CADS A3D MAX

How to model shear walls

Modelling shear walls in A3D MAX

Introduction and synopsis

This paper explains how to model shear walls in A3D MAX using the `wide column – rigid arm' sub-frame described in section 3.5 of CIRIA Report 102 *Design of shear wall buildings* by A.W. Irwin[1]. Besides modelling shear wall behaviour in the structure, the results can be used for designing the walls themselves and their foundations.

Scope

Shear walls are here defined as loadbearing walls of reinforced concrete, masonry or composite steel plate and concrete which provide the principal resistance to lateral loads due to wind or seismic effects on the building structure. Shear walls may support all of the vertical loads as in crosswall and cellular construction or may provide bracing to a structure which is consists mainly of beam and column frames in steelwork or reinforced concrete.
This paper does not consider `infilled frames' whereby bracing is provided by masonry panel walls built into a steel or concrete frame. This form of construction can be modelled using equivalent `compression-only' diagonals and may be subject of a separate paper.

General concept

Each plane wall length is modelled as a rectangular section column member of the same storey height positioned at mid length of the wall. The section depth is equal to the wall length and the width is equal to the wall thickness. Whilst beams and columns are normally modelled with sufficient accuracy using 1-D bar elements, this is not adequate for walls and so the extent of the wall panel is modelled by two rigid arms at floor level extending from the centroid to the ends of the wall. The rigid arms can support direct distributed loads from floor panels and point loads from beams. The tee-shaped sub-frame so formed represents the comparatively rigid wall panel in similar fashion to the assembly of 2D plate elements used in alternative analysis methods but permits much simpler post-processing for design of the wall and foundations because the analysis output for the wall is a simple set of forces and moments acting on the whole panel. Wall sub frames can be joined using common nodes and coupling beams as appropriate to model 2D coupled walls and 3D core assemblies as described below.

Single plane shear wall

Fig 1 (below) shows an elevation of a four storey single plane shear wall with attached cantilever beams. Fig 2 shows the line elements used to model this part of the structure (the `stick model'). Vertical members numbered 277-280 are the wide column member type defined in the Member type dialog as shown in fig 3. Horizontal members 281 to 288 are the `rigid arms' defining the extent of the wall panel. Strictly speaking they should be defined as a member type with very large major axis inertia and very small minor axis inertia and torsion constant. However it is convenient to use the rectangular RC section type with depth say 0.75 x storey height and width equal to the wall thickness. This produces an acceptable 3D rendered graphic and doesn't significantly affect the numerical results. Horizontal members 289 to 296 are `real' RC beams which in this case cantilever from the ends of the wall to support the corners of the floor slabs. The subframe was created by first defining the member types in the Member type dialog, then defining the nodes (joints) of the subframe by co-ordinates and finally using the Quick Member tool to join up the nodes with members of the relevant type. Other methods can be used to the same effect.

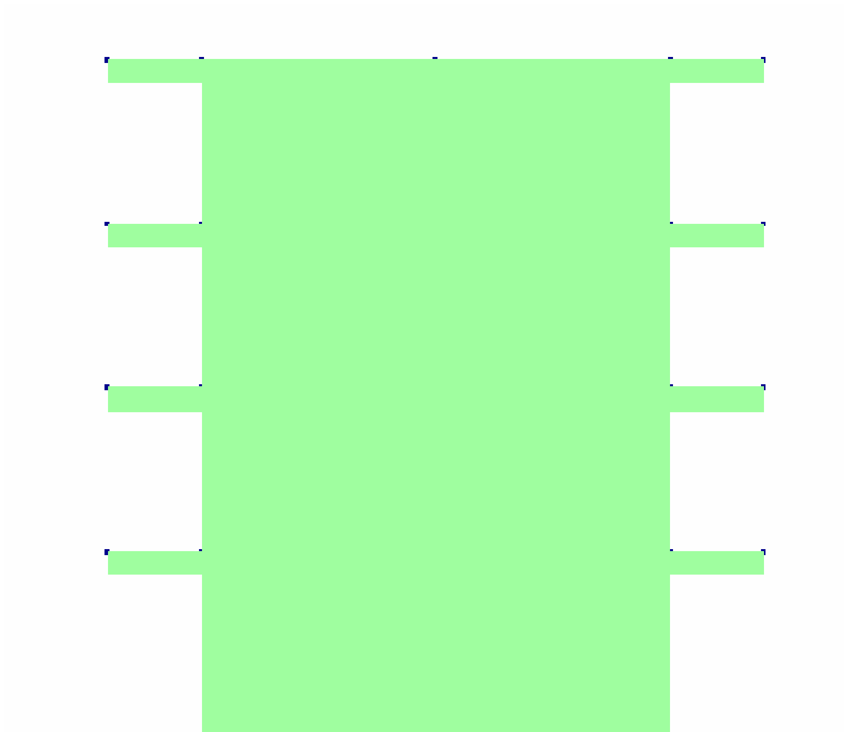


Fig 1: Elevation of four storey single plane shear wall with attached cantilever beams.

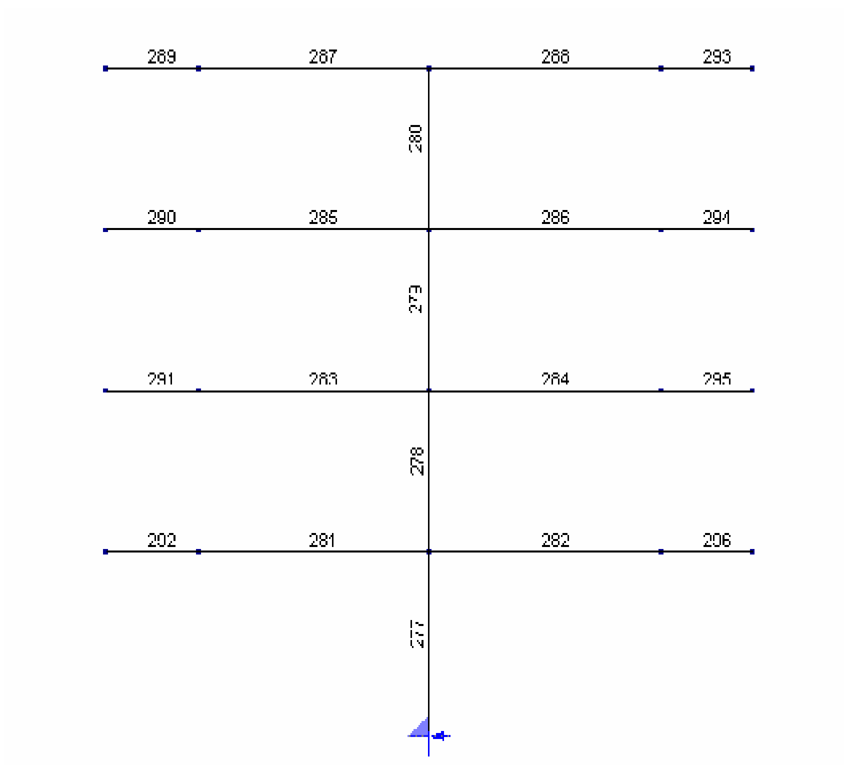


Fig 2: Stick model of single plane shear wall and cantilever beams.

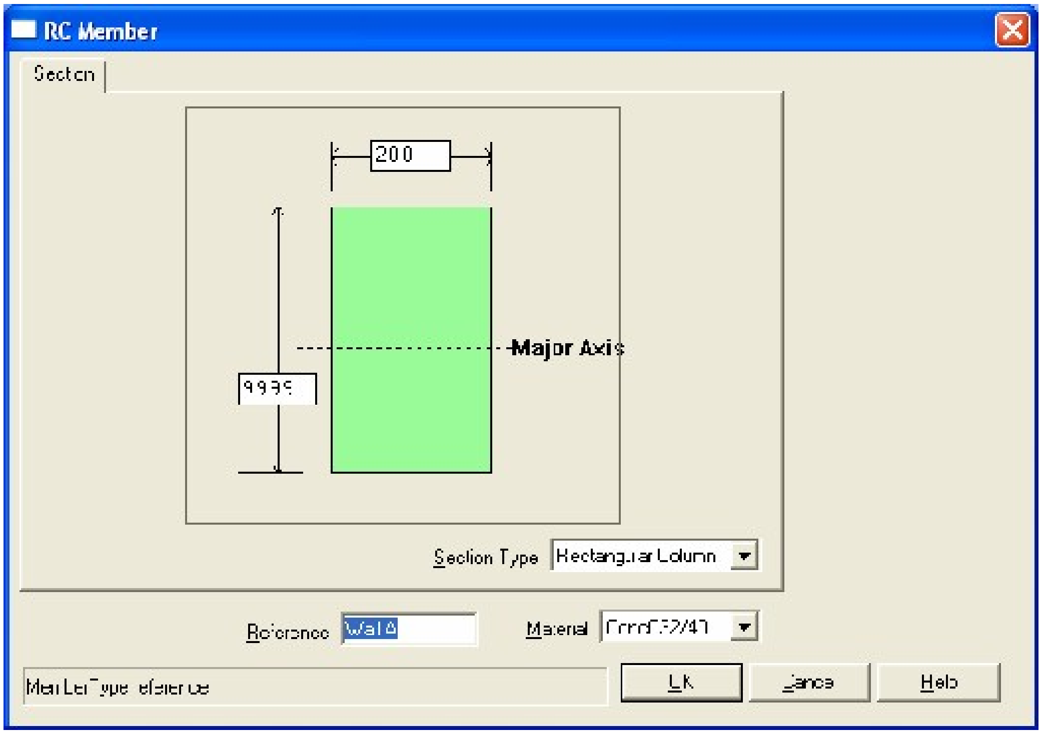


Fig 3: The vertical member type representing the wall is defined as a rectangular column.

Two shear walls coupled in plane

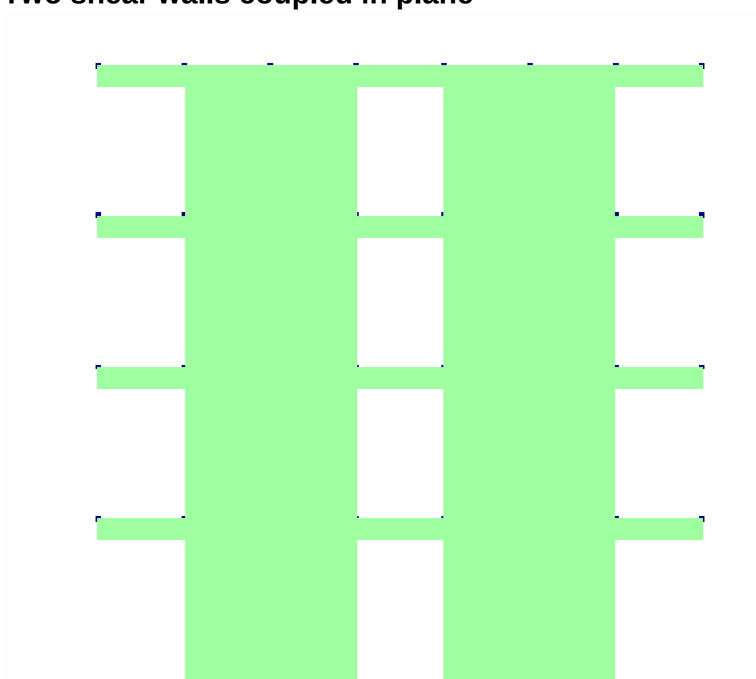


Fig 4: Elevation of two plane coupled shear walls with attached cantilever beams.

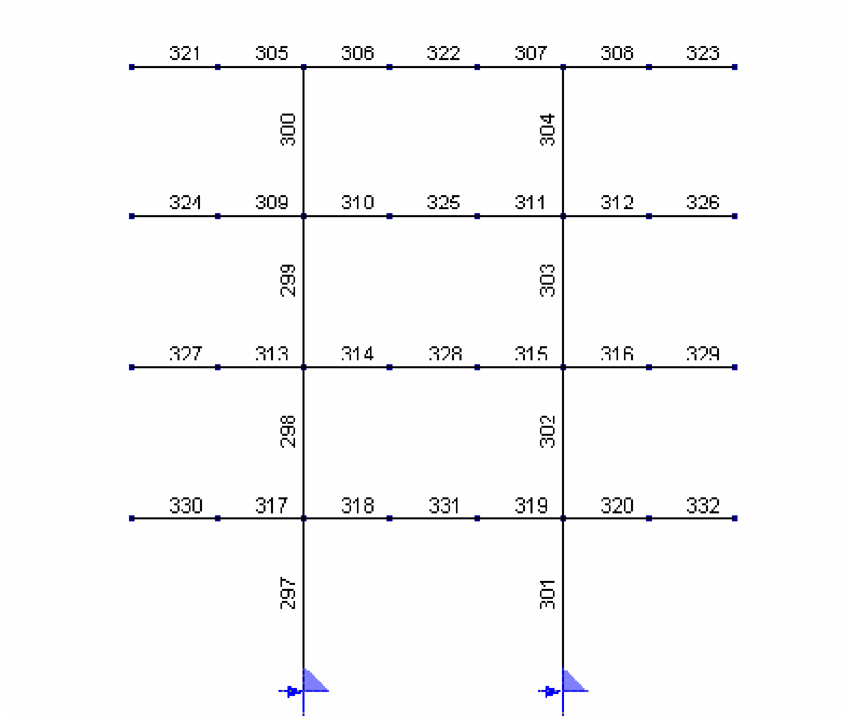


Fig 5: Stick model of two coupled shear walls

Figs 5 and 6 show the solid and stick models of a pair of coupled shear walls.
Members 297 – 304 are vertical wide column members positioned at the wall centrelines and modelling the principal flexural action of the walls.
Members 305 - 320 are the `rigid arms' positioned at or just below floor level defining the wall lengths and collecting floor loads.
The cantilevers at the ends support the corner edge of the floors as in the single wall example and are not part of the shear wall subframe.
Members 322, 325, 328 and 331 are floor beams coupling the two walls together. They have the effect of modifying the simple vertical cantilever behaviour of a single wall some way towards rigid frame action depending on relative stiffnesses. In some circumstances, where beams are not desirable, the coupling action of an in situ RC slab can be modelled by means of a slab member. The effective width of slab may be estimated using the graphs in fig 16 of CIRIA 102. Obviously whether beams or slabs are used to couple shear walls, they must be reinforced to sustain the moments and shear forces output by the analysis. In some extreme circumstances the force effects in the coupling beam or slab may be unsustainable and it may be necessary to resort to specific pin-jointed details or to use plastic analysis to justify a feasible section design.

Intersecting shear walls

Two or more plane sub-frames like those discussed above may be assembled into 3D groups of walls by making the rigid arms share nodes in the 3D frame model. Fig 6 shows two such walls forming an offset T shape in plan. Note that one set of rigid arms has been split to provide the node connection between the two walls. Obviously this would not be necessary for a symmetrical T plan shape.

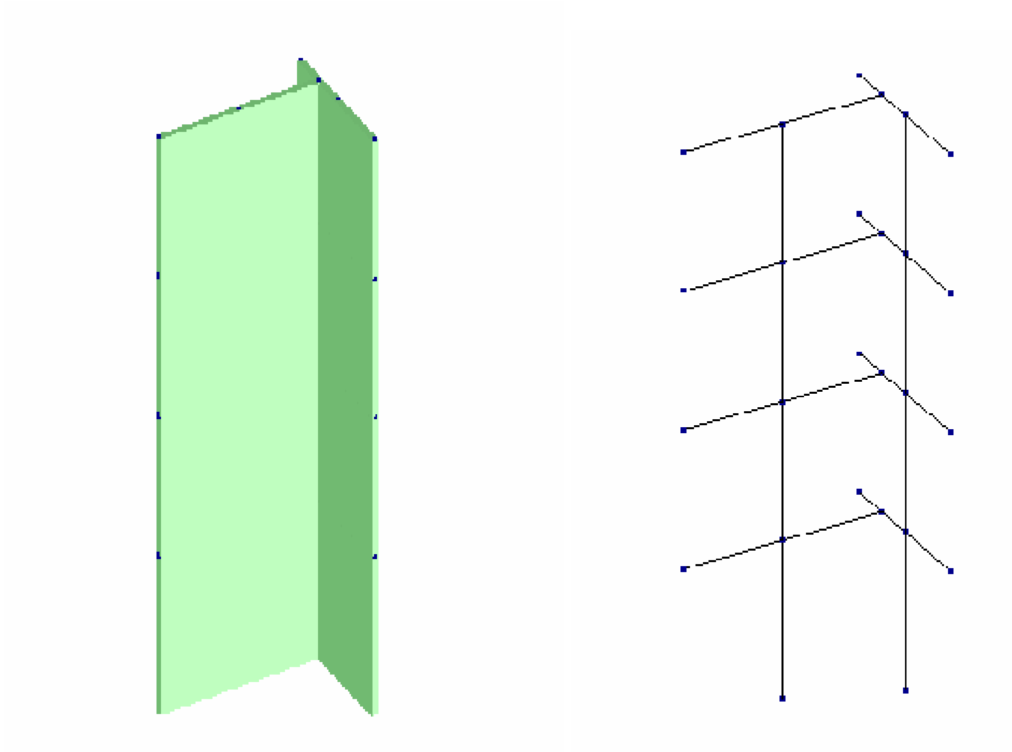
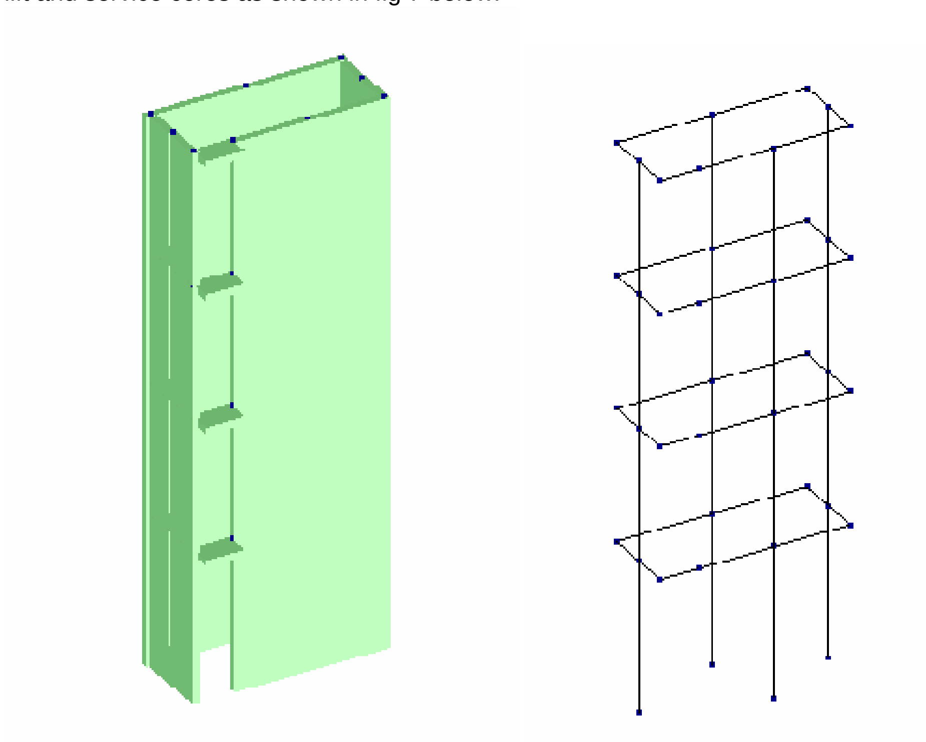


Fig 6: Solid and stick models of intersecting shear walls

Core walls

More complex rectilinear assemblies of sub-frames can easily be constructed to model stair, lift and service cores as shown in fig 7 below.



Building frame model with shear walls

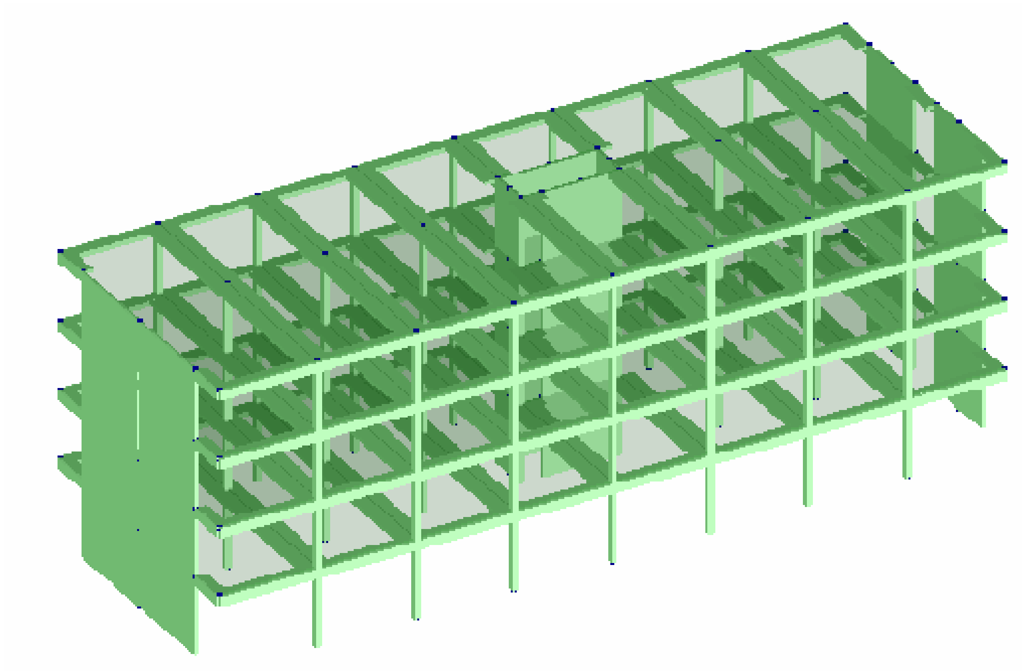


Fig 8: Solid model of RC frame with three groups of shear walls

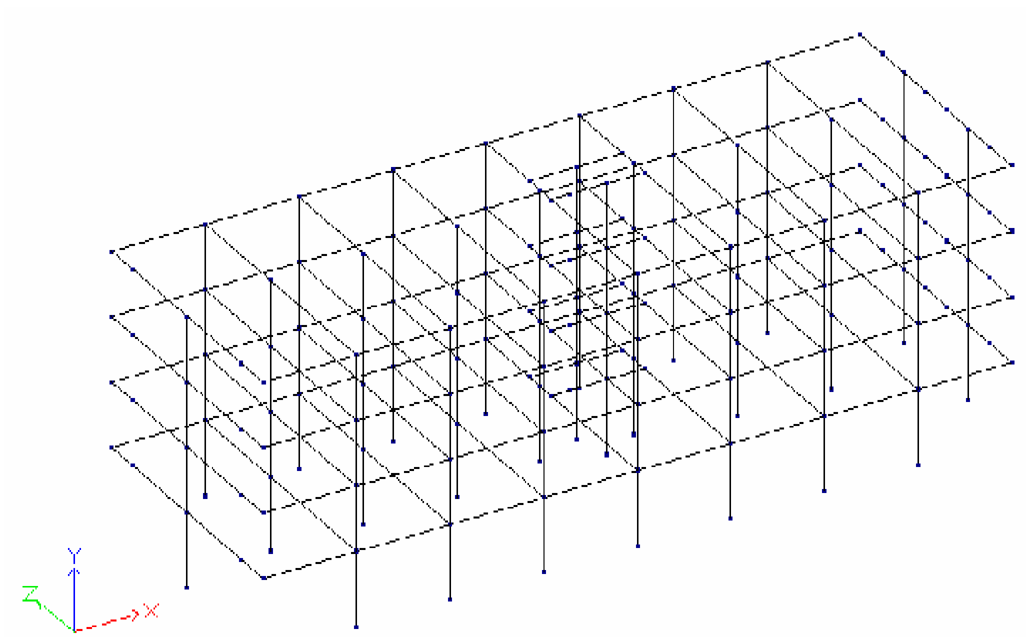


Fig 9: Stick model of RC frame with three groups of shear walls

Figs 8 and 9 illustrate a four storey frame of two spans (6 and 8 m) and 8 bays of 5m with a simple cantilever shear wall at the left end, two coupled shear walls at the right end and a stair core near the centre of the building. The model was initially constructed using the multi storey frame generator and then modified to add the shear wall sub-framing at each end using joint co-ordinates and the Quick member tool. Part of the interior was then deleted to insert the stair core sub-framing and then reinstate the main beams connecting to the corners of the core. The floors and roof loading were applied using the Automatic panels tool after selecting the floor members at each level. The `rigid' panel option was selected to model the diaphragm action of insitu RC floor slabs. Fig 10 shows the panels and the two-way load distribution scheme adopted.

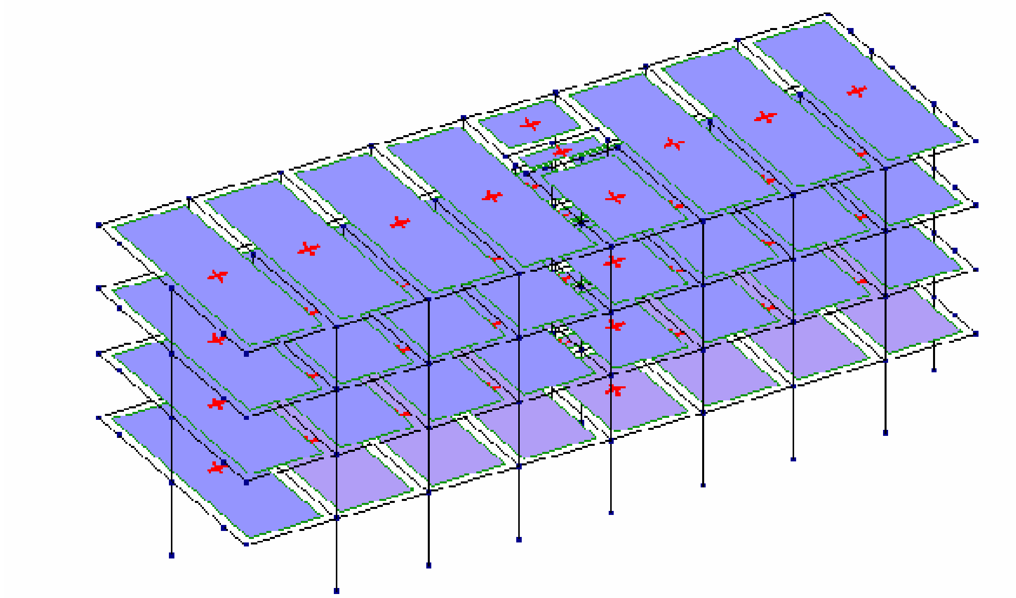


Fig 10: Stick model of frame showing floor and roof panels.

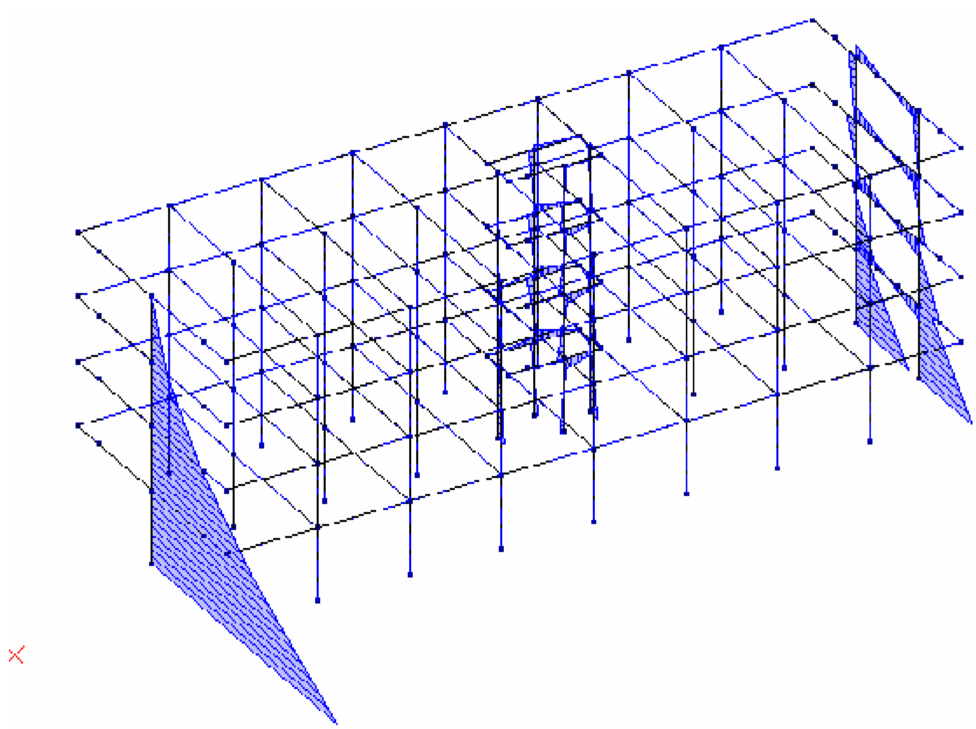


Fig 11: Moments due to wind acting in the Z direction.

Fig 11 shows the moments in the shear walls due to wind blowing in the Z direction. Because of the great stiffness of the shear walls there are negligible bending effects in the columns and beams due to wind. The left shear wall moment diagram shows the expected simple cantilever form whilst the right hand coupled shear walls show a typical `wind portal' form at higher levels changing to cantilever action in the lower storeys. The stair core takes a smaller part in resisting loads in the Z direction due to its relatively shallow section depth. However the axial force diagram in fig 12 clearly illustrates the `flange action' of the core wall panels perpendicular to the wind direction. Similarly tension and compression forces are induced in the windward and leeward walls of the right hand coupled shear wall system due to rigid frame action.

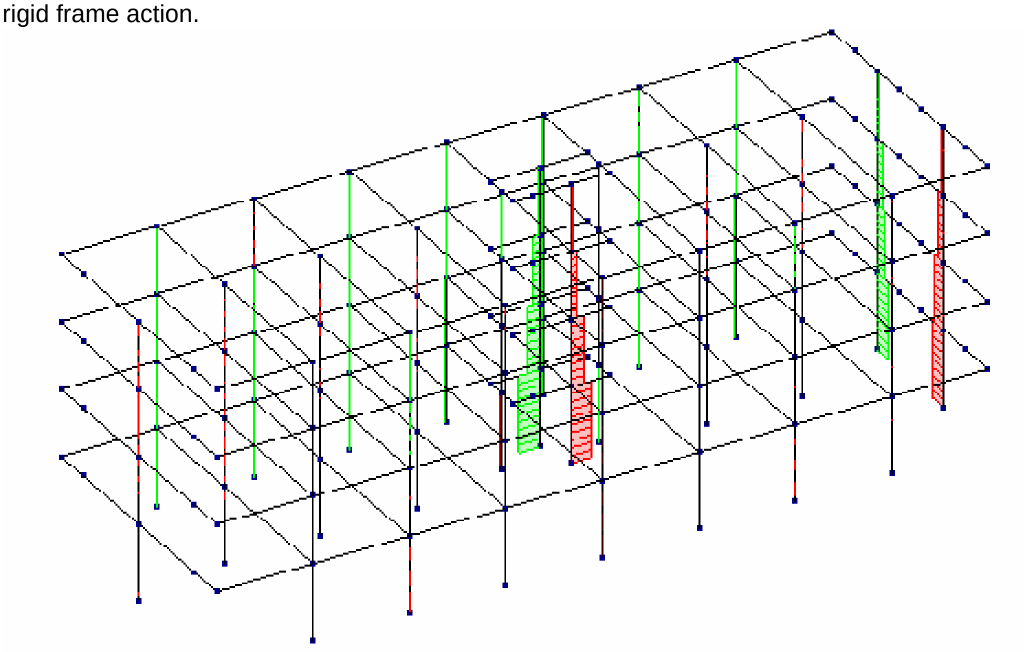


Fig 12   Axial forces due to wind acting in the Z direction.

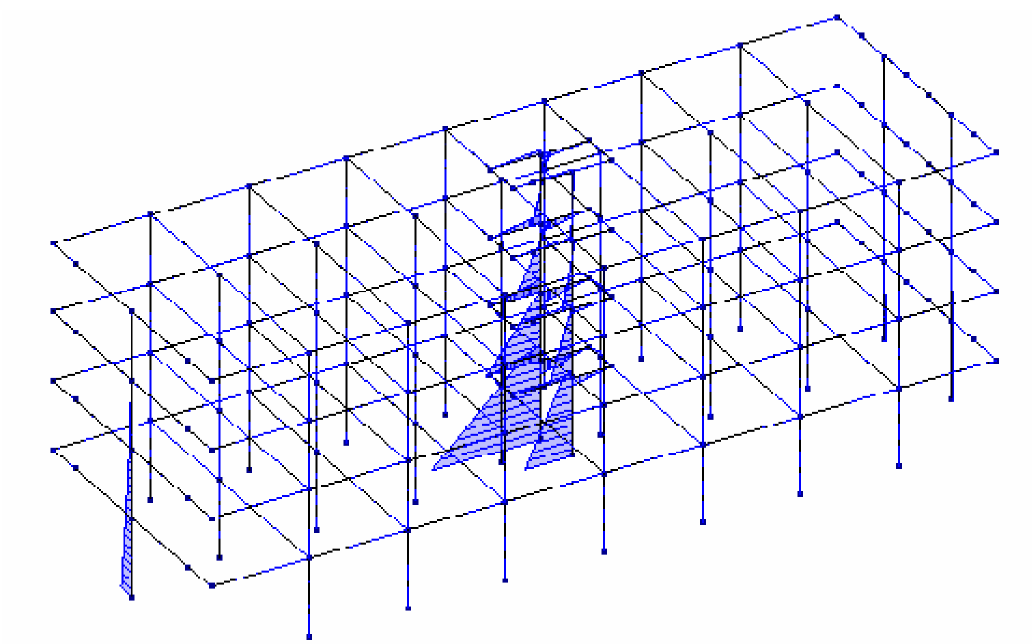


Fig 13   Moments due to wind acting in the X direction

Fig 13 shows the stair core to be dominant in resisting wind acting in the X direction. However a significant moment is induced in the left shear wall which is transverse to the wind direction. This is because of the asymmetry of the stair core both in shape and position in the structure. Asymmetry of wind load or resisting elements causes the floor diaphragms to twist in plan generating force effects in the plane shear walls transverse to the wind direction.

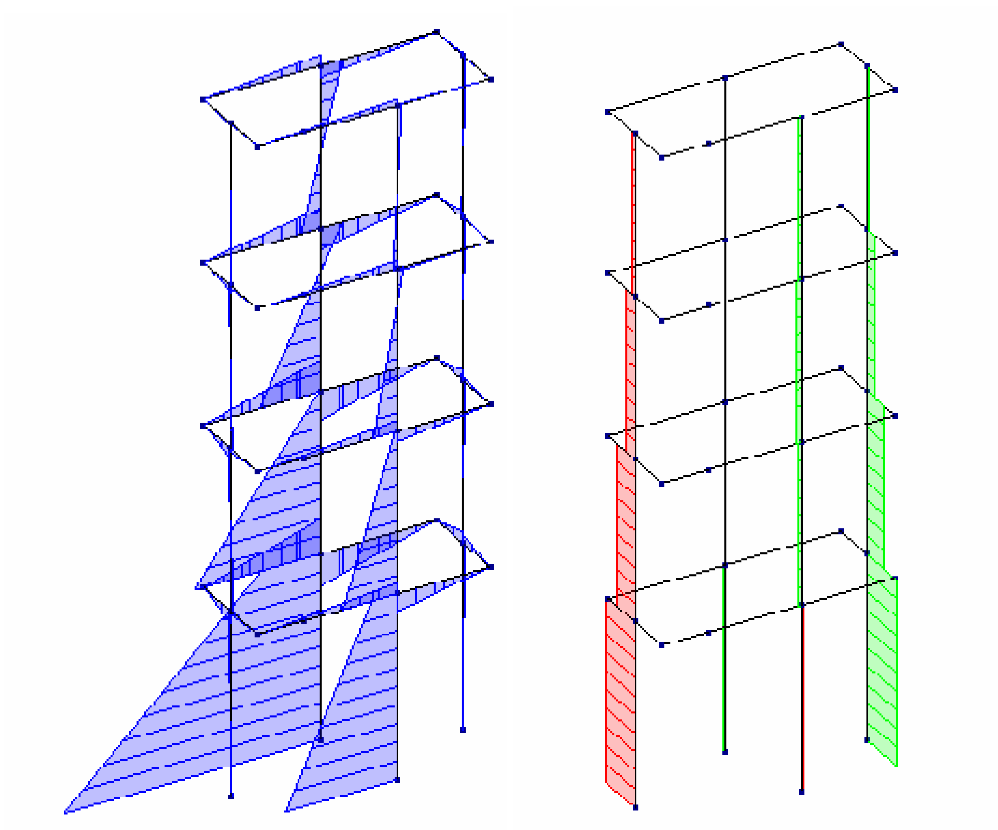


Fig 14   Moment and axial force diagrams for the stair core due to wind acting in the X direction.

Fig 14 shows how the short end walls of the stair core act as `flanges' of the `box' section carrying axial force whilst the long side walls act as `webs' carrying moment.
The resultant vertical stresses are essentially flexural stresses due to bending of the core section and can be read from the *Member effects* results table when the `stresses' option is selected or hand calculated from first principles. It will be found that the extreme fibre stresses in the `webs' do not match the extreme fibre stresses in the `flanges' at their intersection points. This is because the flange forces are not uniformly distributed as assumed in linear elastic calculation but vary due to the `shear lag' effect as in any flanged section. Stresses are maximum in flanges at the junction with the web and least mid way between webs.

Shear walls under dead and imposed loads

The principal purpose of shear walls is to resist horizontal loads due to wind and earthquake effects and to generally provide lateral and/or longitudinal stability. The resultant in-plane moments and shears due to wind, seismic and notional loads are readily appreciated. However, the presence of significant in-plane moments and shears in the walls under vertical dead and imposed loads alone may be initially and superficially surprising. There are two main reasons for this.

Firstly there may be an in-plane moment due to unsymmetrical vertical loads applied directly to the shear wall `arms' by slab and beam reactions. Secondly, and perhaps less obviously, the shear wall and floor diaphragm system may restrain several unsymmetrical beam and column frames from sway under vertical loads as shown in the following example.

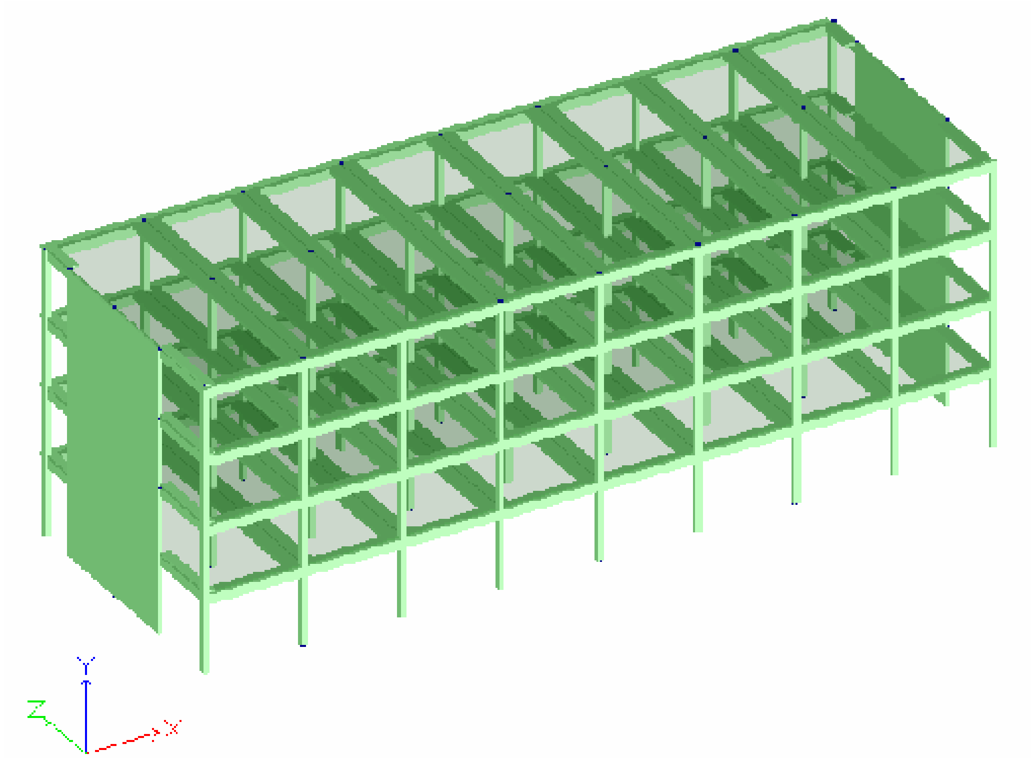


Fig 15: Four storey frame with simple symmetrical cantilever shear walls at each end.

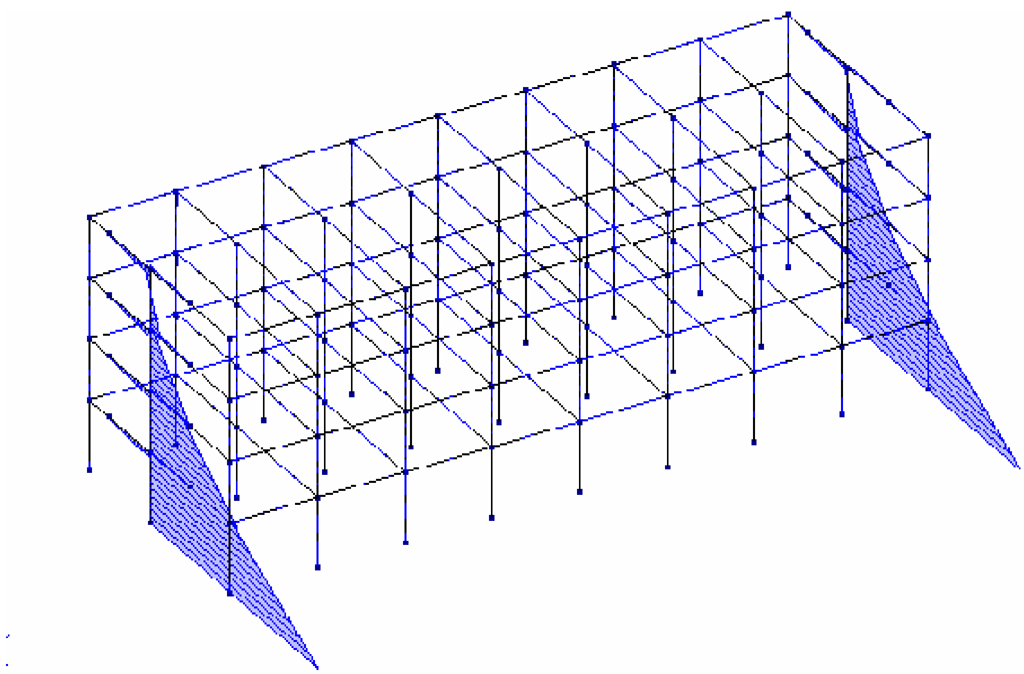


Fig 16: Moments due to wind acting in the Z direction.

Fig 15 shows a very simple frame with symmetrical cantilever shear walls at each end. The internal beam and column frames have two spans of 8 m and 6 m respectively and are therefore unsymmetrical and as isolated 2D frames would sway slightly.
Fig 16 shows the moments in the shear walls due to wind acting in the Z direction which are in the anticipated form. Fig 17 shows the moments due to dead load only with substantial in-plane bending of the shear walls to resist the accumulated sway restraint forces transmitted through the floor diaphragms from the internal frames.

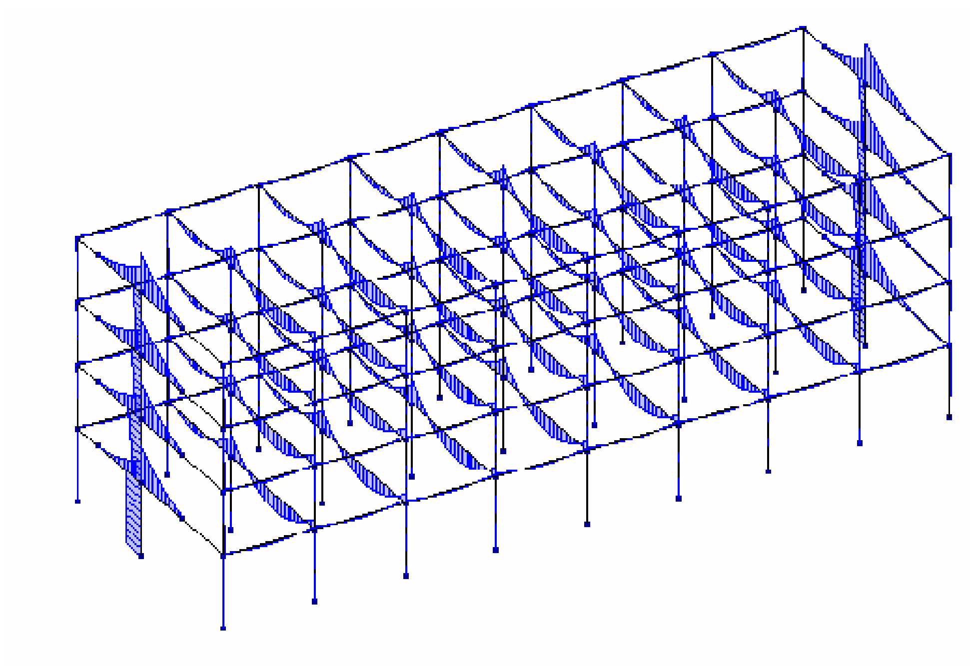


Fig 17: Moments due to dead load only.

The form and magnitude of the shear wall moments due to dead load can be validated by analysing a typical internal frame in 2D with floor level horizontal restraints. The restraint reactions from this analysis are multiplied by 3.5 frames and applied to a 2D model of an end frame containing a shear wall. The moment diagram resulting can be compared to the similar diagram produced by the 3D analysis as shown in fig 18.

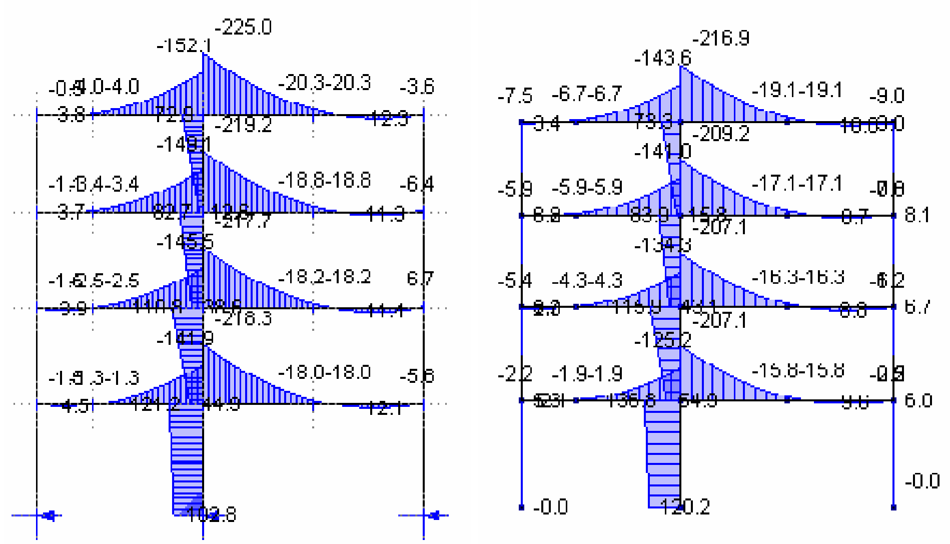


Fig 18: 2D and 3D analyses compared

Shear deformation

For normal beam and column members shear stiffness and deformation are either negligible or very small so are usually neglected in elastic analysis with significant savings in computation time. For wide members like walls, shear stiffness and deformation are potentially significant. However McLeod[2] states that neglect of shear deformation has little effect on the distribution and magnitude of force effects in a shear wall structure. Also, whereas neglect of shear deformation may lead to underestimation of deflections in very low shear walls (by as much as 50 to 80% for height/length = 1.0), these deflections are unlikely to be of any significance. For taller walls the error in neglecting shear deformation is less reducing to 2 to 10% for height/length = 5. Fig 3 in reference 2 could be used to factor the modulus of elasticity for cases where deflection is likely to be significant.

References

1: Irwin, A.W. *Design of shear wall buildings,* CIRIA Report 102
Construction Industry Research and Information Association, London 1984

2: McLeod, I.A., *Structural analysis of wall systems.*
The Structural Engineer, 55 (no 11) November 1977, pp 487-495.