SEISMIC DESIGN OF GLULAM STRUCTURES

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ABSTRACT

This paper gives an overview of the seismic performance of glue laminated (glulam) timber frame buildings. It describes the wide range of connections that can be used in glulam frames, for both single storey and multi-storey buildings, with particular reference to seismic loading.

Several new connections incorporating epoxied steel bars are described in detail. Testing of these connections under simulated seismic loading is reported, with recommendations for seismic design.

A design procedure is given for low rise multi-storey glulam frame buildings.

INTRODUCTION

There is increasing world-wide interest in timber buildings, especially single storey portal frame buildings and multi-storey buildings in the two to six storey range.

Single storey portal frame buildings

The use of timber for industrial buildings in New Zealand is increasing, mostly in single bay portal frame buildings with spans up to 40 m. Columns and rafters are made from glue laminated (glulam) timber. These portal frames require strong moment-resisting connections at the knee joint where maximum moment occurs.

This paper describes a wide range of available connections, and gives test results for several connections incorporating epoxied steel bars.

Multi-storey timber buildings

Multi-storey timber buildings fall into two distinct categories; residential and commercial. Buildings for residential uses such as apartments, townhouses, hostels etc, generally have a large number of rooms and small span floors. These buildings can be built entirely from sawn timber in stud and joist sizes with lateral loads resisted by structural walls [1]. Moment resisting connections are not necessary.

Commercial office buildings require large internal spaces for open plan offices and flexible partitioning. Feasibility studies [2] have shown that commercial office buildings are technically and commercially feasible up to about eight storeys. Such buildings require long span floor systems supported on glulam timber beams and columns. Lateral loads can be resisted by structural walls around a service core, or glulam frames, or a combination of these two. A major constraint in the design of multi-storey glulam buildings has been the lack of a reliable high strength system of connecting beams to columns. This paper describes many possible types of connections, and gives results of recent tests on connections using epoxied steel bars.

MOMENT RESISTING CONNECTIONS

Structural form

A major constraint on the design of timber and glulam structures is the difficulty of making connections, especially moment resisting connections.

In straight beam structures, continuity can be achieved without moment resisting connections by using cantilever and suspended span construction or other imaginative solutions, as shown in Figure 1.

In glulam portal frame structures, much effort has been applied to the development of moment resisting connections to imitate steel frames (Figure 2(a)), but more appropriate and often cheaper structures such as curved glulam frames or triangulated structures require only pinned connections as shown in Figure 2(b) and (c). Innovative designs of triangulated portal frames with steel springs to improve earthquake resistance and to allow for shrinkage and swelling of wood are described by Touliatos [3].

In multi-storey construction, moment resisting connections are essential if frame action is to be used. The alternative is

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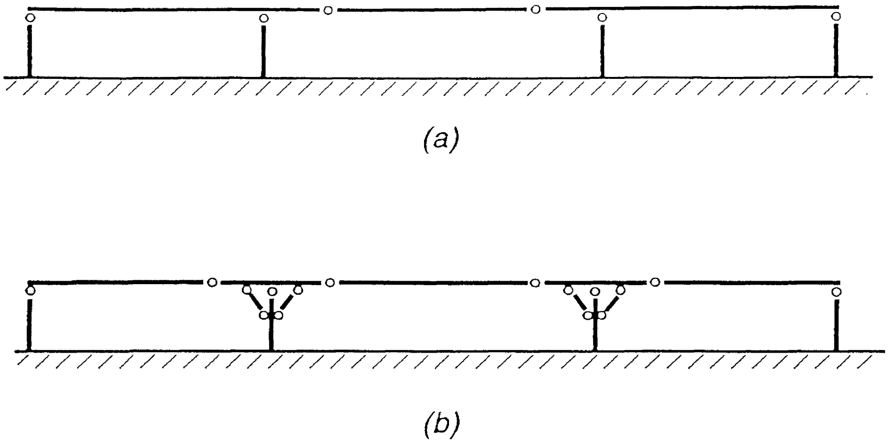


Figure 1 Cantilever and suspended span construction.

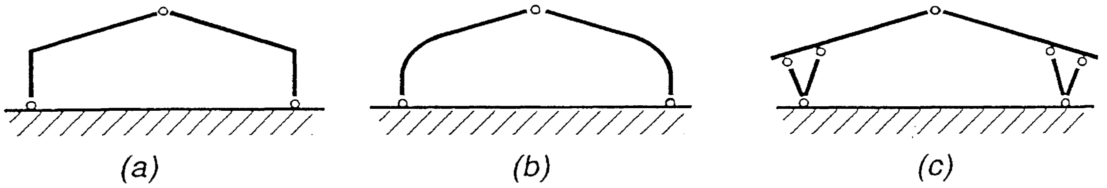


Figure 2 Alternative portal frame systems.

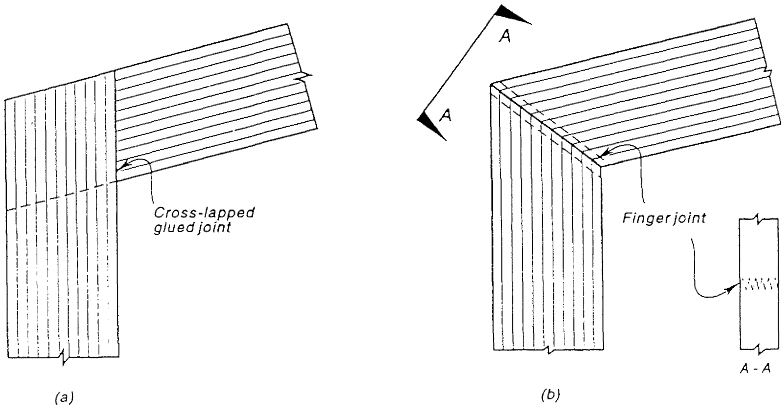


Figure 3 Glued portal frame knee joint connections.

a pin-jointed gravity frame with lateral loads resisted by walls elsewhere in the building.

Glued connections

Glued connections can be used to make strong rigid connections between glulam members such as the cross-lapped portal frame joint in Figure 3(a), promoted by McIntosh Laminates Ltd, or the mitred finger jointed connection in Figure 3(b), widely used in Europe.

Glued connections are attractive and have good fire resistance, but they require extremely high quality control and must be glued in the factory, resulting in transporation problems.

These connections are not suitable for seismic design (unless the structure is designed for elastic response) because of the lack of ductility. When loaded to failure, glued connections tend to exhibit sudden brittle fracture.

Nailed connections

A wide range of moment resisting nailed connections are available, using steel or plywood side plates. Plywood and thin sheets of steel can also be glued together to make composite side plates [4].

For portal frame construction, the connections shown in Figures 4(a), (b) and (c) are widely used in New Zealand. Design methods are described by Walford [5]. These

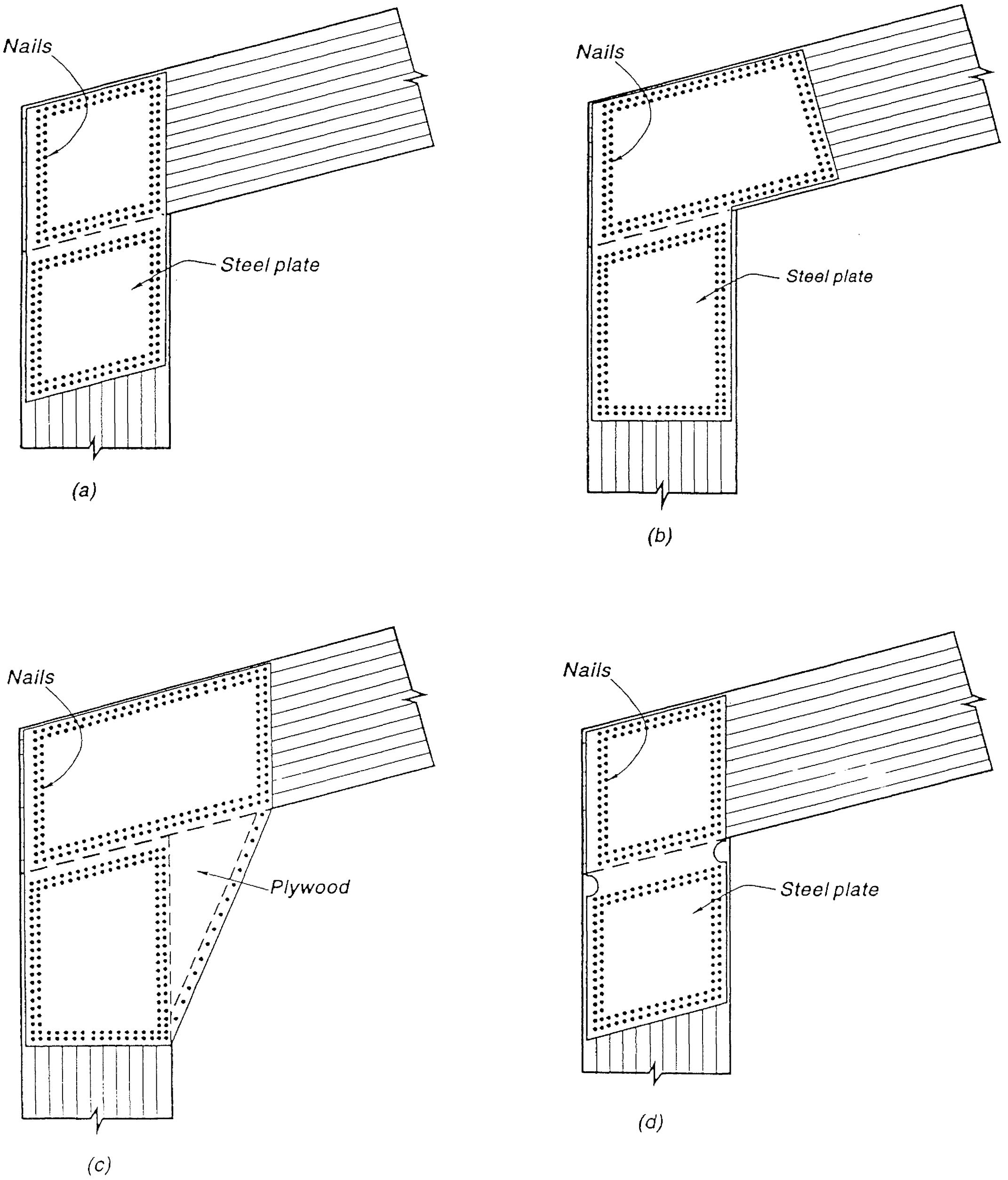


Figure 4 *Nailed portal frame knee joint connections.*

connections are only suitable for deep slender glulam members where there is a large side surface available to accommodate the nails. Plywood plates as shown in Figure 4(c) are cheaper than steel plates because no predrilling is necessary, but limitations in available plywood sizes limit spans to about 20 m.

Portal frame buildings are usually governed by strength, not stiffness, so high strength connections are most important. Ductility is often less important because wind loads are usually larger than seismic loads for these types of buildings.

Experiments have shown that it is difficult to obtain both high strength and high ductility with nailed connections. If only moderate strength is required (up to half or three quarters of the design strength of the glulam members) then the nails behave in a predictable ductile manner under reversed cyclic loading.

However, as more nails are provided in an attempt to develop the full design moment of the connected members, the eventual failure mode tends to be a brittle fracture in one of the members before significant ductile yielding occurs in the nails. This is due to several factors, including a non-triangular bending stress distribution adjacent to large concentrations of nails, and the weakening effect of nail holes in highly stressed wood of low fracture toughness (i.e. juvenile wood or compression wood) [6].

The best method of providing ductility in nailed sideplate connections is to use a necked steel plate as shown in Figure 4(d). Such a plate can be designed for yielding to occur in the plate before critical stresses are reached in either the nails or the glulam. Several New Zealand buildings have been designed using this technique.

For multi-storey construction, steel sideplate connections such as those shown in Figure 5 are possible. Figure 5(a) shows large steel side plates similar to those used in portal frame construction. These are functional, but unattractive and expensive because of the large number of holes to be drilled. They also have poor fire resistance.

Figure 5(b) is a cut-away view of a double beam system, where each beam is fixed to a steel plate which has been nailed to the wide face of the column.

A test of this connection produced excellent ductile behaviour, with the hysteresis loops shown in Figure 6. Note that the moment capacity of the nail groups was well below the design strength of the glulam column, leading to excellent ductility with no danger of wood failure in the column.

These pinched loops are typical of ductile nailed systems such as steel plates or plywood diaphragms. The shape of the loops has very little effect on seismic response, provided that large displacements can be sustained without sudden failure [7].

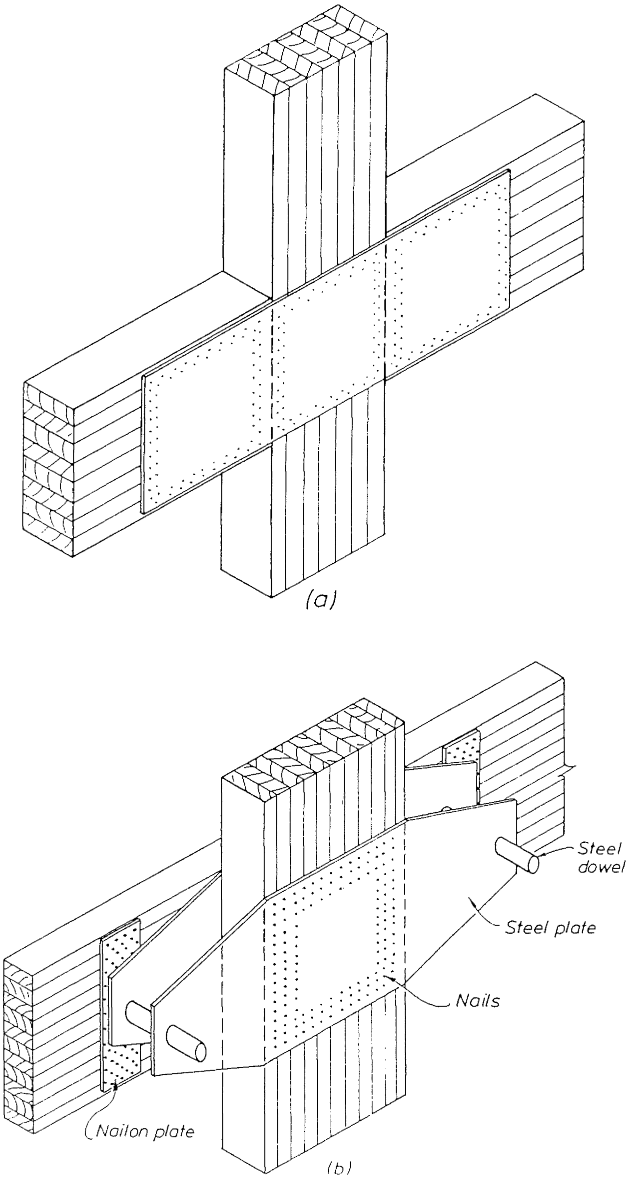


Figure 5 ***Nailed beam-column connections for multi-storey buildings.***

Dowel connections

A popular type of connection in Europe is a circle of dowels connecting a double column to a single central rafter, as shown in Figure 7(a). The dowels are straight smooth steel rods driven into pre-drilled holes in the glulam. These connections are attractive and easily assembled, but it is difficult to develop the full strength of the glulam members, and problems with splitting failures in service have been reported. This type of connection can also be used in multi-storey construction as shown in Figure 7(b).

Tests in Italy [8] have achieved excellent ductile behaviour with pinched loops as shown in Figure 7(c), and the results have been used to analyse the seismic performance of multi-storey glulam frame buildings.

Similar connections can be made with a steel plate in a central slot in the glulam member, again connected with dowels. Extensive testing of this type of connection has been made in Japan [9], using bolts for site connection as shown in Figure 8(a). Typical hysteresis loops are shown in Figure 8(b) and a three storey office building using this type of connection is shown in Figure 8(c).

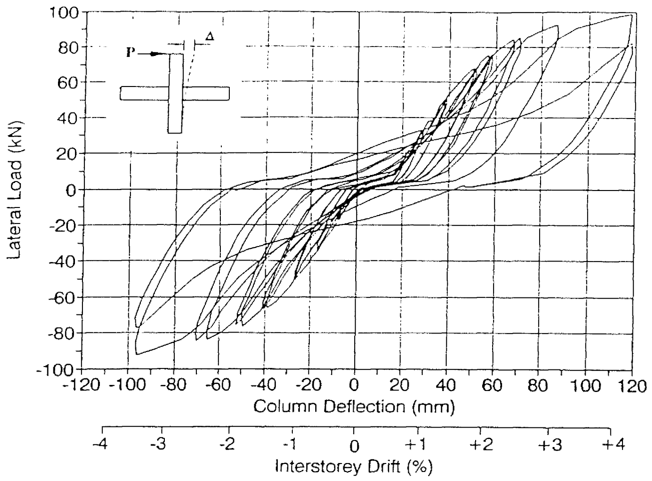


Figure 6 *Hysteresis loops for nailed double beam connections.*

Epoxied connections

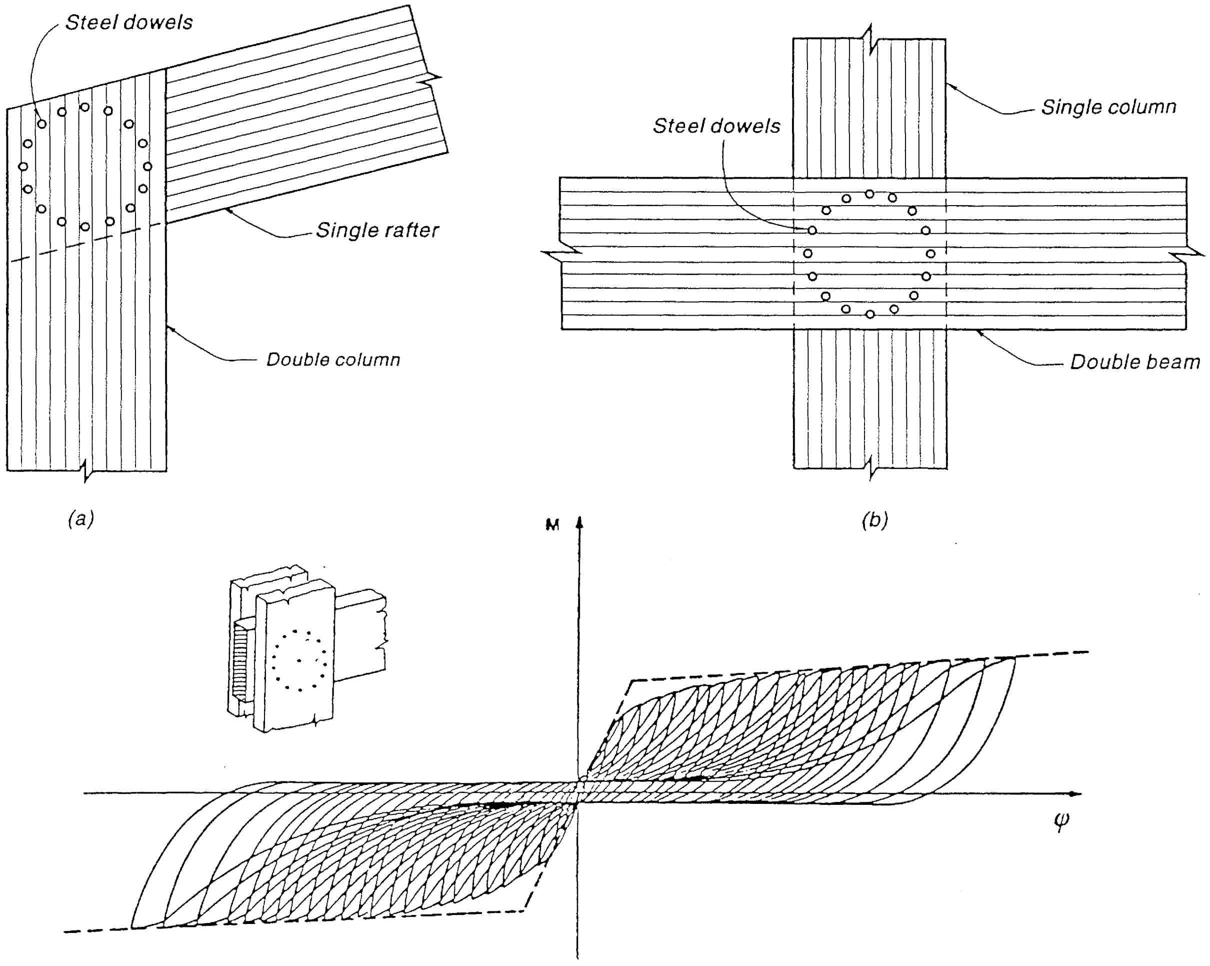
Connections with epoxied steel rods are receiving increasing attention. Early use of these connections in New Zealand is described by McIntosh [10].

A major experimental study by Riberholt [11] formed the basis for designs of sports stadium roof structures by Buchanan and Fletcher [12] and extensive testing by Townsend [13] summarized by Buchanan, Moss and Townsend [14].

Townsend [13] derived the following equation from an empirical analysis of pull out test results of deformed reinforcing bars embedded in dry glulam timber :

$$F = 9.2\ d\ l_g\ (r_d)^2\ \sqrt{r_e}$$

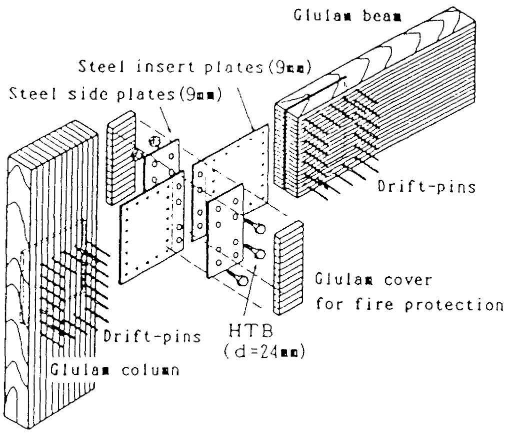
where F = pullout force (N)
d = bar diameter (mm)
l_g = embedment length (mm)
r_d = ratio of hole diameter to bar diameter
r_e = ratio of edge distance to bar diameter, where the edge distance is measured from the bar centre-line



(a)

(b)

Figure 7 *Steel dowel connections.*



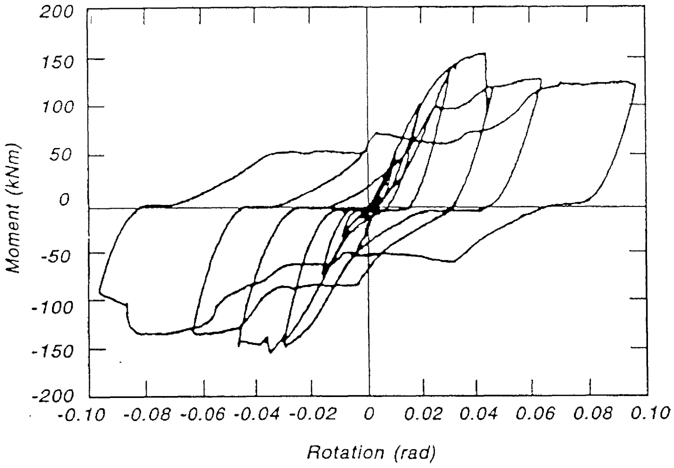


Figure 8 *Japanese connections using bolts and steel dowels.*

Townsend also reported tests of several full size beams spliced with high strength deformed bars. The splices were either at mid-span or offset from mid-span. The beam splice connections performed very well, generally being stronger than the beams themselves. Experiments on epoxied steel rods are continuing at the University of Canterbury.

Several alternative designs are possible for portal frame structures. Three arrangements tested by Buchanan and Townsend [15] are shown in Figures 9(a), (b) and (c). Figure 9(a) shows a knee joint with epoxied bars passing through the rafter. Figure 9(b) shows a mitred connection with steel bars welded to a steel plate along the mitre. In a third design, the entire joint region consists of a steel bracket as shown in Figure 9(c). The threaded ends of the epoxied bars pass through holes in the flanges of the steel knee, and are connected with nuts and washers.

Tests of these three connections are described later in this paper.

In an Australian development, Gardner [16] uses a haunched rafter as shown in Figure 9(d) to overcome the weakening caused by drilling right through the member at the critical section. Other proposals for portal frame knee joints are those in Figure 9(e) [11] and Figure 9(f) [17].

For multi-storey construction, several alternatives are possible. The most simple connection uses straight steel bars passing from beam to beam through the column as shown in Figure 10(a). This type of connection has been used in the two storey building in Christchurch shown in Figure 11 [12].

Alternative designs using fabricated steel brackets are shown in Figures 10(b) and 10(c). Advantages of steel brackets are that all glueing can be done in the factory with only bolted connections on site, and with capacity design for seismic loading, all ductile yielding occurs in the brackets. A cut-away view of a double beam system with yielding steel brackets is shown in Figure 10(d).

Tests on all of these epoxied systems for multi-storey building are described below.

EXPERIMENTAL PROGRAMME

Overview

This paper summarizes several series of tests on moment resisting connections between glulam members using epoxied steel bars. Full details of the tests are given by Buchanan and Townsend [15] and Fairweather [18].

Materials

Glulam

All of these tests used dry untreated radiata pine timber, from several different manufacturers. Average test density was approximately 500 kg/m^3. The grade was specified as No.1 framing grade or better but the grade, strength and stiffness varied considerably between manufacturers. The average modulus of elasticity was 7.6, 8.8 and 11.2 GPa for wood from Christchurch, Nelson and Auckland, respectively. Most beams were laminated from 45 mm thick laminations containing frequent fingerjoints. The cross section depth for all beams and columns was approximately 500 mm. Most

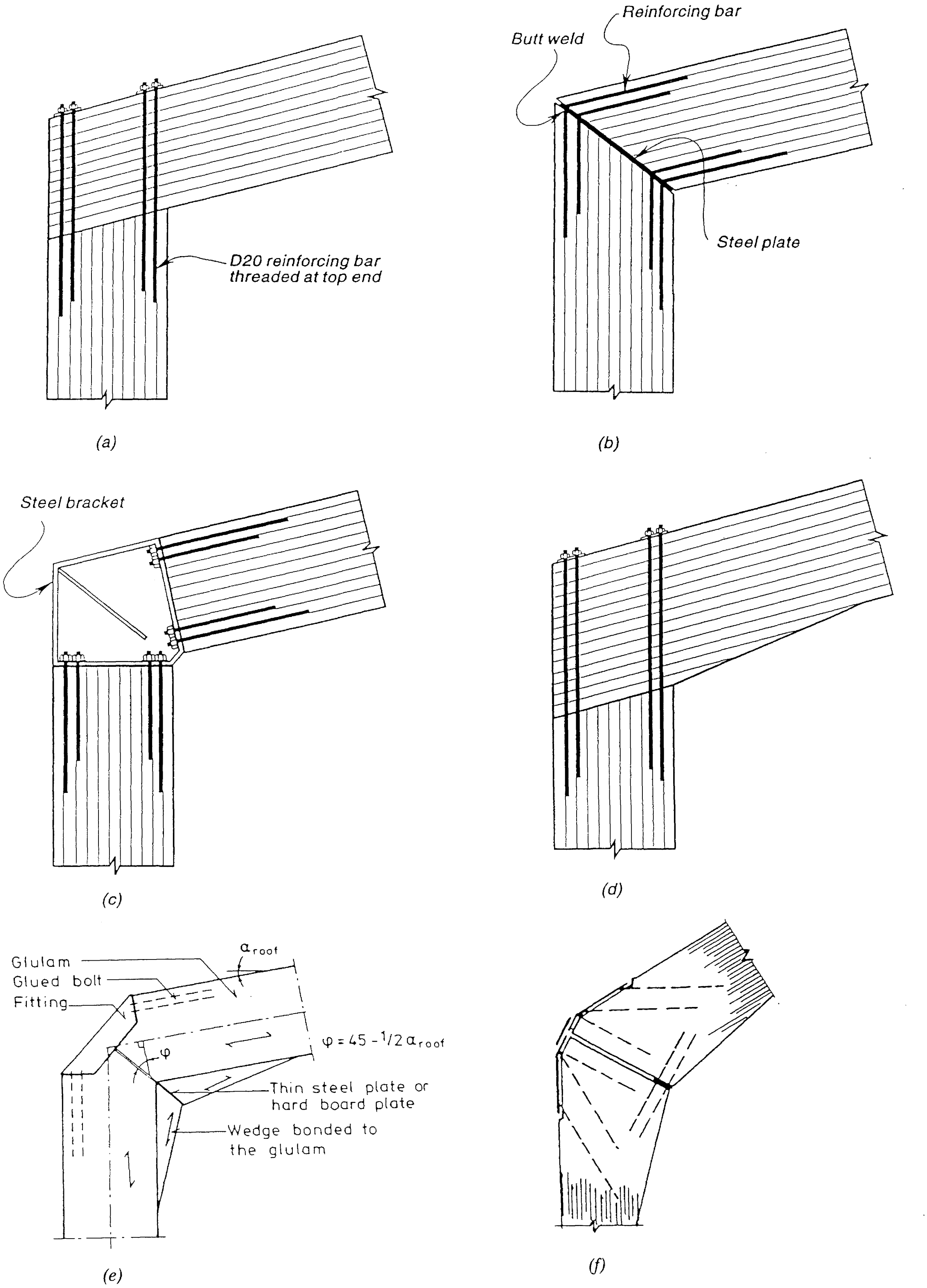


Figure 9 *Portal frame knee joints using epoxied steel rods.*

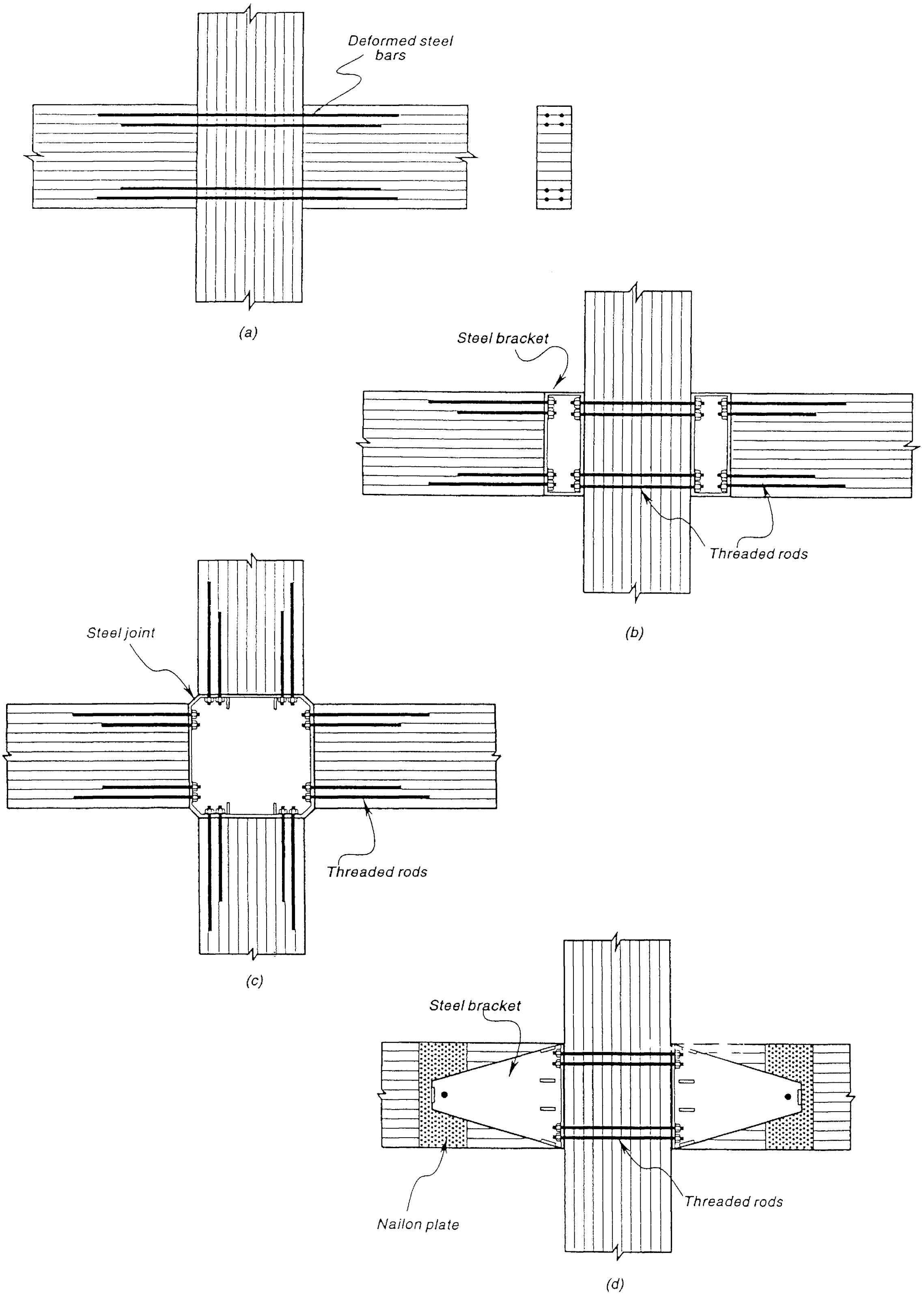


Figure 10 ***Multi-storey beam-column connections using epoxied steel rods.***

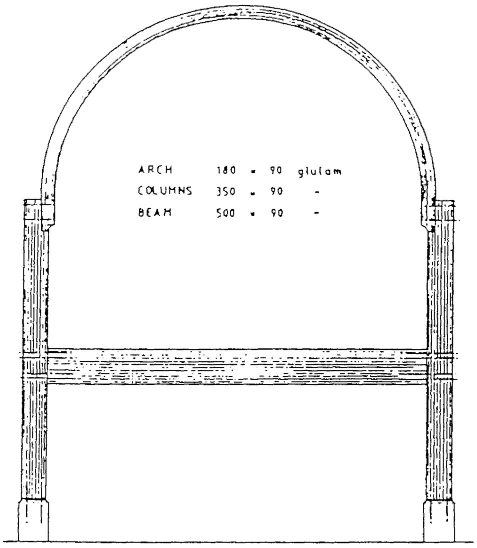


Figure 11 *Two storey building with epoxied rods in the beam-column joint.*

beams were 135 mm wide, except 90 mm for the double beam systems. The columns in the multi-storey connections were 180 mm wide.

Steel rods
Steel rods were either deformed reinforcing bars or high strength threaded rods, 16 or 20 mm diameter. Steel reinforcing bars were grade 300 MPa where bar yielding was anticipated under simulated seismic loading, grade 430 MPa elsewhere. Most of the threaded rods were AISI grade 4140 with a yield stress of 680 MPa. Smooth steel rods were not used because it was considered important to rely on a mechanical anchorage rather than an adhesive bond between the epoxy and the steel.

Epoxy
Several types of epoxy were used. The pullout tests and portal frame connections used Ciba-Geigy K2005, and Ciba-Geigy K80 epoxies. Both gave very good results, although there are indications that the K80 may not perform as well under moist conditions. The beam-column tests used West System Z105 epoxy from Adhesives Technologies Ltd which also gave very good results.

Drilling and placing of dowels

The dowel holes were drilled with a hand held air drill and a wood auger as shown in Figure 12. A steel angle was used for alignment. The drilling method requires some precision and skill, especially when drilling through the column as this fixes the position of the beams. A larger hole size provides better tolerances but requires more epoxy and can weaken the glulam member. The holes were 22mm diameter for the D16 bars and 26 mm diameter for the D20 bars and 20 mm threaded rods. An 8 mm diameter pilot hole was drilled 30 mm from each end of the dowel hole as shown in Figure 13.

The embedment lengths were established using reasonable safety factors on the pullout equation developed by Townsend [13]. The bar lengths were staggered to reduce stress concentrations in the glulam at the ends of the bars. No pullout failures were anticipated as it was not an objective of these tests to investigate pullout strength. Embedment lengths were generally 400 mm for outer bars and 300 mm for inner bars.

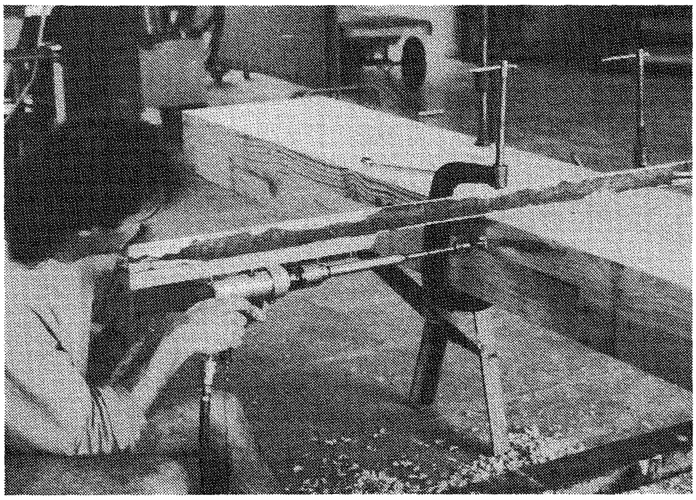


Figure 12 *Drilling of the glulam beam for epoxied steel rods.*

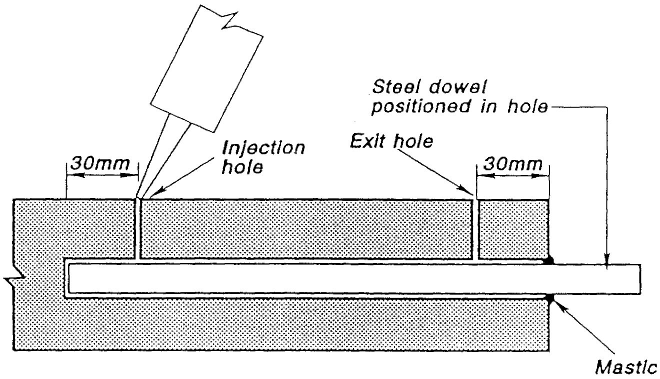


Figure 13 *Arrangement for placing epoxy around steel bar in glulam.*

The epoxy was injected into the pilot hole under gravity, until it began to flow out of the airing hole. The epoxy level in the pilot hole was kept topped until the epoxy began to harden. After some time the epoxy level dropped in the pilot holes. This drop in level is due to the epoxy requiring time to fill all the voids between the wood and the steel and some absorption into the timber.

Seismic testing

Most assemblies were tested under reversed cyclic loading to provide information about expected seismic performance.

Testing at low load levels was under load control with the load being increased in regular steps, with generally two cycles at each load level. The yield load was taken as the load at the first deviation from linear elastic behaviour.

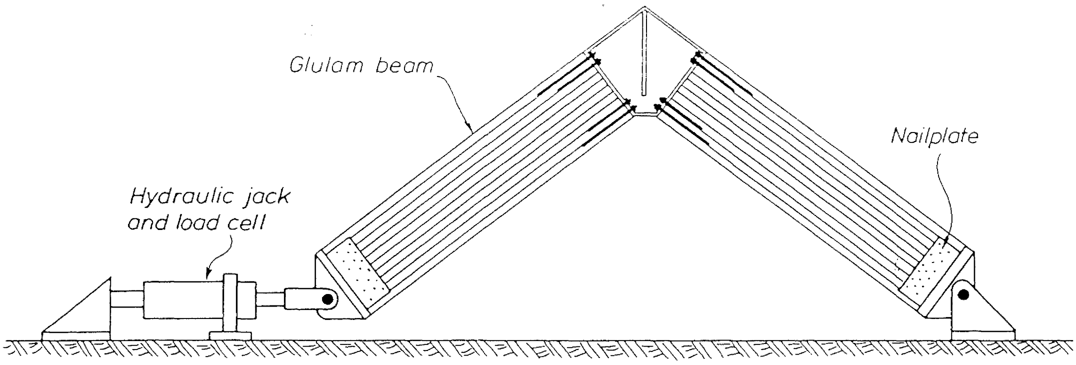


Figure 14 Testing arrangement for portal frame knee joints.

After yielding the testing was changed to a displacement control regime, generally with two load cycles being attempted at ductility levels of ±2, ±4, ±6 and ±8 until failure occurred.

Testing apparatus

The portal frame connections were tested using the apparatus shown schematically in Figure 14. This arrangement gives a reasonable approximation to the bending moments, shear forces and axial loads expected in a portal frame building.

The beam column connections for multi-storey buildings were tested using the apparatus shown schematically in Figure 15. This testing rig allowed a horizontal force to be applied to the top of the column, simulating lateral loading on a multi-storey frame with points of contraflexure at the mid-height of the columns and mid-span of the beams.

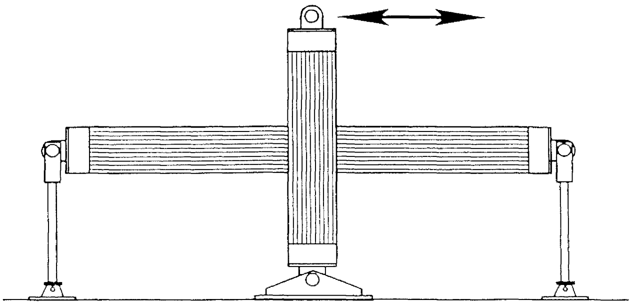


Figure 15 Testing arrangement for beam-column connections.

PORTAL FRAME CONNECTIONS

Epoxied rods across the grain

Five tests were carried out on connections of the type shown in Figure 9(a). None exhibited significant ductility because of premature wood failures. Several different wood failure modes occurred, all associated with drilled holes through the rafter or with finger-joints in the laminations. No pullout of epoxied bars occurred.

A load deflection plot for a typical test is shown in Figure 16. The test was terminated when the rafter failed in the opening mode as shown in Figure 17. Some yielding of the steel bars had occurred, but insufficient to give significant ductility.

On the basis of these tests, it is recommended that any design be based on the nett area of any drilled cross section, with seismic design forces based on elastic response.

Mitred connections

The mitred connection shown in Figure 9(b) was developed to avoid the weakening of the rafter member caused by drilling across the grain. Three mitred connections were tested. All used the same connection to the glulam as shown in Figure 9(b), although the steel plate along the mitre was modified to permit site bolting in some specimens.

The closing and opening modes of loading demonstrated very different behaviour. In the closing mode, the full strength of the members was developed and the outer bars yielded in axial tension.

In the opening mode, a split occurred near the inner bars at low load levels, in all three tests. This split grew during subsequent cycles, leading to a premature flexural failure of the glulam as shown in Figure 18.

This splitting failure is attributed to dowel action in the steel bars. It was initially assumed that all of the moment would be resisted by axial forces in the bars as shown in Figure 19(a). However because some of the moment is resisted by dowel action (shear forces in the bars), tension perpendicular to grain stresses develop as shown in Figure 19(b). The resulting splitting leads to premature failure.

This type of connection should only be used where flexural stresses in the opening mode are low, and where ducitlity is not required.

Steel knee joint

The steel knee joint shown in Figure 9(c) was developed to overcome the problems described with the above two connections.

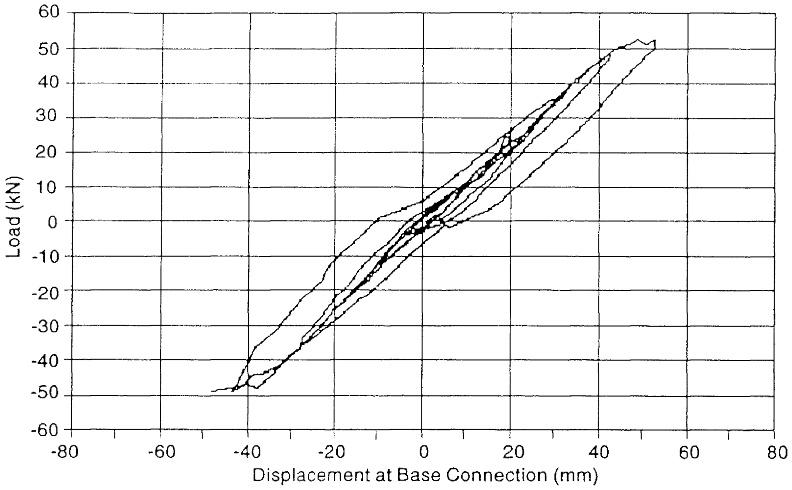


Figure 16 Hysteresis loops for knee joint with epoxied rods.

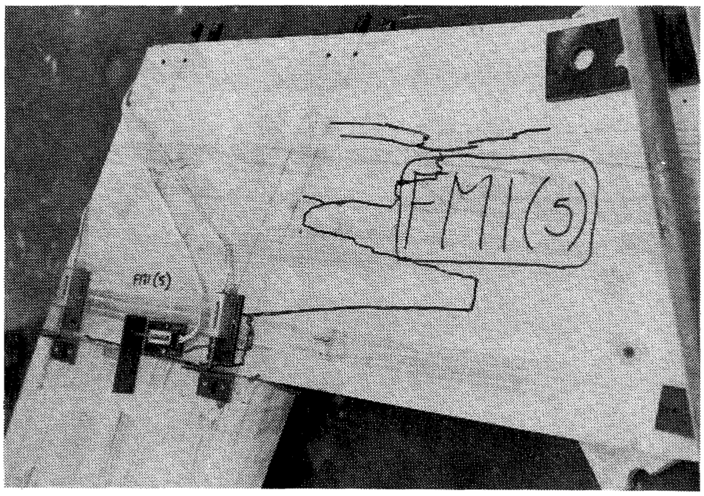
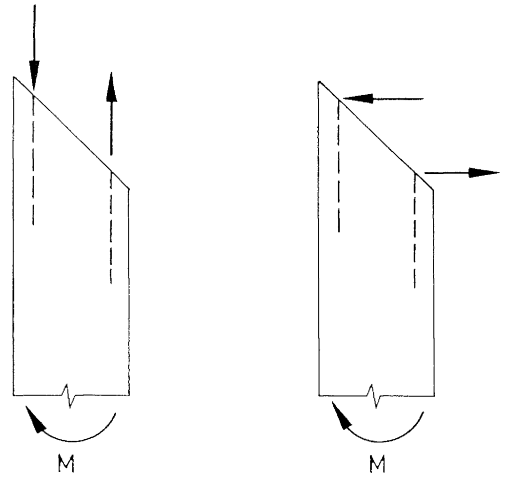


Figure 17 Failure of knee joint with epoxied rods.



(a) *(b)*

Figure 19 Moment resistance in mitred knee joint.

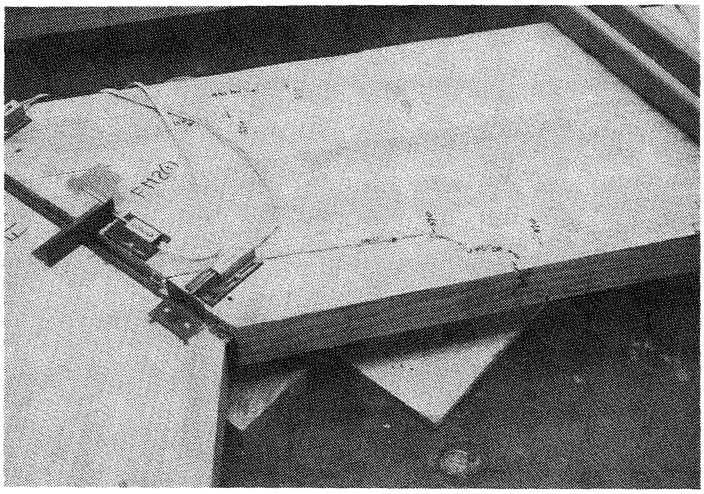


Figure 18 Failure of knee joint with mitred steel connection and epoxied rods.

Advantages of the steel knee joint include the following :

- All epoxying is done in the factory. Only bolted connections are needed on site.
- Holes are drilled into the end grain, not across the grain.
- Glulam ends are cut square, preventing the problem that occurred with the mitred joints.
- Capacity design can be used to ensure ductile yielding in the steel knee.

This type of connection is the subject of a patent application (No. 236583) by Hunter Laminates Ltd who have constructed several large industrial buildings using this system.

Three joints were tested. All exhibited reasonable ductility. A typical load-displacement plot is shown in Figure 20. Yielding occurred both in the flange of the steel knee, and in the steel bars, which were 20 mm diameter grade 275 deformed reinforcing bars with threaded ends.

The pinched shape of the hysteresis loops is primarly due to slop in the system resulting from yielding of the steel bars. This could be reduced by over-designing the bars using capacity design to ensure that yielding occurs only in the knee joint. Local bending of the flanges of the steel knee can be seen in Figure 21.

A design guide [19] has been produced for this type of connection, giving design actions, and bar and joint geometries for a range of glulam beams from 270 x 90 mm to 900 x 180 mm. The guide gives details of the design procedure for both elastic and ductile design, using both deformed reinforcing bars and high strength threaded steel rods.

For the derivation of seismic loads, elastic design is recommended for all designs where gravity loads or wind loads govern. This will be the case for most light portal frame buildings. Ductile design can be used where lower seismic loads will result in smaller sizes and lower costs, or where a ductile structure is desired for other reasons including uncertainty about seismic loads.

A maximum ductility factor of 2.0 is recommended for establishing design forces. Larger ductilities were achieved in the tests, but could not always be sustained for a large number of cycles. For ductile design, a capacity design procedure is necessary to ensure that no failure occurs in the glulam members or in the steel bars when large deformations occur in the steel knee joints.

Experiments with steel brackets gave an empirical relationship between the yield force in the bracket and the flange plate thickness [18]. The ultimate force in the brackets under large deformations is approximately twice the yield force, so an overstrength factor of 2.0 should be applied in the design process.

MULTI-STOREY CONNECTIONS

Epoxied bars

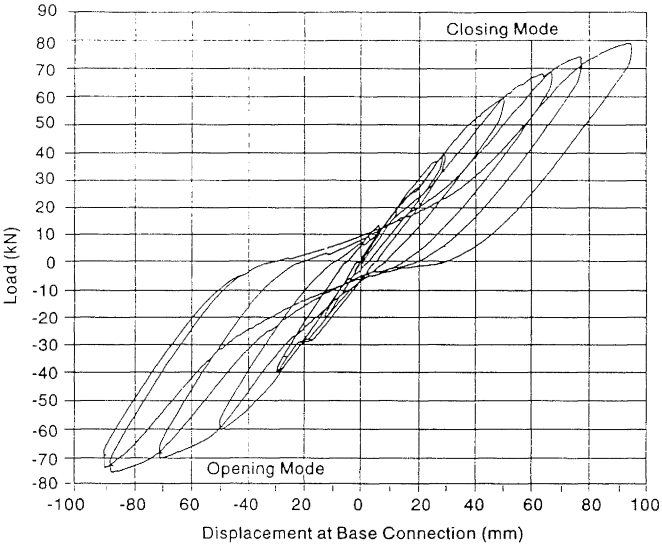


Figure 20 Hysteresis loops for steel knee joint.

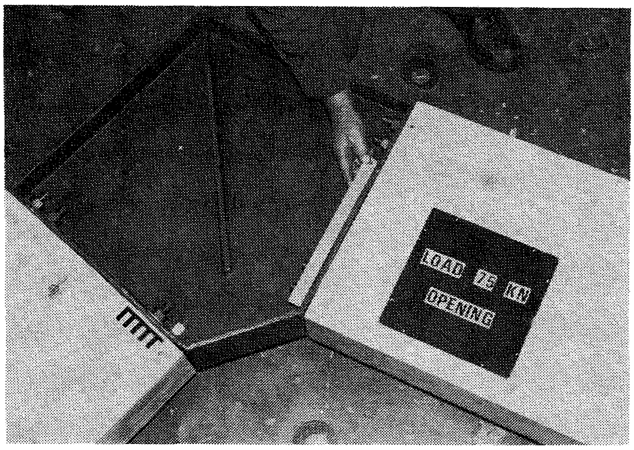


Figure 21 Detail of steel knee joint during the test.

Epoxied steel bars can be used to make beam column connections of the type shown in Figure 10(a). Four assemblies of this type were tested. Three of these exhibited linear elastic behaviour followed by sudden brittle fracture of the glulam column. In some cases the steel bars were deliberately overdesigned, and in others the failure was a result of poor quality glulam timber. A typical column fracture is shown in Figure 22.

The fourth unit was designed by capacity design to ensure that ductile behaviour occurred. The bar arrangement is shown in Figure 23. Before epoxying the bars, a 40 mm length of each bar at the column face (20 mm into the column and 20 mm into the beam) was wrapped with tape to prevent bonding, thus giving a longer length over which yielding could occur.

The assembly at the end of the test is shown in Figure 24, and the load deflection plot is shown in Figure 25. The excellent hysteresis loops show ductile behaviour similar to that of a steel structure.

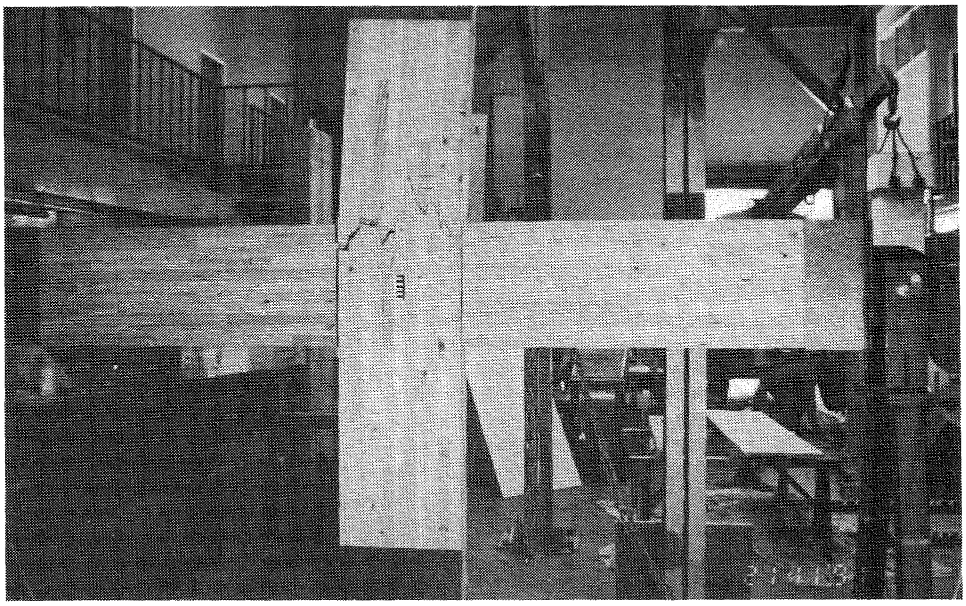


Figure 22 Failure of glulam column with epoxied steel bars.

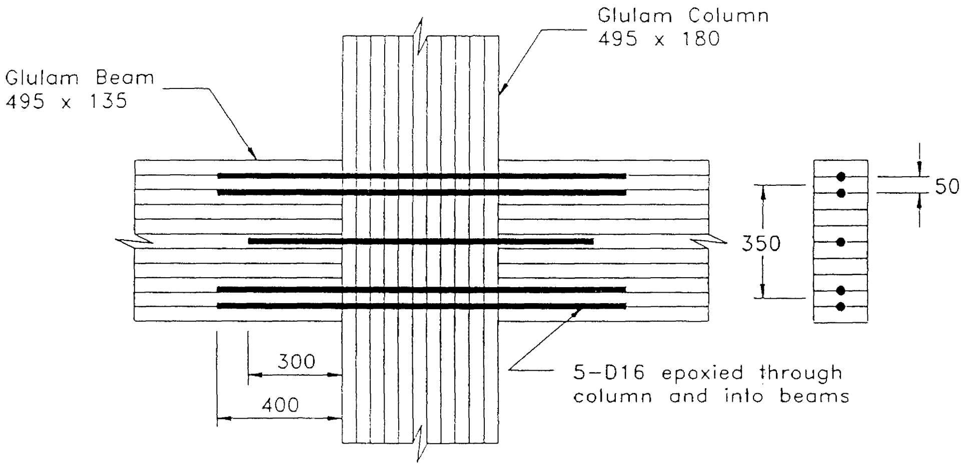


Figure 23 Arrangement of epoxied steel bars in ductile beam-column connection.

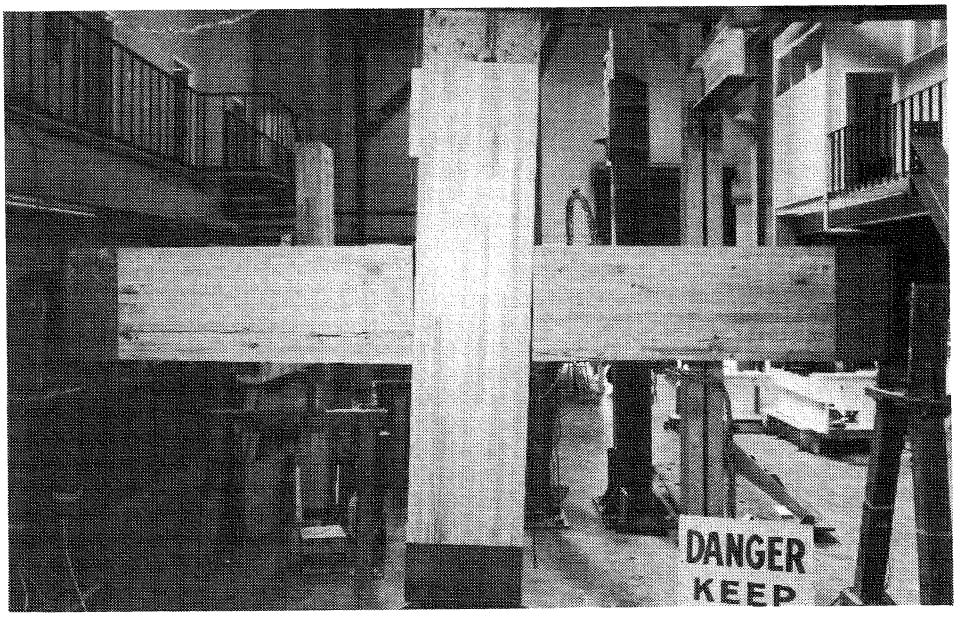


Figure 24 Beam-column assembly at end of test.

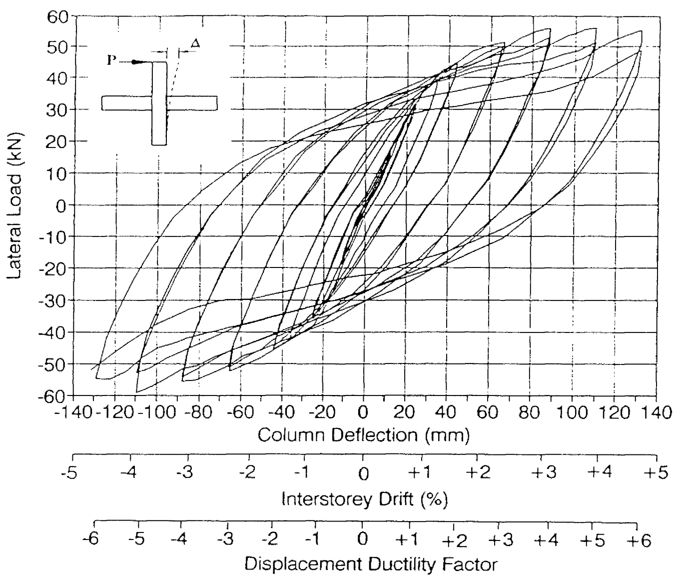


Figure 25 *Hysteresis loops for beam-column assembly with straight epoxied bars.*

Failure occurred after reaching a displacement ductility factor of 6.0 when local buckling of a compression bar caused a horizontal split in one of the beams. The split is visible on the left hand side of Figure 24. There is no easy way to confine these bars in the glulam to prevent buckling, as would be done in a reinforced concrete structure.

Several components of deformations within the assembly contribute to the overall displacement of the column. These have been estimated at several load levels as shown in Figure 26. The formulae for calculating these components are derived by Fairweather [18].

Steel beam brackets

Figure 10(b) shows steel beam brackets which can be used to connect beams to the faces of a continuous column. The beams have steel bars epoxied into the end grain and the column has steel bars epoxied through the joint region. All site connections are bolted. Capacity design can be used to ensure that ductile yielding occurs in the beam brackets. The design procedure is the same as for the steel knee joint for portal frames.

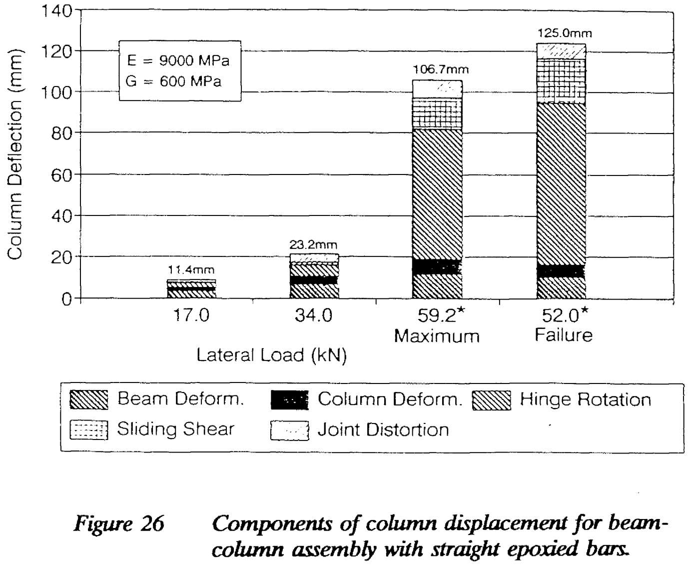


Figure 26 *Components of column displacement for beam-column assembly with straight epoxied bars.*

Figure 27 shows an assembly at the end of a test. Local bending of the flanges of the bracket are shown in Figure 28, and the load deflection plot is shown in Figure 29. Most of the column displacement was due to local bending of the steel beam bracket.

Excellent behaviour to a ductility of ±6.0 was achieved. Local splitting of the steel flange near the weld to the web reduced the load slightly in the last cycle, but no catastrophic failure occurred.

Another assembly was tested with a very thin (1 mm thick) web plate in an attempt to achieve ductility through panel zone yielding in shear, but this was not successful due to tearing of the thin plate at the welds.

Steel joint

Two units were tested with a large steel joint bracket as shown in Figure 10(c), similar in concept to that developed for portal frame knee joints. The test specimen is shown in Figure 30, at the end of the test.

The intention in these tests was to achieve ductility through local flange bending as observed in the tests described above. However in both tests a premature splitting failure of the column did not permit ductility larger than 2.0 to be achieved. The split in the lower column is clearly visible in Figure 30. A typical load displacement plot is shown in Figure 31.

The reasons for the splitting failure in the column are not clear, but may be related to the tolerances for the bolt holes in the flange plate. If the holes are too far apart, as shown in Figure 32(a), all of the shear force is resisted by dowel action on the tension bar, producing tension perpendicular to grain (splitting) stresses. To prevent this, the holes should be drilled with centres slightly closer together than the bar centres so that the shear force is resisted by the compression bars as shown in Figure 32(b). Other possible solutions, which have not yet been tested, include a short transverse bar as shown in Figure 32(c) or diagonal reinforcing as shown in Figure 32(d).

Double beam systems

Double beams running past a central column have several advantages in multi-storey glulam frame construction, including the following :

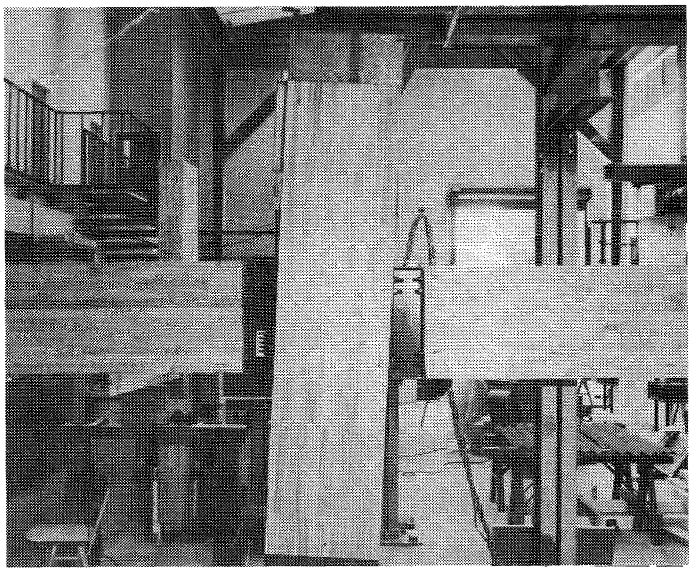


Figure 27 Beam-column assembly with structural steel beam brackets.

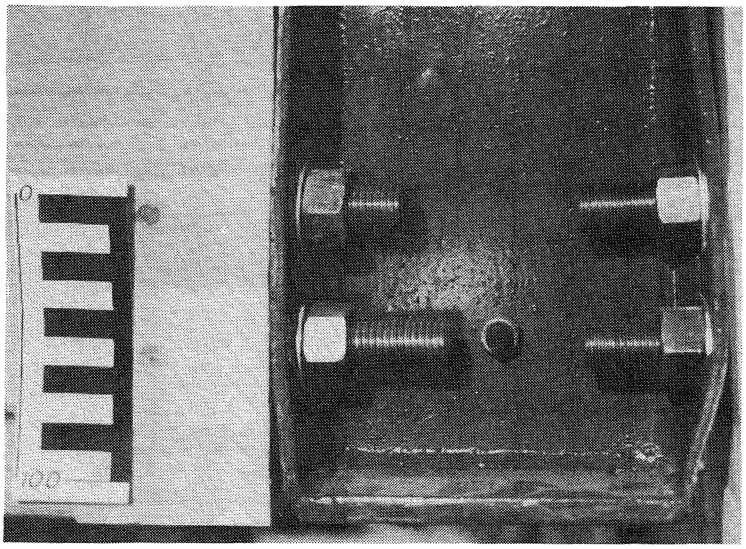


Figure 28 Detail of local bending of steel beam brackets at end of test.

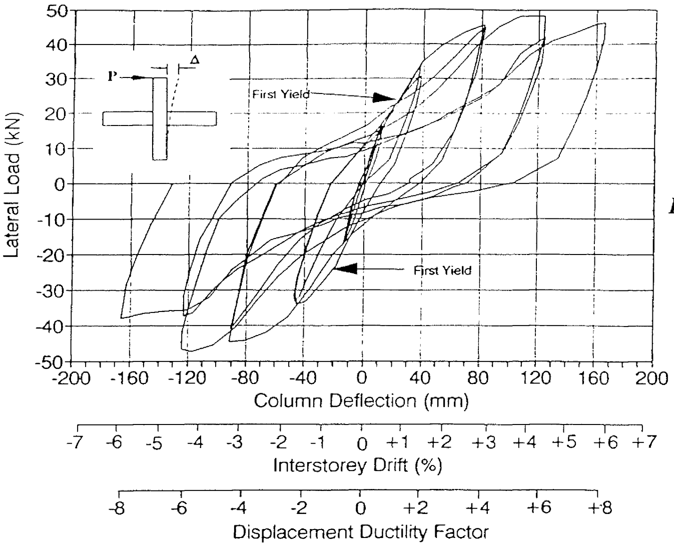


Figure 29 ***Hysteresis loops for beam-column assembly with steel beam brackets.***

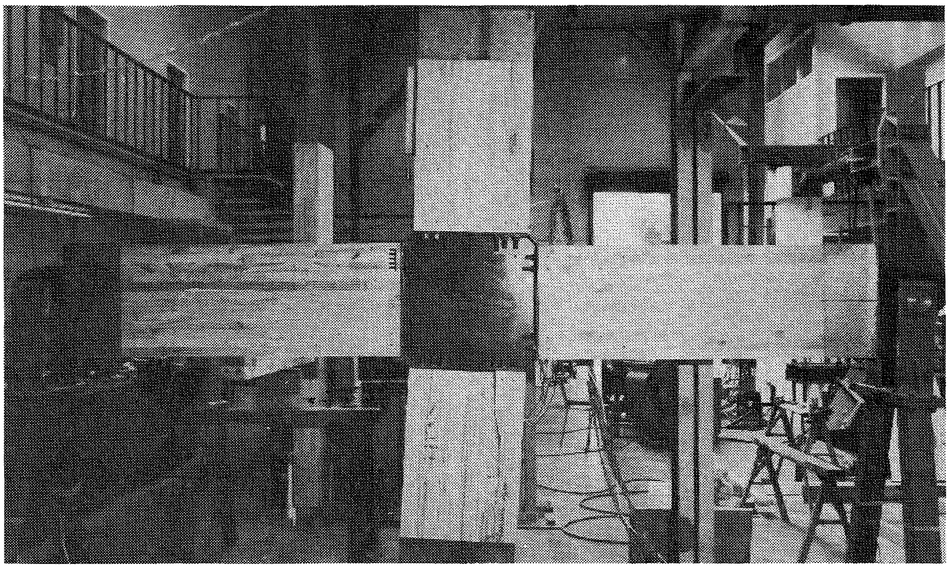


Figure 30 ***Beam-column assembly with structural steel joint.***

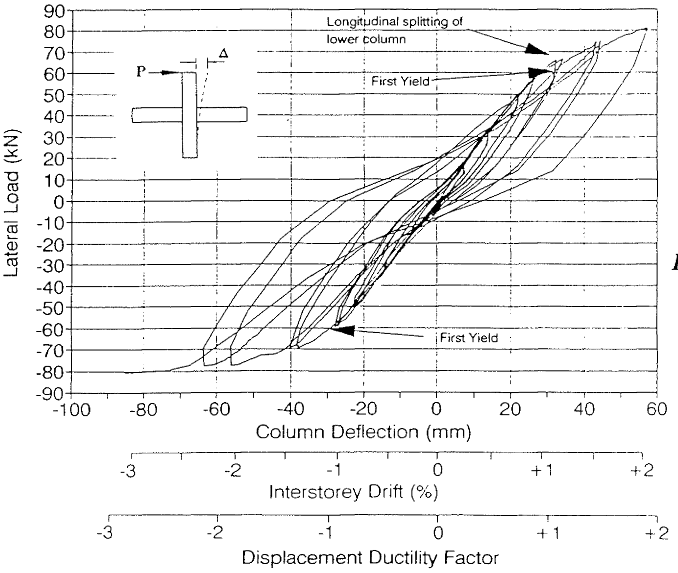
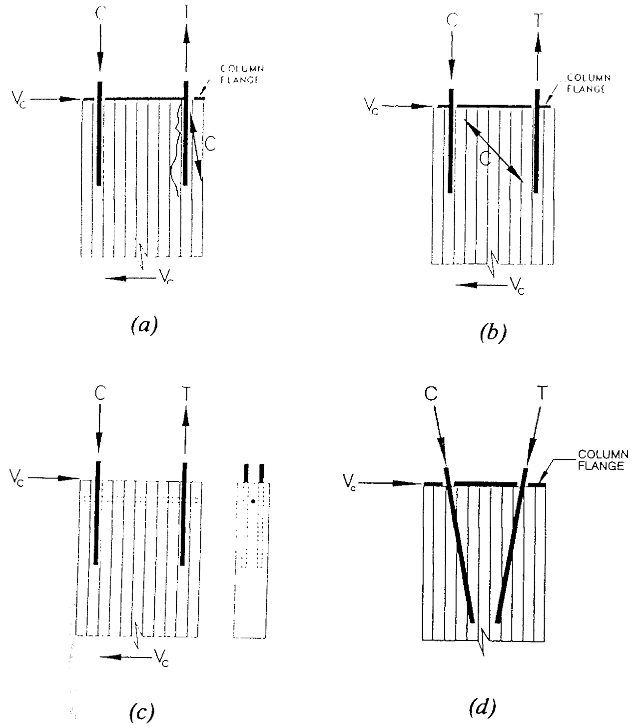


Figure 31 ***Hysteresis loops for beam-column assembly with structural steel joint.***



(a)

(b)

(c)

(d)

Figure 32 *Mechanisms of shear transfer from epoxied bars to steel plate.*

- Beams can be spliced at midspan or where bending moments are low. They are continuous in the area of maximum moment.
- Slender beam sections can be used because the two beams can be braced together for lateral stability.
- The two beams can be boxed together with fire resisting material, so that only the outside face and soffit is exposed to fire.
- The span of the floor joists is reduced.

One test was carried out on the connection shown in Figure 10(d). This connection has fabricated steel brackets fixed to the narrow faces of the column with epoxied rods. These brackets are fixed to the beams with large steel pins, at mid-depth of the beam to avoid any reduction in beam strength. Nailon plates are fixed to the inside faces of the beams so that forces transferred by the steel pins bear against the steel nailon plates and not the timber.

The connection was designed so that ductile yielding would occur as local bending in the steel brackets. Figure 33 shows the assembly at the end of the test, and the load deflection plot is shown in Figure 34, with loading up to ductility of ±8.0. This test confirms that excellent ductility can be achieved with capacity design and careful detailing to ensure that the chosen mechanisms occur as intended.

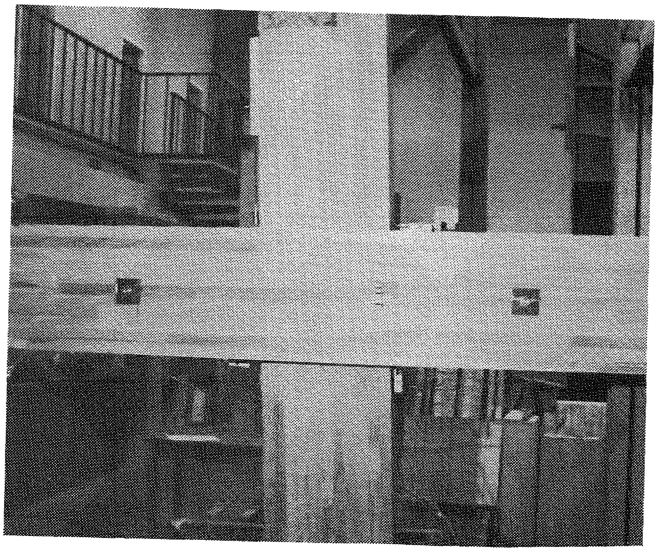


Figure 33 *Double beam system at end of test.*

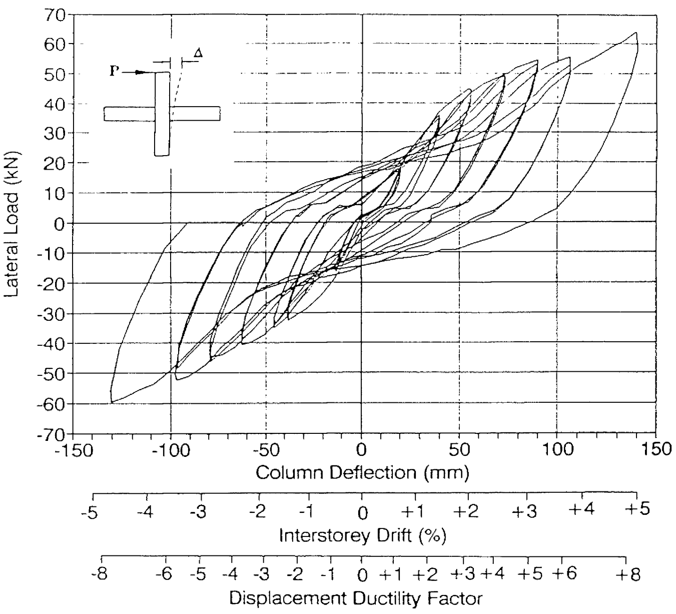


Figure 34 *Hysteresis loops for double beam system with yielding steel brackets and epoxied steel rods.*

MULTI-STOREY BUILDING DESIGN

This section describes a procedure for designing multi-storey timber buildings for lateral loads. The discussion is restricted to moment resisting frame buildings which would incorporate the type of connections tested in this project [18]. The procedures are illustrated for a hypothetical four storey building.

The emphasis of this section is on earthquake loading, to assess ductility demands and hence evaluate the results of the testing programme. Wind loads are not considered in detail because they are generally less than the earthquake loads and they do not rely on ductility in the connections.

Design is in accordance with the New Zealand loadings code [20], using the equivalent static force method.

Design assumptions

The case study building is a typical office building situated in the suburbs of Christchurch. The building site has flexible subsoils. The zone factor Z is 0.8 for Christchurch and the risk factor R is 1.0 for a normal office building. This timber building is not wind sensitive; it is situated in wind region VI with the terrain around the building being category 3.

For this design, the multi-storey timber building has been designed to resist the lateral loads induced during earthquakes and wind using moment-resisting frames positioned around the perimeter of the building. The major column dimension is oriented parallel to the perimeter frame direction, with two columns in each corner of the building. The beam-column connections are assumed to be rigid. The perimeter frame columns are fixed to the foundation. The foundation beam is assumed to be rigid.

The internal beams and columns are designed to support the gravity loads only. Square columns are used to improve the buckling resistance under high gravity loads and ensure that the columns do not attract excessive moment induced by lateral loads. The internal beams connected to the exterior columns in the weak direction are pinned to prevent biaxial bending in the perimeter columns.

Figure 35 shows the floor plan, end elevation and seismic weights of each floor. From the floor plan, it can be seen that the structure is much stiffer in the north-south direction than the east-west direction. This results in a lower period of vibration, hence higher earthquake forces in the north-south direction.

Because of the rectangular shape, wind forces are higher in the east-west direction than in the north-south direction. These factors result in lateral load design being governed by earthquake for north-south loading and wind for east-west loading.

Basic design procedure

The following design procedure was used to find the minimum member sizes required to resist lateral loads induced by earthquakes and satisfy deflection limits using the loading code [20] for a selected structural ductility factor.

1. Assume member sizes for the beams and columns.

2. Determine the seismic weights W_i for each level of the structure.

3. Estimate the fundamental building period.

ULTIMATE LIMIT STATE

4. Determine the lateral force coefficient $C = C_h S_p R Z L_u$ for the ultimate limit state using the structural performance factor $S_p = 0.67$, the risk factor R for this building, the zone factor Z for this site and the ultimate limit state factor $L_u = 1.0$. The basic seismic coefficient is based on the selected structural ductility factor, the fundamental building period and the subsoil at the building site using the spectrum shown in Figure 36.

5. Calculate the base shear force $V = C\Sigma W_i$. Using the equivalent static method determine the earthquake forces F_i at each level for the ultimate limit state.

6. Analyse the structure using elastic analysis with the load combination G & Q_u & E_u where G = dead load, Q_u = seismic live load and E_u = earthquake forces, for the ultimate limit state. Determine the design bending moments, shears and axial forces for the members.

7. Revise the member sizes based on the member actions from step 6, ensuring that the design stresses are not exceeded.

8. Find the building deflections due to the load combination G & Q_u & E_u and the member sizes found in step 7.

9. Use the Rayleigh method to recalculate the fundamental building period using the member sizes from step 7 and the building deflections from step 8. If the calculated building period is close to the assumed period in step 3, then continue the design. Otherwise assume new member sizes and go back to step 1.

SERVICEABILITY LIMIT STATE

10. Determine the lateral force coefficients for the serviceability limit state using the serviceability limit state factor, $L_s = 1/6$, and the basic seismic coefficient C_h based on a structural ductility factor of 1 (i.e. elastic response).

11. Using the equivalent static method of analysis, determine the earthquake forces at each level for the serviceability limit state.

12. Analyse the structure using elastic analysis with the load combination G & Q_s & E_s where Q_s = seismic live load and E_s = earthquake forces for the serviceability limit state. Determine the resulting building deflections.

13. Check that the interstorey building deflections satisfy the limits recommended by the code. If the deflections are too large, increase the member sizes accordingly.

Ductility

The major unknown quality in the above design procedure is the structure ductility factor. The designer has the choice of using a ductility factor of unity (assuming elastic response) resulting in certain member sizes or using a larger ductility factor which will give smaller seismic forces, smaller members, but possible problems with excessive deflections.

The loading code [20] allows ductility factors of up to 4.0 to be used for moment resisting timber frames. If a ductility factor larger than unity is used, then the designer must ensure that the structure has the ability to undergo the necessary inelastic displacements.

To investigate the ductility factor, alternative designs were carried out using factors of one (elastic response), two and four.

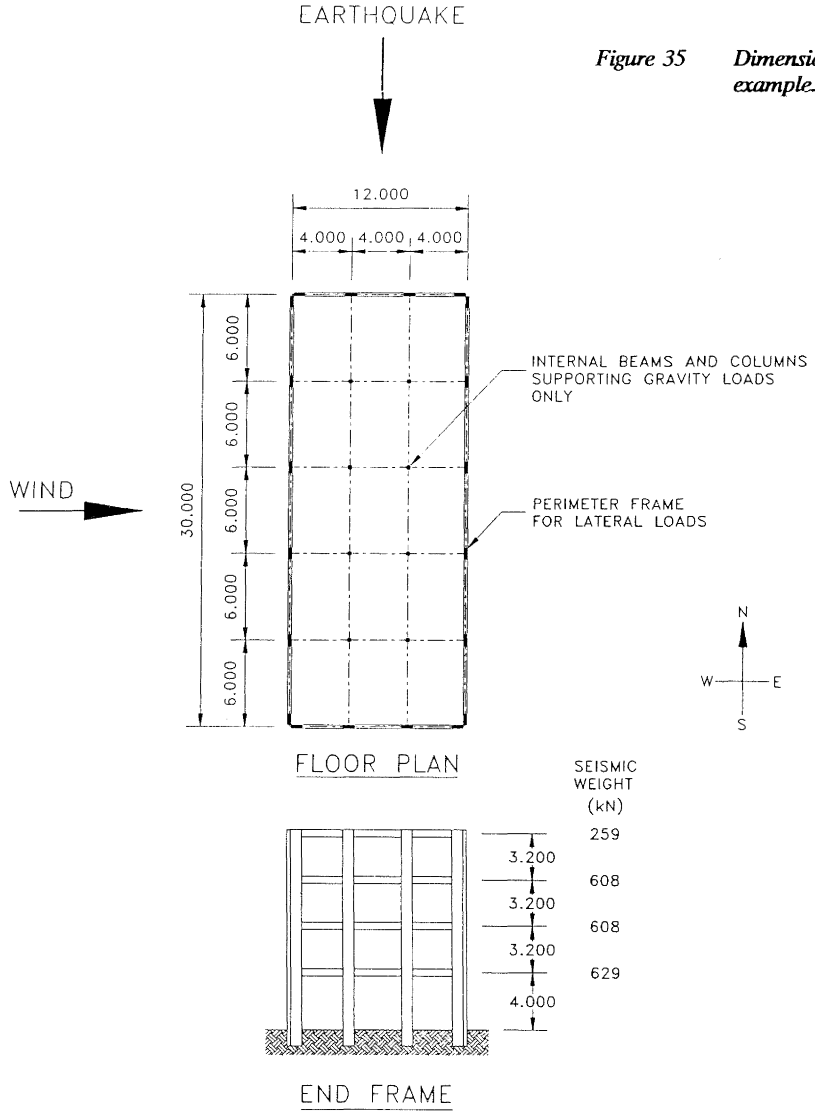


Figure 35 ***Dimensions of four storey building used as design example.***

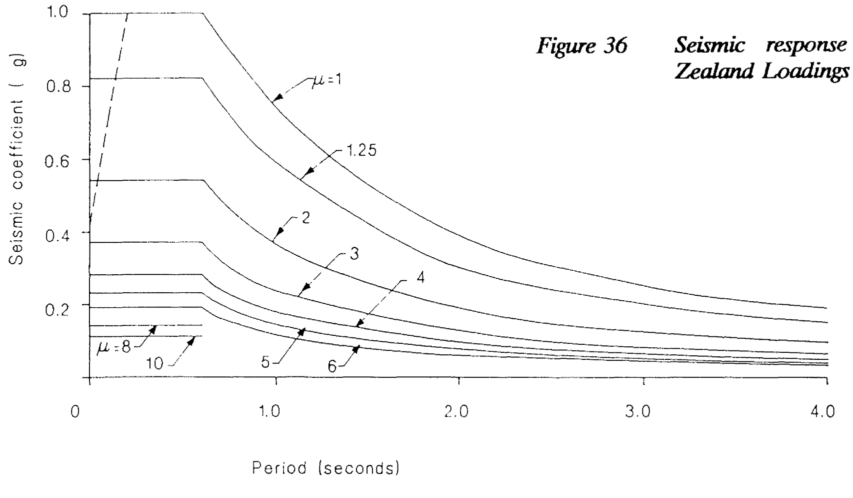


Figure 36 ***Seismic response spectrum from the New Zealand Loadings Code [20].***

Frame stiffness

Analysis of the structure uses an elastic analysis software package which calculates flexural deformations of the beams and columns assuming a rigid joint and no shear distortion. From testing, it was found that a significant percentage of the total column deflection was due to deformations in the connections and shear distortions both in the members and in the joint region. At the limit of elastic behaviour, approximately half of the total deflection is due to member deformations and half due to rotation of the members and joint shear distortion.

To investigate these effects, the design was performed twice, once with full member properties (100% stiffness) then with the flexural stiffness of the beams and columns reduced by half (50% stiffness) to simulate joint and shear distortions.

Results

The results of the design iterations are shown in Table 1, which gives the frame sizes required to keep interstorey deflections less than 1.5% of the storey height in the ultimate limit state.

Consider the results for the north-south frames, using the first design (μ = 1, 100% stiffness) as a reference point. When the assumed ductility is increased from 1.0 to 2.0, member sizes go down because smaller sizes can be used to resist the smaller seismic forces. The smaller forces result mainly from using a lower curve on the spectrum shown in Figure 36, and also because of the increased period.

When the ductility is further increased to 4.0, deflections become a problem and member sizes must be increased slightly. The trend occurs for both 100% and 50% flexural stiffness. For each ductility level, the design using the 50% stiffness level results in slightly longer periods, but larger sections are required to stay with the deflection limits.

Summary

The conclusion from this exercise is that the optimium design can be achieved by assuming a ductility factor in the

Table 1 - Summary of member sizes and periods for the strength and interstorey deflection limits of the ultimate limit state

Ductility μ	Flexural stiffness		N-S Frame	E-W Frame
1	100%	Period	0.85 sec	0.57 sec
		Column size	765 x 180	720 x 180
		Beam size	495 x 135	765 x 135
	50%	Period	0.95 sec	0.80 sec
		Column size	810 x 180	720 x 180
		Beam size	495 x 135	765 x 135
2	100%	Period	0.88 sec	0.80 sec
		Column size	585 x 180	585 x 180
		Beam size	450 x 135	585 x 135
	50%	Period	0.86 sec	0.91 sec
		Column size	720 x 180	720 x 180
		Beam size	585 x 135	630 x 135
4	100%	Period	0.81 sec	0.81 sec
		Column size	585 x 180	630 x 180
		Beam size	495 x 135	540 x 135
	50%	Period	0.81 sec	0.83 sec
		Column size	810 x 180	720 x 180
		Beam size	585 x 135	720 x 135

KEY

☐ Strength governs the design

▨ Stiffness governs the design

intermediate range (say 1.5 to 3). Any attempt to use larger ductility factors will be unsuccessful because member sizes will have to be increased to control deflection, and the resulting strength increase will reduce the ductility demand again.

On the basis of this limited exercise, it is recommended that multi-storey timber frame buildings with ductile connections be designed using a ductility factor of 2.0. This will reduce seismic forces and member sizes below those required for elastic response, and place modest ductility demands on the structural connections.

Many of the better connections tested in this study had no problem achieving repeated load reversals at these (and larger) levels of ductility.

It is important to note that any over-design due to architectural or functional requirements will reduce the ductility demand in buildings of this type. For a more detailed analysis of ductility demand it would be necessary to perform seismic response studies, preferably using non-linear time-history analyses of various buildings exposed to a range of real earthquake records. Such work is beyond the scope of this project, and is not considered urgent because satisfactory designs can be obtained using very modest ductility levels which the connections can easily achieve.

CONCLUSIONS

Moment resisting glulam frames can be used for the lateral load resisting system in single storey and multi-storey timber buildings. The most critical parts of the frames are the connections.

A large number of alternative connections are available, only some of which are suitable for ductile seismic design. In order to achieve ductility in moment resisting connections between glulam members, it is necessary to :

1. Provide steel components that are capable of sufficient ductile yielding and

2. Use a capacity design procedure to ensure that the chosen mechanism can occur with no failure of the wood, the adhesives, or other non-ductile component.

3. Provide careful detailing so that the connection performs as intended.

Large structural ductility factors are not necessary in the design of glulam frame structures because low building masses result in low seismic forces, and interstorey deflections often govern the design process.

ACKNOWLEDGEMENTS

Acknowledgements are due to many people and organizations including the following: The New Zealand National Society for Earthquake Engineering and the New Zealand Timber Design Society provided research scholarships. Glulam timber was donated by Hunter Laminates Ltd, McIntosh Timber Laminates Ltd and Peter Stevens Ltd, through SETMA. Epoxy was donated by Ciba-Geigy Ltd and Adhesives Technologies Ltd. Initial testing was done by Kevin Townsend. Laboratory assistance was provided by Mark Stuart-Jones, David MacPherson, Norrie Hickey and Geoff Hill. Some of these tests were carried out under contract to Hunter Laminates Ltd.

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