Shake Table Testing of a Low Damage Steel Building with Asymmetric Friction Connections (AFC)

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ABSTRACT: The paper describes the shaking table performance of a half-scale two-storey steel moment frame with asymmetric friction connections (AFCs) at the column bases and at the beam ends. The results showed that the beam ends and the base-column joints exhibited bilinear and self-centring response respectively. Residual drifts were less than 0.2% for shake table trials up to 3% peak interstory drift. Even at a peak interstory drift of 6.5%, the residual drift response was still only 0.7%.

1 INTRODUCTION

In light of recent major seismic events worldwide, it has become apparent that while modern design provisions ensure life safety in modern structures during severe earthquake events, structures may be damaged and require extensive repair or replacement. A recent research effort is to develop new superior solutions that will also minimise the possibility of structural damage. Such low-damage structures may be achieved by developing a behaviour that is elastic and/or by incorporating the use of energy dissipaters such as friction connections.

Friction connections can be categorised as the Symmetric Friction Connection (SFC) and Asymmetric Friction Connection (AFC). In these devices, energy is dissipated by permitting sliding of two surfaces that are in contact with each other. Oftentimes, the sliding force is increased by a clamping force from pre-tensioned high strength bolts. A notable case study is that by Yang and Popov (1995) where the researchers utilized SFCs in the top and bottom flanges of beams in rotational beam-column connections for steel MRFs. The experimental test results showed a non-linear frictional behaviour with limited degradation. However, the performance of this particular SFC, where sliding occurs in the top flange, is affected by an overlaying floor slab (Khoo et al. (2014)). Connections which do not slide at the top flange have also been developed and tested. These rotate about the top flange plate and sliding only occurs in the bottom flange plate thus minimising interactions with the overlaying floor slab and the effects of beam elongation. This occurs in Asymmetric Friction Connections (AFCs) (Clifton (1996, 2005), MacRae et al. (2010)) and Symmetric Friction Connections with sliding on the bottom flange only.

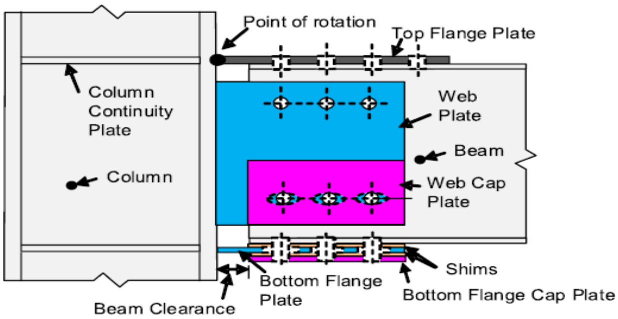


Figure 1. Asymmetric friction connection (AFC) in beam column joint (MacRae et al. 2010).

Past studies of AFC were mostly experimental studies with AFC applied to beam-to-column moment resisting joints (Clifton, 2005; MacRae et al. 2010), braces (Chanchí et al. 2012 & 2014) and base-columns connections (Bourzouie et al. 2015a, b). All of these configurations can possess good seismic performance. However, the dynamic performance of AFCs in entire steel moment resisting frames (SMRFs) has not been validated.

It is clear that experimental validation of the seismic performance of whole AFC frame systems would develop further confidence amongst engineers for their adoption.

This paper seeks to address this need by describing the shake table testing of a half-scale steel frame AFC building. In particular, answers are sought to the following questions:

1. How do these friction connections behave during excitation?
2. What is the peak and residual response for this particular structure?
3. Can it be considered to be a low damage structure?

2 TEST SPECIMEN DESIGN

The building was designed as a full-scale prototype building according to the Equivalent Static Method in NZS1170.5 (2004). It was designed to reach 2.5% drift in a Design Level (DL) earthquake event (1-in-500 year earthquake shaking) based in Wellington, with Z=0.4 and soil type C. The test specimen is composed of two steel frames (yellow frame) with AFC connections in the base column and beam to column joints as shown in Figure 2. In the transverse direction, the two frames are joined by short transverse beams. The length of the beams, columns and the amount of the mass at each floor are provided in

Table **1**. The black beams under the yellow frame are part of a catch frame to provide safety during tests. The white channels on top of the yellow beams forms a tray for the required added mass to satisfy similitude for the shake table tests.

Figure 2. Test building constructed frame, Front Frame, NS direction.

Table 1. Properties of prototype and test buildings

Items	Properties
Inter-storey height [m]	1.6
Bay length [m]	3.2
Building width [m]	2
Mass per floor [ton]	6.25
Column section	100 UC 14.8
Beam section	100 UC 14.8

The prototype was scaled to meet the requirements of the shake table. Considering those limitations of the shake table as well as the added artificial mass, three fundamental Scaling Factors (SF) were determined as the length, mass and stress. These were set to 0.5, 0.25 and 1, respectively. Thereafter, other SFs such as time, acceleration, force were calculated based on the above three factors. In Table 2, the similitude relationships for key quantities often considered in structural engineering are presented. Here, the symbol M refers to the model and symbol P refers to the prototype.

Table 2. Similitude Relationships

Quantity	Symbol	Similitude
Length	L	$L_P = 0.5\ L_M$
Area	A	$A_P = 0.25\ A_M$
Mass	m	$m_P = 0.25\ m_M$
Velocity	v	$v_P = 0.7\ v_M$
Acceleration	a	$a_P = a_M$
Force	F	$F_P = 0.25\ F_M$
Moment	M	$M_P = 0.125 M_M$
Stress	σ	$\sigma_P = \sigma_M$
Strain	ε	$\varepsilon_P = \varepsilon_M$
Time	t	$t_P = 0.7\ t_M$

3 INSTRUMENTATION

For this study, the instrumentation of the test frame consisted of a combination of string pots, strain gauges and potentiometers. The string pots had a maximum displacement of 500mm in each direction from the resting position. The relative displacement of each storey, along with the location of each string pot allowed for the determination of the inter-storey drifts.

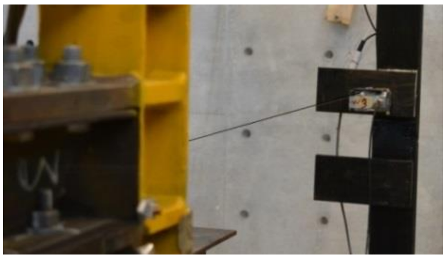
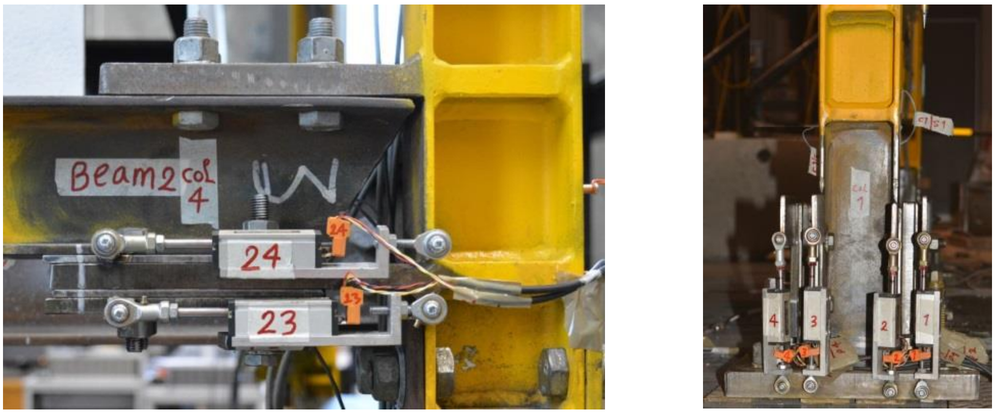


Figure 3. Typical string pot connection to a reference frame

The local behaviour of the beam-column connections and column base connections were recorded by two and four parallel potentiometers respectively. These were placed across each connection interface (Figure 4). The neutral axis and rotation of the connection are subsequently estimated by interpolation assuming a linear strain profile across connections.



(a) Beam Ends **(b) Base column joints**

Figure 4. Potentiometers used for measuring the sliding distance

4 TEST PROTOCOLS

The testing input was a set of 2 earthquakes ground motions selected from NZ local earthquake events as shown in Table 3. The earthquakes were scaled to get peak drifts of about 2.5%, 4.5% and 6.0%. The time-step of each ground motion was reduced by a factor of 0.707 to fulfil similitude requirements and to ensure similar peak acceleration. Figure 5 compares the NZ code (NZ1170.5) spectra with spectral acceleration of ground motions with scale of 100%.

Table 3. Ground motions

Earthquake name	Station Name	Orientation	Date	Time	M_w
Christchurch, NZ	CCCC	N-S	22nd Feb. 2011	12:51pm	6.2
Christchurch, NZ	REHS				

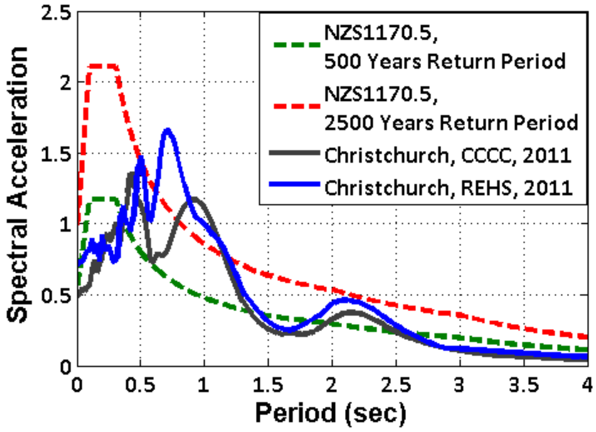


Figure 5. Spectral Acceleration of ground motions compared with NZ Code Spectra (NZ1170.5), (5% damping, Z=0.4, Soil C, S_p=1.0), Time scale=0.7, Acceleration Scale=1.0.

5 EXPERIMENTAL RESULTS

5.1 System Properties

Snap-back tests (free vibration) are performed on the test structure as shown in Figure 6 to determine the natural period and damping ratio of the structural system before the shake table experiments. The fundamental period and damping ratio, calculated by Eq. 1, were found to be 0.45s and 3.4% respectively. Torsional displacements of the frame were negligible in these and the shake table tests.

$$\xi = \frac{\ln\left(\frac{A_0}{A_1}\right)}{2\pi} \quad (1)$$

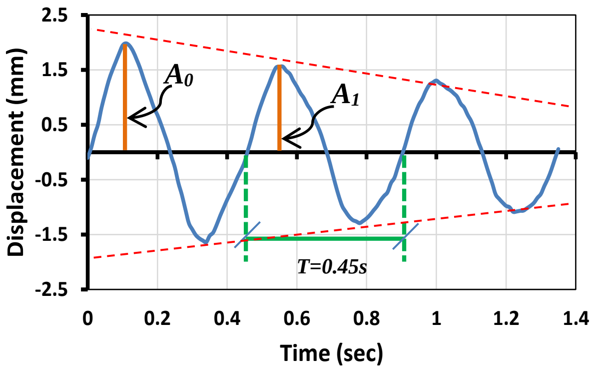
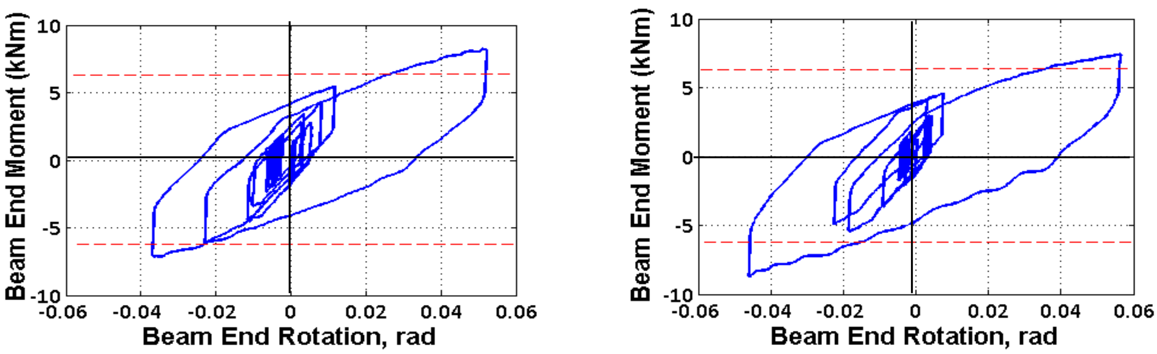


Figure 6. Free vibration tests to determine dynamic properties of the system

5.2 Asymmetric Friction Connection (AFC) Behaviour

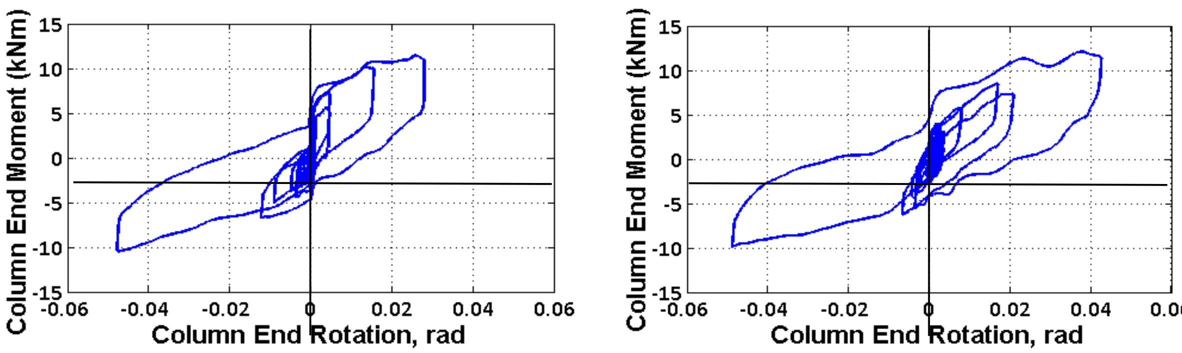
Figure 7 shows that the hysteretic behaviour of the AFC at beam end (previously shown in the Figure 2) (1st floor, North end) are essentially bilinear. Beam initial full sliding occurred at a moment of 6.8 - 7.0 kNm. A sliding bottom flanges force can be calculated by the moment divided by the distance between flanges, M/d 7kNm/0.097mm = 72kN. The effective friction coefficient (MacRae, 2010), computed as the flange sliding force divided by the sum of the bolt proof loads for the M12 Grade 8.8 structural bolts of 45kN, divided by the number of interfaces is = 72kN / (45kN/bolt x 4bolts x 2interfaces) = 0.2. This is close to the value recommended for design of 0.21 and it is greater than the minimum dependable value of ϕ0.21 = 0.7 x 0.21 = 0.15 (MacRae et al. 2010). Red dashed horizontal lines in Figure 7 shows the sliding moment associated with friction coefficient of 0.2. The peak moment (8.5 kN.m) obtained is associated with an effective friction coefficient of 0.24, which is less than the overstrength value recommended for design of ϕ_o0.21 = 1.4 x 0.21 = 0.30 (MacRae and Clifton, 2015).



(a) Christchurch, NZ (CCCC)-Scale 90% **(b) Christchurch, NZ (REHS)-Scale 75%**

Figure 7. Hysteretic Behaviour of AFC at Beam End

Figure 8 shows the hysteretic behaviour of the AFC in a base column joint (previously shown in the Figure 2) (North end). The self-centring characteristics is a result of the axial force on the column. The column base AFC sliding occurred at a column moment of about 7kNm. This indicates that the effective friction coefficient, after considering the effect of axial force, is about 0.2.

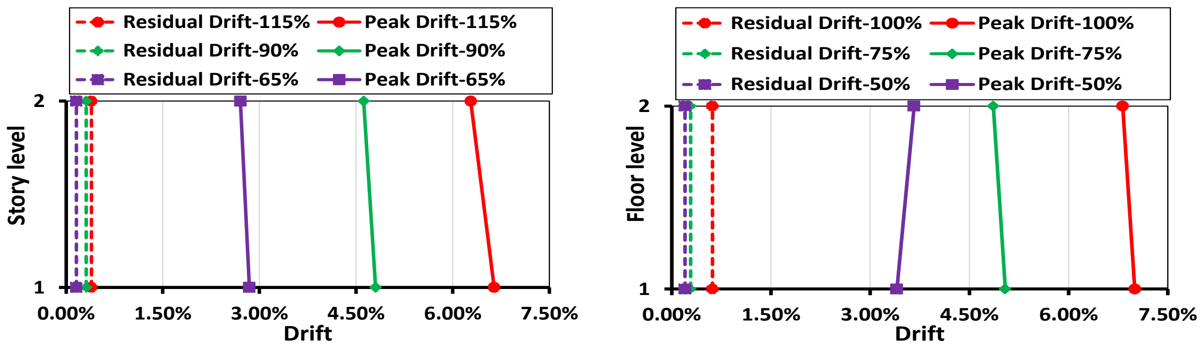


(a) Christchurch, NZ (CCCC)-Scale 90% (b) Christchurch, NZ (REHS)-Scale 75%

Figure 8. Hysteretic Behaviour of AFC in a Base Column Joint

5.3 Peak and Residual Drift Response of the Structure

Figure 9 shows the peak and residual drift response of the structure over the height under different ground motion records with different intensity levels. The frame was put back to its initial straight position, and new bolts were inserted and tightened before each run. The number shown after "Residual and Peak Drift" is the SF of acceleration for the recorded considered. It may be seen that up to peak drift response of 3% the residual drift response is not more than 0.2%, but with increasing the peak drift response up to 6.5%, the residual drift response increases up to 0.7%.



(a) Christchurch, NZ (CCCC) (b) Christchurch, NZ (REHS)

Figure 9. Peak and Residual Drift Response

6 CONCLUSION

This paper describes seismic shake table tests on a half-scale two-storey steel building incorporating asymmetric friction connection (AFC) at the base of steel columns and beam-column joints. Based on the experimental results obtained, the following conclusions can be drawn:

1) The hysteresis loop shapes for beam-column and base-column joints have bilinear and self-centring characteristics respectively. The effective friction values were consistent with that used in design with a nominal value of 0.21 and understrength, ϕ, and overstrength, ϕ_o, values of 0.7 and 1.4 respectively.

2) It also shows that for up to 6.5% peak drift, the residual drift is not more than 0.7%.

3) The overall structural performance was excellent, without any significant member damage. It can be considered to be a low-damage structure. Also, even after very severe shaking where there are some residual displacements, manual restraightening may be carried out.

7 ACKNOWLEDGEMENT

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