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Graeme Beattie and Bruce Deam

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SHEATHING CONTRIBUTIONS TO WALL ELEMENT STRENGTH AND STIFFNESS

By Graeme Beattie and Bruce Deam
Building Research Association of New Zealand

It is commonly known that a degree of interaction exists between the timber framing members and linings or sheathings of timber framed buildings. The purpose of this project was to quantify this by the following:

- The stiffness enhancement of timber studs sheathed with internal and external sheet materials when subjected to simultaneously applied axial and face loading
- The stiffness enhancement of timber top plates provided by internal and external sheet materials
- The strength of connections typically used between top framing plates and truss or rafter members under uplift loading.

Complete composite action occurs when the sheathing and structural member act as one. However, this effect reduces because of slip in the fasteners which attach the sheathing to the structural member. Load sharing occurs when the sheathing redistributes load from a relatively flexible member to adjacent stiffer members. Some prescriptive codes (for example NZS 3604 [SNZ,1990]) contain implicit assumptions about the interaction but the degree of composite behaviour actually present has not been quantified.

STIFFNESS ENHANCEMENT OF STUDS

Only a limited amount of US research has focused on modeling the behaviour of wall elements subjected to either face loading or combined face and axial loading (Gromala and Polensek,1984). A series of research projects, summarised by Deam and King (1994) for the Australian National Association of Forest Industries (NAFI), developed design procedures for walls, floors, rafters, lintels and top plates. The work reported in this paper extends models developed for the NAFI research projects and calibrates them for the loads and building details typically encountered in New Zealand buildings.

The analytical model for the investigation described in this paper was developed specifically to predict the response of a single stud, rather than that of a stud within a wall system, because the stiffness is not significantly affected by load sharing between adjacent studs. It extended an analytical stud model developed by Deam (1993) to predict the lateral deflection of a composite stud and sheathing element subjected to combined face and axial loading, to account for the end conditions normally found in light timber framed construction.

The stud was modeled as a simple beam-column (figure 1) of height, h, and stiffness, EI, which was symmetric about its mid-height. A lateral line load, ws, was used to represent the uniform face pressure, w, multiplied by the stud spacing, s. An axial load, P, (compressive or tensile) was applied to both ends at a distance e from the centre of area of the stud. The neutral axis for the composite stud section was offset by distance ε from the centre of area (ε could also be used to apply additional eccentricity in order to model edge defects such as knots or saw cuts in the stud).

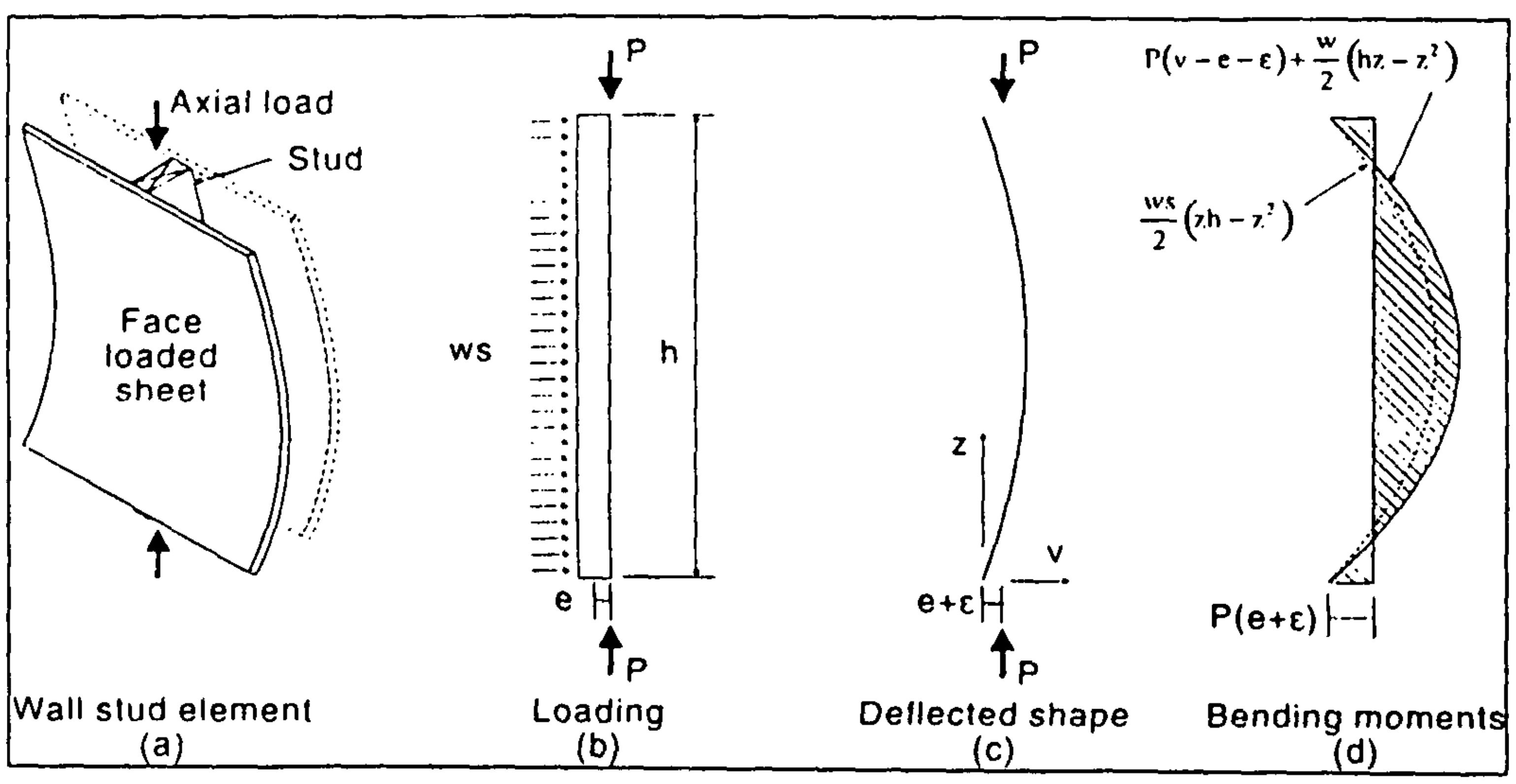


Figure 1 Analytical Stud Model (Deam, 1993)

By equating the Figure 1(d) bending moment to the stud curvature, the lateral deflection, v, is given as (Deam 1993):

$$v = \left(\frac{w}{P\mu^2} - e + \varepsilon\right)\left(\cos\mu z + (\operatorname{cosec}\mu h - \cot\mu h)\sin\mu z - 1\right) + \frac{w}{2P}\left(z^2 - hz\right) \qquad (1)$$

where the multiplier, μ, which makes the length non-dimensional is:

$$\mu = \sqrt{\frac{P}{EI}} \qquad (2)$$

The mid-height stud deflection, δ, and stud base rotation, θ, are obtained from Equation 1 as:

$$\delta = \left(\frac{w}{P\mu^2} - e + \varepsilon\right)\left(\frac{1}{\cos 0.5\mu h} - 1\right) - \frac{wh^2}{8P} \qquad (3)$$

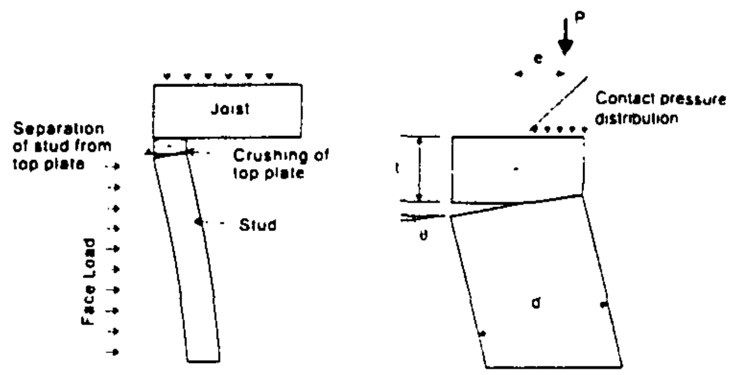
$$\theta = \left(\frac{w}{P\mu^2} - e + \varepsilon\right)\left(\frac{\mu - \mu\cos\mu h}{\sin\mu h}\right) + \frac{wh}{2P} \qquad (4)$$

For tensile axial loads, the eccentricity, e, of the axial load will commonly be half of the stud depth, d, with the axial load being applied through a connector attached to one edge of the stud.

For compressive axial loads, the eccentricity changes as the end of the stud rotates. The stud end remains in full contact with the top (and bottom) plate whenever the stud end rotation is less than a critical angle, θ_{cr}. The contact area reduces (Figure 2(b)) as the rotation increases beyond θ_{cr} and the top plate crushes because it is loaded perpendicular to the grain. (The perpendicular to grain elastic modulus, E_p, for timber is approximately 0.05 to 0.1 times the size of the parallel to grain modulus, E.)

Deam (1993) proposed a simple model for the stud to plate connection (Figure) which assumed the stress increased linearly across the contact area (Figure 2(b)). The rotation of the lower surface relative to the upper surface was modelled using the beam equation. The critical angle was defined as that at which the compression stress at one face was zero. The axial load eccentricity, e (Figure 2), was defined in terms of the axial load, P, the stud depth, d, and width, b, and the top plate thickness, t, and modulus of elasticity, E_p.

The expressions given in Figure assumed that the axial load was applied/resisted by a rigid component material, beyond the outer face of the plate, at both ends. In practice the load will often be applied to the top of the stud by a timber joist or truss chord which is itself loaded perpendicular to the grain and is located somewhere between two adjacent studs. The additional crushing of the joist and the twisting of the top plate are able to be simulated in the stud model by increasing the top plate thickness or reducing E_p. The nails between the top plate and the stud and between the sheathing and the top plate were ignored because the effect was the same when the top plate rotated relative to the support as it was if it remained stationary and the stud rotated.



Stud end conditions
(a)

Pressure distribution
(b)

$$e = \frac{cE_p bd^3}{12t}\theta \quad \text{where} \quad c = 1 \qquad \theta \le \theta_c$$

$$c = 0.5(a+1)(1-2a)^2 \qquad \theta > \theta_c$$

$$\text{and} \quad a = 0.5 - \sqrt{\theta_c / \theta} \qquad \theta_c = \frac{2Pt}{E_p bd^2}$$

Figure 2 Stud end model (Deam 1993)

Both parts of the analytical model were programmed into an Excel (Microsoft Corporation, 1992) spreadsheet. An iterative (secant) method was used to find the eccentricity, e, at the stud end for given face and compressive axial loads by minimising the difference between the rotation calculated using Equation 4 and that calculated using the Figure equations.

Experimental tests were conducted to verify the predictions from the theoretical model.

Seven specimens were constructed. Each specimen consisted of a single 2.4m long stud member fixed to 600 mm long top and bottom plate sections. One side of each specimen was lined with 9.5 mm gypsum plasterboard. Two specimens had 7.5 mm fibre cement board fixed on the other side.

Prior to specimen construction, each stud member was loaded in bending in the same direction as it was to be loaded in the tests to determine its modulus of elasticity without sheathing present.

Tension or compression axial loads were applied to the test specimens using a 150 mm x 50 mm timber stub, representing a truss or rafter, bearing on the top plate.

Each specimen was placed over a suction chamber with the gypsum plasterboard located on the top surface of the specimen, thus placing it in compression under suction loading.
The out-of-plane deflection of the stud was measured for several combinations of axial and suction loads on each specimen and compared with that predicted theoretically.

For a series of axial load levels, an example comparison plot between recorded and predicted mid-span deflection of a specimen with gypsum plasterboard on one face only is presented in Figure 3. The responses for different axial loads are translated horizontally for clarity. The origin for each axial load is indicated on the plot by a vertical line extending from 0 to 0.5 kPa vacuum.

made to contributions to the system strength by the claddings.

Reardon and Xu (1993) showed that the nails between the sheathing and the top plate had a greater effect on the plate stiffness than the composite action of a pair of top plate members.

In this investigation, a simple analytical model of a top plate was formulated to predict its stiffness due to composite action between the top-plate and sheet materials attached to it. In the model, classical beam theory was used to calculate the deflection at a number of selected load points along the length of the top-plate. The load points included the externally applied load (from the truss) and the nail

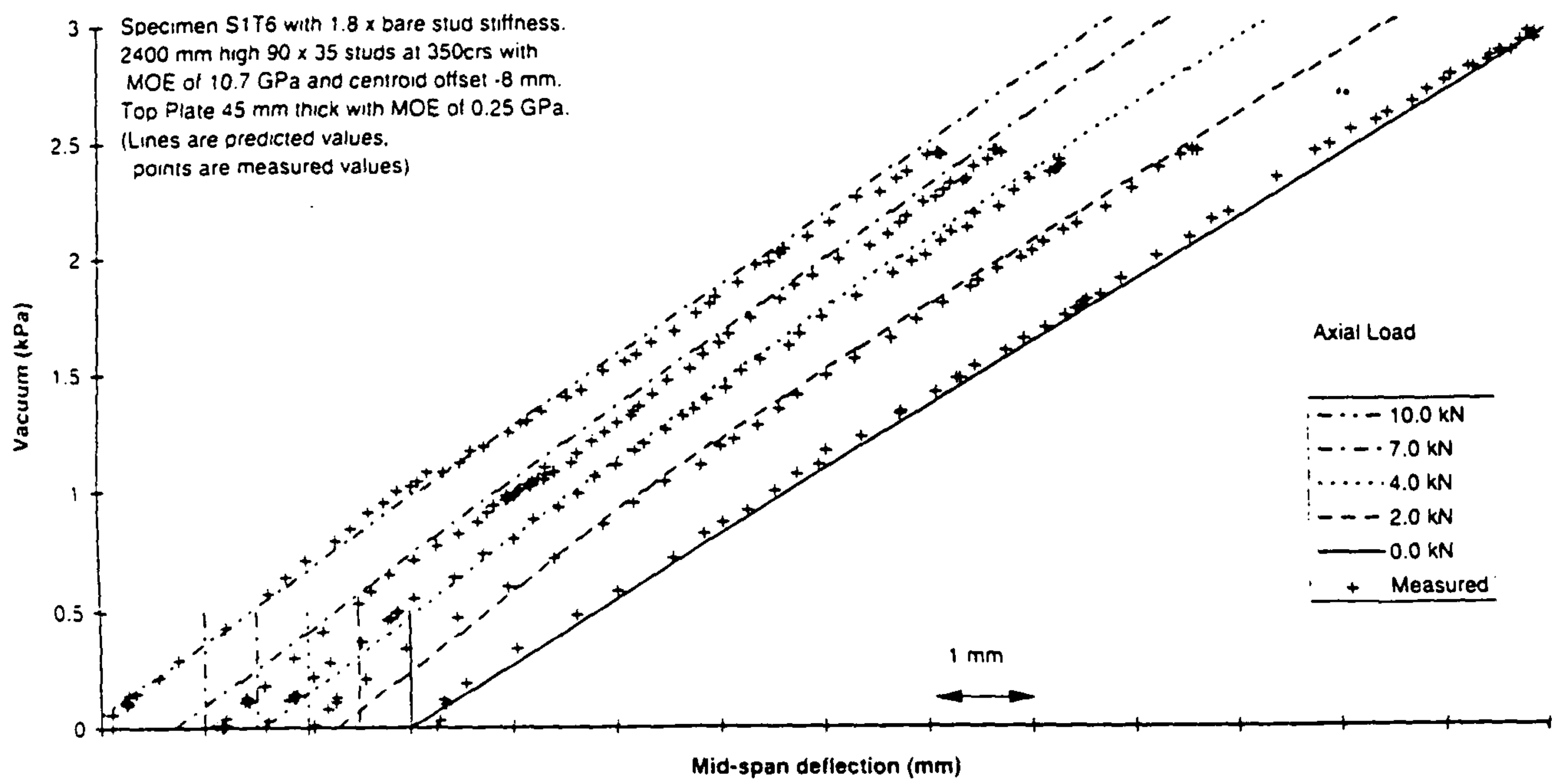


Figure 3 Predicted and experimental results for Specimen with gypsum plaster board on one face only.

The close match between experimental and theoretical results for midspan deflection provide a good validation of the model. Consideration of all the results from the test specimens in the series indicated that a conservative assessments of the stiffness enhancement of the stud were 60% for gypsum plasterboard on one side only and 150% for gypsum plasterboard on one side and fibre-cement board on the other.

TOP PLATE STIFFNESS ENHANCEMENT

Collins (1980) outlined the engineering bases for the plate tables in NZS 3604:1978 (SNZ, 1978). In his investigation, Collins took account of the performance of members used in the practice of the day. He determined that for plates, bending strength was not critical because they were seen to be performing satisfactorily at stresses in excess of what was normally considered acceptable. No specific reference was

positions which produced restraining forces as the top-plate moved relative to the sheet material. Because the nail forces were a non-linear function of the movement between the top-plate and sheet, a spreadsheet-based solving routine was used to iteratively balance the applied force and nail forces. This solver minimised the difference between an initial guess of each nail force and the force calculated using the movement of the top-plate at each nail position to derive a solution. Designers wishing to create a spreadsheet model for their particular application can contact BRANZ.

Two series of experimental tests were conducted to checl the theoretical predictions. In the first, two kiln dried 90 x 4 studs were spaced apart 600 mm and in the second th spacing was reduced to 400 mm. Top plate member: double the stud spacing long, were end nailed to the stud: 9.5 mm gypsum plasterboard and 7.5 mm fibre-cement shee linings were fixed in various combinations to the framin; Figure 4 shows an elevation of a typical specimen.

A point load was applied to the top plate midway between the two studs, firstly when no sheathing materials were present and secondly, with sheathing materials attached. The applied load and midspan deflection of the plate were recorded as the load increased. Loadcells were also installed in notches at the top of the studs to record the proportion of applied load that directly transferred to the studs from the top plate.

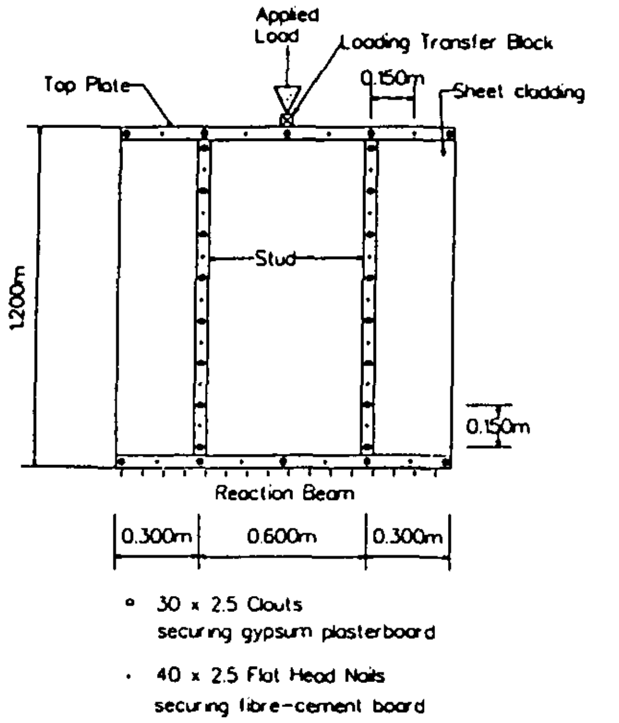


Figure 4 Elevation of top plate test specimen

It was found that only about 5% to 10% of the load was transferred through the sheathing to the studs when the wall was lined with gypsum plasterboard. The fibre-cement sheathing was found to transfer a much greater proportion (20% to 30%) of the load to the studs than gypsum plasterboard because of the stiffer connection at the nails and the greater shear stiffness of the sheet. At midspan deflections below 0.5 mm, more than 50% of the load from the plate was transferred via the sheathing to the studs. For greater deflections, the load carrying effectiveness of the sheathings reduced as a direct plate to stud transfer evolved.

The computer model predicted the deflection of the top plate alone using simple beam theory. This was compared with the experimental results for the top plates used in the specimens. The model was then adjusted to take account of the strength and stiffness of the nails fixing the sheathings to the timber frame. Adjustment of the elastic modulus of the top plate timber, the strength of the sheathing nails and the quasi-stiffness of the sheathing nails was able to be made within the model. Experimentally determined values for these parameters were inserted (e.g. elastic modulus) or estimated from previous work (Thurston, 1995) and the experimental load deflection plots for both the unsheathed and the sheathed specimens were compared with the analytically predicted curves.

A sample load / deflection plot is given in Fig. 5. Quite reasonable agreement was able to be achieved between the predicted and the experimental deflections for the unsheathed frames, provided a constant displacement offset was included to take account of the initial crushing of the top plate on the compression edge of the studs. The crushing is not taken into account in the simple beam theory prediction of the deflection. The displacement offset was approximately 0.7 mm for the studs spaced at 600 mm centres and 1.0 mm for the studs spaced at 400 mm.

There was no local crushing component of the top plate in the sheathed tests because this had already occurred when the plate was loaded with no sheathings attached. The high early stiffness of the nailed connections served to enhance the initial stiffness of the composite.

Fig. 5. clearly shows a divergence of the experimental results as the load increases, confirming the expectation that fibre-cement board sheathing enhances the stiffness of the bare top plate. Using the nail slip and plate bending characteristics previously determined, the model generally predicted a greater stiffness for the sheathed plate than was achieved in the tests.

In specimens with gypsum plasterboard as the only sheathing, once the midspan deflection reached about 1 mm, the stiffness was the same as the unsheathed top plate because the nailed connections had degraded.

TOP PLATE CONNECTION STRENGTHS

According to NZS 3604, the connection between a rafter and top plate need only be two skewed 100 x 3.75 mm nails for most houses in low and medium wind exposure zones. Additional "Z" shaped wire dogs are required when the rafter span exceeds 2.5 metres or the rafter spacing exceeds 900 mm in the high wind zone and for houses with large rafter spans or spacings in the lighter wind zones. The stud to top plate connection need only be two 100 x 3.75 mm nails (end nailed) except that light roofs in high wind exposure zones where the span exceeds 7.2 m, "U" shaped wire dogs or strap connections are additionally required at not more than 900 mm centres along the top plate.

Calculations based on the loadings code, NZS 4203 (SNZ,1992), indicated that for such connections, the wind uplift experienced will often be greater than such a connection could safely resist.

A series of rafter to top plate to stud connections was tested to ascertain the likely strengths of the NZS 3604 connections.

A comparison of the NZS 4203 design loads with the measured strengths showed that:

1. The measured uplift capacity of simulated light framed roof rafters fixed to the top plate with 2 skew nails exceeds the calculated uplift in low and medium wind exposures and within the span and spacing limits of NZS 3604 (SNZ, 1990).

2. The measured uplift capacity of simulated light framed roof rafters fixed to the top plate with 2 skew nails plus 2 wire dogs exceeds the calculated uplift in a low wind exposure and within the span and spacing limitations of NZS 3604 (SNZ, 1990).
3. All other combinations of NZS 3604 allowable rafter spans and spacings and wind exposures less than "very high" for light roofs are predicted to fail in the ultimate design wind event.

CONCLUSIONS

The conclusions to be drawn from the research are as follows:

1. The out-of-plane stiffness of 2.4 m high 90 mm x 35 mm studs is increased by approximately 60% when a gypsum plasterboard lining is added to one side of the frame.
2. The addition of gypsum plasterboard to one side of the frame and fibre cement board to the other increases the stiffness by about 150%.
3. Sheathings provide a beneficial influence on the weak axis bending of top plates and a theoretical model, which took account of the modulus of elasticity of the plate and the strength and load-slip characteristics of the nails, provided a slightly unconservative prediction of load deflection behaviour of a system comprising a top plate and sheathings.

The experimental tests conducted on representative rafter to top plate to stud connections showed that the strength of the joint between the rafter or truss and the top plate was less than required to resist the expected uplift loads calculated in accordance with NZS 4203 (SNZ,1992) in all but a few cases where the contributing roof area was small or the wind exposure was low. The Standards New Zealand committee reviewing NZS 3604 has been notified of the findings.

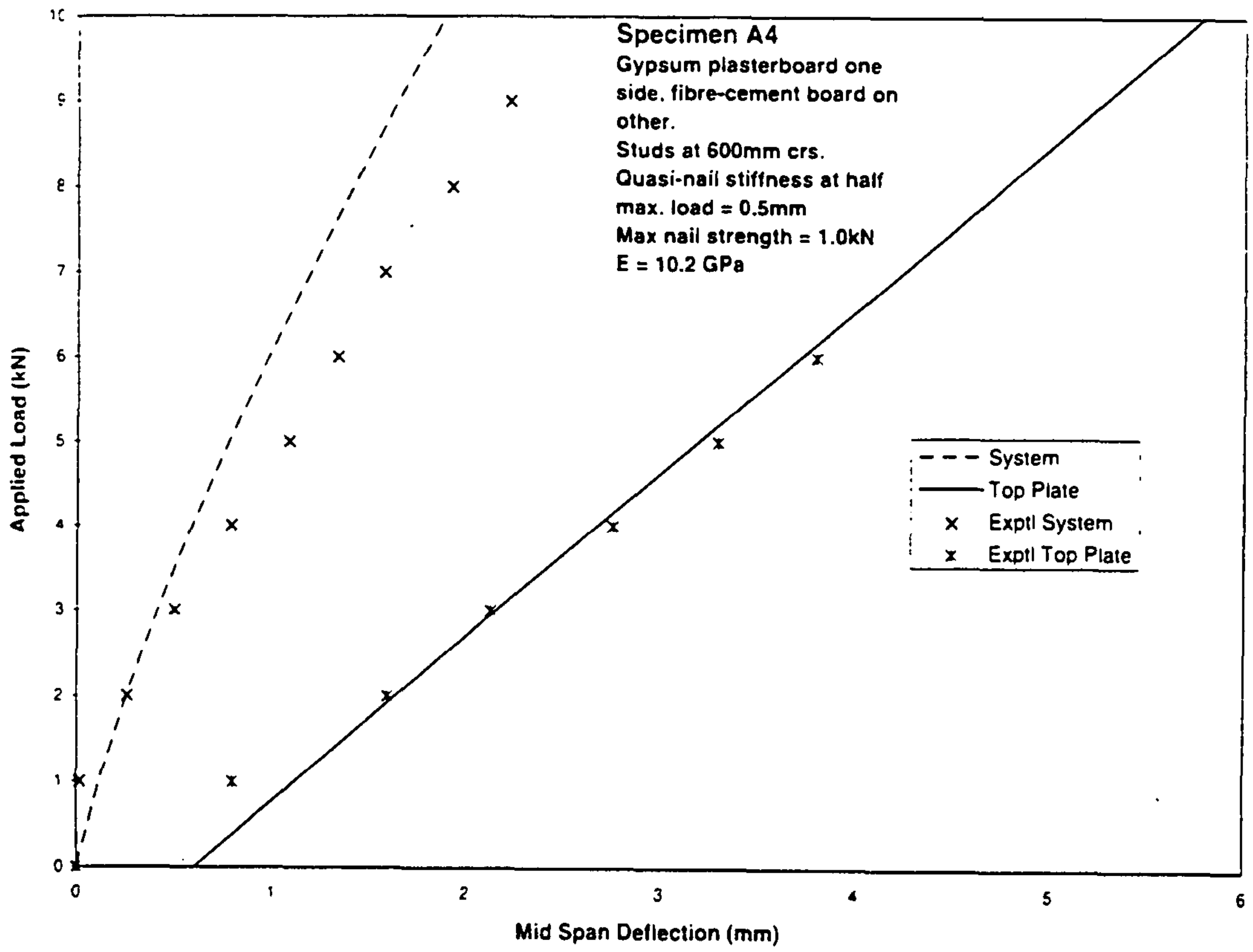


Figure 5 Sample Stiffness Comparison Plot

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Moonshine Road, Judgeford
Postal Address - Private Bag 50908, Porirua
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REGIONAL ADVISORY OFFICES

AUCKLAND
Telephone - (09) 526 4880
FAX - (09) 526 4881
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FAX - (04) 235- 6070
Moonshine Road, Judgeford

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