

## Dynamic Response of Timber Frames with Semi-Rigid Moment Connections

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### ABSTRACT

To avoid brittle failure in timber members or in connected materials, timber frames should be designed such that ductile failure in mechanically fastened connections prevails under extreme loading conditions such as those arise from earthquakes. Whether failure is brittle (eg. in member) or ductile (eg. in connection) depends to some extent on the characteristics of the connections of the frames. This paper discusses the results from a numerical study using a non-linear frame analysis model. The study investigated the effects of moment-rotation characteristics of timber connections on seismic responses of a timber frame. Of particular interest is the potential mode of failure in the frame. This study shows that potential failure modes and response levels are affected by factors such as stiffness and degree of "pinching" of the hysteresis loops.

### INTRODUCTION

In most design codes (eg. NRC, 1990) the seismic design methodology generally recognises the ability of structures to absorb excitation energy during earthquakes through plastic deformation in its ductile components. Different ductility classes can be found in various design codes. For timber structures, the classification into ductility classes is a rather complex exercise for two reasons:

1. Timber products generally behave linear-elastically until fracture. The only potential source of ductility in timber structures is therefore at the connections containing metal fasteners subjected to forces applied lateral to the axis of the fasteners.
2. Even in timber connections containing metal fasteners, failure may occur primarily in the brittle timber material or in the ductile fasteners.

It is clear that the ability to predict failure modes in timber connection is a prerequisite for classification of timber structures into ductility classes. A series of research studies are currently underway at the University of New Brunswick to develop technical information in this area. Using a 2D frame analysis model and single fastener-joint data as primary input, this paper examines the influences of connection characteristics on the seismic response of a moment-resisting timber frame designed according to the Canadian timber design code (CSA, 1990). Of particular interest is where failure would occur under an over-load situation.

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## ANALYSIS MODEL

Timber products can generally be regarded as behaving linear-elastically until fracture, whereas timber connections with metal fasteners exhibit hysteretic behaviour under dynamic loading conditions. In most cases the load-slip (or moment-rotation) response of a connection exhibits a curvilinear relationship even at low load levels, and the hysteresis loops generally have a high degree of pinching. Therefore accurate prediction of seismic response of timber frames through numerical modelling relies heavily on the ability of a model to accurately describe the hysteresis behaviour of the connections. The general-purpose frame analysis program DRAIN-2D (Powell, 1975) has been used extensively for prediction of seismic response of steel and reinforced concrete structures. When the program was developed, allowance was made for subsequent addition of new element type for accurate description of component behaviour, if the need arises. Taking advantage of this capability, Ceccotti et al (1994) developed an element subroutine to represent the hysteresis behaviour of timber connection. The hysteresis model has undergone a few stages of development and its final form includes a tri-linear envelope curve and the hysteresis loop which is described by a series of straight lines. These lines are defined by the degrading stiffness ( $\tan \gamma$ ), unloading stiffness ( $\tan \alpha$ ), slip stiffness ( $\tan \delta$ ) and slip moment ( $M_s$ ), as is illustrated in Fig. 1 (Ceccotti et al, 1994). Ceccotti (1988) used the "improved" DRAIN-2D program to derive design information in support of Eurocode development in Europe for seismic design of timber structures. The "improved" program is adopted here to study the influence of connection characteristics on frame response.

## FRAME STRUCTURE AND COMPONENT CHARACTERISTICS

The structure considered here is a portal frame consisting of one glued-laminated timber beam and two columns made of the same material. The span of the beam is 10m and the height of the column is 5m. The frame was designed based on a factored uniformly distributed (static) load on the beam of 10 kN/m. Based on this design load, suitable sections for the beam and columns respectively are: 80mm x 532mm Douglas fir 24f-E and 80mm x 418mm Douglas fir 16C-E (CSA, 1990). An elevation of the structure is shown in Fig. 2. The beam and columns are connected by moment transmitting connections containing 12.7mm ( $\frac{1}{2}$  inch) bolts and 6mm mild steel side plates. The bolts are arranged in a circular pattern as illustrated in Fig. 2. The stiffness of the connection can be adjusted by varying the radius and the number of bolts in a bolt group. In this study two types of connections are investigated, Fig. 2. The moment-rotation hysteresis behaviour of each connection group is estimated from the load-slip hysteresis of each bolt using a computer model developed by the first author. The procedure in developing the moment-rotation parameters required as input into DRAIN-2D is as follows:

1. A hysteresis model similar to the one proposed by Dolan (1989) is used to model the load-slip response of each type (eg., parallel or perpendicular to grain loading) of single-fastener joint under reversed cyclic loading.
2. Determine the distance of each bolt from the centroid of the bolt group.
3. Subject the bolt group to a sinusoidal rotation-controlled excitation.
4. At each rotation step, determine the moment required to produce this connection rotation by summing the products of the force acting on each bolt and distance from the bolt to the centroid of the group. This approach is similar to that proposed by Smith (1987) for monotonic loading.
5. Estimate the parameters of the moment-rotation relationship required for input into DRAIN-2D (i.e.,  $\tan \alpha$ ,  $\tan \beta$ ,  $\tan \gamma$ ,  $\tan \delta$ ,  $\tan \theta$ ,  $M_{s1}$ ,  $M_{s2}$  and  $M_s$ ).

The design static moments and forces in the beam and columns are influenced by the stiffness of the semi-rigid connection. Figure 3 shows how the beam moment at mid-span varies with connection stiffness under the static design load of 10 kN/m. In reality due to the nature of the material, it was found that the connection stiffness tends towards the lower tail which implies that the static moments under design load are not sensitive to changes in connection stiffness. As will be discussed later the same cannot be said about the influence of its characteristics on seismic response of the frame.

The reason for specifying the bolt size (12.7mm) and member thickness (80mm) is that it is the only cyclic load test information on single-fastener bolted joints available to the authors (Gutshell and Dolan, 1994). In addition, in terms of fastener diameter, this bolt size generally represents the transition from a predominately ductile fastener failure to a timber failure in timber connections. Gutshell and Dolan's data include both parallel and perpendicular to grain tests. An examination of the data indicates that the shapes of the curves are similar, with the only major differences being in the initial stiffness and the maximum displacement reached before failure. In deriving the moment-rotation behaviour, it was assumed that approximately half of the bolts in a group are loaded parallel and half are loaded perpendicular to grain. Figure 4 shows the generated moment-rotation behaviour of a connection containing 19 bolts and with a radius of 150mm.

#### **INFLUENCE OF CONNECTION CHARACTERISTICS ON SEISMIC RESPONSE OF FRAME**

The frame was analysed with 30% of the design static load acting on the beam. Two earthquake ground excitation records were used: El-Centro North-South and Orion Blvd. North-South which occurred in year 1940 and 1971 respectively in California, USA. Response of the frame was calculated for the first 24 seconds. The peak accelerations for the two records were 0.348g and 0.255g respectively. In total ten combinations of earthquake record, ground excitation multiplier, connection "stiffness" and slip moment  $M_s$  were studied. These combinations and peak response values are summarised in Table 1.

The first four combinations (A,B,C and D) study the effects of connection "stiffness". In A and B each connection contains one "ring" of bolts of radius 150mm (Type I) while in C and D a connection contains two "rings" with radii of 75mm and 150mm (Type II). Figures 5 and 6 show comparisons of the horizontal deflection of the beam under the two earthquake records respectively. It was found that in all cases maximum deflection exceeds the limit of 0.1m (0.02 x height) specified in the National Building Code (NRC, 1990). The maximum combined (bending and axial) stress in each case are : 12.7, 19.8, 12.3 and 24.0 MPa. According to data published by Foschi et al (1989) the characteristic bending strength of 24f-E glulam beams is approximately 34.7 MPa. Thus the stresses induced by these earthquakes would be well below the actual strength of the material. For the connection, Gutshell and Dolan (1994) reported a mean maximum displacement before fracture in their single-fastener tests to be 10.3mm parallel to the grain. If a coefficient of variation of 20% is assumed for their data, this translates into a 5th percentile maximum rotation of 0.046 rad for the two types of connections. In all four cases, the maximum rotations are : 0.021, 0.043, 0.017 and 0.046 rad. Thus despite the large horizontal displacements, the connections appear adequate. It must be mentioned that under the Orion record one of the frame's natural frequencies appears close to the dominant frequencies of the excitation, which leads to magnification of the response magnitudes, as shown in Fig. 6.

In the next two combinations (E and F), combinations A and C above were re-analysed with a ground excitation multiplier of 2. This was done to evaluate the comparative responses of the members and

connections under an overload condition. It was found that while, as expected, the displacement, connection rotation and member stresses all increased substantially, the percentage increases in deformation and rotation are much higher than those for member stresses. In fact while the member stresses are still below the estimated material strength, the rotations of the connection exceed the estimated "safe" limit of 0.046 rad. This behaviour is a result of the reduction in connection stiffness with increasing rotation. Thus it appears that under an overload condition, the studied frame will likely have a "fracture failure" in the connection. This however would have occurred after the frame has undergone extremely large horizontal movements.

The next two combinations (G and H) were similar to combinations A and B except that slip moment  $M_s$  of the connection hysteresis loops was artificially increased from 370 Nm to 5000 Nm. This represents the case where more slender fasteners than 12.7mm diameter bolts were used. The use of more slender fasteners leads to a "fatter" hysteresis loop which can absorb more energy, presumably through more plastic bending of the fasteners. Figure 7 shows a comparison of the horizontal deflection responses between A and G for the El-Centro record. It is noted that the responses differ very little initially, but the latter part of the response appears to have been significantly dampened by the "fatter" hysteresis loop. More importantly, the maximum rotation in the connection and the member stresses in the case of G are lower than those in A. Thus, in contrast to the conclusions reported by Buchanan and Dean (1988), the shape of the hysteresis loop appears to play an important role in seismic response of timber structures. Though not shown here, the difference between the responses was even greater between B and H for the Orion record.

The last two combinations (I and J) study the difference in response when the stiffnesses of all straight line segments of the hysteresis loop and envelope curve were artificially increased by a factor of 100 when compared with combinations A and B respectively. This represents a situation where a heavily over-designed connection was used. It can be noticed from Table 1 that while the deformation levels have decreased substantially, the member stresses have increased by a significant amount. Thus for a timber frame structure the likelihood of having brittle failure in members increases with connection stiffness. Structures with "very stiff" connections are likely to have brittle fracture of material in their members under extreme loading conditions.

## CONCLUSIONS

The following conclusions apply to the timber frame structure with semi-rigid connections considered here:

1. The shape of the hysteresis loops of the connection influences the seismic response of the frame. "Fatter" loops provide additional damping effect on the response magnitudes.
2. When subjected to extreme loading conditions, the potential mode of failure of frames with connections having "normal" stiffness characteristics will be in the connections at large displacement levels. For frames with "stiff" connections, the potential failure mode is likely to be brittle fracture of the member.

## ACKNOWLEDGEMENTS

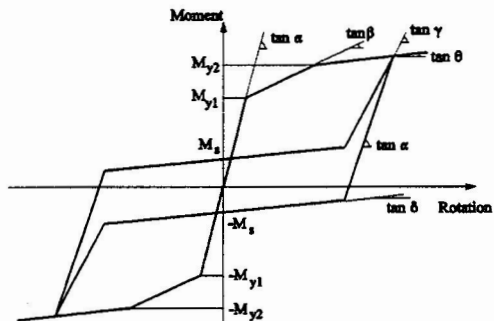
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**Table 1 -** Summary of computer analysis of different combinations of excitation record and connection characteristics.

Combination	Ground acceleration record x multiplier	Connection type	$M_s$ (Nm)	Peak response value		
				Member stress (MPa)	Horizontal displacement (m)	Connection rotation (rad)
A	El-Centro x 1	I	370	12.7	0.17	0.021
B	Orion x 1	I	370	19.8	0.343	0.043
C	El-Centro x 1	II	500	12.3	0.144	0.017
D	Orion x 1	II	500	24.0	0.39	0.046
E	El-Centro x 2	I	370	25.7	0.537	0.062
F	El-Centro x 2	II	500	35.7	0.867	0.11
G	El-Centro x 1	I	5000	11.0	0.135	0.018
H	Orion x 1	I	5000	15.6	0.253	0.030
I	El-Centro x 1	"Stiff"	370	26.7	0.138	0.014
J	Orion x 1	"Stiff"	370	23.4	0.075	0.0005



**Figure 1 -** Moment-rotation model proposed by Ceccotti et al (1994).

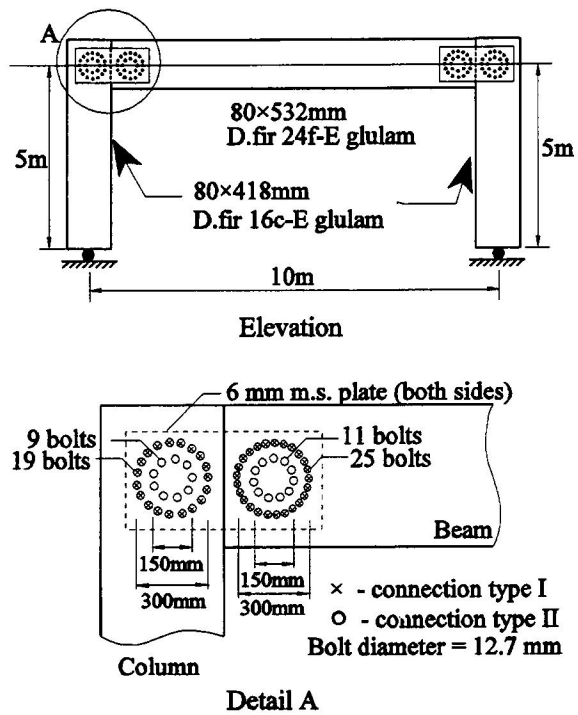


Figure 2 - Details of studied frame.

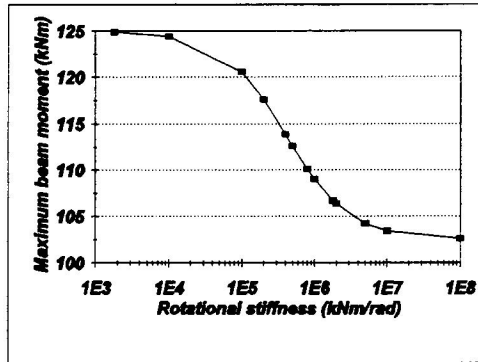


Figure 3 - Maximum beam moment vs rotational stiffness of connection.

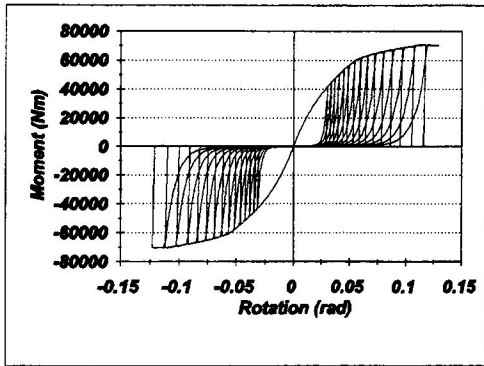


Figure 4 - Generated moment-rotation relationship for connection type II (steel plate-to-column).

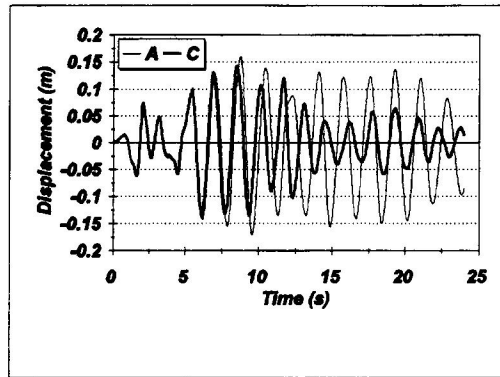


Figure 5 - Horizontal beam displacement for combinations A and C under El-Centro.

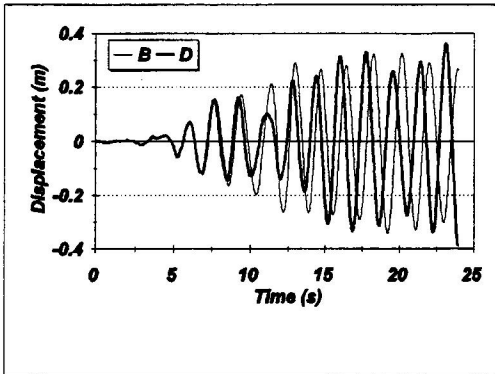


Figure 6 - Horizontal beam displacement for combinations B and D under Orion.

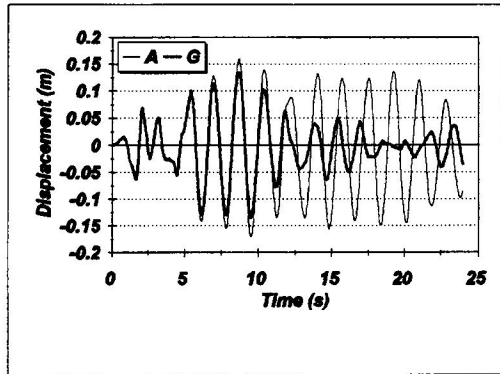


Figure 7 - Horizontal beam displacement for combinations A and G under El-Centro.