CADS A3D MAX

Analysing portal structures in 3D
– why and how
– traps and tips for newcomers

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– why and how - traps and tips for newcomers

Synposis

This paper proposes that three dimensional analysis of portal framed structures by computer is justified because of the potential savings in designers labour and avoidance of numerical errors. In order to realise these savings in practice it is necessary to avoid a number of traps for the unwary which can lengthen the learning curve. The paper examines numerical examples of typical single span, two span and `hit and miss' frames in order to identify the main problems and solutions.

Introduction

Portal frame structures are traditionally designed as an assemblage of 2D main portal frames plus the ancillary end frames, posts, bracing and other secondary members. The advent of computer analysis and design software has not changed this much...yet. It is well known that there is little or no scope for refining a skilful design of main frames based on 2D elastic-plastic analysis. So what is the point of modelling these structures in 3D?

For the typical `prismatic shed' the case for 3D analysis is based on designer productivity rather than economic design of the structure itself. A three dimensional frame analysis will automatically transfer the dead, imposed, wind and snow loads applied to the surfaces of the structure through the framing to the foundations without manual intervention. The members and foundations can all be checked or auto-designed in an integrated process rather than piecemeal.
Two dimensional analysis of the main frame leaves all the secondary members to be designed separately and, perhaps more important, gives the engineer the tedious and error prone task of transferring reactions/loads through the structure to the foundations...for all the relevant load combinations especially those including wind blowing on the end faces. The case for 3D structural modelling is therefore based on reducing designer's labour and errors.

Three dimensional modelling is even more attractive for those portal structures which depart from the simplest prismatic shed format.

Portal structures with omitted columns, colloquially known as `hit and miss frames' require the transfer of loads between frames via valley, ridge or purlin beams. This is dealt with automatically in a 3D frame analysis model.

Hip end roofs are even more suited to 3D modelling whilst the common portal variations of intermediate floors, roof appendages, skew ends, crane gantries etc that come with `real jobs' also favour 3D computer modelling especially for visualisation.

However, three dimensional structural modelling demands considerably more skill and understanding from the user than 2D analysis. Many engineers with experience and confidence in 2D frame analysis and design are frustrated that their first ventures into 3D encounter problems and anomalies that take time to resolve before they become productive. The purpose of this article is to anticipate and discuss some of the issues relevant to 3D portal frame analysis so as to speed up this learning process. This will be done by means of specific examples.

Single span portal frame example

Fig 1 shows a `wire frame' diagram of a typical single span pin based portal frame structure with purlins and sheeting rails supporting metal cladding. Main dimensions:-

Span 32.5 m, 6 bays of 6.0m, Eaves height above slab 6.0m. Base top below floor 0.45 m. Roof slope 6 degrees.

The structural analysis model could incorporate the roof purlins and sheeting rails but this is rarely done because these members can be designed adequately from manufacturers load tables or software. More significantly, the inclusion of (typically) hundreds of purlin and rail members in the analysis model leads to highly inefficient and slow computation and data handling. Normal practice is not to include the purlins and rails as members at the design/analysis stage but to allow for their effects as restraints to the main frame members by other means.
Surface loads are applied as unit area loads (kN/m2) to load application panels or directly to the rafters and columns as distributed loads (kN/m). It is universally accepted that point loads from purlins etc and equivalent distributed loads produce practically the same effects for all but the smallest frames.

Most engineers will instinctively create simple main frame models like that shown in fig 2 in \`stick member' form.

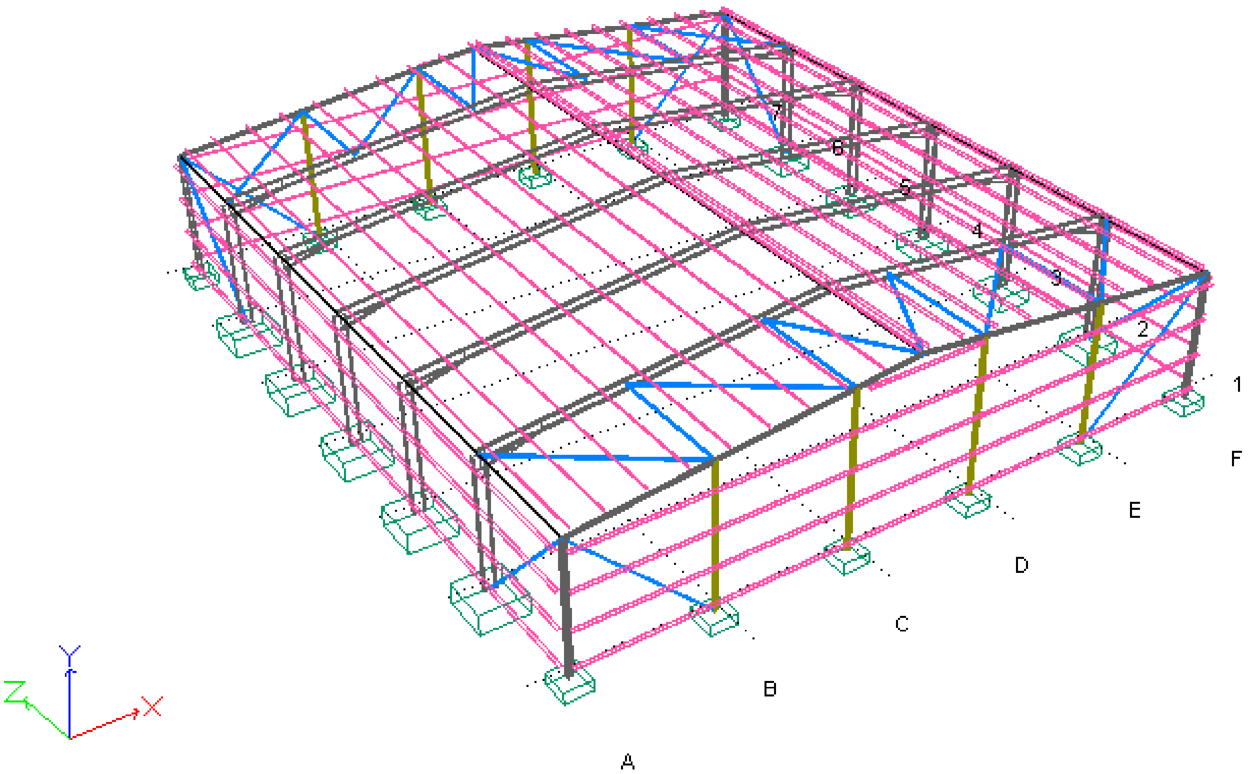

Fig 1: Wire-frame diagram of single span portal frame structure with purlins and rails

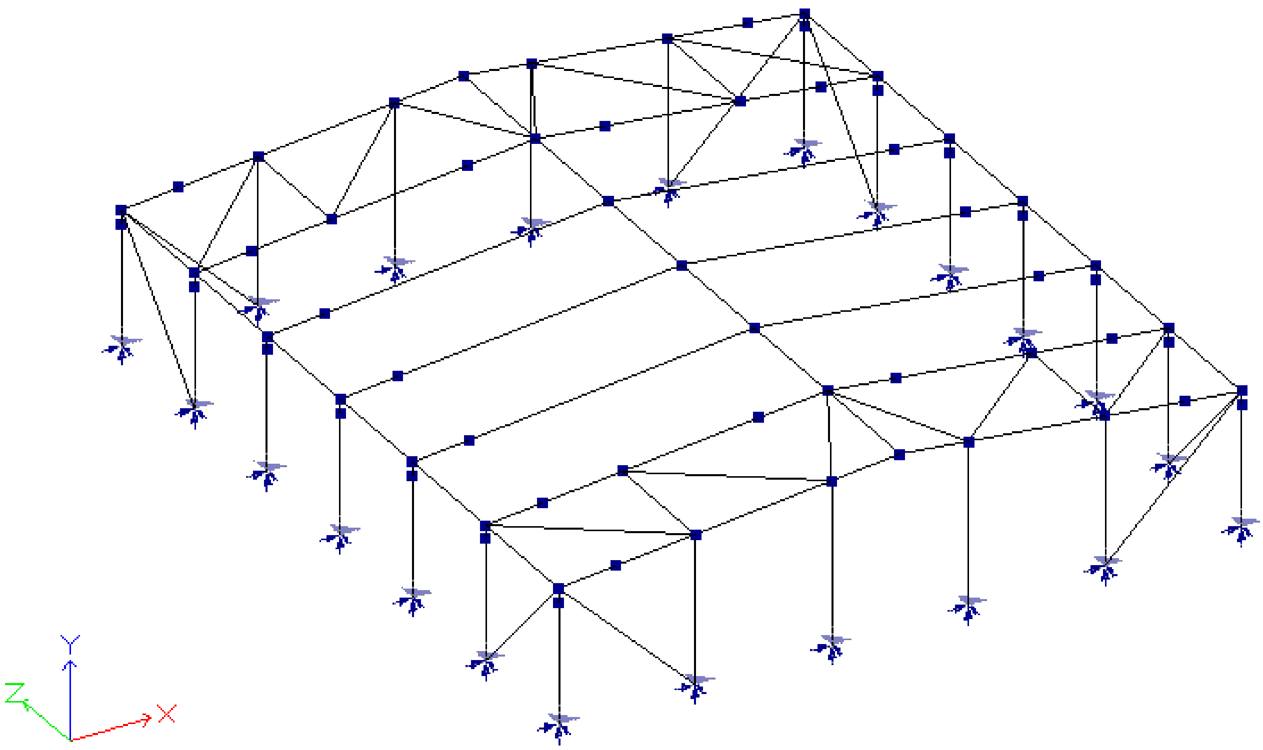

Fig 2: Stick-frame diagram of single span portal structure with purlins and rails omitted.

This simple frame and loads was originally created in CADS SMART Portal 3D [SP3D] including autodesign of all steel sections. It is an entirely conventional steel portal building frame with all the common features. A loaded internal main frame type was exported to CADS A3D MAX as a 2D frame. This 2D frame was then copied 7 times and then longitudinal members, bracing and end posts were added to produce the model shown above. Note that it is advisable not to export one of the frames adjacent to bracing because the 2D file will include axial loads from the bracing which would be duplicated in the 3D model. The end and penultimate frame rafter members were `split' so as to introduce intermediate nodes with which to attach the roof bracing and end posts. The end frames each include 4 permanent gable posts and so the rafter and column sections were reduced accordingly compared with the main frames.

The following members were autodesigned and subjected to second order elastic plastic analysis in SP3D as continuous portal structures and the sections adopted in the A3D MAX analysis:-

Main rafters:	457 x 191 UB 67 + haunch.
Main columns:	533 x 210 UB 92

The end frame sections are as designed in SP3D using conventional simple design methods:-

End rafters:	203 x 203 UC 46 continuous - no haunch.
Corner columns	203 x 203 UC 46
End posts:	152 x 152 UC 37.

The bracing sections are as designed in SP3D using conventional simple design methods:-

Roof bracing:	114 x 3.6 CHS

Side bracing:	114 x 3.6 CHS
End bracing:	76 x 3.6 CHS
Ridge ties:	76 x 3.6 CHS
Eaves ties:	76 x 3.6 CHS

These members are pin-connected to the main frame nodes.

The model could have been created directly in A3D MAX but this would take longer.

Note that the haunches are each represented by two special sub-members to facilitate elastic-plastic hinge analysis with or without second order Pdelta effects. The rafter haunch members are tapered profile with the elastic section properties of the haunch included. The column haunch member is uniform and same as the main column section. Both haunch members are assigned `unlimited' plastic resistance moments so that plastic hinges are only formed in the uniform members of the frame. After analysis, the rafter haunches are checked as part of the rafter members using the A3D MAX Design>Results module or by direct export to CADS Steelwork Member Designer [SWMD] to ensure that, as well as being laterally stable, they do not contain plastic hinges under any design load combination.
Note also that the column support nodes are defined as "pinned bases" with torsional rigidity about the vertical (Y) axis. Absence of this torsional restraint would cause spurious premature `spinning' mechanisms to be reported whenever a plastic hinge is formed in the column at higher level.

Single span 2D frame analysis

In order to provide a benchmark for the 3D analysis, an internal main frame was analysed in 2D. Linear elastic-plastic analysis of the critical load combination 1: 1.4DL + 1.6IL + NL reports first hinge formation at Lf1 = 0.996 and second hinge formation and collapse at Lfc = 1.099 . With second order Pdelta effects included, the hinges form at Lf1 = 0.971 and Lfc = 1.008. The elastic critical load factor for this load combination is Lcr = 8.89. These analysis results are the benchmark against which the 3D analyses are compared.

Fig 3 shows the moment diagram for load combination 1 at load factor 1.0 together with the plastic hinges formed at collapse.

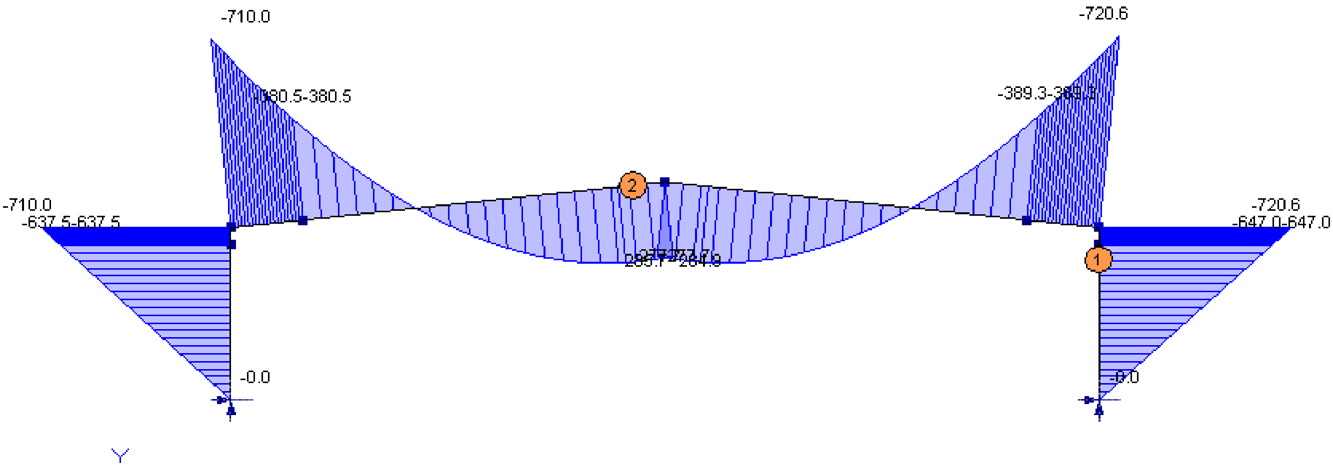

Fig 3: 2D main frame moment diagram at Lf = 1.000 and hinges at collapse for load comb.1: 1.4DL + 1.6IL + NL

Single span 3D frame linear analysis – effect of a ridge member

The behaviour of the 3D model may now be compared with the 2D frame analysis.
The 3D frame was first subjected to linear elastic-plastic analysis. Fig 4 shows the plastic hinges formed at collapse for the critical load combination 1: 1.4 Dead + 1.6 Imposed + Notional horizontal forces in X direction.

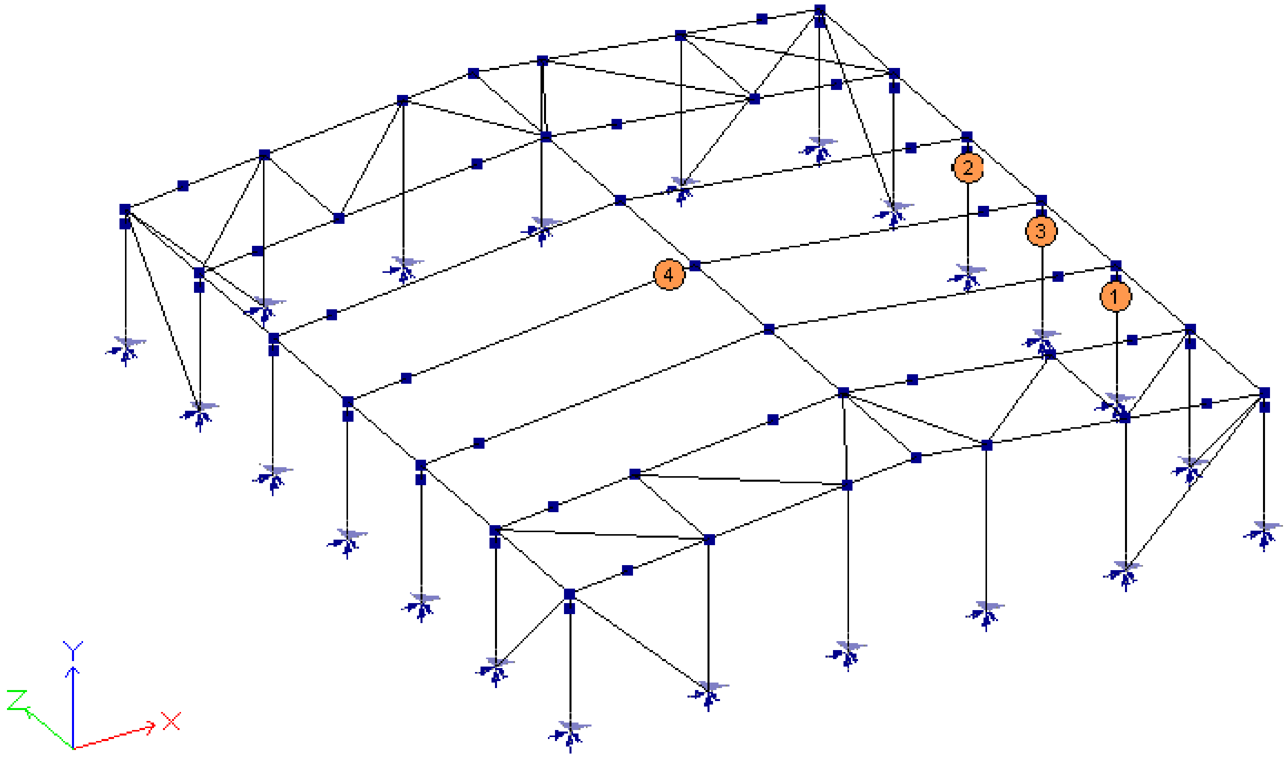

Fig 4 3D frame plastic hinge formation at collapse for load combination 1: 1.4DL + 1.6IL + NL

Three hinges form simultaneously in the right hand columns at Lf1 = 0.996 and collapse occurs on formation of a sagging hinge in the central left rafter at Lfc = 1.099. These results are entirely consistent with the 2D frame analysis results.

However, before proceeding to second order Pdelta analysis it is instructive to look more closely at the other results of first order analysis. Fig 5 shows the moment diagram for load combination 1.

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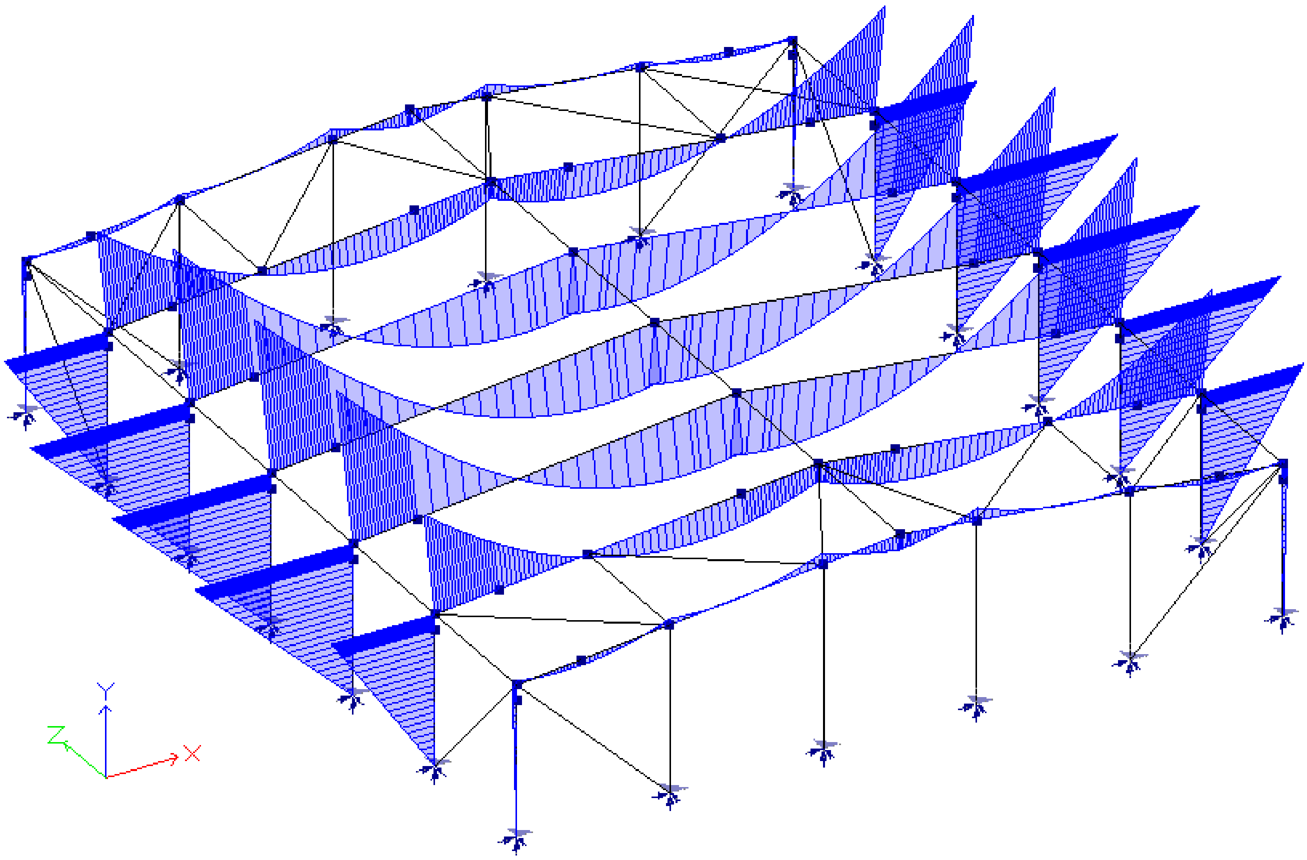

Fig 5: Bending moments for load combination 1: 1.4DL + 1.6IL + NL with ridge member

The diagram above shows bending moments for load combination 1. Note that the moments in the two penultimate frames are significantly less than for the three internal frames. The labels are omitted here for clarity but in fact the eaves moments are 457 and 720 kNm respectively whilst the midspan (ridge) moments are 75 and 278 kNm. It is evident that the penultimate frames are restrained horizontally at the eaves by the `closed tension ring' formed by the two sets of roof bracing and the eaves members. This causes the penultimate frames to act partly as `tied portals' and some load is redistributed to the end frames.
The corresponding axial force diagram in fig 6 shows maximum axial force 263.5 kN in the penultimate frames but 116.7 kN in the inner frames.
The maximum force in the roof bracing is 130 kN. This is much more than the maximum roof bracing force corresponding to 1.40*WNIS (wind near with internal suction) for which the bracing would be designed manually (52.2 kN).
Note that the force in the ridge longitudinal member for load combination 1.4DL + 1.6IL + NL is a substantial 185.4 kN due to its contribution to the `tension ring' system. This member would be considered nominal by conventional design methods and would be omitted altogether by many designers who rely on the ridge purlins to tie/strut the main frames together. These figures appear to indicate that the 3D analysis has shown up a significant load path which is entirely ignored by traditional designers. Taking the results at face value would lead to the conclusion that roof bracing members as normally designed (for end face wind only) are inadequate!

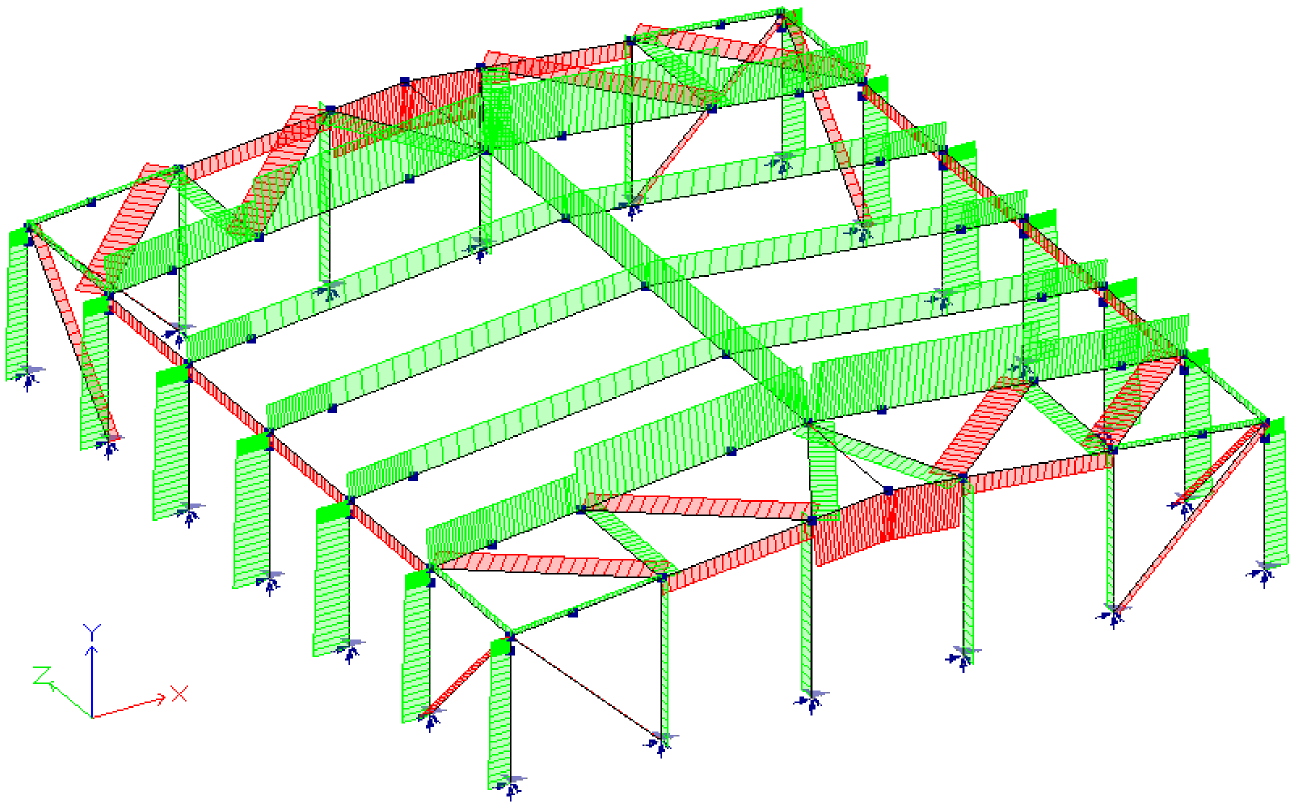

Fig 6: Axial forces for load combination 1: 1.4DL + 1.6IL + NL. – with ridge member

As there is a long history of trouble-free designs by traditional methods it might be deduced that the tension ring system does not develop in practice – at least not to the extent indicated by stiffness analysis. This could be due to yielding of the bracing connections limiting the forces carried by these members. Alternatively it may be speculated that the full design imposed loading does not occur in practice and no real portal structure has been loaded sufficiently to cause visible distress to the bracing members or their connections.

When modelling single span frames the `tension ring effect' can be largely eliminated by simply omitting the ridge compression member which most designers instinctively provide in the analysis model. This dramatically reduces the stiffness of the tension ring system. The results can be seen in fig 7 which shows the axial forces in the 3D frame with the ridge member omitted. The corresponding moments in the penultimate and inner main frames are then similar as shown in fig 8. The collapse load factor is unchanged but more column hinges are formed because the penultimate frames now behave very like the internal frames.

Without the ridge member, the internal main frames look unstable, but as they are only loaded in-plane and buckling effects are ignored by linear analysis, no problems are detected in this mode. However if non-linear Pdelta effects are included in the analysis, this is no longer true and measures are required to take into account the stabilising effects of the purlins, rails and cladding that exist in the real structure.

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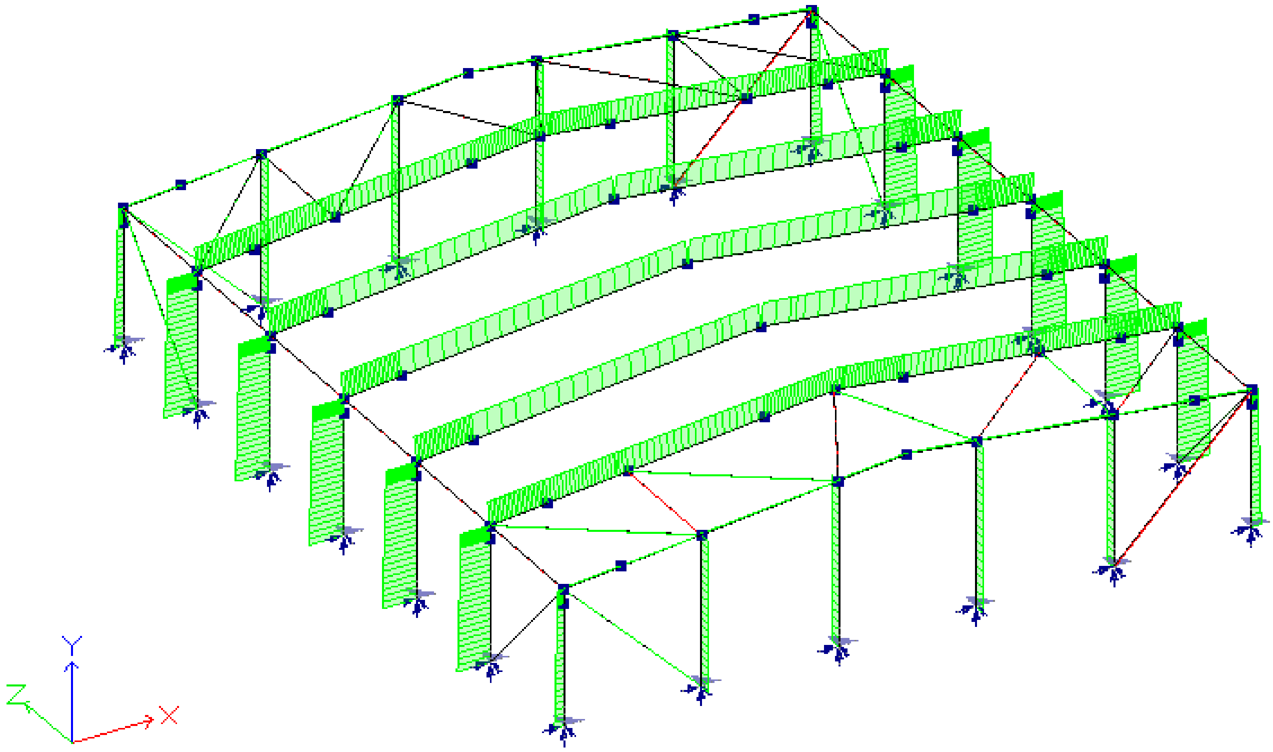

Fig 7: Axial forces for load combination 1: 1.4DL + 1.6IL + NL. – without ridge member

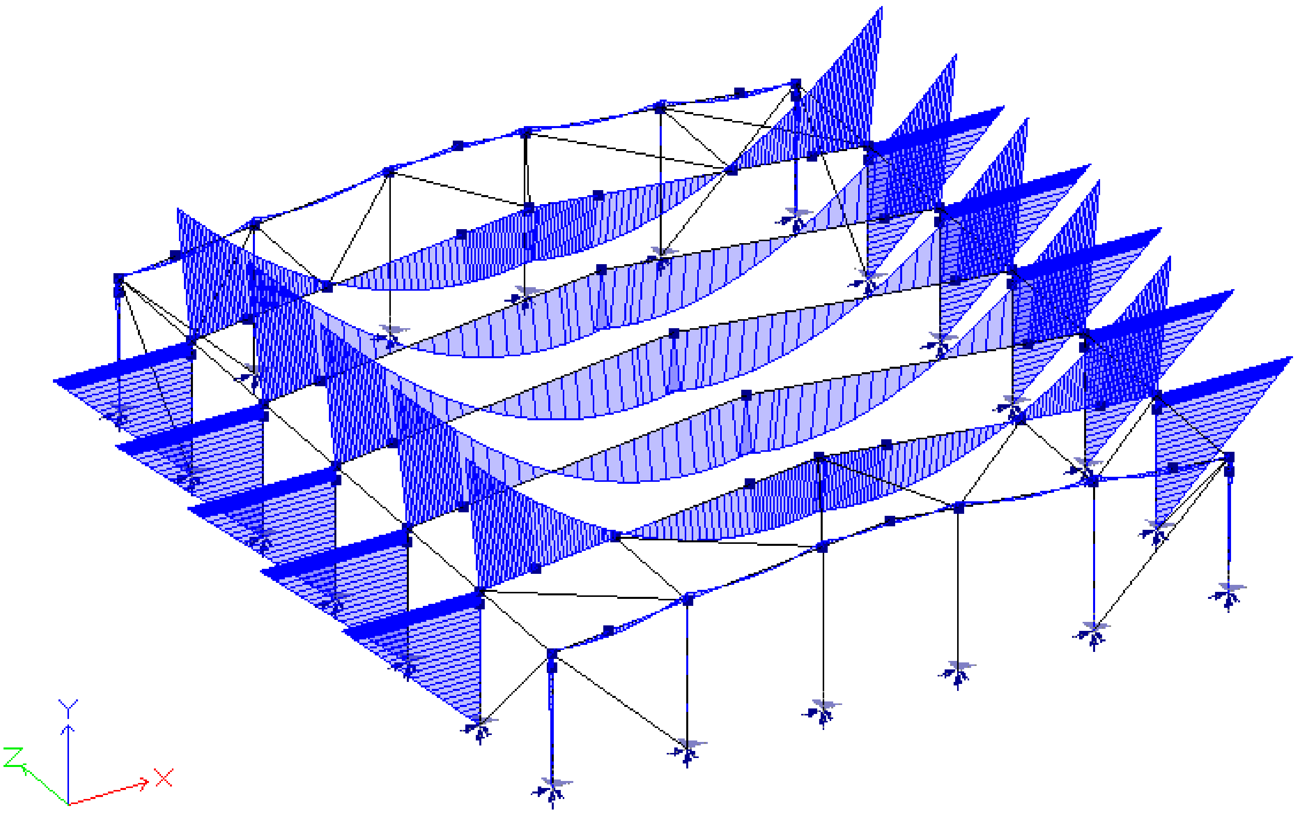

Fig 8: Bending moments for load combination 1: 1.4DL + 1.6IL + NL.– without ridge member

3D frame non-linear Pdelta analysis

Most of the main members in a normal portal frame structure are prevented from buckling about their weak axes by lateral restraints provided by the purlins and side rails acting in conjunction with metal sheeting. Detailed member lateral and torsional stability calculations to BS 5950-1:2000 are carried out by the steelwork design module (SWMD) but for global frame analysis it is sufficient to set these members to “Pdelta effects – major axis only” instead of the default “Both axes” setting. This has the effect of raising the elastic critical load factor for load combination 1 from a spurious 0.09 to 8.93 which agrees closely with the value obtained by 2D analysis. The members to which the “Pdelta effects – major axis only” setting is applied are:

Main rafters; Main columns; End rafters; Corner columns; End gable posts

Obviously this setting should not be applied to members without adequate lateral restraint such as extenal columns in an open sided building or unrestrained internal columns.

After obtaining consistency between 2D and 3D results for the elastic critical load factor, the second order elastic-plastic analysis of individual load combinations can be examined.

Under load combination 1 the 3D frame is reported to collapse at a load factor of 0.971 with just one plastic hinge formed in the right hand column at the underside of the haunch. The 2D frame analysis also formed a first hinge at Lf1 = 0.971 but proceeded to collapse at Lfc = 1.008 with a sagging hinge in the left rafter. What is the reason for the difference?

On reflection it can be appreciated that in the 3D frame the column plastic hinge together with the pinned base and the pin connected eaves ties form a local mechanism in the YZ plane.

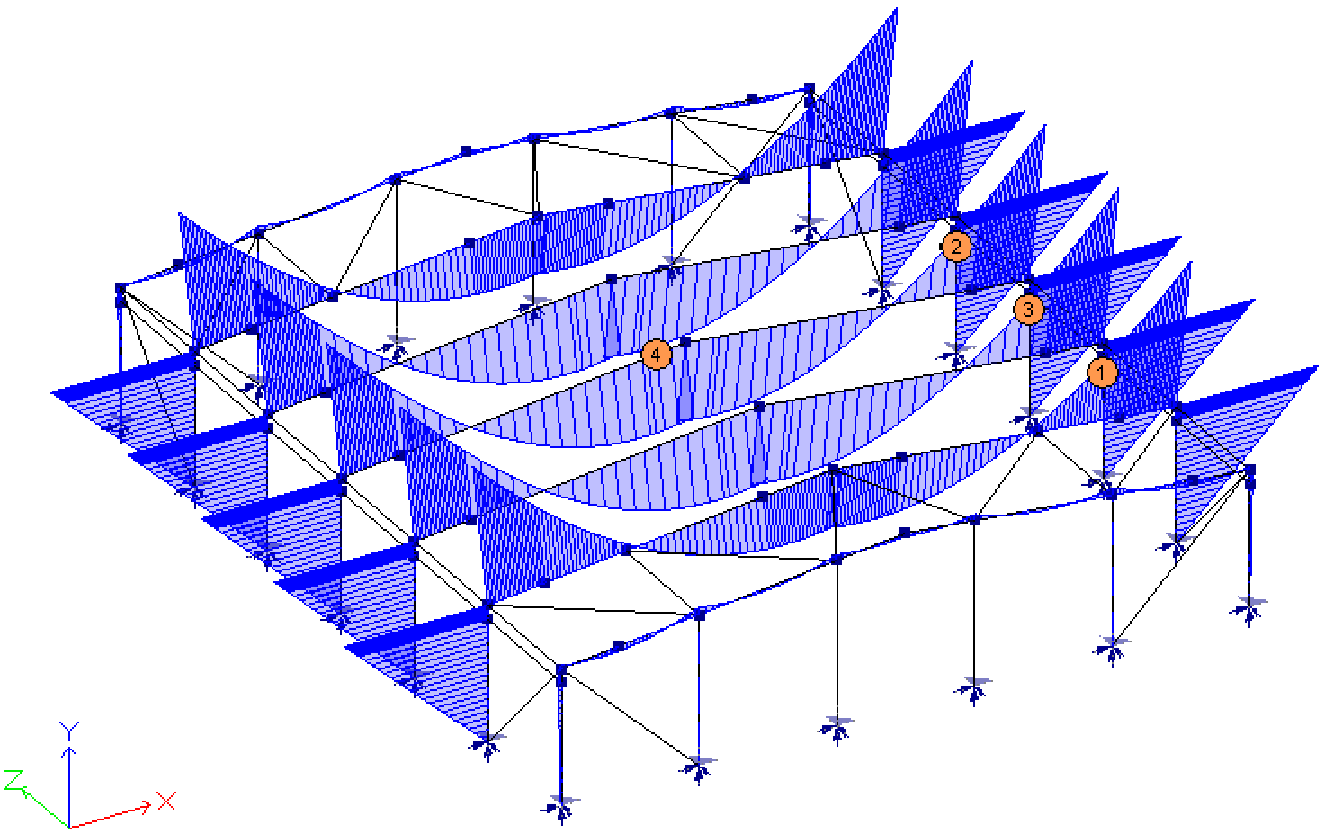

Fig 9: 1.4DL + 1.6IL + NL bending moments at Lf = 1.000 and hinges formed at collapse Lfc = 1.008. Non-linear elastic-plastic analysis with Pdelta effects major axis only for rafters columns and end posts

To avoid this spurious failure it is necessary to restrain the nodes at the underside of the haunches. This can be done by inserting longitudinal members joining the nodes and connecting to the corner column assuming this does not take part in the collapse mechanism or to the corner eaves node. Inserting these longitudinal members prevents the mechanism and is consistent with the normal design requirement to provide a lateral restraint at this

position. This is commonly achieved by placing a sheeting rail and stays at underside of the haunch. With the added longitudinal members Pdelta elastic-plastic analysis of load combination 1 returned Lf1 = 0.971, Lfc = 1.008 in agreement with 2D frame analysis.
Fig 9 above shows the moments at the design load factor = 1.000 together with the hinge locations at collapse.

Two span frame

Having described some traps and tips for 3D analysis of single span portal structures, the next logical step is to examine normal two span frames before moving on to hit-and-miss frames.

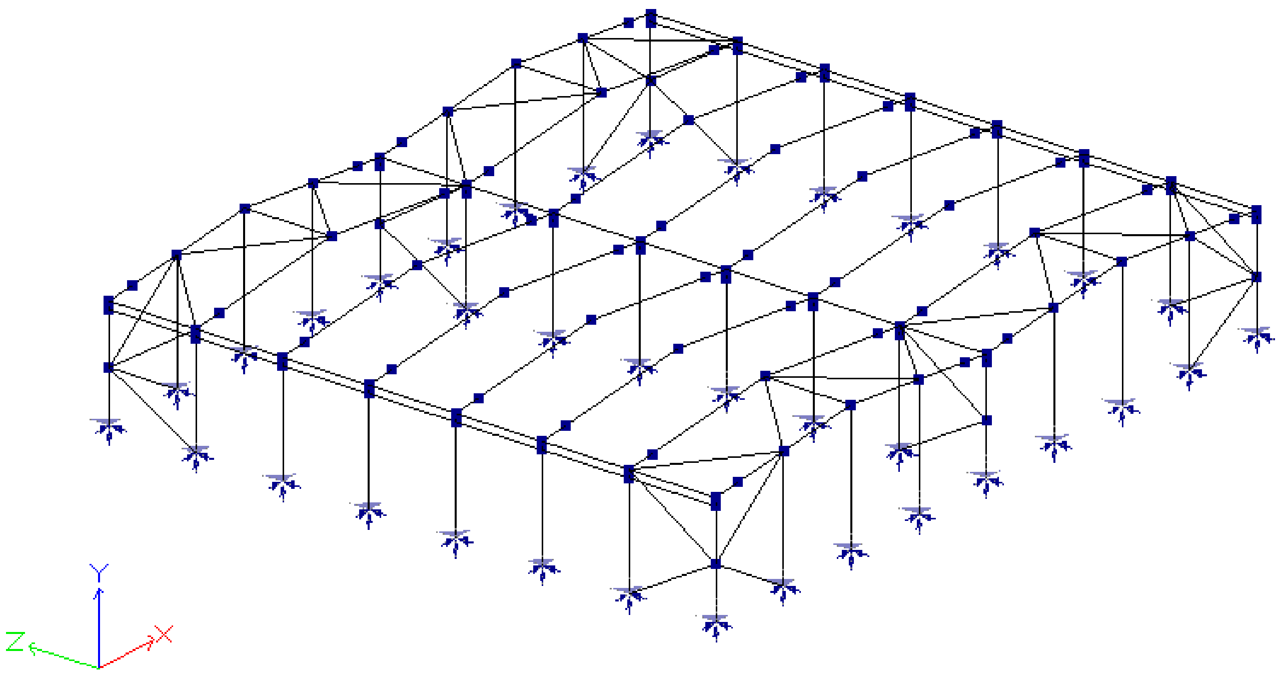

Fig 10: Stick model of two-span frame with purlins and rails omitted.

Fig 10 illustrates the frame chosen for study. Principal dimensions: Both spans 24.0 m, Eaves node 7.0 m above slab, Pinned connections to top of bases 0.45 m below slab. 7 bays of 6.0 m Roof slope 6 degrees. The analysis model incorporates the `tips' learnt on single span frames:

Pinned base supports must have torsional rigidity.
No internal ridge member
Longitudinal restraint members joining external columns at underside of haunch nodes,
Rafters and columns set to "Pdelta effects applied to major axis only."

The member section sizes were arrived at using SMART Portal 3D as follows:-

Main rafters and haunch cutting	356 x 171 UB 51
Main external columns	457 x 191 UB 67
Main internal columns	254 x 254 UC 73
End frame rafters	203 x 203 UC 46
End frame corner columns	203 x 203 UC 46
End frame internal columns	254 x 254 UC 73

End frame intermediate posts	203 x 203 UC 46 (web parallel to Z)

Roof bracing	114 x 3.2 CHS
Longitudinal bracing	114 x 3.2 CHS
Eaves and valley members	114 x 3.2 CHS
End frame bracing	76 x 2.9 CHS

These bracing members are pin-connected to the main frame nodes.

Note that internal longitudinal wall bracing is provided but this can be omitted at the cost of heavier roof and side wall bracing whilst keeping within acceptable deflection limits.

Two span 2D frame analysis

In order to provide a benchmark for the 3D analysis, an internal main frame was analysed in 2D. The critical load combination 1: 1.4DL + 1.6IL + NL has first hinge formation at Lf1 = 0.907 and fourth hinge formation and collapse at Lfc = 1.197 with Pdelta effects suppressed. With Pdelta effects included, the hinges form at Lf1 = 0.890 and Lfc = 1.094. The elastic critical load factor for this load combination is Lcr = 4.7. The lower Lcr relative to the single span frame (8.9) is because the height / span ratio is higher and the internal UC column is significantly less stiff than the UB section outer columns. These analysis results are the benchmark against which the 3D analyses are compared. Fig 11 shows the moment diagram for load combination 1 at load factor 1.0 together with the plastic hinges formed at collapse.

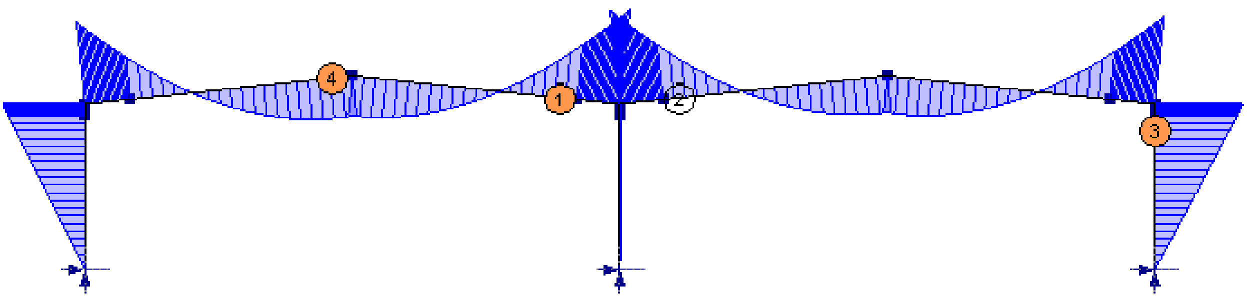

Fig 11: 2D main frame moment diagram at Lf = 1.000 and hinges at collapse for load comb.1: 1.4DL + 1.6IL + NL

Note that the second hinge to form unloads on formation of hinge 3 and collapse occurs as a result of the sagging hinge 4 forming in the left rafter near the middle of span 1.

Two span 3D frame analysis

The behaviour of the 3D model may now be compared with the 2D frame analysis.

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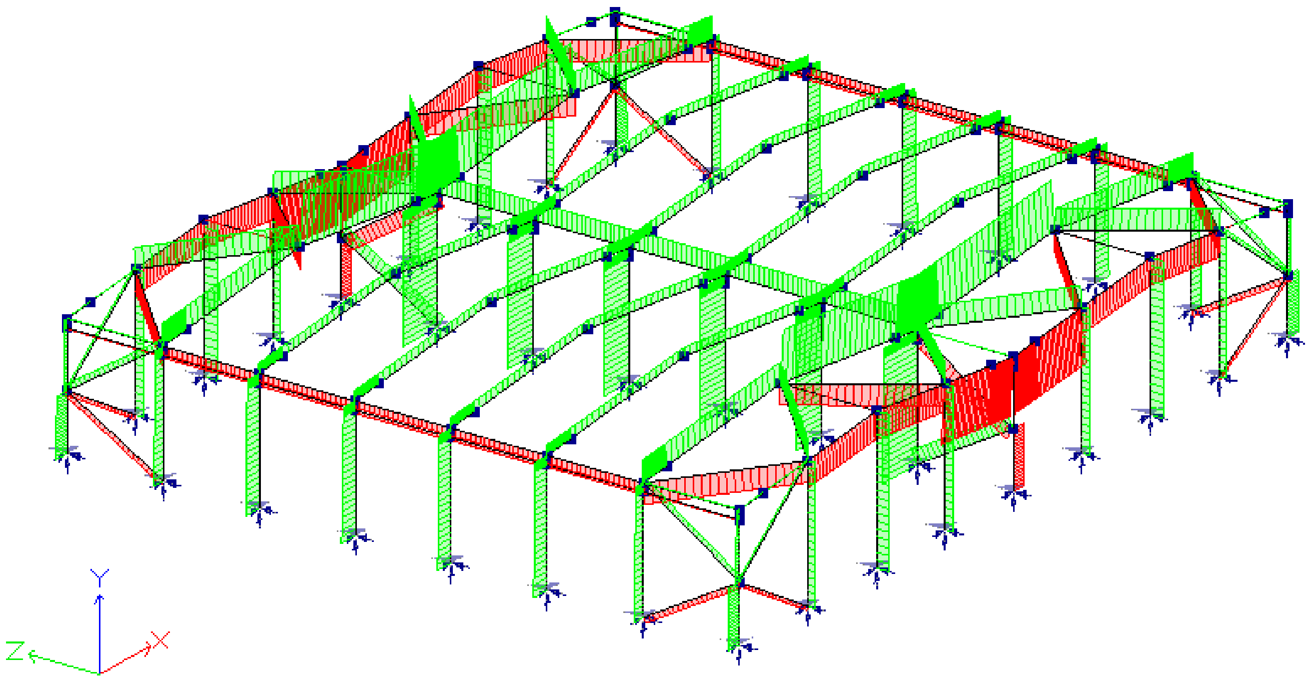

Fig 12 Two span frame with valley members.
Axial forces in members for load combination 1: 1.4DL + 1.6IL + NL

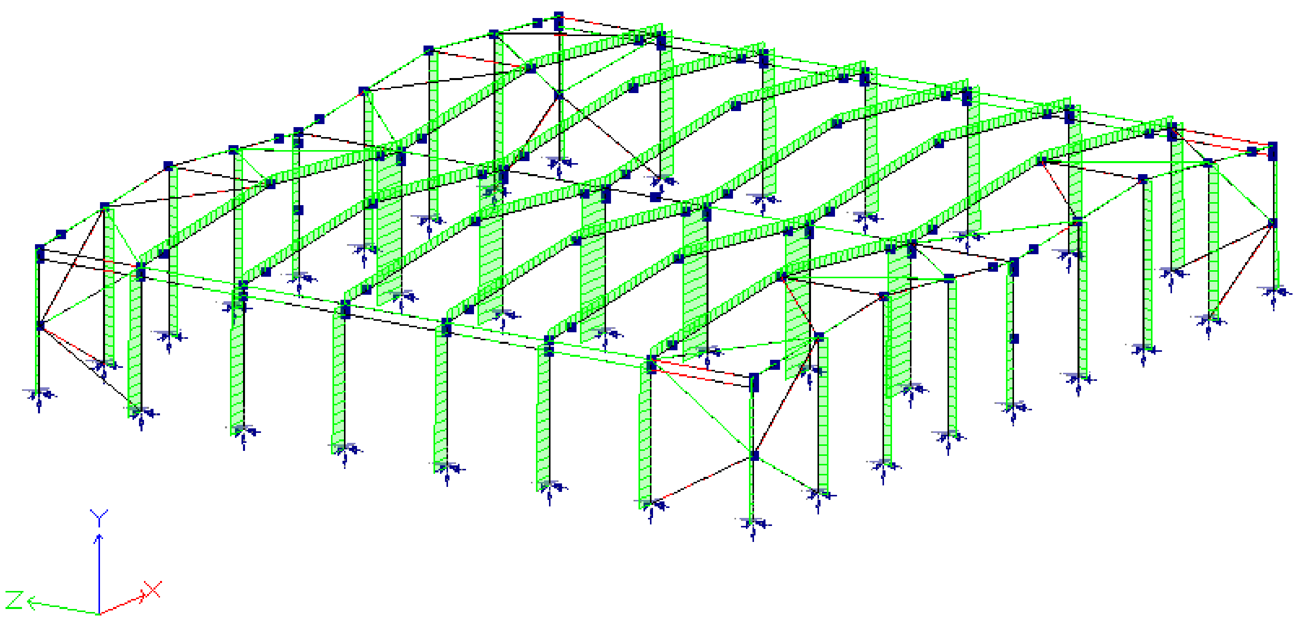

Fig 13 Two span frame with severed valley member. Axial forces in members for load combination 1: 1.4DL + 1.6IL + NL

Fig 12 shows the axial forces in the members for load combination 1: 1.4 Dead + 1.6 Imposed + Notional horizontal forces left to right. In similar fashion to the single span frame with ridge member a closed ring load path is visible in the roof bracing, end rafters, longitudinal eaves and valley members.

The forces in these members under factored dead and imposed loading are several times bigger than the forces due to wind pressure acting on the end(s) of the building. As for the single span frame it could be argued that, unless specifically designed for, these forces will not develop because of yield of the bracing connections .

The ring system may be released by removal of one of the valley members, by specifying valley members of low axial stiffness or by introducing a `cut' or `gap' in one of the valley members. This massively reduces the stiffness of the remaining ring structure and its internal forces and equalises the axial forces in the penultimate and internal main portal frames as shown in fig 13.

The elastic critical load factor for load combination 1 is 4.7 in agreement with the 2D frame result. Linear elastic-plastic analysis of load combination 1 provides a first hinge load factor Lf1 = 0.898 and collapse at Lfc = 1.196, similar to the 2D results (Lf1 = 0.907 and Lfc = 1.197). 23 hinges are formed at collapse because of duplication of the internal frames as shown below in fig 14

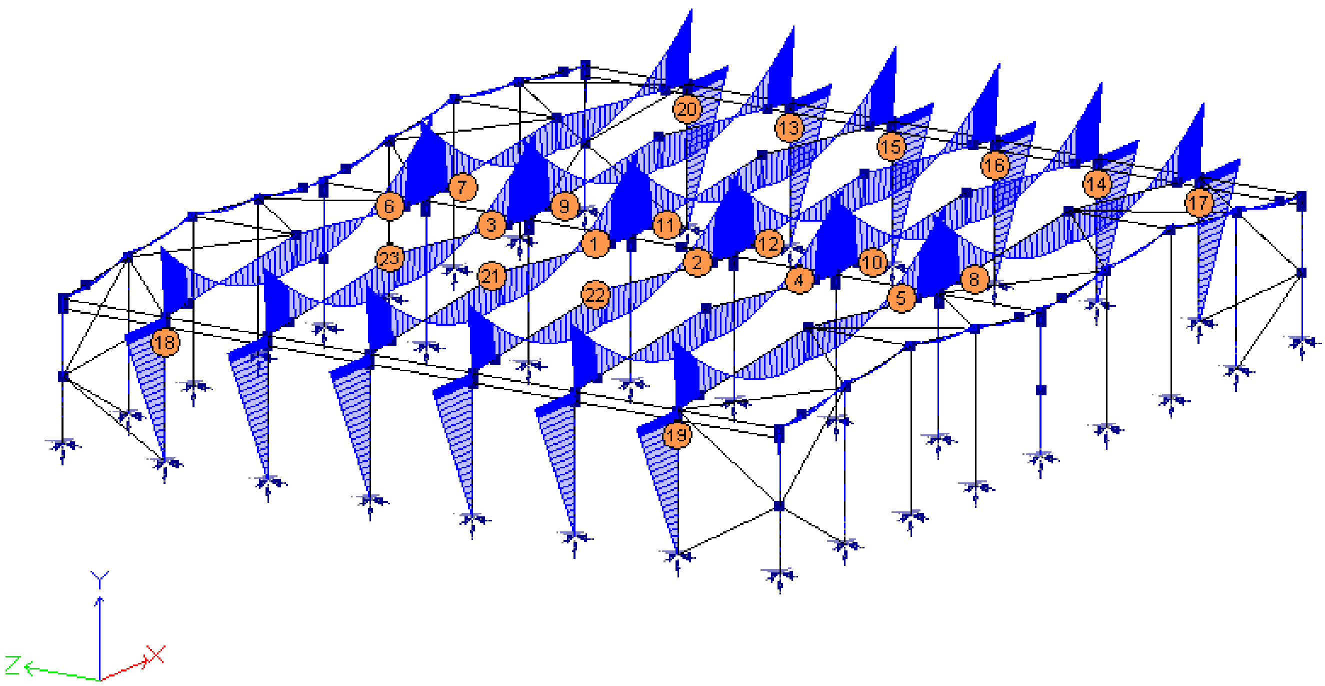

Fig 14 Two span frame moments for load combination 1: 1.4DL + 1.6IL + NL and plastic hinge formation at collapse. Linear elastic plastic analysis.

Non-linear elastic-plastic analysis of load combination 1 provides a first hinge load factor Lf1 = 0.881 and collapse at Lfc = 1.061, with 13 hinges formed. The moments and hinge formation are shown below in fig 15. This may be compared with the 2D result Lf1 = 0.890 and Lc = 1.094. It will be observed that no sagging hinge has been formed at collapse of the 3D structure and the `unloading hinge' reported in 2D does not occur in 3D. This is as yet unexplained. It may be that the collapse mechanism is spurious and a collapse load factor of about 1.094 should be achieved as in 2D. Alternatively the 2D frame may have been close to instability before the last hinge formed and slight differences in the 3D model may be sufficient to trigger buckling. However the 3% difference in collapse load factor is no problem in practice as the required load capacity is obtained.

The modelling devices of omitting, severing or `softening' the valley member are generally satisfactory in respect of force effects and load factors but unfortunately produce problematic deflection results. For this frame under load combination 1: (1.4DL + 1.6IL + NL) the valley nodes move inwards by a not inconsiderable 88 mm. Corresponding unfactored dead and imposed deflections along the valley are 18 and 31 mm.

The alternative device of providing only one set of roof bracing at one end has similar effect of eliminating the `tension ring' load path but generates longitudinal (Z) sway deflections of similar magnitude with movement towards the unbraced end.

The most satisfactory method of avoiding the `tension ring' load path and deflection effects appears to be achieved by placing one set of roof bracing in the centre bay. However this has the disadvantage that longitudinal members are required to join the end posts to the bracing system.

Attempting to replicate traditional 2D design assumptions in a 3D model seems to involve too many unsatisfactory side effects so it may be best to be consistent and design the bracing for the force effects generated by the 3D analysis.

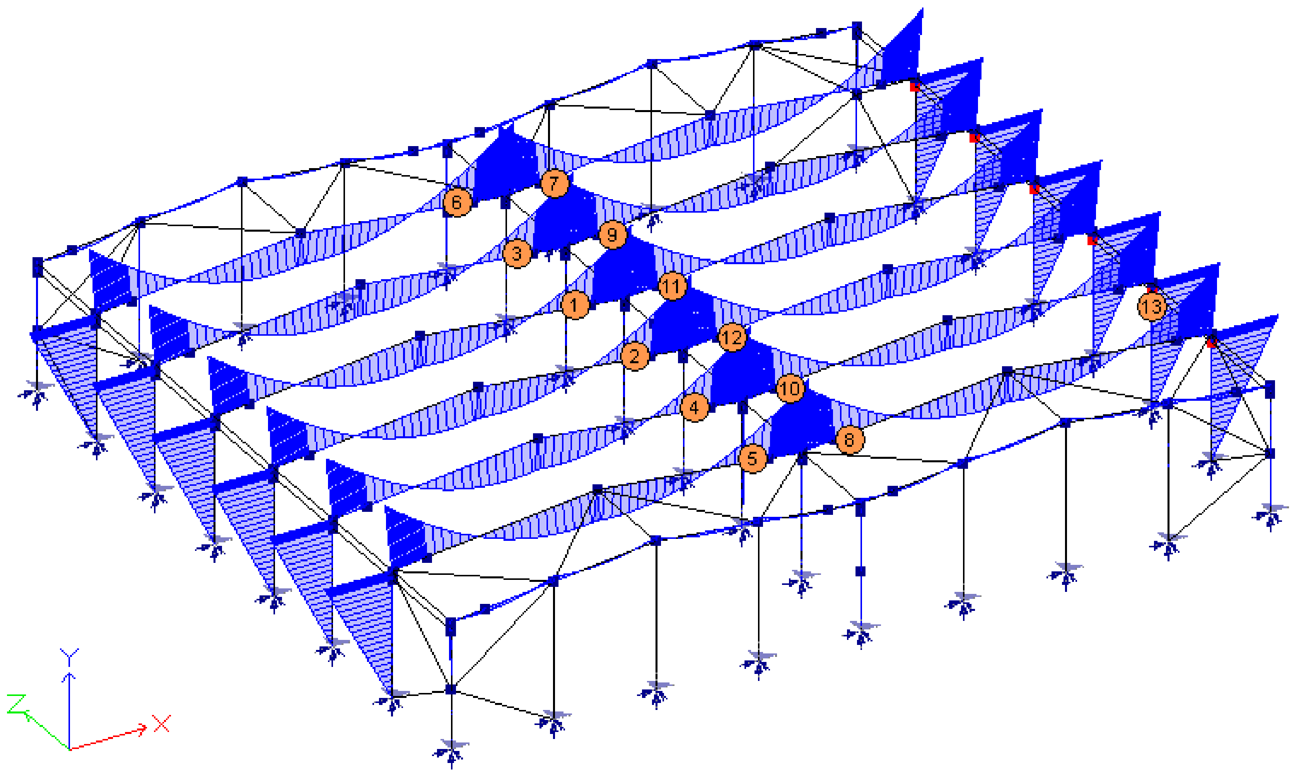

Fig 15: Two span frame moments for load combination 1: 1.4DL + 1.6IL + NL and plastic hinge formation at collapse. Non-linear elastic-plastic analysis with Pdelta effects.

Two span `hit and miss' frame

After exploring single and two span `normal' portal frames in 3D, the next step is to examine `hit and miss' frames – ie frames with some internal columns omitted. For purposes of comparison, the `hit and miss' example is conveniently obtained from the `normal' two span valley frame by deleting four selected internal columns and supports to produce the frame shown in fig 16. The valley beam is made continuous through the internal columns with a 3 bay central span of 18 metres and end spans of 2 bays = 12 metres. The valley beams can also be designed as simply supported or with partial fixity (semi rigid connections) at the designer's discretion.

Note that fully rigid connections will usually require small haunches to develop the moments because extended end plate unhaunched connections would foul the valley gutter.

In this example the valley member has been positioned at the underside of haunch level for the purpose of frame analysis with idealised line member elements. It could also have been placed to join the nodes of the rafter intersections. There is no significant difference to the results.

Note that the orientation of the valley column in each end frame is changed from 0 to 90 degrees so that the flanges face the valley beam to provide some `wind portal' stiffness in the connection.

The practical considerations that led to the severing of the valley member for the `normal' two span frame analysis model do not apply to a `hit and miss' frame. Here the valley member is a primary beam of substantial section which cannot be severed for modelling purposes. Consequently the design of the roof bracing members is governed by dead and imposed load effects resulting from the closed ring load path system in the roof and 168 x 5 CHS sections are required in this case. Fig 17 illustrates the member axial forces for load combination 1.

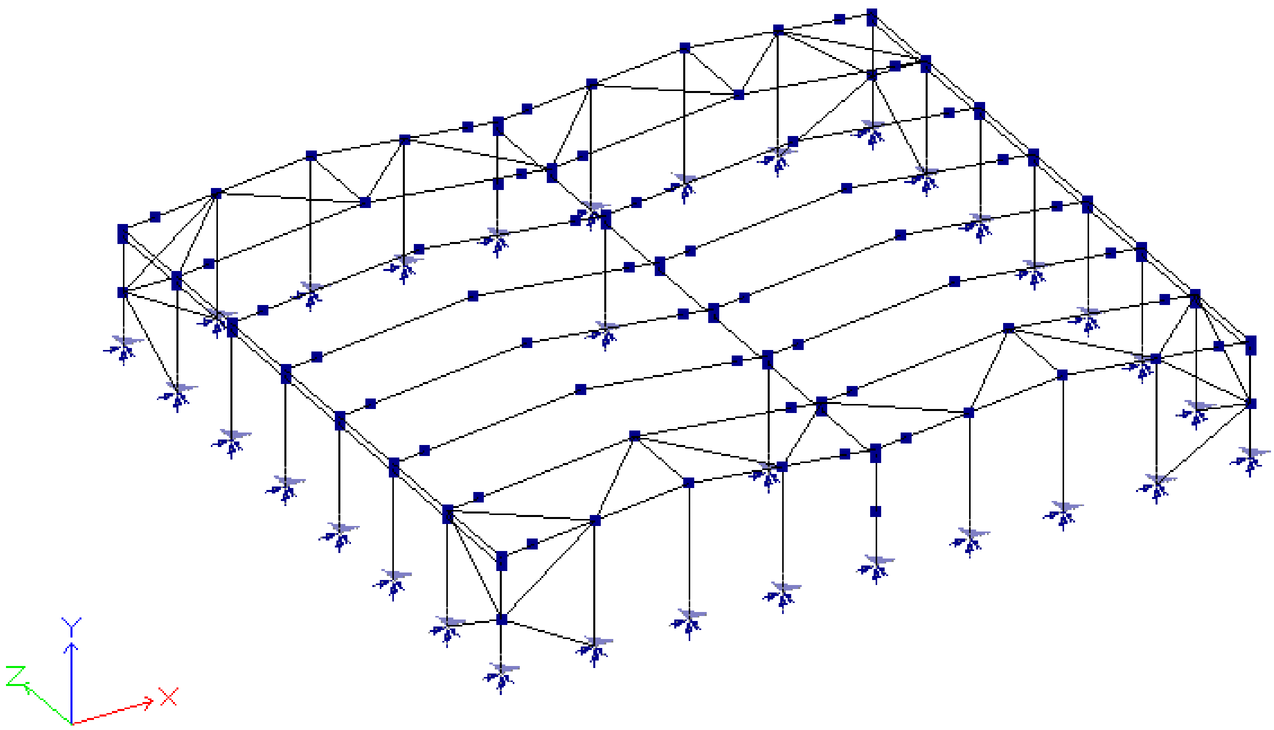

Fig 16 Stick model of two-span `hit and miss' frame with purlins and rails omitted.

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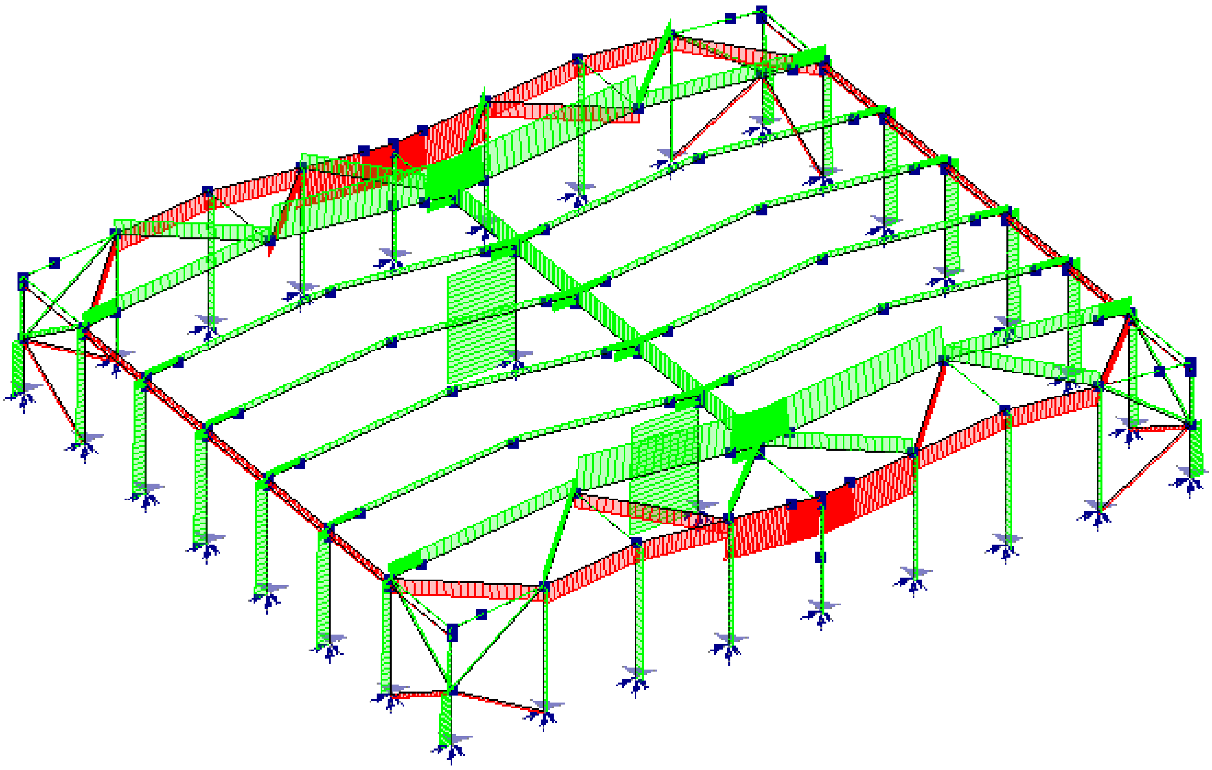

Fig 17: Two span hit and miss frame axial forces for load combination 1: 1.4DL + 1.6IL + NL

In this example the section size for the valley beam was arrived at quite simply by increasing it progressively until it did not control the collapse load factor. Fig 18 shows the hinge formation for load combination 1 with an inadequate 533 x 210 UB 92 section. The collapse load factor is 0.992.

Fig 19 shows hinge formation with 610 x 229 UB 101 continuous valley beam with hinges formed at the supports only at Lfc = 1.052. Fig 20 shows the hinge formation with a 686 x 254 UB 125 section which stays elastic and permits the full collapse mechanism of the central `miss' portal frame to develop at Lfc = 1.072. All these results were obtained by second order elastic-plastic analysis.

This `guess and check' method for establishing an initial valley beam size is quite efficient as the total 3D elastic-plastic frame analysis for several load combinations takes only seconds with current computer hardware and software. After an acceptable minimum collapse load factor has been obtained by 3D second order elastic-plastic analysis, the lateral stability of individual members including the valley beam spans can be checked in accordance with BS 5950-1:2000 using the SWMD module. This may lead to upsizing of some members which have widely spaced lateral restraints or none at all. In this case a check on the valley beam reveals that the mid length between the lateral restraints provided by the supported `miss' frames fails the BS 5950 lateral-torsional buckling checks so that the section must be increased or alternatively additional lateral restraints may be provided to subdivide the buckling length.

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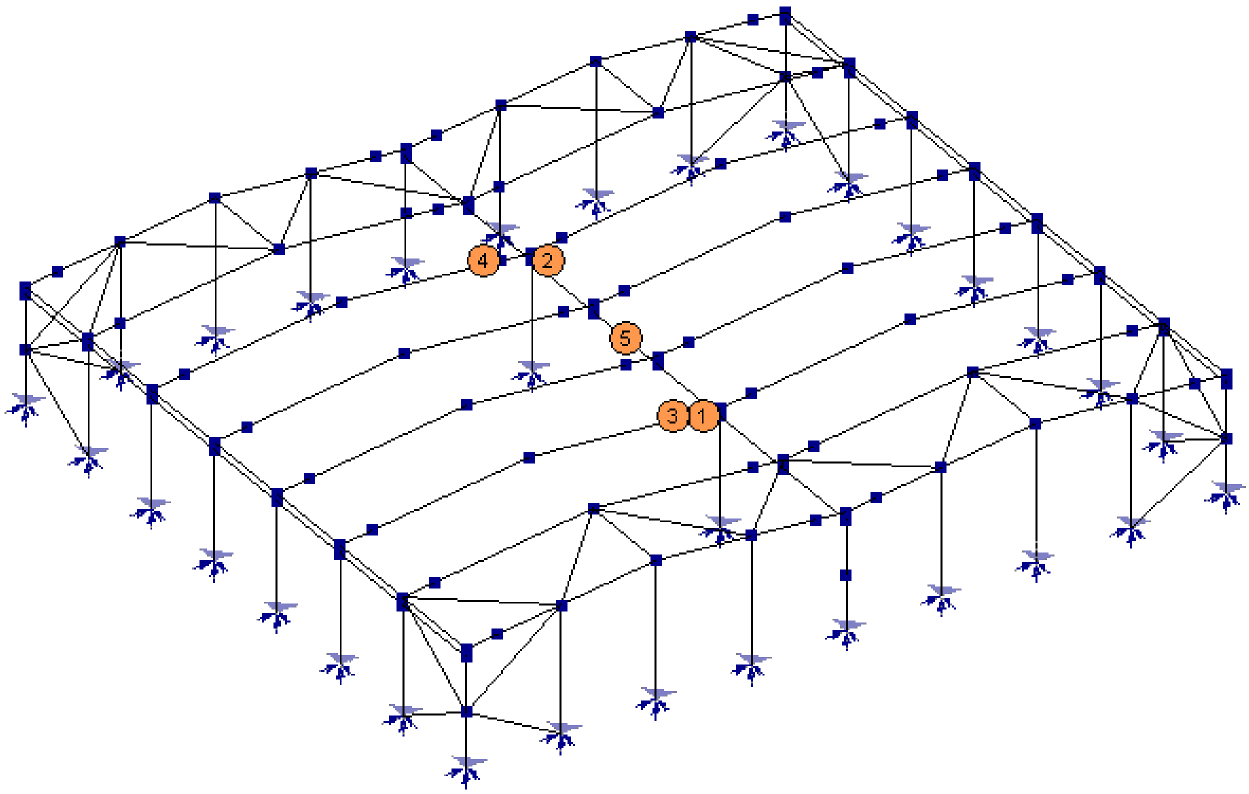

Fig 18: Two span `hit and miss' frame with inadequate 533x210 UB 92 valley beam.
Note three hinge collapse of the middle span of the valley beam with only two rafter hinges formed at Lfc = 0.992.

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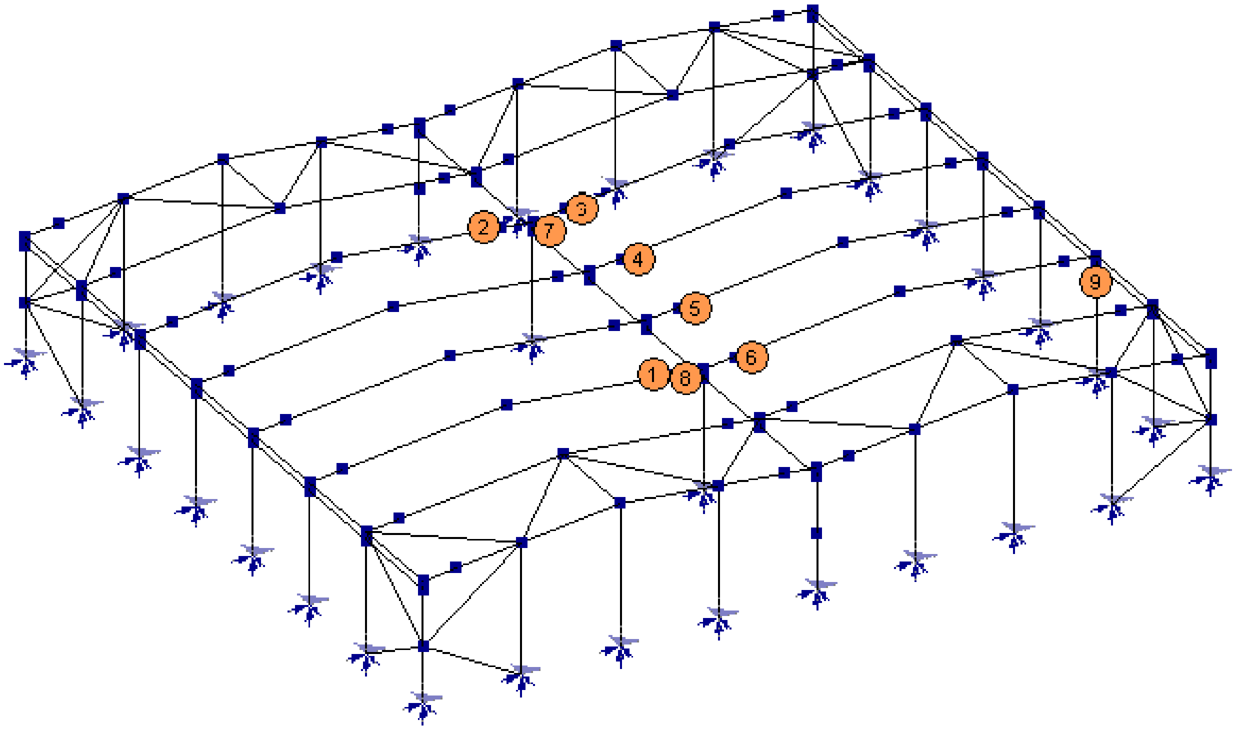

Fig 19: Two span `hit and miss' frame with `adequate' 610 x 229 UB 101 valley beam.
Note plastic hinges formed at supports of the middle span of the valley beam with six rafter hinges and one column hinge formed at Lfc = 1.052

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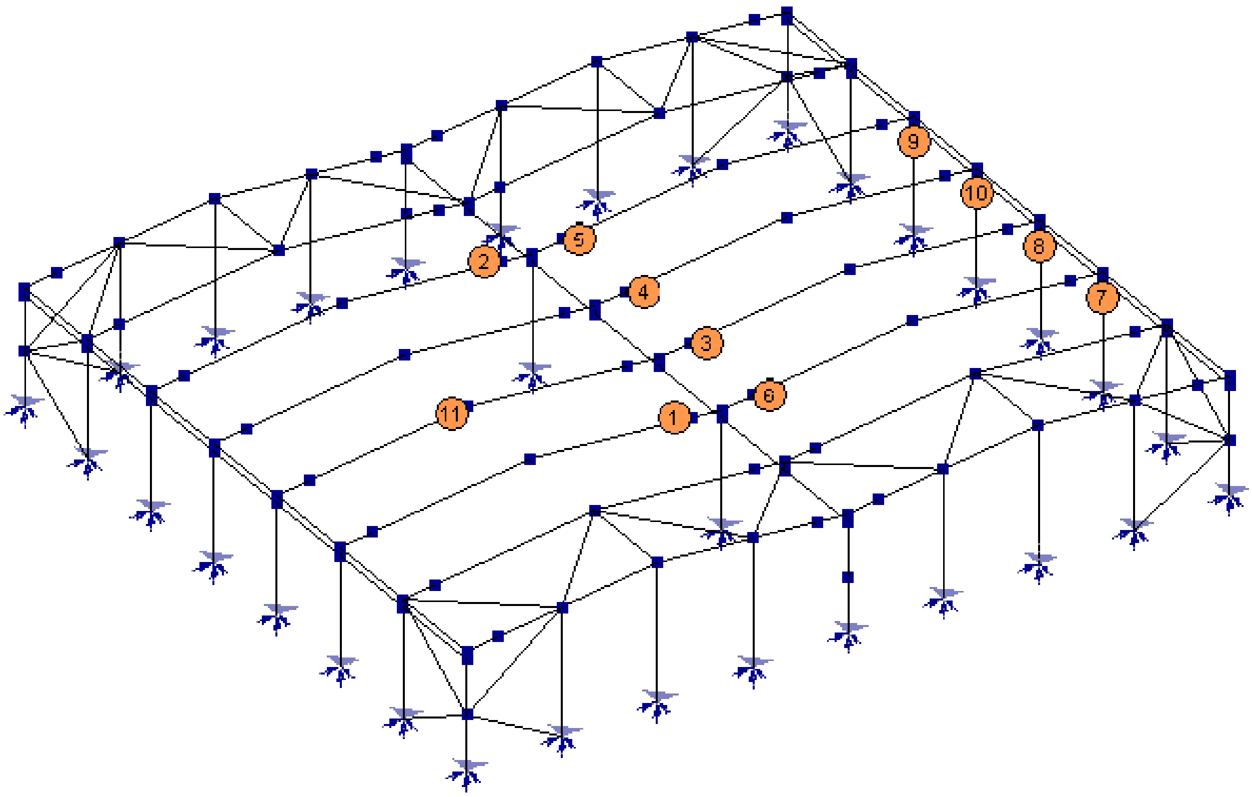

Fig 20: Two span `hit and miss' frame with `more than adequate' 686 x 254 UB 125 valley beam. Note no plastic hinges formed in the valley beam permitting a full collapse mechanism of the central `miss' portal frame at Lfc = 1.068

Before leaving this example it is instructive to consider 2D analysis of the component main frames.

Two span `miss' frame 2D analysis

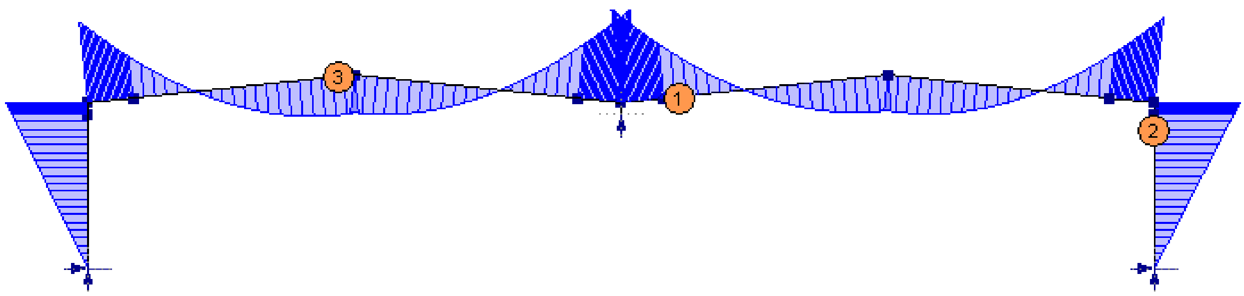

Fig 21: Two span `miss' frame moments for load combination 1: 1.4DL + 1.6IL + NL showing plastic hinges formed at collapse at Lfc = 1.106 (non linear) and 1.193 (linear)

This frame was created simply by removing the internal column member below the haunch and the base node. Note that the valley beam support is assumed to be rigid in the vertical direction and free in the horizontal.

Application of finite vertical spring stiffness has little effect on the design. Application of any horizontal support would have substantial effect on sway stiffness but would need to be backed up by suitable bracing in the 3D model and the real structure. This has not been done here.

Note that the valley support is shown applied to the node representing the underside of the haunch. Removing the haunch stub vertical member from the model and placing the support at the rafter intersection node has negligible effect on stability or elastic plastic analysis results.

The elastic critical load factor for this frame is 3.3 which may be compared with 4.7 obtained for the `normal' two span frame examined earlier. Obviously removal of the internal column affects the result.

Second order effects reduce the plastic collapse load factor from 1.193 to 1.106 but the mechanisms are practically the same.

Two span `hit' frame 2D analysis

This frame is similar to the `normal' two span 2D frame except that the reactions from the valley beams obtained from the 3D analysis have been added to the valley node as point vertical loads. This has only been done for load combination 1 for the purposes of the example. It will be appreciated that adding the valley beam loads for all load combinations would be extremely tedious which is the main attraction of 3D analysis – load transfer is automatic!

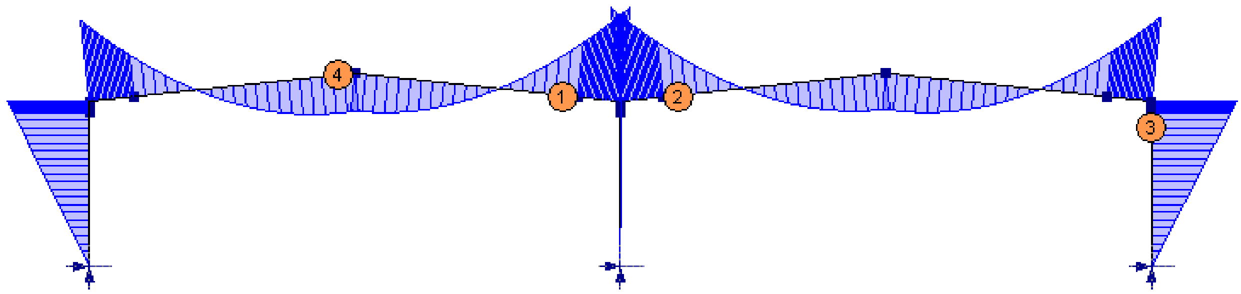

Fig 22: Two span `hit' frame moments for load combination 1: 1.4DL + 1.6IL + NL showing plastic hinges formed at collapse at Lfc = 1.197. Linear elastic-plastic analysis.

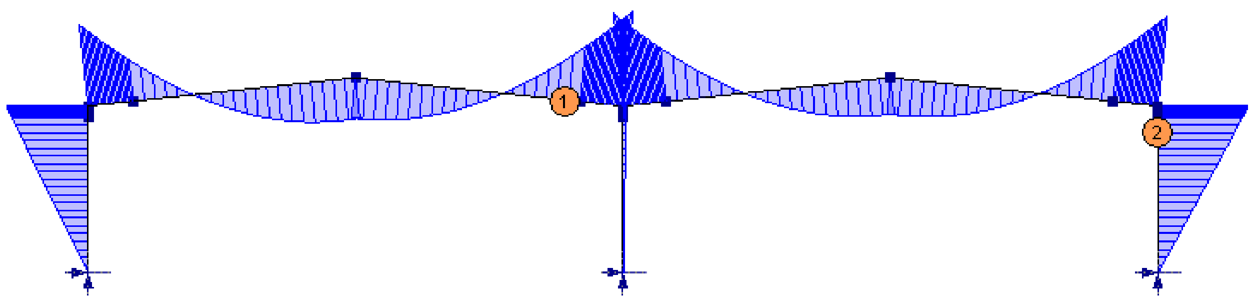

Fig 23: Two span `hit' frame moments for load combination 1: 1.4DL + 1.6IL + NL showing plastic hinges formed at collapse at Lfc = 1.043. Non-linear elastic-plastic analysis including Pdelta effects.

The effect of the valley beam reactions on the internal column stiffness is dramatic - reducing the elastic critical load factor for load combination 1 from 4.7 for the `normal' frame to 2.3 for the `hit' frame. The diagrams above show the effect on the collapse behaviour. Linear elastic-plastic analysis predicts the formation of four hinges at collapse Lfc = Lf4 = 1.197. However second order analysis with Pdelta effects shows that the frame becomes unstable when the second hinge forms in the right hand column at Lf2 = 1.043.

This is a good example of `deteriorated stability' resulting from the interaction of plasticity and high axial force as first reported by Wood and others in the 1950's[1]. The reduction in collapse load factor due to second order effects (13%) for this `hit' portal frame is less than typical multi storey frames but greater than `normal' portal frames.

Comparing 2D and 3D results for the hit and miss frames.

The elastic critical load factor of the 3D frame (2.7) is roughly midway between the value calculated for the `hit' frame (2.3) and the `miss' frame (3.3). The comparison of elastic-plastic analysis results is based on the valley beam size 610x254 UB 125 so that this member remains elastic and takes no part in the hinge mechanism.
For brevity only load combination 1: (1.4Dead + 1.6Imposed + Notional loads left to right) results are compared.

Linear elastic-plastic analysis	2D `miss' frame	2D `hit frame	3D frame
First hinge formation load factor	0.913	0.909	0.906
Collapse load factor	1.193	1.197	1.215
Non-linear elastic-plastic analysis			
First hinge formation	0.896	0.888	0.886
Collapse load factor	1.106	1.043	1.068
Reduction in collapse load factor (due to second-order Pdelta effects)	7.3%	13.1%	13.0%

Conclusions

It is proposed that 3D analysis of portal frame structures using modern computer software can substantially increase designer productivity provided that there are no `hidden snags.' This article has reviewed some examples of single span, two span and `hit and miss' frames to demonstrate modelling problems that can occur and their solution in order to accelerate the learning process for newcomers. The following main points have emerged:-

1: The purlins and sheeting rails are usually omitted from the 3D analysis model for simplicity to reduce computation time and keep data manageable. However, their lateral restraint effects on the main steel members must be dealt with not only in steel member design checks but also in any second-order analysis including Pdelta effects. This can be done by application of the setting "Pdelta effects applied to major axis only" to those columns, rafters and end posts which are restrained by purlins and rails. Failure to apply the `major axis only' setting will also lead to spurious low values of the elastic critical load factor.

2: It is common to provide roof and side bracing systems at or near both ends of a portal frame building. When these are included in a 3D frame analysis, the results indicate that a closed – ring axial load path is set up in the roof planes. The ring structure is very significantly stiffened by the inclusion of longitudinal ridge and valley members in the analysis. The ring system has the effect of restraining eaves spread of the penultimate main frames under vertical dead and imposed loading at the expense of generating forces in the bracing which can considerably exceed the wind forces for which the bracing is traditionally designed.

Perhaps these effects do not develop in real structures because of yield and flexibility of the bracing connections which cannot easily be modelled directly. Alternatively it may be that no real portal structure has ever been subjected to sufficient imposed loading to cause visible distress. In the absence of test data, the design engineer must decide whether to accept the results of the stiffness analysis and consistently design members and connections to suit or make adjustments to the analysis model to simulate the behaviour assumed in manual design. The following are some ways of minimising the `closed ring' effect in the analysis model:-

a: Include only one set of roof bracing. This has the disadvantage that additional tie/strut members have to be added to transmit end post wind reactions to the remote bracing system.

Alternatively:-

b: Exclude ridge members from the 3D analysis model

c: For normal multi-span portals, omit or sever one of the valley members in each line or reduce its stiffness properties. This has the disadvantage of producing spurious excessive deflections in the longitudinal direction.

3: If it is intended to carry out elastic-plastic analysis either with or without second order Pdelta effects, the haunches must be modelled using separate rafter haunch and column haunch sub-members with unlimited plastic resistance moments. This is done automatically for 2D frames generated by CADS SMART Portal.

4: Column base supports must be modelled with torsional restraint (restraint to rotation about the column length axis) even if `pinned'. This is in order to avoid spurious `spinning' instability when a plastic hinge is formed in the column at underside of haunch. This is taken care of automatically by CADS SMART Portal or by selection of the `Pinned base' support type in CADS A3D MAX.

5: For competitively designed structures which are expected to develop plastic hinges in the main columns under the design ultimate loads, provide longitudinal restraint members joining the nodes at underside of haunch and connected to the end frames. These restraint members represent the stayed sheeting rail arrangement commonly provided for column stability and will prevent a local mechanism collapse of pin based columns with rotation about the weak axis.

6: It will usually be necessary to relax or ignore the normal deflection criteria for roof bracing members which are located in the end bays. This is because they have to accommodate the differential deflections of the penultimate and end frames. Articulation of the normal pin joint equivalent fin plate connections will ensure this occurs without adverse effects.

7: `Hit and miss frames' are more sensitive than `normal' portal frames to Pdelta effects and relatively low elastic critical load factors are to be expected. Careful second order elastic plastic analysis can nevertheless justify similar or only slightly increased section sizes compared to normal portal frames.

Definitions

Load factor (Lf)

A multiplier applied to a design combination of loads to indicate the margin of safety against failure or attainment of a limit state. In general load factors less than 1.0 are inadequate and greater adequate. However higher load factors are required for some effects eg: elastic critical buckling.

Collapse load factor (Lfc)

A multiplier which applied to a design combination of loads would result in collapse of all or part of the structure. Collapse may be due to formation of a plastic hinge mechanism, member buckling or an interaction of the two involving buckling with an insufficient hinges to for a complete plastic hinge mechanism.

Elastic critical load factor (Lambda crit.) (Lcr)

A multiplication factor which if applied to a design ultimate load combination would result in elastic frame instability if the material did not yield. It is used in BS 5950 as a measure of the sensitivity of a frame to Pdelta effects in sway mode. Lcr > 10 not sensitive. Lcr = 4 to 10 moderately sensitive. Lcr < 4.0 second order elastic-plastic analysis essential. A3D MAX and some other programs calculate Lamda crit. by repeated second order elastic analyses with incremented load factors. Unfortunately instability may be due to non- sway modes in which case the BS 5950 guidance does not apply. However full elastic-plastic second order analysis is always valid and should be applied if in doubt.

Linear elastic-plastic analysis

Incremental linear elastic analysis of a structure under a load combination. At each increment of the load factor the next plastic hinge is predicted until sufficient hinges are inserted to trigger a plastic collapse mechanism.

Non linear elastic-plastic analysis

Similar to linear analysis but Pdelta effects are included at each stage and collapse may be due to a full plastic hinge mechanism or to instability with insufficient hinges to form a complete hinge mechanism.

Hit and miss framing

Portal frame structure of two or more spans in which some of the internal columns are omitted and replaced by longitudinal valley, purlin or ridge beams which transfer load to the remaining columns.

Reference

Wood, R H, `The stability of tall buildings', *Proc. of the Institution of Civil Engineers, vol 11, 1958, Paper no 6280 pp 62-102*

END.