https://doi.org/10.1061/(ASCE)0733-9445(1999)125:8(870)

Paper Title: Three-dimensional Modelling of Steel Portal Frame Buildings

Authors: Mahen Mahendran[1] and Costin Moor[2]
Physical Infrastructure Centre, School of Civil Engineering
Queensland University of Technology
Brisbane, QLD 4000, Australia

Abstract

The realistic strength and deflection behaviour of industrial and commercial steel portal frame buildings is understood only if the effects of rigidity of end frames and profiled steel claddings are included. The conventional designs ignore these effects and are very much based on an idealised two-dimensional frame behaviour. Full scale tests of a 12 m x 12 m steel portal frame building under a range of design load cases indicated that the observed deflections and bending moments in the portal frame were considerably different to those obtained from a two-dimensional analysis of frames ignoring these effects. Three-dimensional analyses of the same building including the effects of end frames and cladding were carried out and the results agreed well with full scale test results. Results clearly indicated the need for such an analysis and testing to study the true behaviour of steel portal frame buildings. It is expected that such a three-dimensional analysis will lead to lighter steel frames as the maximum moments and deflections are reduced.

1- Associate Professor of Civil Engineering and Director, Physical Infrastructure Centre, School of Civil Engineering, Queensland University of Technology, Brisbane QLD 4000, Australia.

2- Former Postgraduate Research Scholar, Senior Systems Engineer, Queensland Rail, 305, Edward Street, Brisbane QLD 4000, Australia.

1. Introduction

Traditional analysis and design practice does not consider three-dimensional structural behaviour of steel portal frame buildings. Normally designers consider only an internal frame (a two-dimensional analysis), an end frame, purlin and girt systems and profiled steel claddings, all of which are designed independently based on simple assumptions of load transfer from one to another. A code-compliant structural framing can be and often is designed as a collection of two-dimensional vertical and horizontal planes of framing. However, there is no doubt that the three-dimensional behaviour of the structure could not be neglected (Cohen, 1994). The main parameters affecting both the strength and deflection behaviour of steel portal frame buildings under given load conditions are

- Cladding action, rigidity of end frames and bracing
- Joint action (Base fixity, Knee/Haunch flexibility)

Ignoring the effects of these in the analysis of steel portal frames will not give the correct design action effects including deflections, in particular for lateral loading. The presence of stiffer end frames and the diaphragm action of profiled steel roof and wall claddings (first parameter above) causes part of the loads to be transferred to the end walls. As a result, calculated maximum frame stresses and deflections are much less than in the bare frame. This cladding action of resisting in-plane deflection by shear is known as stressed skin or diaphragm action and had been extensively studied by Davies and Bryan (1982) and his co-workers. Their results are used in the design of buildings in the UK, Europe and the USA. However, their work is limited to valley-fixed claddings that are commonly used in these countries under their specific conditions.

In Australia and its neighbouring countries, roof claddings are crest-fixed, and therefore Davies and Bryan's (1982) results cannot be used. Designers also take the view that crest-fixed claddings do not have adequate strength and stiffness to carry the required shear forces in addition to the uplift forces caused by wind actions. Therefore the ability of roof and wall cladding panels or end frame panels to resist the lateral forces due to wind is ignored in the design of buildings. Steel portal frames are designed as two-dimensional bare frames assuming that all the loads are carried by the frames only. However, the claddings do carry in-plane shear/racking forces whether designers acknowledge it or not. Recent tests have shown that even the crest-fixed steel claddings could act as diaphragms while still carrying the transverse forces due to wind action (Mahendran, 1994). Therefore it is necessary that cladding action and end frame rigidity be taken into account in the design of steel portal frame buildings using a three-dimensional modelling approach. Similarly joint action also should be taken into account.

It is considered that an inadequate two-dimensional model ignoring the effects mentioned above will affect the serviceability criteria more than the ultimate strength criteria. In recent times, improved technologies (materials, fabrication and construction methods and analysis and design tools) have in most cases led to lighter buildings for which serviceability criterion has become more critical. But neither the analytical serviceability model nor their limits have been refined which has caused an imbalance between the design quality for the ultimate and serviceability conditions. Recent research by Bernuzzi and Zandonini (1993) and Saidani and Nethercot (1993) in Europe has been investigating these issues relating to serviceability, in particular the limits and the need for more realistic three-dimensional analysis models.

In this investigation on steel portal frame buildings with crest-fixed claddings subject to both gravity and lateral loading, it was considered necessary to conduct a three-dimensional computer modelling and full scale tests of the entire building in order to include the effects mentioned above and to study the true behaviour of steel portal frame buildings. A 12 m x 12 m steel portal frame building with a conventional crest-fixed steel cladding system was used in this investigation. This paper presents the details of the three-dimensional modelling and full scale tests, and their results.

2. Effects of Cladding

2.1 Principle of Stressed Skin Action

Figure 1 shows how horizontal forces, acting on a pitched roof portal frame are carried by the roof cladding action to the more rigid end frames which are stiffened in their own planes by bracing or claddings acting as shear diaphragms. The cladding transfers part of the spread or sway forces from the internal frames to the end frames. Based on the sway and spread flexibility of portal frames and the direction of the forces, the steel cladding panels tend to resist any in-plane displacement together with the supporting purlins or girts like a deep plate girder, spanning between end frames. Edge members (purlins) act as flanges taking the axial tension or compression forces, and cladding acts as a web carrying the shear forces while the end frames take the reactions. This means that the relative horizontal displacement between the frames is dependent on the cladding panel. In general, the end frames are more rigid than internal frames, and thus their deflections are relatively small compared with the internal frames. This causes the panels between the end frames and the first internal frame to be more critical from both strength and deflection points of view. Further, because of this combined action, the deflections of internal frames and thus bending moments are greatly affected by both the frame's sway/spread flexibility and cladding's shear flexibility. Therefore in a computer analysis attempting to model the true strength and deflection behaviour of the entire building, the claddings have to be modelled just like the frames, purlins and girts. However, it is rather difficult to model claddings using conventional frame analysis programs such as SPACEGASS or MICROSTRAN. Therefore it was decided to model the claddings using the equivalent truss theory discussed in the next section.

2.2 Equivalent Truss Member Theory

This theory was used by Davies and Bryan (1982) and Bernuzzi and Zandonini (1993) in their work on steel frames with valley-fixed claddings. According to this theory, a truss member connecting the two opposite corners of two adjacent frames (Figure 8) simulates the effect of a cladding panel in shear.

In the equivalent truss theory, it is considered that the connection between the truss member and the frame is a perfect hinge and the truss member simulating cladding action is acting only in tension. A simple formula for the cross-sectional area A of the equivalent truss member is derived by taking into account the applied force (F), the displacement (D) and the panel dimensions (a x b) (see Figure 2).

Extension in the equivalent truss member $D` = \dfrac{\dfrac{F}{\cos\alpha}\dfrac{a}{\cos\alpha}}{EA} = D\cos\alpha$ (1)

where F = In-plane shear force on the panel

D = In-plane shear deflection of the panel

E = Modulus of elasticity

A = cross-sectional area of the equivalent truss member

α, a = Panel geometry parameters as shown in Figure 2

Since the shear flexibility of the cladding panel c is the shear deflection per unit shear load, equal to D/F, Equation (1) leads to the following.

$$A = \frac{a}{cE\cos^3\alpha} \tag{2}$$

Equation (2) can be used to determine the equivalent member cross-sectional area A, provided the shear flexibility ‘c’ of the cladding panel is known. It must be noted that ‘c’ is the total shear flexibility of the panel including the effects of flexibility due to the sheet deformation and that due to all the screw-fastened connections. Therefore it depends on the sheeting profile (thickness, geometry and modulus of elasticity), fastening arrangements, and aspect ratio of the panel.

3. Shear Tests of Crest-fixed Claddings

For valley-fixed European/UK claddings, Davies and Bryan’s (1982) design expressions can be used to calculate ‘c’. These claddings fastened with self-drilling or self-tapping screws at every or alternate valleys (troughs) also had seam or lap fasteners between sheets at a spacing not exceeding 500 mm. They also included shear connectors between sheeting and rafters in addition to purlin-sheeting fasteners. The recent code of practice for stressed skin design (BS5950: Part 9, 1994) gives details of the basic requirements for stressed skin design using light gauge profiled steel sheeting and recommended design procedures.

In this investigation where crest-fixed claddings were used without lap fasteners and shear connectors, the design expressions given in Davies and Bryan (1982) and BS5959: Part 9

(1994) are not applicable. Therefore full scale shear/racking tests were conducted to determine the ‘c’ values of crest-fixed steel cladding panels. In order to model a typical roof panel in the test building, a 6 m x 6 m crest-fixed trapezoidal cladding was tested as shown in Figure 3. The test cladding had a base metal thickness of 0.42 mm and was made of a high strength steel (G550 with a minimum yield stress of 550 MPa). It was fastened at every crest with No.14 (6.4 mm diameter) x 50 mm self-drilling screw fasteners to 200Z16 purlins at 1.1 m spacing.

The test arrangement used was similar to that used by Davies and Bryan (1982) and recommended by BS5950: Part 9 (1994). It included two 125 x 125 x 6 SHS rafters connected to seven Z20016 purlins as used in the full scale portal frame tests (see Figure 3 (b)). One of them was bolted to the slab using 30 mm diameter bolts and a steel support whereas the second rafter had two roller supports at each end, which allowed free longitudinal movements, but prevented vertical movements. The purlins were connected to the two rafters via special joints, which allowed free rotation of the purlins when one of the rafters moved longitudinally under the shear/racking load. With this arrangement, the entire shear test rig was free to move until the steel cladding was fastened to the purlins. This ensured that the cladding carried the entire shear load applied to the free rafter by a hydraulic jack.

The applied shear load was increased until there were large shear deflections without any increase in the shear load. The failure was due to the tearing failure at the main fastener holes along the lap joints. This was expected due to the lack of lap fasteners and was one of the accepted ductile failure modes in BS5950:Part 9 (1994). The most important parameter required for this investigation, the total cladding shear flexibility coefficient, c, was obtained from the elastic part of the shear load versus shear deflection curves, and was 4.5 mm/kN.

Shear tests were also conducted on panels of different aspect ratios, different profiles and fastening systems. The ‘c’ value for the 6 m x 4 m wall cladding panel was found to be 3 mm/kN. These values have been used in the computer modelling of the full scale portal frame building (see Section 5 later).

Further details of shear tests and results are presented in Mahendran and Subaaharan (1995). Their results clearly indicate that even crest-fixed steel cladding systems have considerable shear strength and stiffness in contrast to the current design approach of ignoring the effects of these claddings. Further work must be carried out to develop suitable design formulae for the shear strength and stiffness of crest-fixed cladding systems.

4. Full Scale Tests of a Steel Portal frame Building

In the past, full scale testing of the entire building has been limited because of the associated complexity and cost. However, it is only through such investigations that new and optimum building systems, design models and design assumptions can be validated, and more importantly the true building behaviour can be studied. The last such work was carried out by Bates et al. (1965) and Bryan (1971) in England, but their work was limited to portal frames with valley-fixed claddings and gravity loading. Recent work (Dowling et al. (1982), Kirk (1986), and Davies et al. (1990)) has been mainly conducted to investigate the behaviour of modern portal frame systems and thus to study the effects of advanced technology and new building systems. To date, little research has been conducted on full scale portal frame buildings clad with crest-fixed steel sheeting and for wind uplift load cases.

The primary objective of the full scale tests was to determine the true three-dimensional portal frame building behaviour by including the effects of claddings and end gable frames under three different design load cases such as Live Loads, Cross wind load and Longitudinal wind load. The full scale test program included a series of tests to study the effects due to

1. Crest-fixed Profiled Steel Claddings (Unclad building versus Clad building)
2. Rigidity of end Gable Frames (Unbraced versus Braced frames)
3. Base fixity (pinned versus normal bases)

Therefore tests were carried out on unclad and clad buildings including unbraced and braced end frames, and columns with pinned and normal bases.

4.1 Test Building

The test building chosen for this project was a steel portal frame building consisting of three steel frames with a 12 m span at 6 m centres representing a typical medium size industrial building. This resulted in approximate plan dimensions of 12 m x 12 m for the test building. The column height at eaves was 4.2 m. The roof pitch was 5°, which gave a ridge height of 4.72 m. The three frames were made of the new hollow flange beam 30090HFB33 manufactured by Palmer Tube Mills Pty. Ltd. (Dempsey, 1993a). The new 300 mm deep beam with two triangular hollow flanges and a slender web was made of 3.3 mm thick Grade 450 steel. The top flange was 90 mm wide and the side flanges were inclined at 30°. It was manufactured from a unique cold-forming and electric resistance welding process. The same test frames were also used in another project investigating the buckling performance of HFB frames (Heldt and Mahendran, 1995) as part of a large research project investigating the use of HFBs in portal frame buildings. The design and construction of the test frames was similar to that recommended by Dempsey and Watkins (1993). A conventional cladding system of trapezoidal sheeting with Z-purlins and girts was used (see Figure 3). The purlins and girts were 200Z16 sections with the former located at 0.9 – 1.1 m spacing and the latter at 1.7 m spacing. This gave a total of seven purlins on the roof and three girts on the wall. Two rows of conventional bridging were provided in both bays to both purlins and girts. Both roof and wall claddings were crest-fixed since it was decided to test only the crest-fixed claddings in shear for the purpose of including the cladding effects. Figure 4 shows the unclad and clad test buildings used in this investigation.

All three test frames were fixed to the strong floor using conventional base supports (referred to as normal bases) as shown in Figure 5 (a). Four M20 4.6/S bolts and a 350 x 150 x 16 mm plate were used for each column base. Although the conventional base supports are usually assumed as pinned supports in designs, the accuracy of this assumption must be investigated. Therefore, tests were also conducted with true pinned base supports shown in Figure 5 (b) in order to investigate the true behaviour of portal frames with pinned bases.

Typical HFB end plate connections recommended by Dempsey (1993b) were used for the ridge (440 x 130 x 16 mm end plate with 8 M16 8.8/S bolts) and knee connections (600 x 130 x 16 mm end plates with 8 M16 8.8/S bolts). Figure 6 shows the knee connection details. Dempsey (1993b) has shown these connections to be rigid (fixed) based on the moment versus rotation curves obtained from experiments on these connections. Therefore in this investigation, the knee and ridge connections were assumed to be rigid.

The behaviour of the bare frame building as well as the clad building was investigated under different load cases in this investigation (see Figure 4). As seen in the figure the end frames were not clad, but instead two 24 mm diameter steel rods were used. These rods were considered to provide the required end frame rigidity during the experiments.

4.2 Simulation of Test Loads

A total of 16 loading points (8 for each bay) split into five independent groups (two acting on side walls and three acting on roof) were used to load the test building (see Figure 7). The loads were simulated as concentrated loads. Servo-controlled hydraulic actuators were used to apply the required wind uplift and racking and gravity loads at the centre of the purlins and girts in ten steps. In each bay, combined use of six actuators and six loading yokes applied the required uplift loads to the purlins at 12 loading points (see Figure 7(a)). Similarly, in each bay two actuators and two loading yokes were used to apply the required racking loads to the girts at four loading points. Since this midpoint loading produced twice the bending moments in purlins and girts compared with that for the uniform load, purlins and girts were oversized compared with the frame members. This also enabled the frame to be more critical for the other project investigating the frame behaviour (Heldt, 1997). This purlin/girt oversizing had no effect on this project as the same purlin/girts were used in the cladding tests and computer modelling.

The hydraulic jacks loading the roof via purlins were connected to the strong floor as shown in Figure 7 (a). This allowed the wind uplift loads to be applied perpendicular to the roof as in a real situation. Similarly the racking loads were applied perpendicular to the side walls using lateral anchor beams as shown in Figure 7 (b). The joints between the members and the hydraulic loading system were designed as pinned connections. In this way, load distribution would occur as assumed.

The tests were conducted using the three important load cases "Live Load", "Longitudinal Wind Load" and "Cross Wind Load". The loads to be applied during these tests were first calculated for the test building using the Australian loading codes AS1170 Parts 1 & 2 (SA, 1989). Since the deflections and stresses in the frame were small at serviceability load levels, ultimate design loads were used as test loads for cross wind and longitudinal wind load cases. The wind uplift and racking test loads were based on a design gust wind speed of 41 m/s.

The frame analyses showed that the live load case produced smaller maximum moments in the central test frame than the wind load cases. Therefore the live load case was factored by 3.5 times in order to reach approximately the same maximum moments as in the other cases. In this manner, the central frame had the same level of maximum stress in all the test load cases. However, preliminary tests were conducted to ensure that the higher level of loading chosen did not cause premature failure of any of the test building components except for some possible localised yielding in the members or connections.

The dead load component of sheeting was not simulated by the hydraulic jacks as part of the three load cases for unclad building. However, in the case of clad building, the sheeting dead load was present. The computer analysis of the test building took into account these variations.

4.3 Test Results

Deflections and strains were monitored at various locations in the central and gable frames. Using the measured strains, stresses and thus moments due to the applied loading were determined. Tables 1 and 2 present these results in Section 5 for the three critical locations (ridge and two knees) of central and end frames. They are presented in a format that suits the discussion of results in Section 6. The strain gauges were located a small distance away from the exact knee and ridge positions for practical reasons. The building was finally tested to destruction as part of the other project on HFB frames, and these results are presented elsewhere (Heldt and Mahendran, 1995). Further details of the full scale test facilities are available in Heldt (1997).

5. Three Dimensional Analysis

The behaviour of the tested portal frame building was analysed using a conventional structural frame analysis program SPACEGASS. Analyses closely followed the sequence and procedure used in the full scale tests. In the first stage, all the three portal frames and the purlins and girts including their bridging were modelled using this program. In order to study the interaction between steel frames and claddings, the claddings were modelled using the "Equivalent truss member" theory described in Section 2. The cross-sectional areas of equivalent truss members for the 6 m x 6m roof panels and the 6 m x 4 m side wall panels were found to be 18.8 and 17.6 mm^2, respectively Therefore, steel members with a rectangular cross-section of 9 mm x 2 mm were used to model the roof and side wall claddings as shown in Figure 8. Two diagonal members were used for each panel, but the frame analysis program disabled one of them if it carried a compression force. This therefore enabled accurate modelling of a cladding panel by a single diagonal tension member using the equivalent truss theory. The braces on the end frames of the test building were 24 mm circular rods. Analytical model included these rods as they were used in the test building. Figure 8 shows the final three-dimensional model including frames, purlins and girts and their bridging, roof and wall claddings and end bracing rods.

The pinned and normal bases of the test portal frames (see Figure 5) were analysed as pinned and fixed bases, respectively. Since the degree of fixity with normal bases was not known, it was decided to model them as fixed bases. The HFB joint behaviour was observed to be rigid (Dempsey, 1993b), and therefore the knee and ridge connections of the frame were modelled as rigid joints. Although the degree of fixity of these joints can be easily changed using the frame analysis program SPACEGASS, no attempt was made in this investigation to study the effects due to different degrees of fixity of these joints. Future research will consider these.

As in the case of full scale tests, three load cases were considered in the analysis. The loads were distributed via the purlins and girts as in the test building. A two-dimensional analysis of the central frame subject to equivalent loads was also conducted in order to compare the effects of three-dimensional modelling. Analytical results from both two- and three-dimensional analyses are presented and compared with full scale results in Tables 1 and 2.

6. Discussion of Results

6.1 Unclad (Bare Frame) Building

Full scale results for the true pinned base supports are essential to verify the accuracy of the computer models used in this investigation. Comparison of test and analysis results was good in this case except for the maximum horizontal deflection at the central frame windward knee for cross wind load (see Tables 1 to 3). However, the agreement between full scale test and analysis results was not good for normal bases because the analyses assumed the normal bases to be fixed supports. Despite the possible support from the thicker concrete slab below the base plate, the normal base conditions used in the full scale test appeared to be closer to that of a pinned base. For the accurate modelling of normal base supports, further research is required.

In general, results indicate that a two-dimensional analysis considering only the central frame is adequate to model the behaviour of unclad building. This is because the use of a three-dimensional model has only led to minor changes to the deflections and moments in the frames (see Table 3). However, the following section illustrates the significant differences in the behaviour of unclad and clad buildings, two- and three-dimensional analyses and thus the need for three-dimensional analyses for clad buildings.

6.2 Fully Clad Building

6.2.1 Cross Wind Load

Pinned Bases

As seen in Tables 1, 2 and 4, the results from the three-dimensional analysis of a steel portal frame building including the effects of crest-fixed steel claddings and end frame rigidity agreed well with the full scale test results of the same building. For example, the analytical windward knee and ridge deflections for the central frame with pinned bases were 14 mm and 56 mm compared with full scale test values of 14 mm and 55 mm, respectively. Similarly, the corresponding analytical and full scale test moments at these locations were 31.7 and –25.2

kNm, and 29.7 and –24.9 kNm, respectively. Both analysis and testing showed that there were significant differences between the bare frames and fully clad frames when they were under lateral loads due to cross wind (compare P3 values in P3-B-C values in Tables 1 and 2). When the frame was fully clad, the critical windward knee moment was reduced from 44.6 kNm to 29.7 kNm in the tests and from 41.9 to 31.7 kNm in the analyses for the case of central frame with pinned bases. In the design of most of the common industrial and commercial buildings, cross wind load is the governing load case. Therefore it is important that the significant effects observed due to the presence of claddings and end frame rigidity are taken into account in the analysis and design of these buildings.

As seen in Tables 1 and 2, there was a noticeable load transfer from the central frame to the end gable frame when claddings and end frame bracing were added to the test building. At the same time, within each of these frames, the difference between the moments at the windward and leeward knees was reduced by a factor of 5.6 to 1.3 (based on test values). As seen in Table 4, two-dimensional analyses were unable to predict these observed changes to frame behaviour and therefore must be considered inadequate.

Table 5 shows the observed reduction and redistribution of maximum moments and deflections in the central frame due to the presence of claddings and end frame bracing based on both the full scale tests and computer analysis for the cross wind load case. As seen in Table 5, the reduction in critical windward knee moment from the analysis was 24%. It was associated with an increase in leeward knee moments. This resulting moment distribution is quite beneficial as it creates a uniform moment distribution within each of the central and end frames, leading to reduced design moments. Obviously all these are associated with corresponding changes to deflections of frames as seen in Table 1. Table 5 shows the significant reductions to the horizontal deflections at the central frame windward and leeward knees (more than 64%). These reductions are quite significant from a serviceability design viewpoint. The ridge moment was not included in Table 5 as there were only minor changes due to the addition of claddings and end frame bracing.

As seen in Tables 1 and 2, the results from full scale tests and three-dimensional analyses agreed reasonable well not only for unclad and full clad buildings, but also when only the cladding or the end frame bracing was added (see P3-C and P3-B cases in Tables 1 and 2). From these results, it can also be seen that adding the cladding alone did not cause significant reductions to deflections and moments. This is because the cladding was effective only when end frame bracing was added to the test building. This observation agrees well with the basic stressed skin behaviour (Davies and Bryan, 1982). The use of three-dimensional modelling will therefore be most useful when the buildings have both cladding and end frame bracing. Most industrial and commercial buildings will have some form of end frame bracing and hence stressed-skin action of steel cladding will be significant. Appropriate three-dimensional models including both cladding and end frame bracing could then be used to model the true behaviour and to gain the benefits due to the structural action of cladding and bracing.

A three-dimensional analysis of the test building with end frame bracing and roof cladding, but without side wall claddings was also conducted to study the influence of side wall claddings. However, the results indicated that side wall claddings have very little influence on the frame behaviour under cross wind loads. This implies that it may not be necessary to include side wall cladding in the analytical model for cross wind load.

Normal Bases

Although the unclad building model results showed that modelling normal bases as fixed supports is inadequate, the same approach was used for clad buildings. Agreement between the results from full scale tests and analysis was not as good as in the case of true pinned base supports (see Tables 1 and 2). Although there was noticeable stressed skin action in the test building, the cladding action was not as effective in reducing the windward knee moments and deflections as with the pinned base support case. In fact the results from the analysis assuming fully fixed bases produced considerably smaller reductions to windward knee moment. Since full scale test results gave larger reductions than the analysis, it confirms the previous observation that the normal bases used in the test building cannot be assumed as fixed supports. The normal base supports have to be considered to be equivalent to one between pinned and fixed bases. Earlier research by Melchers and Maas (1994) and Robertson (1991) have investigated the effect of normal base fixity in detail and their results support the findings from this investigation. Robertson's (1991) results were for an experimental building whereas Melchers and Maas (1994) used in situ observations of a 25-year old, 30 span steel portal frame building under wind load conditions. Their results showed that normal bases (referred to as "nominally pinned bases") can exhibit considerable rotational stiffness depending on the level of loading.

6.2.2 Longitudinal Wind and Live Loads

Since the behaviour of test building was similar under longitudinal wind and live loads, the results are discussed together in the same section. Deflection and moment results for longitudinal wind and live load cases are given in Mahendran and Moor (1997) and Moor

(1997). Comparison of results from the three-dimensional analyses and full scale tests show good agreement between the two results. This confirms the accuracy of three-dimensional modelling of clad and unclad steel portal frame buildings used in this investigation for longitudinal wind and live loads.

The results showed insignificant differences in the maximum deflections and moments when the base fixity was changed from pinned to fixed bases. Similarly, the end gable frame and claddings had little effect on the central or end gable frame behaviour under longitudinal wind and live loads. This agrees well with the observations made by Davies and Bryan (1982) that buildings with a flat roof slope (in this case only 5°) will not gain from the presence of claddings when vertical and symmetrical load cases are considered.

6.3 Overall Remarks

Although the claddings used on the test building were crest-fixed and not valley-fixed as expected for stressed-skin action, there was noticeable stressed-skin behaviour in the test building. The action of these crest-fixed steel claddings with braced end gable frames has produced a reduction of 33% in the maximum moment and 72% in the maximum horizontal deflection (Table 5). When the test building was subjected to repeated simulation of loads, the measured deflections and moments in the frames indicated no loss of strength or stiffness. This implies that the various components of the test building, in particular the crest-fixed steel sheeting and its connections remained elastic during the loads simulated in the tests. It must also be noted that the portal frame system was subjected to its ultimate design load levels, but the cladding remained an integral part of the structure. Therefore designers could safely assume continued stressed skin action of crest-fixed steel claddings during fluctuating wind loads and in three-dimensional models including cladding effects.

The current analysis and design of steel portal frame buildings based on a two-dimensional analysis of bare steel frames completely ignores the ever-present stressed-skin behaviour of claddings and thus the resulting spread of moments and deflections from windward to leeward locations and from internal frames to end gable frames. All these mean that it is not based on the true three-dimensional behaviour of the entire steel portal frame building system. In this process the current analysis and design procedures do not take advantage of the reduced moments and deflections mentioned above. Although this investigation considered only a specific steel portal frame building, it is believed that the above comments are equally applicable to all other steel portal frame buildings.

This investigation clearly indicates that it is very important that a three-dimensional analysis taking into account the effects of claddings and end frame rigidity is used in the design of steel portal frame buildings, particularly for lateral loading due to cross wind. This can be achieved using the method described in this paper, provided the shear flexibility values (‘c’) are available for various cladding systems. Another investigation has obtained these values for some of the commonly used claddings in Australia (Mahendran and Subaaharan, 1995).

Analytical study in this investigation considered only the idealised rigid knee joints and pinned and fixed base supports. Tests clearly showed that conventional base plates could not be assumed as fixed supports. They have to be considered equivalent to one between pinned and fixed bases. Further research should be conducted to improve the modelling of the knee and base plate connections in portal frame buildings. Such improved analysis will reduce further the gap between assumed and real behaviour.

7. Conclusions

This paper has described a research project on the three-dimensional behaviour of steel portal frame buildings. Effects of crest-fixed steel roof and wall claddings, end frame rigidity and appropriate conditions of base fixity were included in the study of a 12 m x 12 m steel portal frame building through full scale tests and three-dimensional computer modelling under a range of load cases such as longitudinal and cross wind loads and live loads. A series of tests were conducted on unclad and clad test buildings with and without end frame bracing and with pinned and normal base supports. Experimental and analytical results clearly showed the significant differences between the assumed two-dimensional behaviour and the true three-dimensional behaviour. The maximum moments and deflections were significantly reduced by the addition of claddings and end frame bracing, particularly under lateral loads due to cross wind. This demonstrated that three-dimensional modelling of steel portal frame buildings including the effects of both the structural and non-structural components is necessary for a realistic and efficient design from both strength and deflection points of view. The method of including the effects of claddings in the three-dimensional analysis using the equivalent truss member theory and shear flexibility values of claddings obtained from shear tests is explained in this paper. Details of shear tests of claddings, full scale building tests, two- and three-dimensional analyses and their results are also included in the paper.

8. Acknowledgements

The authors wish to thank the Queensland University of Technology (QUT) for providing financial support through the QUT Postgraduate Research Award (QUTPRA), Palmer Tube Mills Pty. Ltd for providing the test building, and the Physical Infrastructure Centre and the School of Civil Engineering at QUT for providing the necessary facilities and support to conduct this project.

9. References

Bates, W., Bryan, E.R. and El-Dakhaakhni, W.M. (1965) Full-scale Tests on a Portal Frame Shed, The Structural Engineer, 43, No.6, pp.199-208.

Bernuzzi. C. and Zandonini, R. (1993) Serviceability and Analysis Models of Steel Buildings, Proc. International Colloquium on Structural Serviceability of Buildings, International Association for Bridge and Structural Engineering, Goterborg, pp.195-200.

Bryan, E.R. (1971) Research into the Structural Behaviour of a Sheeted Building, Proc. of the Inst. of Civil Engineers, Vol.48, pp.65-84.

Cohen, J.M. (1994) The Northridge Warning: Has 3-D Design been lost? Civil Engineering, ASCE, Dec.

Davies, J.M., Engel, P., Liu, T.T.C. and Morris, L.J. (1990) Realistic Modeling of Steel Portal Frame Behaviour, The Structural Engineer, Vol.68, No.1, pp.1-6.

Davies, J.M. and Bryan, E.R. (1982) Manual of Stressed Skin Diaphragm Design, John Wiley & Sons, New York.

Dempsey, R.I. (1993a) Hollow Flange Beam Member Design Manual, Palmer Tube Technologies Pty. Ltd., Brisbane.

Dempsey, R.I. (1993b) Development of Structural Connections for Hollow Flange Beams, ME Thesis, Queensland University of Technology, Brisbane.

Dempsey, R.I. and Watkins, R.L. (1993) Hollow Flange Beam Portal Frame Buildings, Palmer Tube Technologies Pty. Ltd., Brisbane.

Dowling, P.J., Mears, T.F., Owens, G.W. and Raven, G.K. (1982) A Development in the Automated Design and Fabrication of Portal Framed Industrial Buildings, The Structural Engineer, Vol.60a, No.10, pp.311-319

Heldt, T.J. and Mahendran, M. (1995) Full Scale Tests of an HFB Portal Frame Building, Proc. of the Int. Conf. on Structural Stability and Design, A.A. Balkema, Sydney, Oct., pp.477-483

Heldt, T.J. (1997) The Use of Hollow Flange Beams in Portal Frame Buildings, PhD thesis, Queensland University of Technology, Brisbane.

Kirk, P. (1986) Design of a Cold-formed Section Portal Frame Building System, Proc. 8th Int. Conf. on Cold-formed Steel Structures, University of Missouri-Rolla, St.Louis, MO, pp.295-310.

Mahendran, M. (1994) Behaviour of Corrugated Steel roof Cladding under Combined Wind Uplift and Racking, Civil Eng. Transactions, Vol.36, No.2, pp.157-163

Mahendran, M. and Moor, C. (1997) Three-dimensional Modelling of Steel Portal Frame Buildings, Research Monograph 97-5, Physical Infrastructure Centre, Queensland University of Technology, Brisbane.

Mahendran, M. and Subaaharan, S. (1995) Shear Strength and Stiffness of Crest-fixed Steel Claddings, Research Report 95-22, Physical Infrastructure Centre, Queensland University of Technology, Brisbane.

Melchers, R.E. and Maas, G. (1994) Column Base Restraint Effect for a Steel Portal Frame, The Structural Engineer, Vol.72, No.4, pp.61-67

Moor, C. (1997) Three Dimensional Analysis of Steel Portal frame Buildings, ME thesis, Queensland University of Technology, Brisbane.

Robertson, A.P. (1991) A Study of Base Fixity Effects on Portal Frame Behaviour, The Structural Engineer, Vol.69, No.2, pp.17-24.

Saidani, M. and Nethercot, D.A. (1993) Structural Serviceability of Buildings, Proc. International Colloquium on Structural Serviceability of Buildings, International Association for Bridge and Structural Engineering, Goterborg, pp.111-118

Standards Australia (SA) (1989) Loading Codes: Wind Load, Sydney.

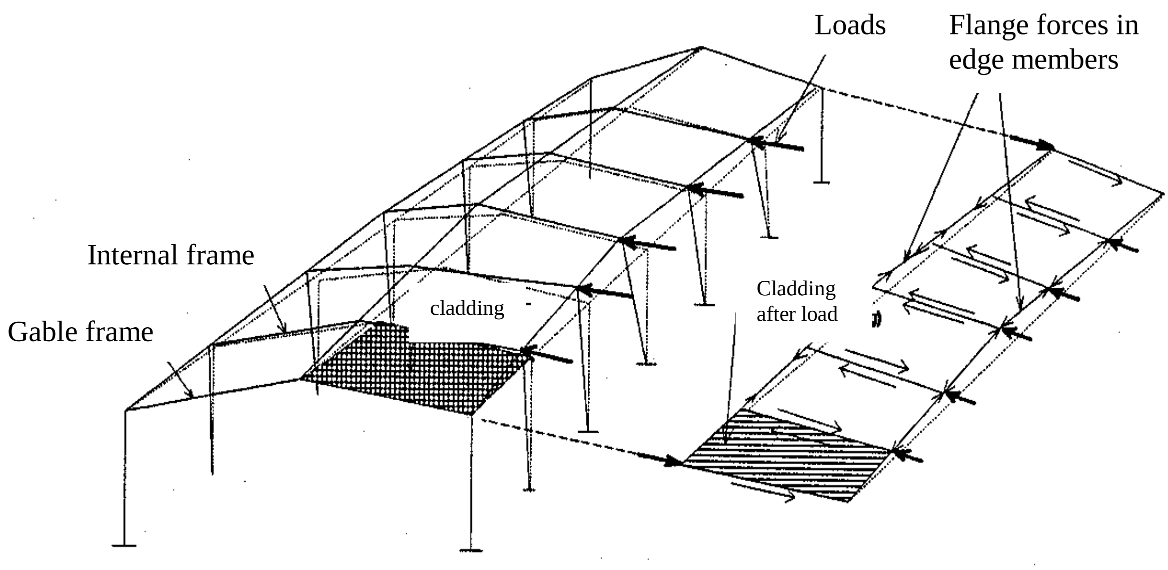
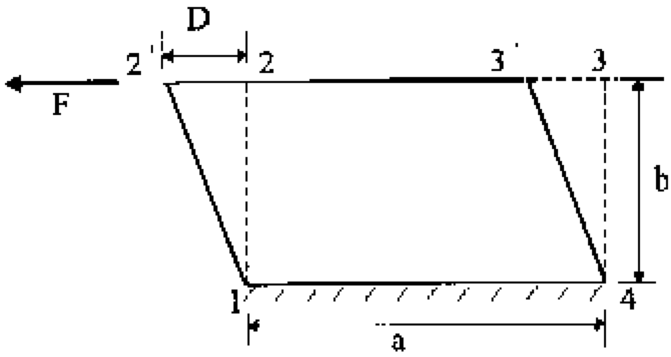
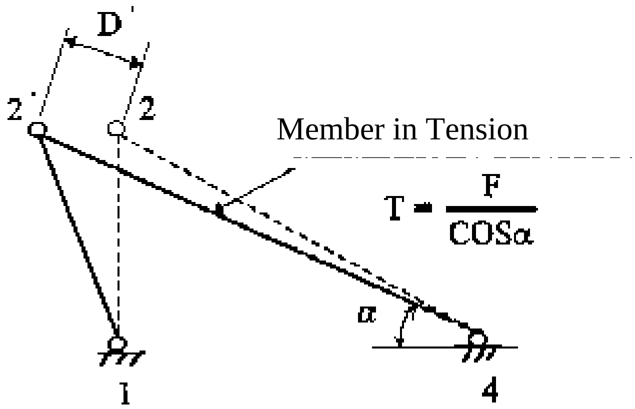


Figure 1. Stressed Skin Action



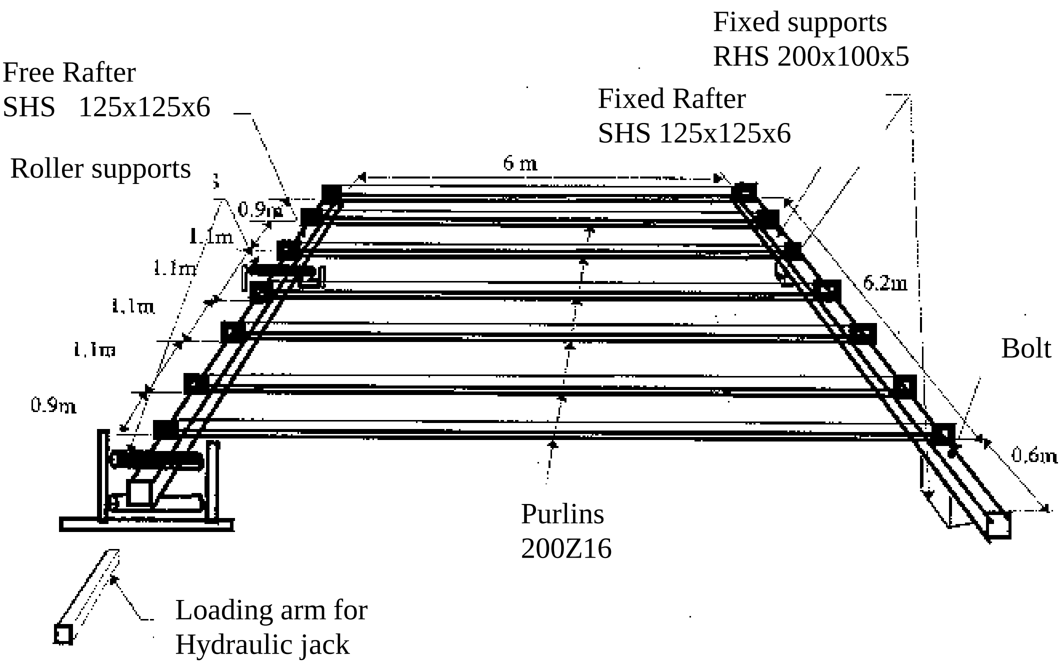
(a) Clad panel



(b) Equivalent truss member

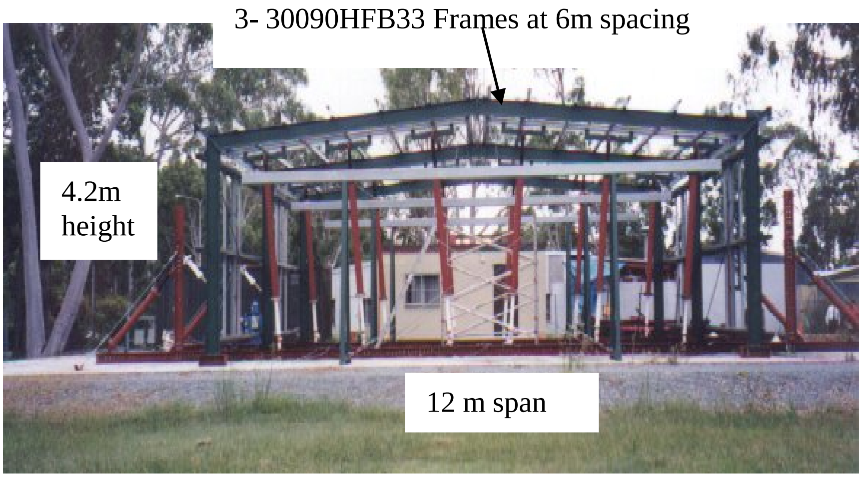
Figure 2. Equivalent Truss Member

(a) Shear Test



(b) Schematic diagram of Test Rig

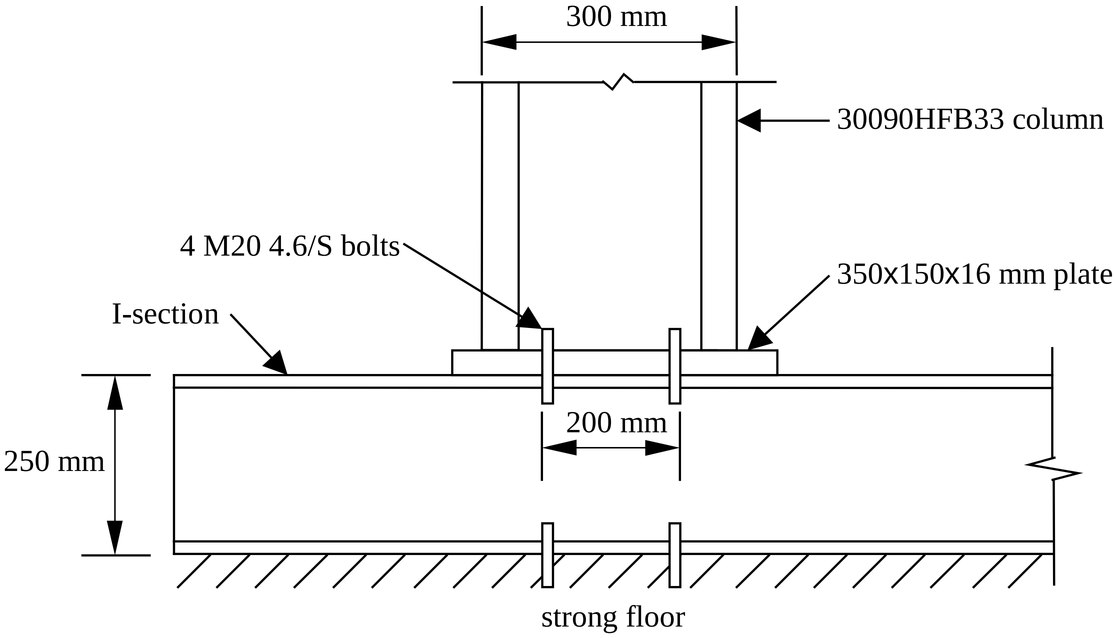
Figure 3. Shear Tests of Crest-fixed Claddings



(a) Unclad Building

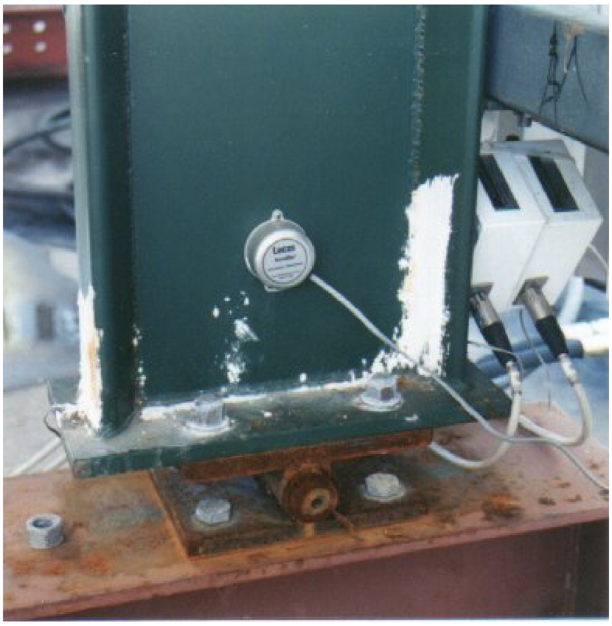
(b) Clad Building

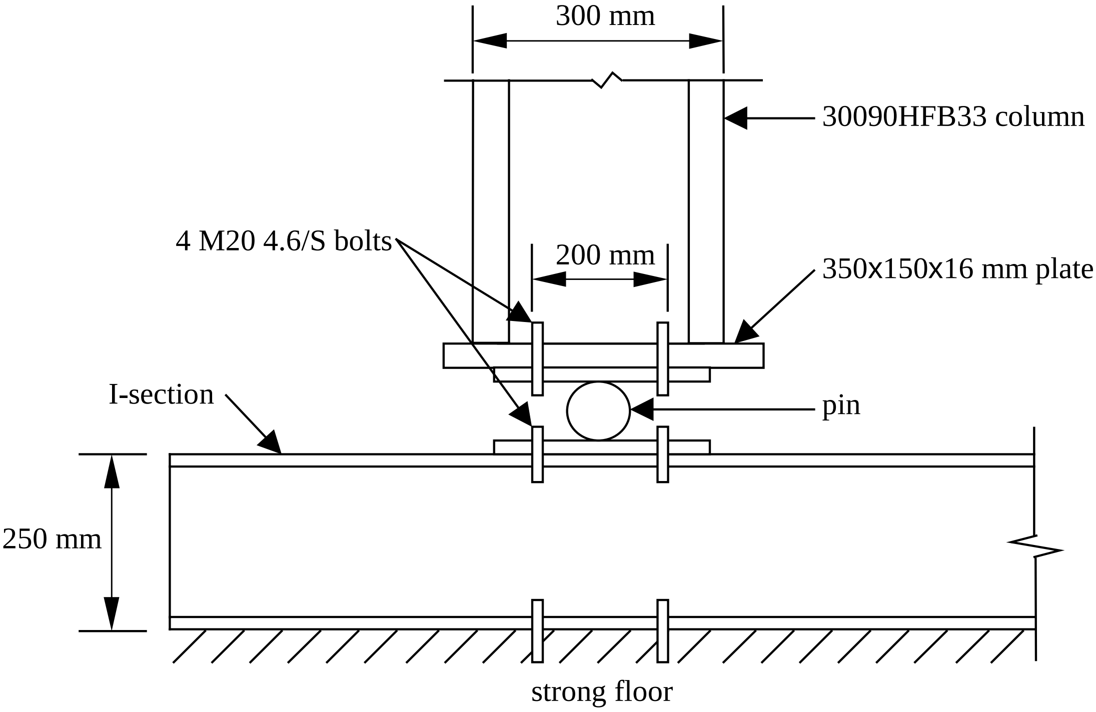
Figure 4. Test Building (12m x 12m)



(a) Normal Bases

Figure 5. Base Supports of Test Frames





(b) Pinned Bases

Figure 5. Base Supports of Test Frames

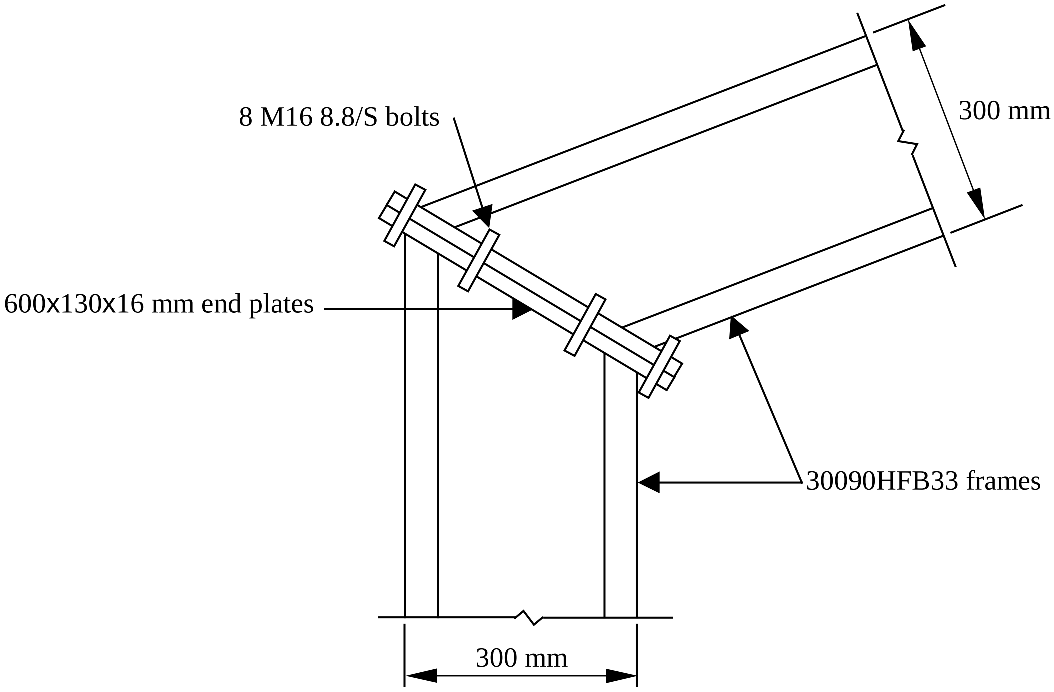
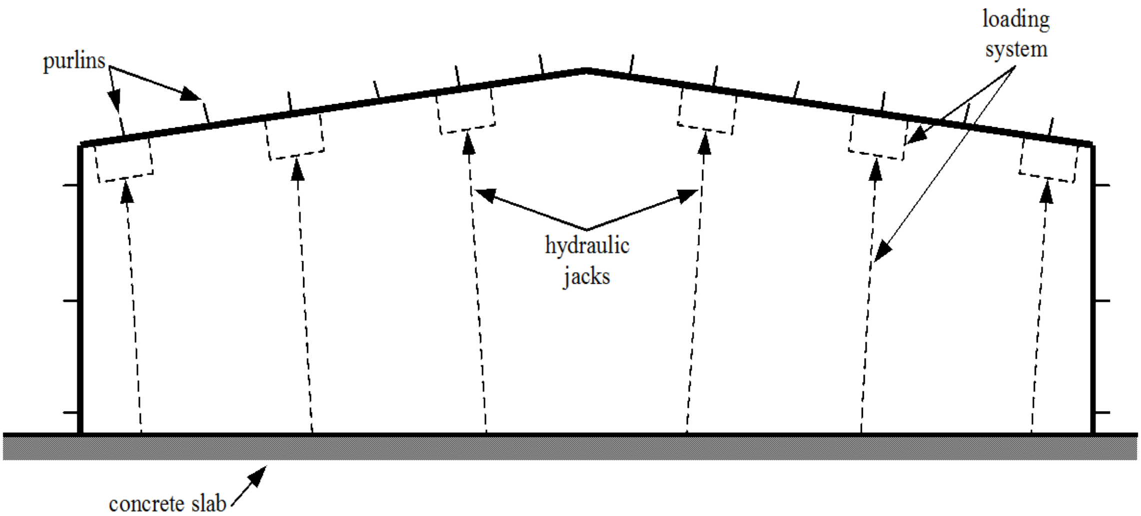
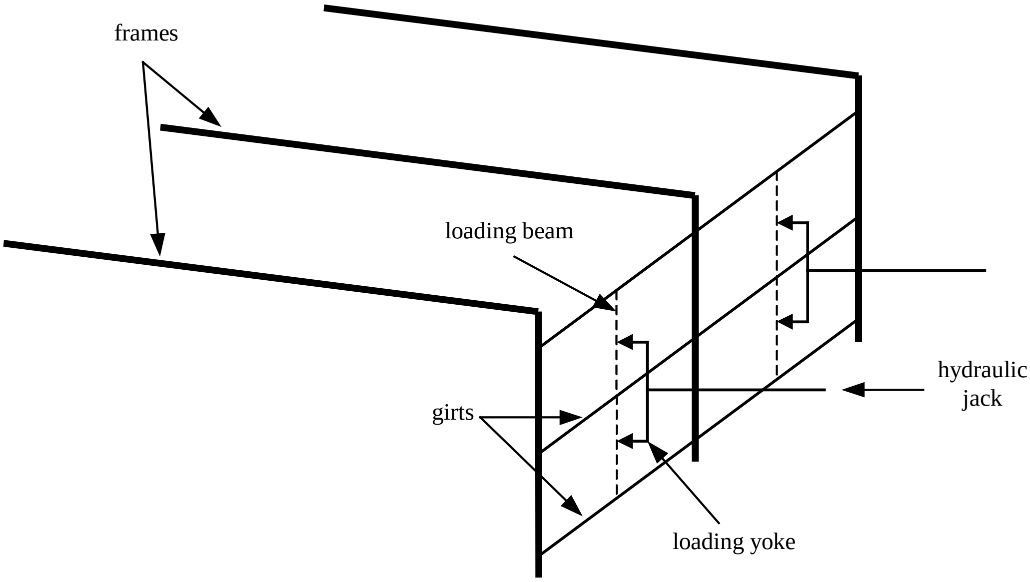


Figure 6. Knee Connections of Test Frames



(a) Roof Loading System

Figure 7. Load Simulation



(b) Side Wall Loading System

Figure 7. Load Simulation

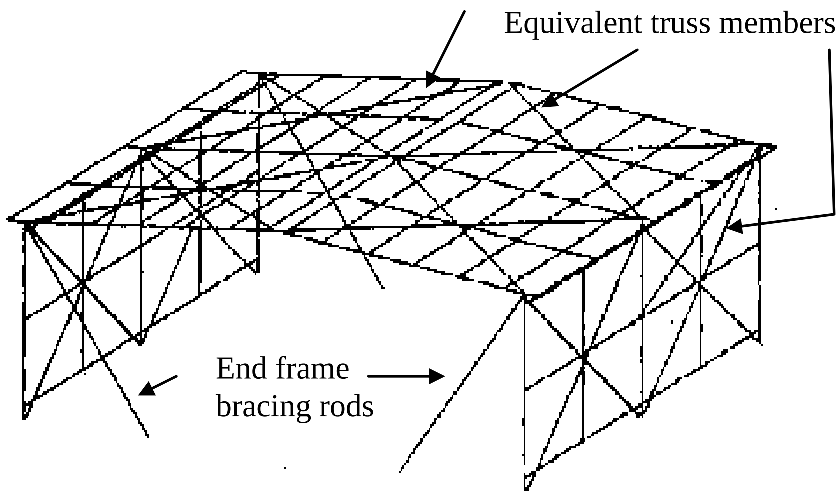


Figure 8. Three-dimensional Model of the Test Building

Table 1. Frame Deflections (mm) for Cross Wind Load

Location	Windward knee		Ridge		Leeward Knee	
Test code	Full scale test	Comp. Analysis	Full scale test	Comp. Analysis	Full scale test	Comp. Analysis
CENTRAL FRAME PINNED BASES						
P3 (3 frames)	51	39	55	56	40	30
P3-C (Cladding)	35	37	52	55	24	28
P3-B (end frame braced)	39	20	54	56	27	10
P3-B-C (end frame braced + cladding)	14	14	55	56	3	5
2-D ANALYSIS		53		49		45
END FRAME PINNED BASES						
P3 (3 frames)	28	30	21	20	24	27
P3-C (Cladding)	29	31	19	20	26	28
P3-B (end frame braced)	1	5	19	22	1	1
P3-B-C (end frame braced + cladding)	3	5	19	22	1	1
CENTRAL FRAME NORMAL BASES						
F3 (3 frames)	22	9	48	48	12	2
F3-C (Cladding)	14	9	50	47	3	1
F3-B (end frame braced)	21	9	49	48	11	1
F3-B-C (end frame braced + cladding)	10	8	50	47	1	0
2-D ANALYSIS		9		41		2
END FRAME NORMAL BASES						
F3 (3 frames)	6	5	17	16	4	2
F3-C (Cladding)	7	5	18	17	5	2
F3-B (end frame braced)	1	3	16	17	1	0
F3-B-C (end frame braced + cladding)	2	3	18	17	1	0

Note: P3 & F3 – Unclad building (bare frames) with pinned and normal bases, respectively
P3-C & F3-C – Clad building with pinned and normal bases, respectively
P3-B & F3-B – Unclad building and end frames braced with pinned and normal bases, respectively
P3-B-C & F3-B-C – Clad building and end frames braced with pinned and normal bases, respectively
The term ‘Normal Bases’ is used in this table instead of ‘Fixed bases’ as Full scale test results are for Normal bases. However, computer analysis results are based on Fixed bases

Table 2. Frame Moments (kNm) for Cross Wind Load

Location	Windward knee		Ridge		Leeward Knee	
Test code	Full scale test	Comp. Analysis	Full scale test	Comp. Analysis	Full scale test	Comp. Analysis
CENTRAL FRAME PINNED BASES						
P3 (3 frames)	44.6	41.9	-26.6	-25.0	8.0	12.8
P3-C (Cladding)	38.7	40.9	-25.3	-24.7	13.3	13.4
P3-B (end frame braced)	40.6	34.0	-25.3	-25.6	12.0	20.9
P3-B-C (end frame braced + cladding)	29.7	31.7	-24.9	-25.2	22.5	23.0
2-D ANALYSIS		42.8		-23.2		4.4
END FRAME PINNED BASES						
P3 (3 frames)	19.3	21.6	-8.7	-10.8	-0.8	-1.8
P3-C (Cladding)	21.5	22.2	-9.4	-11.0	-2.4	-2.2
P3-B (end frame braced)	9.5	11.8	-8.5	-10.8	10.6	9.3
P3-B-C (end frame braced + cladding)	10.5	12.0	-8.0	-11.0	9.3	9.4
CENTRAL FRAME NORMAL BASES						
P3 (3 frames)	36.0	34.2	-23.0	-22.2	16.8	23.4
P3-C (Cladding)	32.3	33.7	-23.6	-21.9	21.1	23.5
P3-B (end frame braced)	35.3	33.8	-23.6	-22.2	17.1	23.8
P3-B-C (end frame braced + cladding)	30.6	33.1	-23.8	-21.9	22.9	24.3
2-D ANALYSIS		28.4		-19.1		20.3
END FRAME NORMAL BASES						
P3 (3 frames)	11.9	12.5	-7.7	-8.3	6.0	7.1
P3-C (Cladding)	13.0	12.8	-8.5	-8.4	5.0	7.1
P3-B (end frame braced)	9.8	11.6	-7.5	-8.5	9.1	8.5
P3-B-C (end frame braced + cladding)	10.3	11.8	-7.9	-8.6	8.9	8.6

Note: P3 & F3 – Unclad building (bare frames) with pinned and normal bases, respectively
P3-C & F3-C – Clad building with pinned and normal bases, respectively
P3-B & F3-B – Unclad building and end frames braced with pinned and normal bases, respectively
P3-B-C & F3-B-C – Clad building and end frames braced with pinned and normal bases, respectively
The term 'Normal Bases' is used in this table instead of 'Fixed bases' as Full scale test results are for Normal bases. However, computer analysis results are based on Fixed bases

Table 3. Comparison of Results from 2-D and 3-D Analyses and Full Scale Tests for Unclad Building

Frame	Results From	Deflections windward Knee	Deflections ridge	Deflections leeward Knee	Moments windward Knee	Moments ridge	Moments leeward Knee
Central Frame Pinned Bases	Tests	51	55	40	44.6	-26.6	8.0
	3-D analysis	39	56	30	41.9	-25.0	12.8
	2-D analysis	53	49	45	42.8	-23.2	4.4
Central Frame Normal Bases	Tests	22	48	12	36.0	-23.0	16.8
	3-D analysis	9	48	2	34.2	-22.2	23.4
	2-D analysis	9	41	2	28.4	-19.1	20.3

Table 4. Comparison of Results from 2-D and 3-D Analyses and Full Scale Tests for Clad Building

Frame	Results From	Deflections			Moments		
		windward Knee	ridge	leeward Knee	windward Knee	ridge	leeward Knee
Central Frame Pinned Bases	Tests	14	55	3	29.7	-24.9	22.5
	3-D analysis	14	56	5	31.7	-25.2	23.0
	2-D analysis	53	49	45	42.8	-23.2	4.4
Central Frame Normal Bases	Tests	10	50	1	30.6	-23.8	22.9
	3-D analysis	8	47	0	33.1	-21.9	24.3
	2-D analysis	9	41	2	28.4	-19.1	20.3

Table 5. Effects of Cladding and End Frame Bracing on Central Frame Deflections and Moments for Cross Wind Load and Pinned Bases

	Windward Knee		Leeward Knee	
	Tests	3-D analysis	Tests	3-D analysis
Deflections (mm)	51 to 14 (72%)	39 to 14 (64%)	40 to 3 (92%)	30 to 5 (83%)
Moments (kNm)	44.6 to 29.7 (33%)	41.9 to 31.7 (24%)	8.0 to 22.5 (181%)	12.8 to 23.0 (80%)