite sections, for instance H-sections, boxes, channels etc, which are separated by major and regular openings (for instance a corridor). The capacity of such compound sections can then be determined using software such as AdSec if alternate methods of post-processing are not available.

## 5.3 Stress concentrations in 2D elements

Often high peak stresses occur when interrogating the results associated with 2D elements and in some cases, these may lead to post-processed results resulting in large areas of reinforcement being required or in the extreme element being shown to be un-reinforceable for the reported stress field.

Often these peaks will be numerical effects due to the mesh geometry and element density. In situations where these peaks are representative of real stress concentrations, in reality, the concrete is likely to crack in such locations and the stresses redistributed. This behaviour is typically not modelled by software in general use.

Thus, although such peak stresses can often be ignored, or discounted, it is important to investigate the reason why they occur in a given analysis to make an informed decision, which in some instances may mean ignoring the stress concentration but introducing additional reinforcement to control the cracking due to stress re-distribution.

## 5.4 Global checks

The principal checks that affect the global behaviour of the stability system are discussed in this section.

### 5.4.1 Lateral displacement

The provision and sizing of the stability system will often be stiffness governed. In this respect, the lateral behaviour of the structure must be considered and limits placed on such movement. Two principal methods are used in practice: limiting the story height:drift ratio and through lateral acceleration acceptance criteria. In reality, concept and scheme level calculations may be carried out to the former with detailed scheme and design level calculations being influenced by the latter. Both options are considered below:

#### Height : drift ratio limits

Lateral displacement is typically checked for story drift and whole-building drift. In the absence of further guidance, it is suggested in BS8110-2 (3.2.2.2) that storey drift be limited to (story height)/500 whilst CIRIA Technical Note 107 suggests that overall deflection be limited to height/1000 for short-term loading. Note that these limits apply to the worst point on a typical level, e.g. at a remote edge of the floorplate, not simply at the top of the core.

In many cases, particular limitations on building movement will be imposed by the cladding system. This should be investigated at an early stage in the design process and the assumptions made by the designer communicated to the client and design team, both internal and external. Similarly, bespoke architectural features such as expressed lift shafts or stairs are likely to be movement sensitive and, if not isolated in some way from the primary stability system, should be checked.

Such building drift criteria are typically used to (nominally) control lateral accelerations and second order effects alongside damage to cladding and architectural finishes. However, as long as it can be shown that these effects are considered explicitly, there is no technical reason to limit building deflection to such codified ratios.

It is often difficult to find detailed guidance beyond the above and in part, the imposed limits are often related to previous project experience where they have been found to be acceptable. However, in such cases, the designer should fully understand the origin and reasons for the use of non-codified limits to confirm their suitability for a given design situation.

#### Lateral acceleration acceptance criteria

Provided that limits imposed by the façade system on differential movements are met, an alternative to height/drift ratios is to limit building movement based on acceleration serviceability criteria. In slender buildings, there is a susceptibility to oscillation caused by the action of wind. Such lateral oscillation can often be felt by occupants in strong winds, and can result in psychological or physiological discomfort.

Acceleration serviceability criteria have been developed from surveying people living and working in tall buildings and through a series of both motion simulator and full-scale motion studies, and are related to the peak level of acceleration likely to be experienced within a certain period (usually 1 year or 10 years). The acceptability of the predicted acceleration is a function of both ‘perception’ (what people feel) and ‘acceptability’ (some people will feel more acceleration than others will, but may also be more tolerant of it). Acceptability of the motion can be dependent on the building’s intended use and its intended occupants.

Accelerations felt by occupants of the building can be reduced either through changes to the building shape or by adding specialist damping systems. Damping systems come in a variety of forms, from simple passive systems to much more complex active systems. Space availability, cost, maintenance and response reduction are issues to consider in deciding on appropriate damping systems. The damping ratios chosen for use in an analysis are likely to have a significant influence on the modelled behaviour.

Detailed guidance on acceptance criteria is outside the scope of this Note; similarly, there is no single set of agreed damping values. For further details, contact Arup’s Advanced Technology & Research group.

### 5.4.2 Vertical displacement and axial shortening

Although usually not critical, vertical displacement should be considered by the designer particularly with respect to differential movements between members of significantly different stiffnesses and/or adjacent elements of vertical structure with markedly different stresses such that differential shortening becomes an issue. When a steel frame is used with a concrete core, the difference in short and long-term shortening of the different materials should be accounted for.

Detailed treatment of this topic is beyond the scope of this note and this aspect of the design should be considered in early design reviews and specialist advice sought if necessary.

### 5.4.3 Torsional stability

As vertical load is applied, the structure will try to twist and this twisting is resisted by the vertical elements of the structure, principally by the structural walls. Thus, it must be checked that the resistance provided to twisting should exceed the destabilising twisting moment. A method of carrying out such a check is given in 2000 NST 07.

Although the destabilising loads will be greatest at the bottom story of the structure, it is also likely that the resistance provided by the walls will also be at a maximum. As such, it cannot be said that the critical point to carry out this check is at the bottom story of a structure. It is suggested that the torsional stability should be checked at those floors in a building where the torsional stiffness provided by the stability walls is reduced, i.e. as the building height increases, it is often the case that the stability walls are reduced in thickness, length, and number.

Note that all vertical elements must be modelled to evaluate the destabilising effect due to their axial load combined with their eccentricity on the elements resisting the twisting effect.

### 5.4.4 Secondary effects

The issues surrounding secondary effects on concrete cores are covered in 2004 SGN 02.

Significant second order effects are most likely to develop in stability systems resisting low wind loads whilst stabilising heavy floors. Many stability systems in low-rise structures fall within this category.

## 5.5 Local checks

The principal checks that affect the local behaviour of the stability system are discussed in this section.

### 5.5.1 Wall slenderness

Ideally, walls should be schemed to avoid the need to design for the additional stresses due to slenderness. However, in some instances, e.g. where lateral restraint does not exist, or in the temporary condition, slenderness must be considered.

1999 NST 11, 2002 NST 04, and 2007 NST 09, address the analysis and design issues surrounding wall slenderness with the latter two notes considering outstand walls.

Also, note that in the permanent condition the effect of penetrations on the restraint provided to a given wall, Fig 19, must also be considered when carrying out these checks.

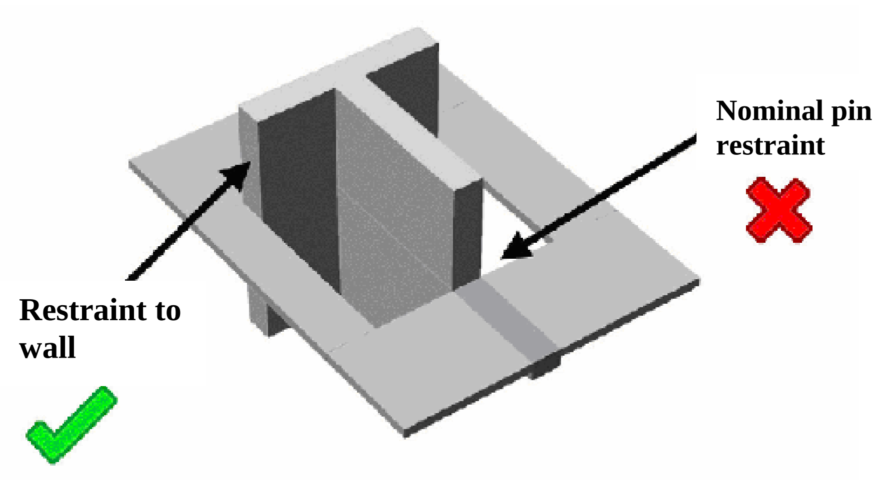



Fig 19. Lateral restraint to walls.

1999 NST 11 was originally written for the design of panels in offshore gravity base structures, which are subject to in-plane normal forces and shears and are designed as thin as possible for buoyancy reasons; the approach is also suitable for core walls with at least simple supports on two opposite sides.

2002 NST 04 was originally written for a cruciform core that extended for several levels through an atrium with no support from floors. The symmetric arrangement meant that all four walls could buckle rotationally the same way, which means that rotational restraint cannot be taken at the centre. The approach described in this Note is appropriate for cantilever walls where there is no rotational restraint on the vertical edges and stability is only achieved through torsion.

2007 NST 09 is appropriate for cantilever walls where rotational restraint is available on the laterally restrained vertical edge and stability can be achieved through bending.

### 5.5.2 Buckling

Local buckling of the wall typically relates to slenderness issues as described in previous sections and should be checked accordingly.

### 5.5.3 Flexure, shear, torsion, and axial stress

The stresses induced by the destabilising forces resisted by the stability system, whether at a given level or at the foundations, must be considered in the design of these elements, e.g. by superposition of stresses as appropriate.

The actual design process is as per standard practice and in accordance with the appropriate codes of practice. However, care should be taken to account for simultaneous effects, e.g. axial stress superimposed on an element under shear will either enhance or reduce the section capacity depending on whether it is tensile or compressive. The effects of load reversal must also be considered when carrying out such design checks.

# 6 Seismic analysis

As noted elsewhere, consideration of seismic effects and the implications to analysis and design are generally outside of the scope of this Note. This section is therefore intended to be a brief introduction to this subject and to highlight some of the main issues. Specialist advice should be sought if seismicity is being considered on a project.

## 6.1 Scope

This section contains items specific to seismic analysis. In general, most of the issues listed elsewhere in this Note will apply in addition to these specific measures. This section concentrates on linear seismic analysis. For non-linear analysis, specialist literature such as ASCE 41-06 and guidelines by the Council on Tall Buildings and Urban Habitat (CTBUH) should be consulted.

## 6.2 Core layout

Care should be taken choosing core layout to:

- minimise stiffness eccentricity
- minimise and preferably eliminate transfer structures
- minimise axial load in shear walls.

Note that where shear walls carry a significant proportion of gravity load (typically more than 20%), then this system cannot be classified as a dual system where vertical loads are carried by the frame whilst lateral loads are resisted by both frame and structural walls. In such cases, walls should be classified as a bearing wall system (US codes) or wall system (Eurocode).

Structures with a complex and/or highly irregular form, e.g. outriggers, are not covered by the design codes and reference should be made to guidelines produced by CTBUH as to how the analysis should be carried out.

## 6.3 Cracked stiffness of elements

Since seismic loading will usually take concrete elements beyond their elastic limit, modelling their effective cracked stiffness is a key feature of the analysis models. Stiffness listed in Fig 20. are taken from FEMA 356.

| Component | Flexural Rigidity | Shear Rigidity | Axial Rigidity |
|---|---|---|---|
| Beams—nonprestressed | $0.5E_cI_g$ | $0.4E_cA_w$ | — |
| Beams—prestressed | $E_cI_g$ | $0.4E_cA_w$ | — |
| Columns with compression due to design gravity loads $\geq$ 0.5 $A_g f'_c$ | $0.7E_cI_g$ | $0.4E_cA_w$ | $E_cA_g$ |
| Columns with compression due to design gravity loads $\leq$ 0.3 $A_g f'_c$ or with tension | $0.5E_cI_g$ | $0.4E_cA_w$ | $E_sA_s$ |
| Walls—uncracked (on inspection) | $0.8E_cI_g$ | $0.4E_cA_w$ | $E_cA_g$ |
| Walls—cracked | $0.5E_cI_g$ | $0.4E_cA_w$ | $E_cA_g$ |
| Flat Slabs—nonprestressed | See Section 6.5.4.2 | $0.4E_cA_g$ | — |
| Flat Slabs—prestressed | See Section 6.5.4.2 | $0.4E_cA_g$ | — |

Fig 20. Cracked stiffness.

It is also possible to check these stiffnesses by considering moment – curvature diagrams or secant stiffness which may be investigated, e.g. in AdSec.

When core sections are modelled using shells or several beam elements, it is difficult to alter the bending stiffness without altering the axial stiffness of the elements. Since the effect of altering bending stiffness is often more significant than approximations in modelling, this would favour a structural model with fewer beam elements, e.g. a single stick to model each core.

The dynamic modulus of elasticity of concrete is usually higher than the long or short-term modulus and specialist advice should be sought to determine an appropriate value.

In areas of severe seismicity, the engineer must also consider the ductility of the structure overall which, in the case of very tall buildings, may be limited.

## 6.4 Diaphragm flexibility

It should be identified whether the floor diaphragm is to be considered as rigid. If not rigid, then the in-plane flexibility should be modelled using 2D elements or a series of 1D beam elements.

Care should also be taken to get a reasonable distribution of mass on plan, as this will affect the torsional response.

## 6.5 Further Information

For further information and guidance on seismic analysis and design, contact Arup Advanced Technology and Research.

# 7 Summary

The representation and simplification of any structure to allow analysis and design to be carried out is inevitably subjective with a wide range of options being available and often with equal validity. This Note has presented a range of such options and examined some of the issues surrounding various modelling methods in addition to considering the typical checks that are required.

Due to their complexity, some topics such as foundation-soil interaction, stiffening effects of the core walls on the foundations, differential shortening, etc., have been omitted from detailed consideration. Indeed, they warrant specific Guidance Notes in their own right. Specialist advice should be sought if such areas are of concern and be considered in subject specific technical reviews at an early stage in a given project.

# 8 References

## 8.1 Arup references

| Number | Description |
|---|---|
| 1999 NST 11a | Slender concrete walls and slabs subjected to in-plane normal and shear forces |
| 2000 NST 07 | Torsional stiffness of structures |
| 2002 NST 04 | Slender concrete outstand walls |
| 2003 NST 03 | Effects of eccentricity on the elastic critical load for buildings |
| 2004 NST 04 | Combining the properties of a number of stability elements |
| 2006 NST 01 | Modelling edge columns on slabs represented by 2D elements: Part 2 |
| 2007 NST 09 | Slender concrete outstand walls with moment restraint |
| 2014 NST 05 | Column loads through floors |
| 1997 SGN 01 | Good practice guide to calculations (SGN 1.2) |
| 1997 SGN 08 | Hand analysis of simple, linked, and coupled shear walls (SGN 4.2) |
| 2004 SGN 02 | Guidance note on secondary effects on concrete cores |
| 2005 SGN 01 | Structural modelling manual |
| RGN 1 | Project reviews guidance note |

The following presentations and general notes are referenced within this Note:

Modelling Cores – Structural Surgery Presentation – Ian Feltham & Andrew Fraser

Lessons from the design of the GLA building RC core – Andy Pye

## 8.2 Third party references

The following references are suggested as sources of further information:

Theory of elastic stability – Timoshenko

Roark's formulas for stress and strain – Young & Budynas

Design for movements in buildings – CIRIA TN107

Movement, restraint and cracking in concrete structures – Concrete Society TR67

Design of shear wall buildings – CIRIA R102

Council on Tall Buildings and Urban Habitat (http://www.ctbuh.org)

Stability of buildings Parts 1 and 2: General philosophy – IStructE 2014

Stability of buildings: Part 3: Shear walls – IStructE 2014

# Appendix A

## Details of GSA models

# A1 Details of GSA models

A number of GSA models are used to illustrate various aspects of core behaviour in Section 2.9 and brief details of these models and the associated hand calculations are provided for reference purposes.

## A1.1 C-shaped and square core

The models are 50m high with a story height of 5m.

Both models are based on a 10m x 10m centreline perimeter with the section thickness being 300mm.

The default GSA long-term concrete properties are used.

Rigid arms are either story height or half story height sections depending on their frequency and are modified as described in Section 2.9.3

All nodes at the base of each model are encastre.

A lateral load of 1000kN is applied to the top of the section at an eccentricity of 14.675m from the centroid of the C-section and 10m from the centroid of the box section.

## A1.2 Hand calculation of rotations

In Section 2.9, it is noted that a simple hand calculation was carried out to estimate rotation, θ, with the formulae being taken from Roark's Formulas for Stress and Strain:

For a C-shaped thin-walled section restrained against twisting and warping at the support:

$$\theta_{max} = \frac{T}{C_w E \beta^3}\left[\beta L - \mathrm{Tanh}(\beta L)\right]$$

where

$$\beta = \left(\frac{KG}{C_w E}\right)^{\frac{1}{2}}$$

For a thin-walled closed box:

$$\theta = \frac{TL}{KG}$$

where: T is the torsional load; $C_w$ is the warping constant; K is the torsional stiffness constant; L is the length; G is the shear modulus; E is Young's modulu

# Appendix B

## Simplifications for hand calculations

## B1 Introduction

Whilst useful to allow simple checking to be carried out, it will often be as easy to construct models that are slightly more complex by making use of the various software packages available or through tools such as the Arup Advanced Properties Spreadsheet, hence benefiting from increased accuracy.

However, understanding the background into the simplifying assumptions, and their limitations, is useful in the development of a wider understanding and hence is presented in this Appendix.

## B2 Theory

For reference, Table B1 from Notes on Structures 2003 NST 05 is reproduced below. For the examples considered, the relationships of interest are highlighted. However, it is important to understand that the expressions derived are specific to the assumed deflected shape as described below.

Table B1. Flexural and shear deflections, slopes and end rotations.

| L, flexural stiffness EI, shear stiffness $GA_s$ | Flexural | | Shear | | |
|---|---|---|---|---|---|
| | End deflection | End slope and rotation of cross-section | End deflection | End slope | End rotation of cross-section |
| M | $\frac{ML^2}{2EI}$ | $\frac{ML}{EI}$ | 0 | 0 | 0 |
| W | $\frac{WL^3}{3EI}$ | $\frac{WL^2}{2EI}$ | $\frac{WL}{GA_s}$ | $\frac{W}{GA_s}$ | 0 |
| w | $\frac{wL^4}{8EI}$ | $\frac{wL^3}{6EI}$ | $\frac{wL^2}{2GA_s}$ | $\frac{wL}{GA_s}$ | 0 |
| W | $\frac{WL^3}{12EI}$ | 0 | $\frac{WL}{GA_s}$ | $\frac{W}{GA_s}$ | 0 |
| | Mid-span deflection | End slope and rotation of cross-section | Mid-span deflection | End slope | End rotation of cross-section |
| W | $\frac{WL^3}{48EI}$ | $\frac{WL^2}{16EI}$ | $\frac{WL}{4GA_s}$ | $\frac{W}{2GA_s}$ | 0 |
| w | $\frac{5wL^4}{384EI}$ | $\frac{wL^3}{24EI}$ | $\frac{wL^2}{8GA_s}$ | $\frac{wL}{2GA_s}$ | 0 |
| M, M | $\frac{ML^2}{8EI}$ | $\frac{ML}{2EI}$ | 0 | 0 | 0 |
| M; L, R | $\frac{ML^2}{16EI}$ | L: $\frac{ML}{6EI}$ | 0 | 0 | $\frac{-M}{LGA_s}$ |
| | | R: $\frac{ML}{3EI}$ | | | $\frac{M}{LGA_s}$ |
| $M_0$, M; L, R; $M_0 = \frac{M(1-6k)}{2(1+3k)}$; $k = \frac{EI}{L^2GA_s}$ | $\frac{ML^2}{32EI}$ | L: 0 | $\frac{9M}{32(1+3k)GA_s}$ | $\frac{3M}{2(1+3k)LGA_s}$ | 0 |
| | | R: $\frac{ML}{4EI}$ | | $\frac{3M}{4(1+3k)LGA_s}$ | $\frac{9M}{4(1+3k)LGA_s}$ |

As shown on the following pages, the above relationship is used to determine an equivalent stiffness for the lintel assuming it is a solid section of wall smeared over the height of the opening and the appropriate shear area factor.

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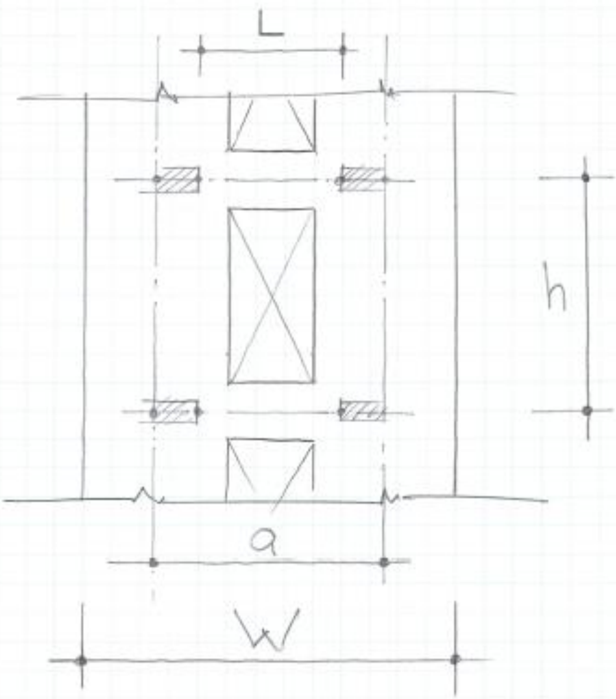
| ARUP — Calculation sheet | Job No. | Sheet No. | Rev. |
|---|---|---|---|
| | Member/Location | | |
| Job Title | Drg. Ref. | | |
| | Made by ASF | Date Oct.'08 | Chd. |

LAYOUT



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ARUP | Calculation sheet

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Job Title

Drg. Ref.

Made by ABF Date Oct. '08 Chd.

# EQUIVALENT WALL THICKNESS.

- DETERMINE THE EQUIVALENT THICKNESS OF WALL TO REPRESENT THE LINTEL:

$$\Delta_S = \Delta_B$$

$$\Rightarrow \frac{WL^3}{12EI_b} = \frac{W/L}{GA_s} \quad (\text{REF. 2003 NBT 5b})$$

$$\Rightarrow A_s = \frac{12EI_b}{GL^2}$$

$I_b$ = LINTEL I

$$\Rightarrow t_{eff} = \frac{12EI_b}{GL^2h} \cdot \frac{6}{5}$$

USE $t_{eff}$ TO DETERMINE J:

$$J = \frac{1}{3}\Sigma\,(d_s \cdot t^3)$$ THIN WALLED <u>OPEN</u> SECTION.

<u>OR</u>

$$J = \frac{4A^2}{\Sigma\left(\frac{d_s}{t}\right)}$$ THIN WALLED <u>CLOSED</u> SECTION,

OAP20

| ARUP — Calculation sheet | Job No. | Sheet No. | Rev. |
| --- | --- | --- | --- |
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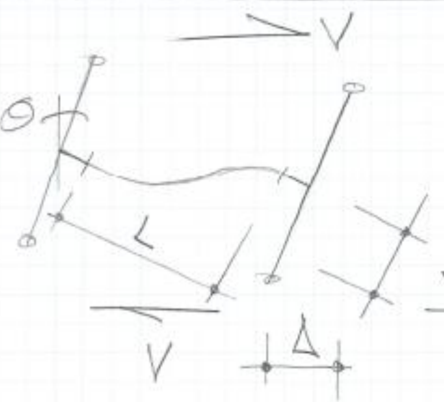
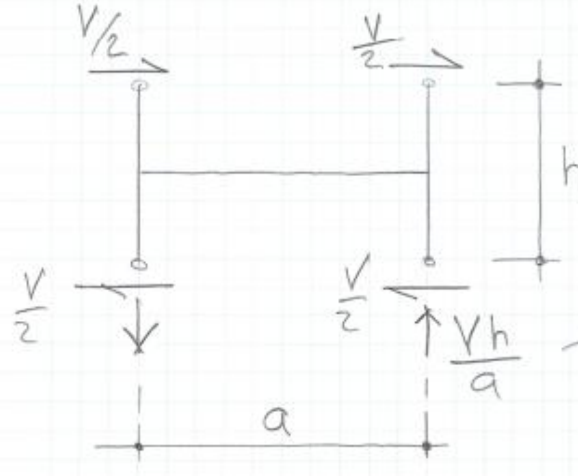
| | Made by [illegible] | Date Oct. '98 | Chd. |

# PANEL DEFORMATION.




$$\frac{wL^3}{12EI} = \underbrace{\frac{Vh}{a}}\cdot\frac{L^3}{12EI}$$





$$\theta = \left(\frac{Vh}{a}\cdot\frac{L^3}{12EI}\right)\Big/a = \frac{Vh}{a^2}\cdot\frac{L^3}{12EI}$$

$$\Delta = \theta h = \frac{Vh^2}{a^2}\cdot\frac{L^3}{12EI} = \frac{Vh}{GA_s}$$

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$$\Rightarrow A_S = \frac{12 EI a^2}{GL^3 . h} \qquad G = \frac{E}{2(1+\nu)}$$

$$= \frac{2.4 bd^3 a^2}{L^3 . h}$$

∴ SHEAR AREA FACTOR, $K = \frac{\text{SHEAR AREA}}{\text{TOTAL AREA}}$

$$= \frac{A_s}{W . b}$$

WHERE W = TOTAL LENGTH OF WALL.

$$\therefore K = 2.4 \frac{a^2}{W.h} \cdot \left(\frac{d}{L}\right)^3$$

To summarise the calculations on the previous pages, the effective thickness, $t_{eff}$ , may be used to determine the torsion constant, J, as follows:

$t_{eff} = 12EI_b \;/\; GL^2\,h$ where $I_b$ is the second moment of area of the lintel beam

Thus, for a thin walled open section: $J = 1/3\ \Sigma(d_s\, t^3)$

For a thin walled closed section: $J = 4A^2 \,/\, \Sigma(d_s/t)$

Considering the penetrations through a wall to be vertical, i.e. height > width, and the shear stiffness of the wall governed by the lintels, the shear area factor $K = 2.4\ a^2 /(Wh)\ (d/L)^3$.

See the sketches above for an explanation of the notation used.

## B2.1 Circular core

The simplification presented above may be applied to rectangular cores. Occasionally the question also arises as to the treatment of a pierced tubular core.

The following text is reproduced (with slight re-formatting) from the SSN Structural Forum where Ian Feltham addressed this issue:

"The shear area of a tube without openings is equal to half of its cross sectional area, so the shear stiffness, $GA_s$ , is given by $G.\pi.r.t$, where 'G' is the shear modulus of the concrete wall, which can be taken as E/2.4, 'r' is the radius to the centreline of the wall, and 't' its thickness.

Isolated openings will have little impact on the shear stiffness but if there are lines of openings above each other, (the engineer) will have to estimate an effective thickness of solid wall that would give the same shear deflection as the lintels. For a storey height 'h' and lintels of effective span 'L' and flexural stiffness EI, the effective thickness, $t_{eff}$ of the wall is approximately $12EI/(GhL^2)$. It would be conservative to use $G.\pi.r.t_{eff}$ for $GA_s$ , otherwise use the Advanced Section Property spreadsheet to estimate the shear stiffness in specific directions.

The torsional stiffness, GJ, is given by $4G.\pi^2.r^4 \,/\, \Sigma\{\text{length/thickness}\}$, where length is the length around the perimeter on the centreline of the wall of thickness, or equivalent thickness, t. So with no openings, $GJ = 2G.\pi.r^3t$ and with one line of openings,
$GJ = 4G.\pi^2.r^4 \,/\, \Sigma\{L/teff + (2\pi.r\text{-}L)/t\}$."

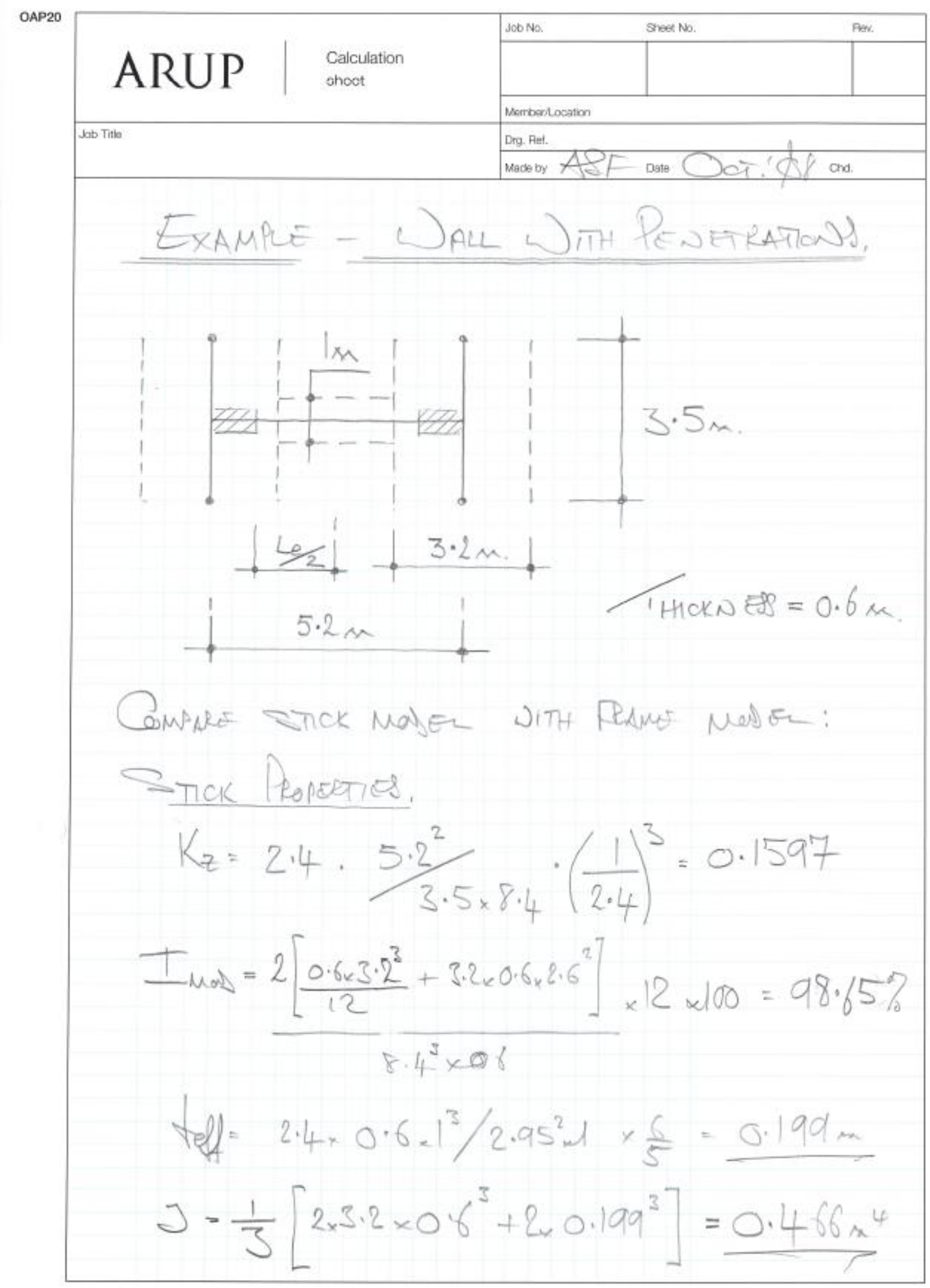
Reference:
http://forums.intranet.arup.com/Thread.cfm?CFApp=26&&message_id=76573&_#Message76573

# B3 Example

An example of a pierced wall is presented to illustrate the simplification process.

## B3.1 Example – wall with penetrations

As a simple example, a pierced wall is examined using a single stick and a sub-frame type model.  The properties for the single stick are as follows: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EXAMPLE – WALL WITH PENETRATIONS.



COMPARE STICK MODEL WITH FRAME MODEL:

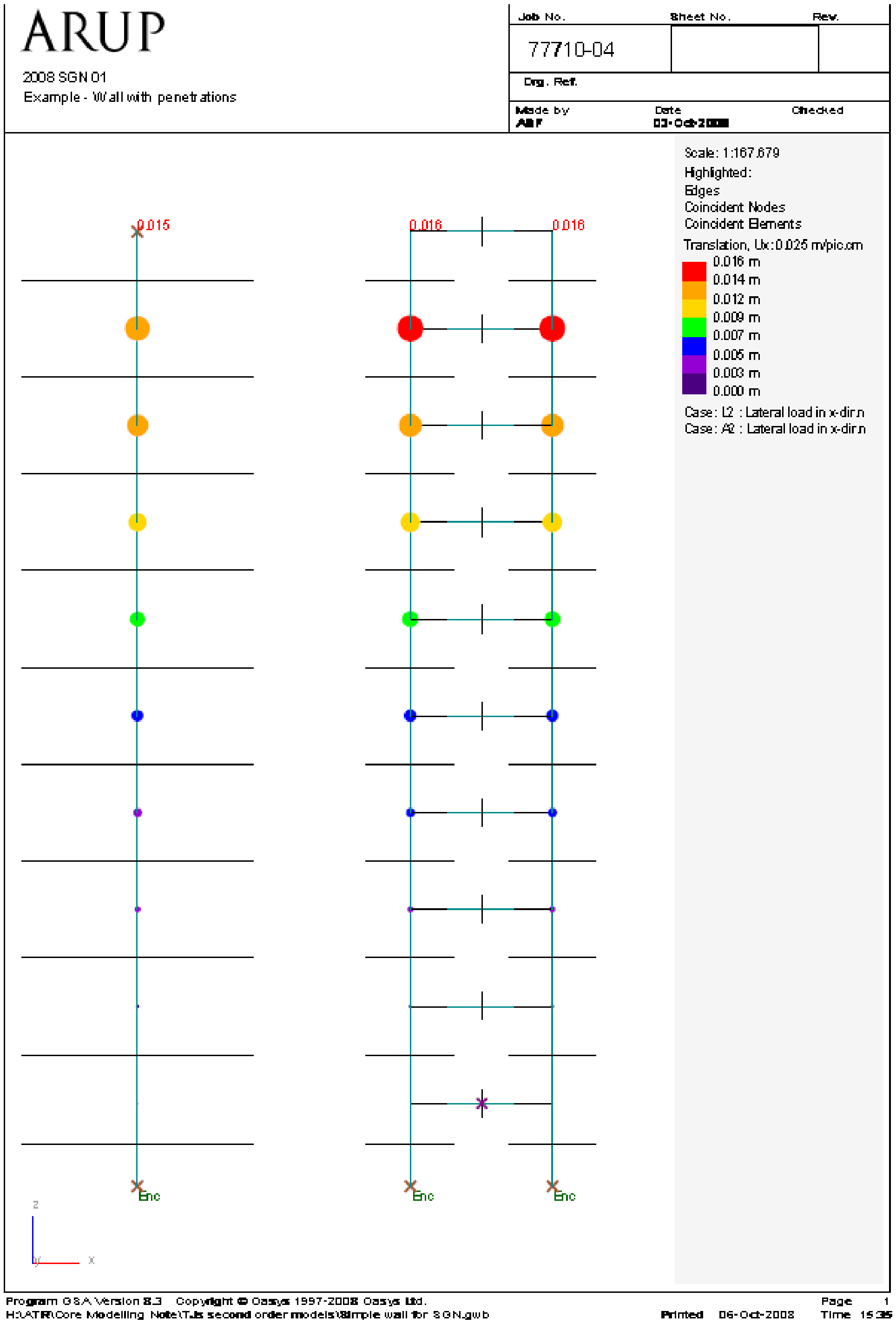
STICK PROPERTIES.

$$K_z = 2.4 \cdot \frac{5.2^2}{3.5 \times 8.4} \cdot \left(\frac{1}{2.4}\right)^3 = 0.1597$$

$$I_{MOD} = \frac{2\left[\frac{0.6 \times 3.2^3}{12} + 3.2 \times 0.6 \times 2.6^2\right]}{8.4^3 \times 0.6} \times 12 \times 100 = 98.65\%$$

$$t_{eff} = 2.4 \times 0.6 \times 1^3 / 2.95^2 \times 1 \times \frac{6}{5} = 0.199\,m$$

$$J = \frac{1}{3}\left[2 \times 3.2 \times 0.6^3 + 2 \times 0.199^3\right] = 0.466\,m^4$$




Although there is good agreement between models, the assumptions made in the simplification to a single stick must be remembered, namely: the walls are stiff compared to the lintels, vertical penetrations are assumed, and the assumption of embedment depth in the frame model is correct. Also, remember that these models illustrate reasonable agreement when compared against each other, which is not in itself a guarantee that the model is an accurate representation of the physical reality.