Shear Wall Analysis – New Modelling, Same Answers.

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Introduction

Engineers now routinely have access to highly capable 3D analysis packages often including the ability to use finite elements ("FE" shell elements). More and more structures are being analysed in 3D, and as suppliers of such software we at CSC are certainly being faced with increasingly frequently asked questions relating to the modelling of shear and core wall systems using shell elements within the context of a 3D model.

It seems that while there are many texts dealing with the theoretical aspects of finite elements and FE analysis, there are almost none that provide practical advice to engineers wishing to make use of this technology. Personally I had not identified any useful text until the recently published "Finite Element Design of Concrete Structures" (ref 1) which I would thoroughly recommend to anyone seeking a more pragmatic engineering view of the issues together with a very realistic review of the options which should be considered.

In this Technical Note we will show that shell elements can be used but that they do not necessarily yield different answers to traditional idealisations of walls.

Analytical Idealisations

At this point it is appropriate to re-quote a quote in a previous Technical Note titled "Structural Engineering Modelling and Analysis" (ref 2):

"Engineering (and some may think FE practice also) is the art of modelling materials we do not wholly understand, into shapes we cannot precisely analyse, so as to withstand forces we cannot properly assess, this in such a way the public and (hopefully) the customer has no reason to suspect the extent of our ignorance."

Before looking at the analytical idealisations we should consider the above in the context of concrete shear walls.

If deflections and the distribution of forces are important to you then you need to get all aspects of your model right, this means:

- Accurate material properties for each member.
- Accurate section properties for each member.
- A good arrangement of members to idealise the overall physical geometry.

BS8110 indicates a potentially broad range of properties for concrete of any given grade (For example, the short term Young's modulus for C40 grade concrete is suggested as being somewhere between 22 and 34kN/mm^2.) This then needs to be adjusted to allow for load duration (and perhaps other factors as well). The gross section properties of elements may need to be adjusted to allow for cracking. Therefore there is a good deal of judgement involved in the selection of the section and material properties, this directly affects results and must be borne in mind when debating the intricacies and relative merits of alternative idealised models: sensitivity studies may be appropriate.

If deflection is not a concern and the aim is to produce design forces then BS8110 logically suggests that consistent section and material properties should be used in analysis. This is the approach taken in this technical note, so although deflections are used as a primary basis of comparison, it is only for demonstrating that the models (or idealisations) are equal to each other.

Example 1 – A Simple Single Wall Panel

To be convinced that complex core wall systems can be satisfactorily idealised and analysed using simple beam element models we will make a series of comparisons starting with a very simple example. Consider a wall 35m high (10 storeys at 3.5m), 6m long and 200mm thick. An axial load of 1000kN and a lateral load of 100kN are applied at the top of the wall.

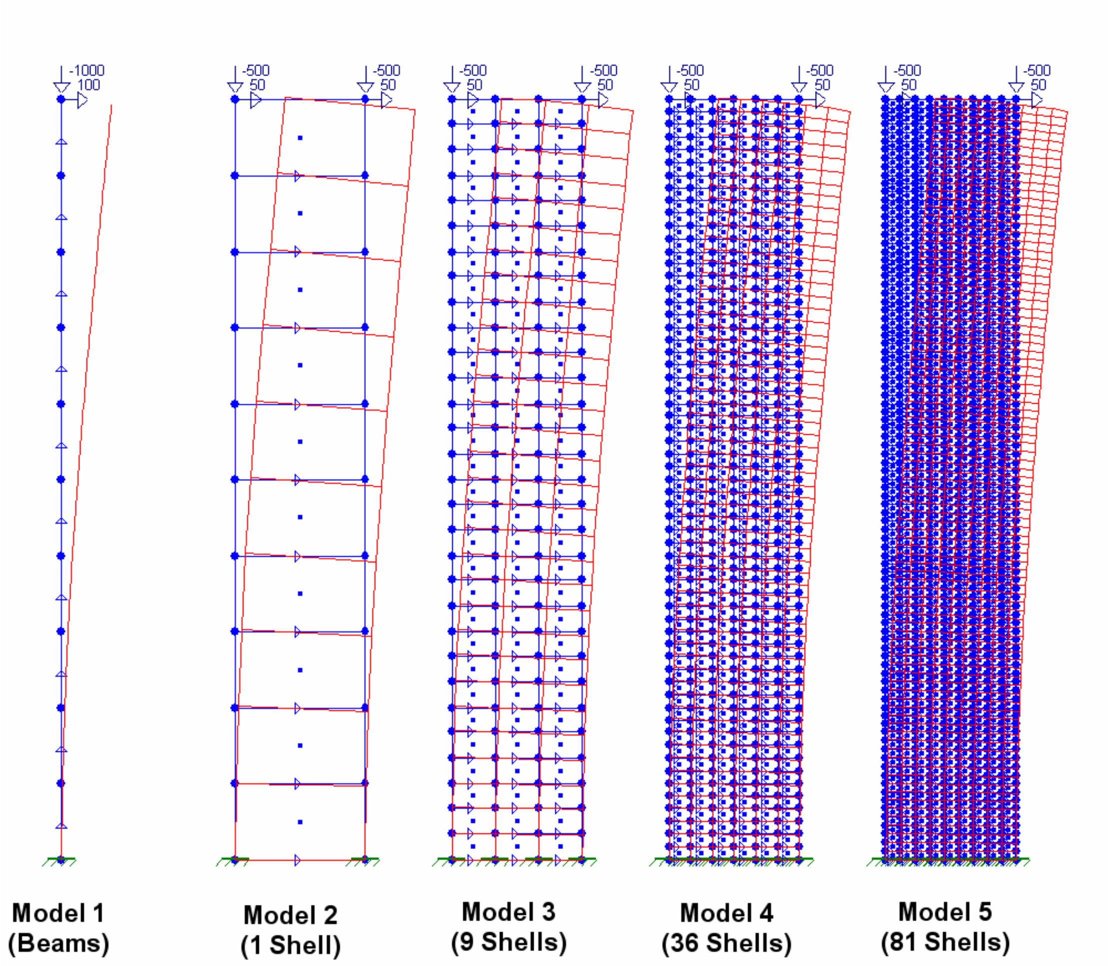

Fig 1 –Analysis Models for a Simple Wall Panel viewed in S-Frame

Model 1 is a simple beam model, the section properties of the beam are simply the properties of a 200mm wide and 6000mm deep section. Set beside this are a series of increasingly finely meshed versions of the model using shell elements. The quoted number of shells refers to the number of elements used on a floor-to-floor basis. The lateral deflections for each of these models are given in Table 1.

	Beam	1 Shell	9 Shell	36 Shell	81 Shell
Lateral Deflection (mm)	31.2	29.7	31.0	31.2	31.2

Table 1 – Lateral Deflections for Analysis Models of a Simple Wall Panel (see fig 1)

The points to be drawn from this simple comparison are:

1. When considering global effects such as building sway the beam element model gives the same result as the most finely meshed shell models. You should not enter the world of shell elements expecting different answers or even a new level of accuracy.
2. The results for the shell models vary slightly as the meshing is increased. This is the first indication of mesh sensitivity and it is worth noting that different software packages using different shell element types and formulations will display differing degrees of mesh sensitivity. When you use shells it is your responsibility to check the results you are going to rely on are not sensitive to increased meshing.
3. In this example an engineer will probably be happy to accept the result given by the "9 Shell" model and this does conform to a widely held view that shear walls can be adequately modelled with shells sized at 1/3 or 1/4 of the floor-to-floor height.
4. As well as agreeing on the deflection estimation, the beam model also produces more readily usable design information, axial forces, shear forces and bending moments for the panel as a whole are readily available. When shells are used all sorts of contour diagrams are available, but if you are going to want to know the design forces applicable to an entire wall panel at some stage of design or checking then the shell nodal results need to be re-integrated along desired cut lines. (Some software provides features to do this for you.)

Example 2 – Single Wall Panel with Openings

Some engineers may be surprised at the accuracy of the good old fashioned beam model in the previous example but still feel that the only way to take account of openings in walls is to resort to the use of shells. In this example we will look at the same wall panel, with the same loads applied, but with significant (door) openings cut out of the wall at every floor level. Firstly we will examine the results obtained from a series of increasingly finely meshed shell models as shown in Figure 2.

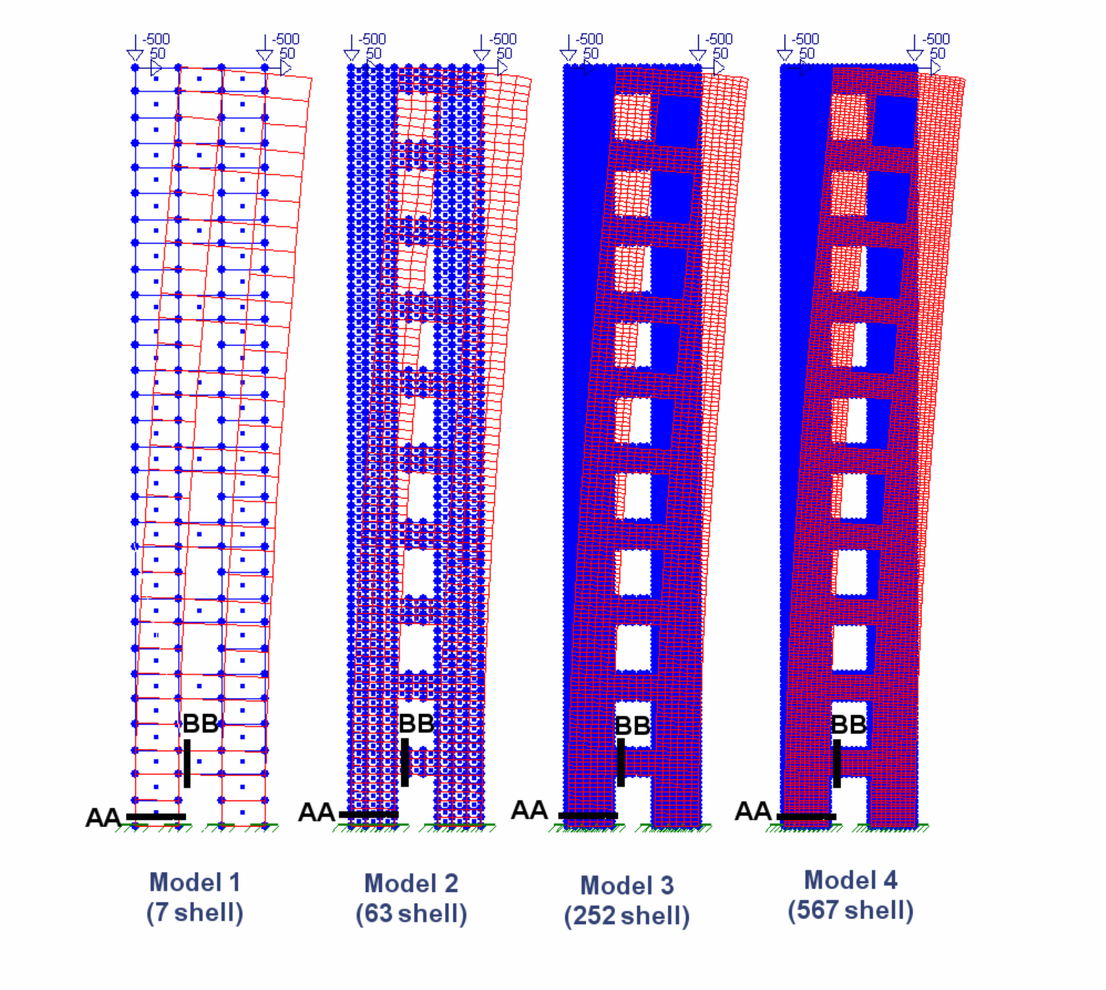

Fig 2 – Shell Models for a Wall Panel with Openings viewed in S-Frame

Table 2 indicates results for these models this time considering sway at the top of the panel and re-integrated section forces reported by S-Frame at sections A-A and B-B.

Model	Sway (mm)	Section A-A			Section B-B		
		Axial (kN)	Shear (kN)	Moment (kNm)	Axial (kN)	Shear (kN)	Moment (kNm)
7 Shell	33.6	269	35	194	17	69	66
63 Shell	36.0	256	40	225	12	60	58
252 Shell	36.6	253	41	232	11	58	57
567 Shell	36.8	252	41	235	10	58	56

Table 2 – Comparison of results for Shell Models of a Wall with Openings (see fig 2)

In this case the model using 63 shells per floor would probably be considered reasonable (the results vary by less than 5% compared with those of the most finely meshed model). Note that rather than thinking in terms of having 3 or 4 shells between floors this suggests we should be looking for at least 3 or 4 shells across the width of each section that is being meshed, including the coupling beam at section B-B.

We can now make comparisons with a series of models that utilise beam idealisations. Figure 3 shows 4 variations of this type of model that will be considered.

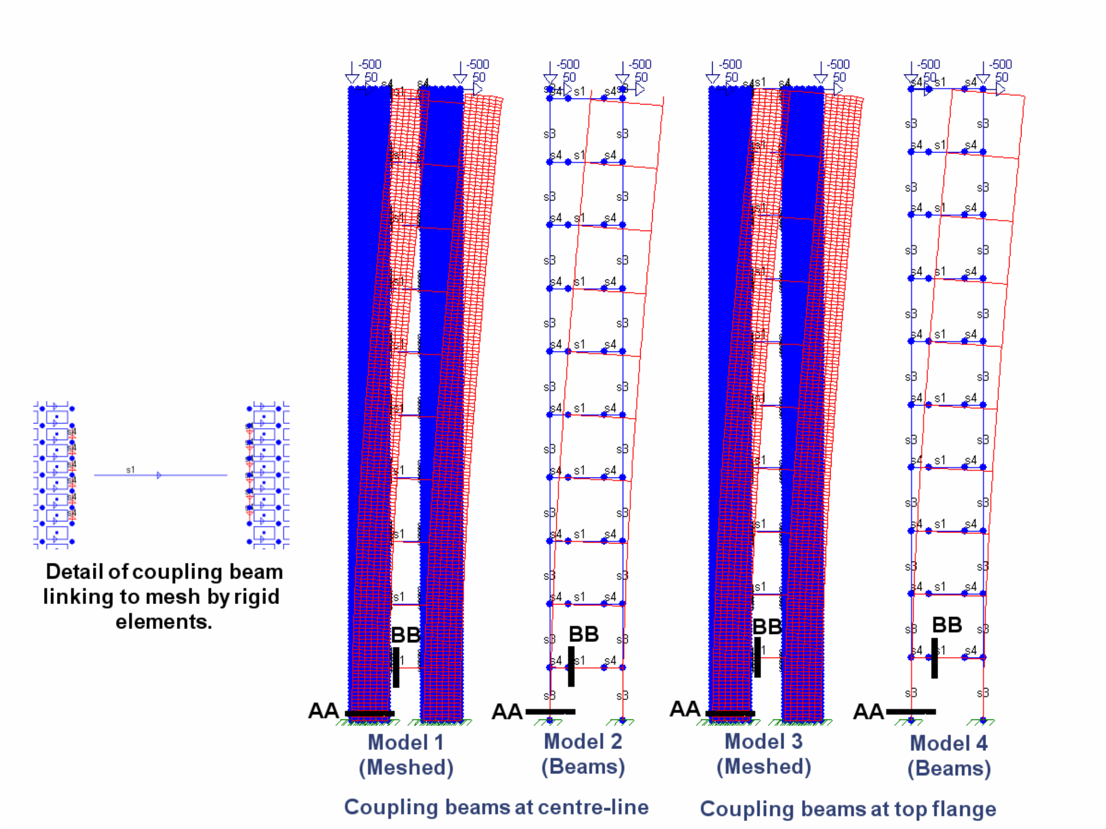

Fig 3 – Optional Idealisations of a Wall Panel with Openings

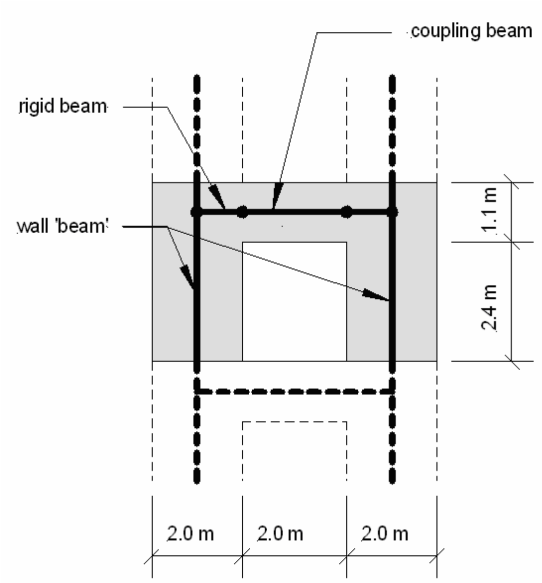

Fig 4 – Detail of typical beam idealisation for Model 2

Model	Sway (mm)	Section A-A			Section B-B		
		Axial (kN)	Shear (kN)	Moment (kNm)	Axial (kN)	Shear (kN)	Moment (kNm)
Model 1	36.4	256	37	229	14	57	55
Model 2	36.2	256	50	239	0	61	61
Model 3	36.5	255	40	234	11	57	53
Model 4	36.2	251	50	249	0	67	67

Table 3 – Comparison of results for alternative models of a wall with Openings (see fig 3)

Some discussion of each of these models is appropriate:

MODEL 1 – Meshed wall panels to each side of the opening with a beam element used for the coupling beam between the panels. The coupling beam has the properties of a rectangular section 200mm wide and 1100mm deep (the full depth of concrete between the openings) and it is positioned on its physical centreline. The results for this model shown in table 3 agree well with the results of the fully meshed versions of the models in table 2. Note that at each end of the coupling beam rigid elements are extended up and down the face of the meshed wall to the full physical depth of the coupling beam as indicated in the detail shown in Fig 3. If this is not done the stiffness of the connection between the beam and the shells becomes dependent on the shell size and will tend not to be sufficiently stiff. As an example, if these rigid elements are deleted from this model the deflection of the wall increases from 36.4 to 118mm and the moment generated at section B-B reduces substantially. While this model demonstrates that good results can be achieved, it also underlines a potential pitfall of mixing beams and shells in analysis models.

MODEL 2 – Beam elements are used throughout this model. Referring to figs 3 and 4, the sections used are:

Walls (S3) – Modelled as 2m deep, 200mm wide sections positioned on their physical centrelines.

Coupling Beams (S1) – The coupling beams are the same as for model 1 but only extend to the face of the wall.

Rigid Beam (S4) – Connects the end of the coupling beam to the centreline of the wall.

The results in table 3 indicate that this simple model also compares well to the meshed models in table 2. The only exception is that the shell models indicate an axial load within the coupling beam while the beam models do not. This is explicable and relates entirely to the vertical load in each meshed panel – when a meshed panel is compressed vertically the sides of the panel expand laterally (as dictated by Poisson's ratio) and these opposing lateral expansions are resisted by the coupling beam. This effect is negligible and would normally be ignored therefore this difference is not considered significant.

The main concern expressed by engineers considering this sort of of shear wall modelling relates to the properties of the rigid beam – how rigid should it be and will the wall not become unreasonably stiff if it is made too rigid? In practice you will tend to find that as you stiffen this beam the results converge on those of a meshed model and then become relatively insensitive to further increases in the stiffness of the rigid beam. If you attempt to make these elements infinitely stiff you may introduce numerical problems in the analysis, so ideally they need to be made “relatively” rigid. It is also noted that while the rigid beams should be relatively rigid in the plane of the wall, they should not be rigid out of plane and a little more caution is required where these rigid beams interact with each other as part of a core wall. Some suggestions on the selection of rigid beam properties are given in the next example where a core wall is considered.

MODELS 3 & 4 – These are essentially repeats of models 1 and 2 but the coupling beams and rigid arms are lifted and idealised at the top of the coupling beam rather than on its centreline. Clearly this is a less accurate idealisation but it is often much more convenient to model everything in one floor at a common level and top of structure is often chosen for this purpose. It is interesting to see that this common idealisation has very little impact on the results in this example.

Example 3 – A Simple Core Wall System

The methods shown to this point will extend very successfully to the analysis of 3-dimensional core walls. We will consider here a simple C-shaped core slightly offset from the centre of a simple 10 storey building – see fig 5.

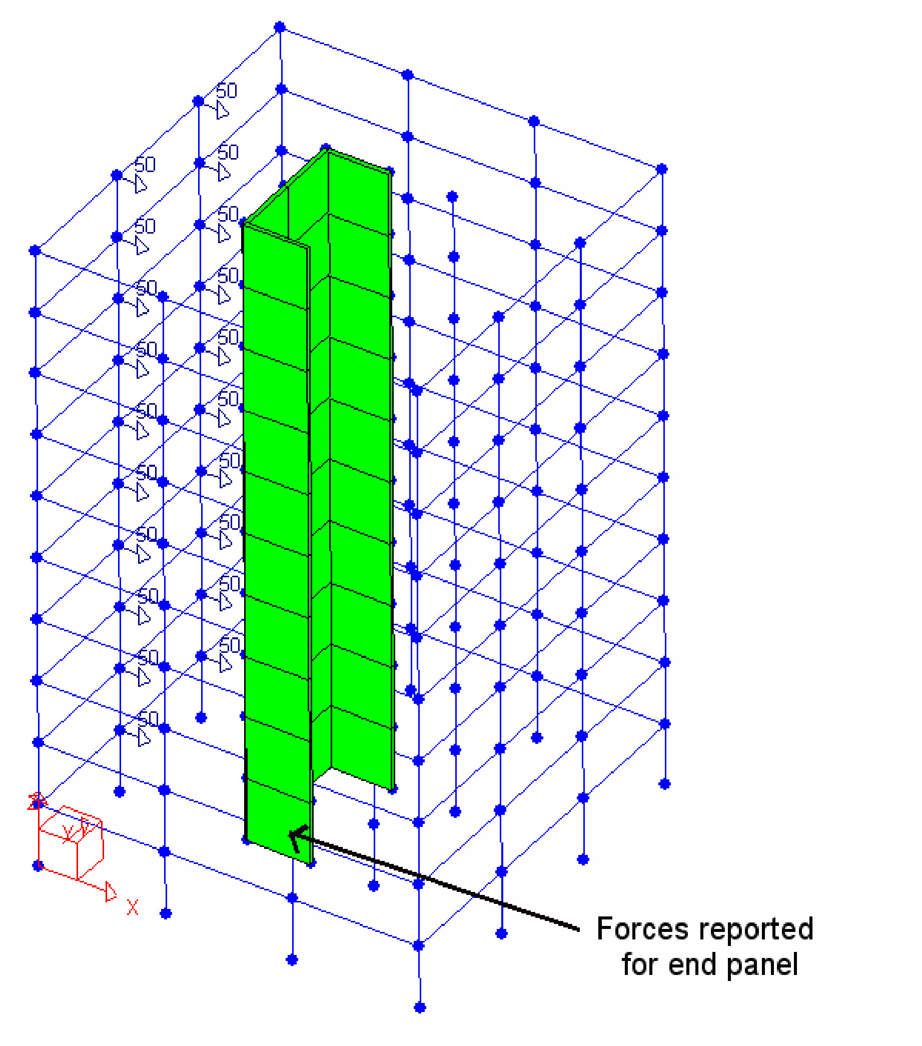

Fig 5 – Simple Building with C-Shaped Core Wall viewed in S-Frame

The columns around the core are held in position by floor diaphragm action. In this model two load cases are considered.

Sway in X – refer to figure 5 – loads are applied in the X direction (parallel to the flanges of the core).

Sway in Y – similar loads are applied in the Y direction – since the effective line of action of this load is eccentric to the centre of resistance provided by the core we expect this case to result in twisting at each floor level.

As before we can look at the model using a series of increasingly finely meshed versions of the core and compare these results with those given by a beam idealisation. Comparisons of deflections and panel design forces are summarised in Table 4.

Model	Loads applied in X				Loads applied in Y			
	Max Sway in X (mm)	End Panel Forces			Max Sway in X (mm)	End Panel Forces		
		Axial kN	Moment kNm	Shear kN		Axial kN	Moment kNm	Shear kN
Meshed model – 1 shell over floor-to-floor height.	127.2	-2883	3886	527	+/- 373.7	39	5040	890
Meshed model – 3 shells over floor-to-floor height.	130.7	-3030	3659	520	+/- 381.6	-49	4803	843
Meshed model – 9 shells over floor-to-floor height.	131.4	-3061	3609	508	+/- 383.5	-74	4743	810
Beam Model	132.6	-2963	3763	525	+/- 381.9	33	4892	848

Table 4 – Comparison of results for alternative models of a Core Wall (see fig 5)

Once again the beam model is in excellent agreement. With loads applied in the X direction all the models sway without twisting. When an eccentric load is applied in the Y direction all the models sway and twist. As expected the twist is symmetrical as is indicated by the +/- values of sway in X reported in table 4.

The beam model is shown in outline in figure 6. Each panel of the C shape core is modelled using a vertical beam element defined at its centre. These 3 lines of vertical beam elements are then linked together by rigid arms (rigid beams) at each floor level. It is important that these rigid arms are given properties that are relatively rigid in the plane of each wall panel but not out of plane. To achieve this, a reasonable starting point is to assume that the rigid arm properties are based on a section with depth equal to the floor-to-floor height (or the vertical spacing of the rigid arms). These properties might then be adjusted as follows:

Ix – Torsion Constant (out of plane effects) – reduce significantly, say by factor of 10

Iy – In Plane Stiffness – increase significantly, say by factor of 10 or 100

Iz – Out of Plane Stiffness – no adjustment.

Ax – Gross Area – no adjustment

Ay – Out of Plane Shear Area – no adjustment

Az – In Plane Shear Area – set to zero to eliminate in-plane shear deformation.

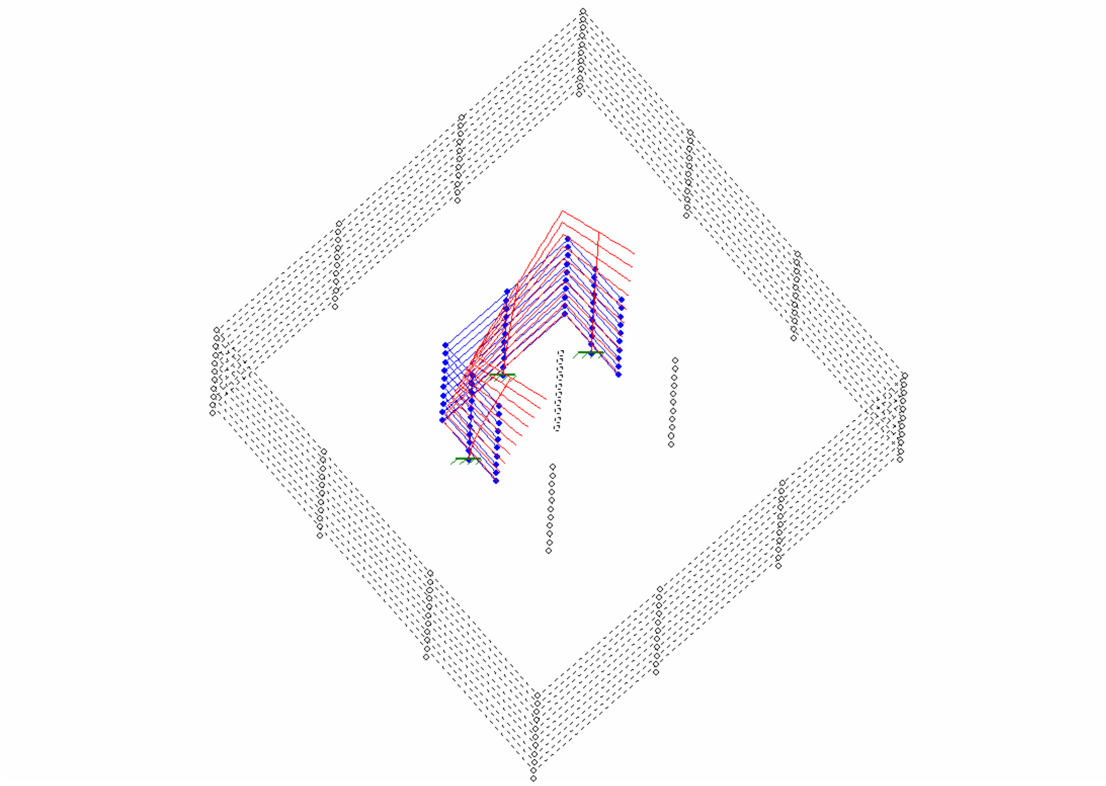

Fig 6 – Core Wall Idealised with beam elements and rigid arms also showing deflection and twist due to eccentric lateral loading.

Concluding Notes

Clearly the ultimate aim is to design and construct a building, analysis may only be a small part of that process. The quality and detail of the analysis work should be in some way proportionate to the anticipated design requirements and challenges. Bear in mind that just 25 years ago very few engineers had access to any sort of computing capability.

This technical note focuses on analysis model idealisations. Regardless of the model used the input relating to section and material properties will have a direct effect on the results. When dealing with concrete frames and shear walls the estimation of such properties dictates that all resulting deflections and forces should be regarded as best estimates and engineers should consider the use of alternative runs to assess sensitivity to design assumptions.

For a low rise buildings simple idealisations, or even hand calculations, are still appropriate. You should seriously question whether any sort of FE analysis of the walls in a low-rise building is appropriate and cost effective.

Idealisations using beam elements can be shown to extend effectively into all sorts of complex geometries. In many cases it can take longer to construct these models however there is certainly an advantage in that the forces reported for the beam elements are more readily understood and usable than many of the complex contour diagrams that can be displayed for shell models. Arguably there is another hidden advantage in the use of beam idealisations for shear walls. While deciding on and creating the idealised model you tend to develop a feel for the structure and an expectation of its response to loading. If it does not respond in the way you expect you will start to investigate. When working with shells this sort of intuitive feel is not as readily developed. Regardless of which way you have idealised the structure, if you have doubts about the results the best thing you can do is model it another way and compare.

You should not think that the world of shell elements offers a new level of accuracy – in many cases it might better be regarded as a new way to get the same answers, or perhaps more worryingly as a new way to make some new mistakes?

References

1. Finite Element Design of Concrete Structures, by G.A. Rombach.
   Thomas Telford, 2004
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   The Structural Engineer – 3rd Feb 2004