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Short-term cyclic performance of metal-plate-connected wood truss joints

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Abstract. The objective of this research was to evaluate the performance of metal-plate-connected truss joints subjected to cyclic loading conditions that simulated seismic events in the lives of the joints. We also investigated the duration of load factor for these joints. We tested tension splice joints and heel joints from a standard 9.2-m Fink truss constructed from 38- × 89-mm Douglas-fir lumber: 10 tension splice joints for static condition and for each of 6 cyclic loading conditions (70 joints total) and 10 heel joints for static condition and for each of 3 cyclic loading conditions (40 joints total). We evaluated results by comparing the strengths of the control group (static) with those of the cyclic loading groups. None of the cyclic loading conditions showed any strength degradation; however, there was significant stiffness degradation for both types of joint. The results of this research show that the current duration of load factor of 1.6 for earthquake loading is adequate for these joints.

Key words: wood engineering; wood connections; duration of load; seismic loads; tension splice joint; heel joint; fink truss.

1. Introduction

Metal-plate-connected (MPC) wood trusses are widely used in the construction industry in the United States of America. Their design requirements are based on static and monotonic loading conditions, as described by industry standards (*TPI 1-1995* 1995). Very little, however, is known about the cyclic characteristics of MPC wood trusses. Because the connections are primary factors in the overall response of a structural assembly, results from connections tests are essential for

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understanding the behavior of wood buildings during earthquakes. Test results from this study might also provide insight into the duration of load (DOL) behavior of MPC joints.

The study of the performance of MPC truss joints subjected to cyclic loading is a fairly new research area. Most research associated with MPC joints deals with the response to static loads (Gupta *et al.* 1996) and the structural properties resulting from static tests, such as ultimate load, design load, and stiffness. In static tests, however, the loads do not represent the short, random, reversing loads experienced during an earthquake, and the joints subjected to the tests are not the actual connections used in the structural assemblies.

Cyclic loading of joints has received some attention recently (Emerson and Fridley 1996, Kent *et al.* 1997, Redlinger 1998). Emerson and Fridley (1996) tested tensile joints under cyclic loading and observed loss in initial stiffness, but no drop in strength. Kent *et al.* (1997) tested MPC joints under a historical (Northridge), artificially generated earthquake simulation, sequential phased displacement (SPD) loading, and cyclic loads. They found that the earthquake simulations caused no strength degradation, but that earthquake loadings affected the axial stiffness of the heel joints. The damage that accumulated in the connection during the SPD load depended on the level of displacement. Finally, Kent *et al.* (1997) found that large cyclic loads cause significant strength loss in MPC joints. Redlinger (1998) tested joints under simulated hurricane and impact loads and observed stiffness increase in the joints but no strength degradation. For tension splice joints, the accelerated ramp load produced the same results as the static ramp load in one-tenth the time.

Currently, there is no standard test method for evaluating DOL of MPC joints. Dolan *et al.* (1996) conducted the most recent DOL research on nailed and bolted joints, applying several types of cyclic loading conditions to evaluate the DOL factor. By comparing capacities and ductilities of joints with and without prior cyclic loading, they concluded that a DOL of 1.6 for seismic loading of nailed and bolted joints is adequate. Their tests were based on Dolan's (1989) study on shearwalls, which concluded that 6-8 cycles (at 1 Hz) at the design load, as described by the *National Design Specification for Wood Construction* (1997), represented the accumulated damage during a "reasonable" seismic event.

Based on their assumptions, Dolan *et al.* (1996) developed a load controlled loading function that approximated a representative 10-year loading designed to represent several seismic events in the life of the structure. Assuming that this loading represented the cumulative loading during a 10-year period, the researchers produced a test that evaluated the DOL factor used in design. Their tests included a loading function that cycled at 1.0 times the design load for 30 seconds, at 1.6 times the design load for 15 seconds, and at 2.0 times the design load for 8 seconds, all cycles at 1 Hz. They surmised that the first stage (30 seconds at 1.0 times the design value) simulated four events of minor magnitude (i.e., minimum design level), the second stage (15 seconds at 1.6 times the design value) simulated two seismic events at the design level, and the third stage (8 seconds at 2.0 times the design value) simulated a single major seismic event. They concluded that these tests conservatively estimated the loading during the structure's lifetime. Therefore, a similar approach was used here for MPC joints.

The objective of this study was to evaluate the performance of MPC truss joints under cyclic loads that simulated earthquake loadings in the joints. We used loading conditions similar to those of Dolan *et al.* (1996) to evaluate DOL of MPC joints. We compared the behavior of MPC wood truss joints that had undergone static tests with the behavior of joints subjected to cyclic load histories at different load levels. We evaluated the strength and stiffness of MPC tension splice joints and heel joints for cyclic loading conditions that simulated seismic events in the life of a joint. We used the strength degradation in joints to investigate the DOL factor of 1.6 for seismic loading for MPC joints.

2. Materials and methods

2.1 Materials

We obtained machine stress rated (1800f – 1.6E), 38- × 89-mm (referred to as 2 × 4) Douglas-fir lumber 3 m long, conditioned it to 14% moisture content, and measured the modulus of elasticity (MOE) of each piece with an E-Computer (Metriguard, Model 390, Pullman, Washington). Making two or three joints from each board, we connected the tension splice joints with two 2 × 4s 0.5 m long, and 76- × 102-mm metal connector plates (MCPs), and the heel joints with two 2 × 4s 0.5 m long, and 76- × 127-mm MCPs. The heel joints were fabricated at a slope of 4:12. A typical metal connector plate is shown in Fig. 1. Properties and dimensions of the MCPs are given in Freilinger (1998). Many other types of MCPs used in truss fabrication along with various joint configurations are given in Gupta *et al*. (1996).

We employed a sample size of 10 for each joint and loading condition, and determined specific gravity and moisture content according to ASTM Standards D2395 (ASTM 2000a) and D4442 (ASTM 2000b), respectively.

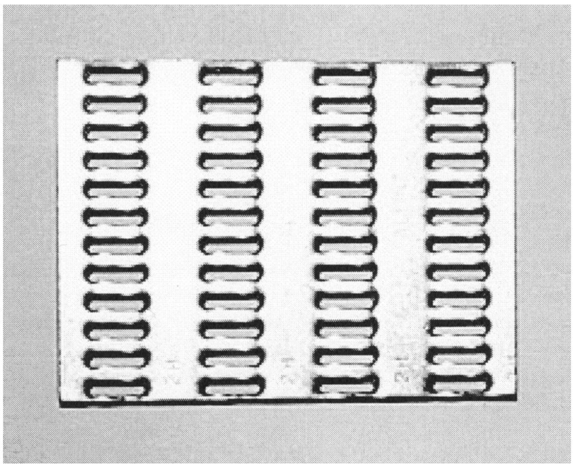

Fig. 1 A typical metal connector plate

2.2 Testing apparatus

We tested the joints with a trapezoid-shaped frame similar to that developed by Gupta and Gebremedhin (1990) and applied the loads with a 49 kN capacity hydraulic actuator. Fig. 2 and Fig. 3 show the schematics of the test setups for the tension splice joints and heel joints, respectively.

We measured the axial load on the tension splice joints with a load cell and tested the heel joints with two similar Sensotec load cells. We measured the relative displacements on either side of each tension splice joint (Fig. 2) with direct current linearly variable differential transducers (LVDTs). We

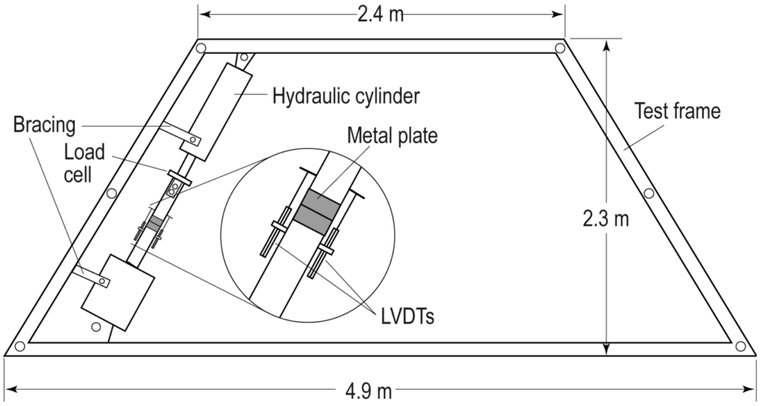

Fig. 2 Schematic of test setup with tension splice joint

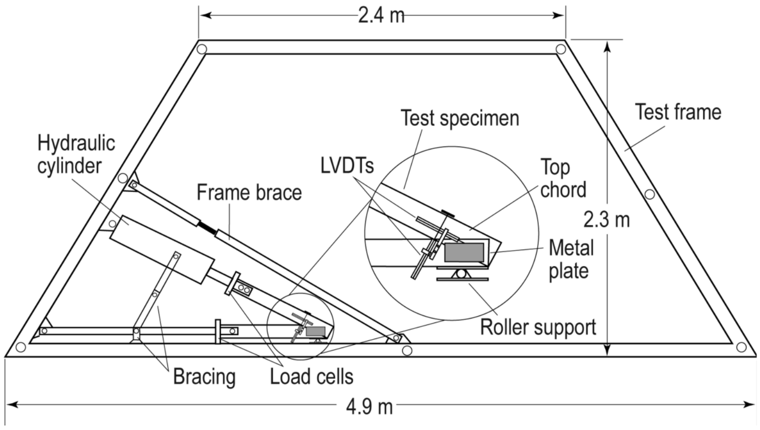

Fig. 3 Schematic of test setup with heel joint

tested the heel joints with two LVDTs, one to measure the axial displacement across the metal plate and the other to measure the rotation of the top chord away from the bottom chord of the joint (Fig. 3).

2.3 Test procedures

We tested the joints under eight loading regimes (Table 1), the first of which was a static loading test that functioned as a control. The next seven loading regimes tested the joints under various cyclic loading conditions that simulated seismic events in the life of a joint. These cyclic loading regimes are shown and explained in Table 1. Each cyclic loading regime is designated by letter ‘C’ and consisted of ramping the joint to dead load, one to three cyclic loading regimes, designated by a number after the letter ‘C’ and then ramping to failure if joint survived the cyclic loading. Each number after the letter ‘C’ indicates the multiple of design load. An example loading function, C132, is shown in Fig. 4. Letter ‘C’ in C132 indicates cyclic loading, number ‘1’ indicates cycles of one times the design load for 30 sec at 1 Hz, number 3 indicated 1.33 times the design load for 15 sec at 1 Hz, and number 2 indicates two times the design load for 8 sec at 1 Hz. All other cyclic loading regimes are explained in Table 1.

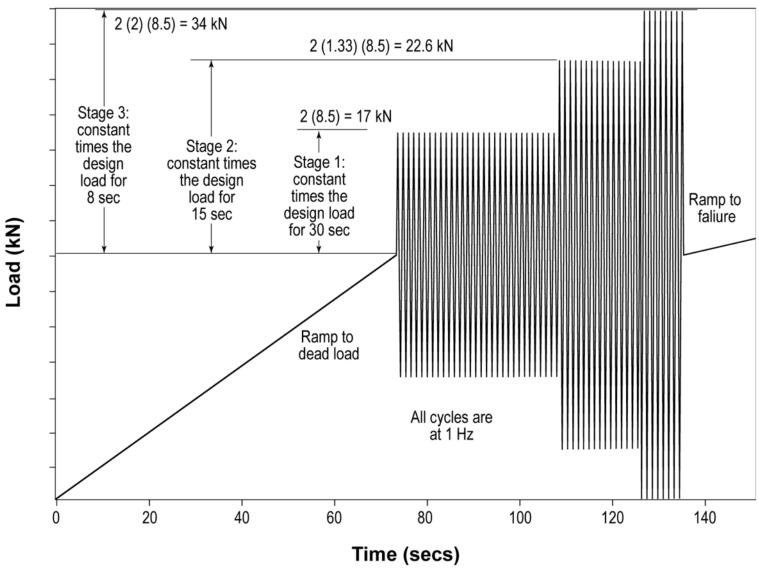

Fig. 4 Example cyclic loading function (C132). Constant refers to the factor multiplied by the allowable design load to determine severity of the represented seismic event.

Table 1 Loading conditions for MPC joints

Joint type	Static	C1	C6	C16	C162	C132	C8	C18
Tension splice	√	√	√	√	√	√	√	-
Heel	√	-	-	√	√	-	-	√

Static: Ramp at 3.5 kN/min to cause failure in 7-10 minutes.
C1: Ramp to dead load + 1.0 DL for 30 seconds (1 Hz) + ramp to failure
C6: Ramp to dead load + 1.6 DL for 30 seconds (1 Hz) + ramp to failure
C16: Ramp to dead load + 1.0 DL for 30 seconds (1 Hz) + 1.6 DL for 15 seconds (1 Hz) + ramp to failure
C162: Ramp to dead load + 1.0 DL for 30 seconds (1 Hz) + 1.6 DL for 15 seconds (1 Hz) + 2.0 DL for 8 seconds (1 Hz) + ramp to failure
C132: Ramp to dead load + 1.0 DL for 30 seconds (1 Hz) + 1.33 DL for 15 seconds (1 Hz) + 2.0 DL for 8 seconds (1 Hz) + ramp to failure
C8: Ramp to dead load + 1.8 DL for 30 seconds (1 Hz) + ramp to failure
C18: Ramp to dead load + 1.0 DL for 30 seconds (1 Hz) + 1.8 DL for 15 seconds (1 Hz) + ramp to failure
DL: Design Load

Our tests were similar to those of Dolan *et al.* (1996), except that we ramped the joints to dead load, similar to Kent *et al.* (1997), before applying the cycles, in order to produce a more realistic loading (i.e., dead load is always present) on the joints. The tension splice joint dead load was 4 kN; the heel joint dead load was 7 kN. Similar to Dolan *et al*. (1996), 1.0 times the design load simulated a minor event, 1.33, 1.6, and 1.8 times the design load approximated possible design events, and 2.0 times the design load simulated a major event.

We tested the tension splice joints under static condition and six cyclic loading conditions (Table 1), and the heel joints under static condition and three cyclic loading conditions, based on the results for the tension splice joints. If a joint survived the cyclic tests, it was then ramped to failure.

2.3.1 Static tests

We applied a tensile static ramp load of 3.5 kN/min to 10 tension splice joints. The tensile ramp load was to cause failure in 7 to 10 minutes. For the heel joints, we applied a compressive static ramp load to the top chord to cause failure in 7 to 10 minutes.

2.3.2 Cyclic tests

The C1 loading condition represented four minor seismic events (1.0 times the allowable design load for 30 sec at 1 Hz) in the life of a joint. It provided a baseline comparison for all the cyclic tests. We designed C6 to investigate the current DOL factor of 1.6. The test replicated the loading caused by four design level events in the life of the truss. This test was the simplest evaluation of the DOL factor and did not include any additional minor or major loading events in the life of the joint. C16 provided more conservative evaluations of the DOL factor by adding four minor events before the two design level events. Similar to Dolan *et al*. (1996), C162 provided the most conservative investigation of the DOL factor of 1.6. The first step of the loading simulated four minor seismic events, the second step simulated two design events, and the last step simulated a major seismic event in the life of the structure. C132 was based on the same loading model as C162, but the magnitude of the second stage of loading was determined from a DOL factor of 1.33, rather than 1.6.

C8 evaluated the possibility that the DOL factor could be as large as 1.8. This loading simulated four design events at a higher severity of 1.8 times the design load in the life of the structure, based on a DOL factor of 1.8. C18 provided a similar evaluation for a DOL factor of 1.8. This loading simulated four minor events and two design events in the life of a structure, based on a DOL factor of 1.8.

The design loads for both joints were obtained from the static tests without consideration of the dead load. Therefore, when joints were loaded to certain times the design load after loading them to dead load, the actual cyclic load is higher than the constant times the design load. Although this was not the original intent of the cyclic loading, the actual cyclic loading is more conservative (i.e., higher than intended), and results still apply to all cyclic loading conditions used here.

2.4 Property evaluation

For static tests, we determined the strength and stiffness from load deflection data. The strength of the tension splice joint was taken as the maximum load sustained by the joint. We considered the strength of the heel joint to be the maximum force in the top chord. The design load for both joints was calculated by dividing the average strength of each joint by 3, as described by industry standards (*TPI 1-1995* 1995). We determined the stiffness at the design load, coincidentally 8.5 kN for both joints. The stiffness was determined by dividing the design load by deflection at the design load. For tension splice joints, the deflection was the opening of the joint (average of deflection from two LVDTs shown in Fig. 2). For heel joints, the deflection along the top chord (deflection from LVDT along the top chord in Fig. 3) was used to determine the stiffness.

For cyclic tests, the strength was taken as the maximum load from the ramp load applied after the cyclic loading (Fig. 4). In cases where the joints failed during the cyclic loading, we considered the maximum load experienced by the joint during the cycles as the strength and included it in the strength average for that joint. The stiffness for the joints was calculated at the design load determined from static tests), using the ramp load function after the cyclic tests. This represented a secant line drawn from the origin of the load deflection curve to the design load. For these tests, this stiffness included the increased deflection caused by the cyclic loading, which was applied before the design load. This was done to estimate the stiffness of a joint after it had been subjected to a series of simulated earthquakes. We could not use some of the joints for stiffness calculations because they failed to reach the design load following the cycles; therefore, no ramp load after cycles and no design load to calculate stiffness. A few other joints had bad deflection data, so we could not determine their stiffness and included only their strength in the analysis.

The comparisons tested the null hypothesis that the difference in two population means is zero. We calculated p-values to determine significant differences by means of a t-test. The following comparisons were made at a 95% confidence interval: (1) strength of joints from static tests with strength of joints from cyclic tests and (2) some select comparisons among cyclic tests. Although we performed no statistical tests for stiffness because of the small sample size, stiffness values for various tests might indicate degradation in stiffness due to cyclic loading.

Energy dissipation and hysteretic stiffness (stiffness for each cycle) were also calculated. Other material properties measured were modulus of elasticity, specific gravity, ring count, percent latewood, grain orientation, and moisture content. These quantities are not discussed in this paper, but are given in Freilinger (1998).

3. Results and discussion

3.1 Tension splice joints

Table 2 shows average strength, stiffness, moisture content, and specific gravity for tension splice joints tested under static condition and six types of cyclic loading condition. The three most common failure modes were tooth withdrawal, plate failure, and combined tooth withdrawal and wood failure. Only one joint failed in the plate. The rest of the joints failed initially because of tooth withdrawal, with various amounts of wood failure. Kent *et al.* (1997) and Gupta and Gebremedhin (1990) observed similar failure modes. Table 3 shows p-values for various comparisons of tension splice joint strength. These are discussed in the following sections.

Table 2 Tension splice joint test summary[a]

Test	Strength[b] (kN)	Stiffness[b] (10^5 N/mm)	Moisture content (%)	Specific gravity
Static	25.6 (10, 12%)	0.47 (10, 17%)	13	0.49
C1	25.1 (10, 24%)	0.33 (8, 43%)	12	0.47
C6	25.0 (10, 29%)	0.20 (8, 43%)	13	0.51
C16	27.2 (10, 18%)	0.19 (9, 39%)	12	0.49
C162	23.0 (10, 21%)	0.12 (4, 25%)	12	0.48
C132	20.3 (8, 4%)	0.09 (4, 19%)	13	0.51
C8	21.3 (9, 25%)	0.27 (1, –)	13	0.51

[a]Freilinger (1998) gives the details of each test and the failure mode for each joint.
[b]Values in parentheses are numbers of observations used to calculate averages and coefficients of variation, respectively.

Table 3 P-Values for comparisons of tension splice joint strength

Test	Static	C6	C162
C1	0.803	0.983	DNC
C6	0.812	-	DNC
C16	0.396	0.443	0.014
C162	0.027	DNC	0.000
C132	0.000	DNC	0.218
C8	0.051	0.219	DNC

DNC = did not compare.

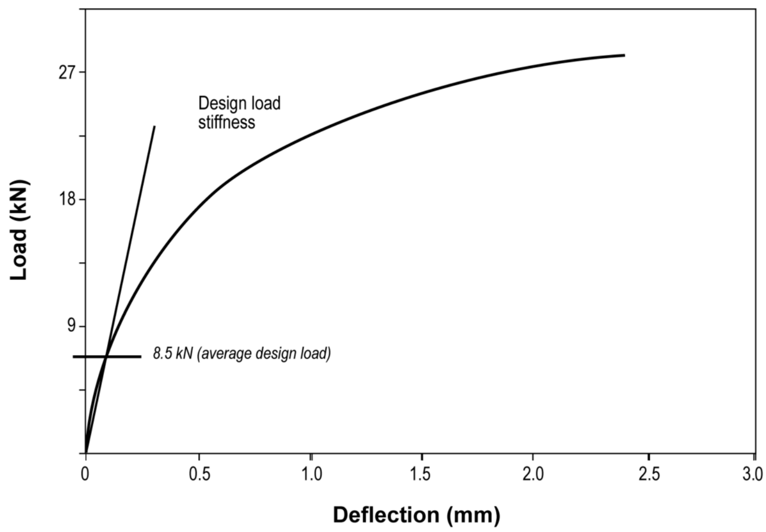

Fig. 5 Load deflection curve for a typical static tension splice joint test

3.1.1 Static test

The average strength, stiffness, coefficient of variation (COV; Table 2), failure modes, and load deflection curve were similar to the results reported in the literature (Gupta and Gebremedhin 1990, Kent *et al*. 1997, Redlinger 1998). A typical load deflection curve is shown in Fig. 5.

3.1.2 Cyclic tests

C1 test: Nine joints survived the cyclic tests and were then ramped to failure. One joint failed during the last cycle. The maximum load for the failed joint was only 12.7 kN; the average maximum load for all 10 joints was 25.1 kN, which explains the high COV. Fig. 6 shows a typical load deflection curve and the isolated hysteresis loops for a typical C1 cyclic test. Because the load deflection curves for other cyclic tests were similar to the C1 curve, only select curves for the other tests are included here. Others are provided in Freilinger (1998). We observed no strength degradation in joints (p-value = 0.803) in comparison with average static strength.

C6 test: Eight of the joints survived the cyclic tests and were ramped to failure following the cycles. Two of the joints failed during the cycles. The high COV was related to the failure of these two joints, which were much weaker than the others in the sample. There was no evidence (p-value

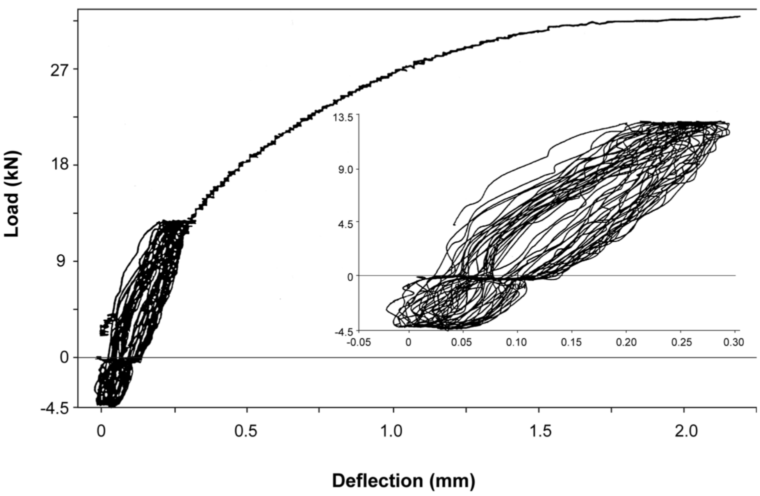

Fig. 6 Complete load deflection curve for a typical C1 tension splice joint test

= 0.812) that strength degradation (compared to static) occurred in this test (Table 3), which implies that a DOL factor of 1.6 might be a reasonable assumption. The remaining cyclic tests on the tension splice joints were more conservative and therefore increased our confidence in a DOL factor of 1.6. Since C1 and C6 had no effect on strength, first stages of loading had no effect on the strength of the joint. The comparison between C1 and C6 also indicates that no degradation (p-value = 0.983) occurred (Table 3).

C16 test: Nine joints survived the cyclic tests and were ramped to failure following the cycles. One joint failed during the second step of the load case (hence low strength), which, along with one high strength load, caused the high COV. C16 showed no evidence of strength degradation (p-value = 0.396) compared with static condition (Table 3). C16 produced some high strength values and the average was actually higher than the value for the static ramp loading, although not statistically (Table 3). We observed no degradation (p-value = 0.443) between C16 and C6 (Table 3). This observation suggests that a DOL of 1.6 based on the C16 test, which approximates only design level events and 1.6 times design load events, is adequate. However, since C16 did not include any major event, a load case that includes this might need to be investigated to have a higher level of confidence in a DOL of 1.6. Load case C162 was used to evaluate just this.

C162 test: Four joints survived the cyclic tests and were ramped to failure following the cycles. Two failed during the second load step and two failed during the third load step. Comparison of C162 with the static ramp load suggests that significant degradation (p-value = 0.027) occurred (Table 3). Since the first two stages were the same as C16, and C16 showed no strength degradation compared to static strength, we can conclude that most of the degradation occurred during the third and final stage of loading. A load deflection plot for C162, shown in Fig. 7, also supports this. Fig. 7 shows that joints had reached maximum load during the last stage of the loading regime. That is, 2 times the design load for 8 seconds (one major event in the life of a joint) can cause strength damage in the joint. Dolan *et al.* (1996) found no strength degradation for nailed and bolted joints subjected to similar loading regimes. Therefore, MPC joints do not provide the same conservative level of confidence that nailed and bolted joints demonstrate. If our results had shown no strength degradation for C162, we could have reasoned that a DOL factor of 1.6 was adequate (to the same

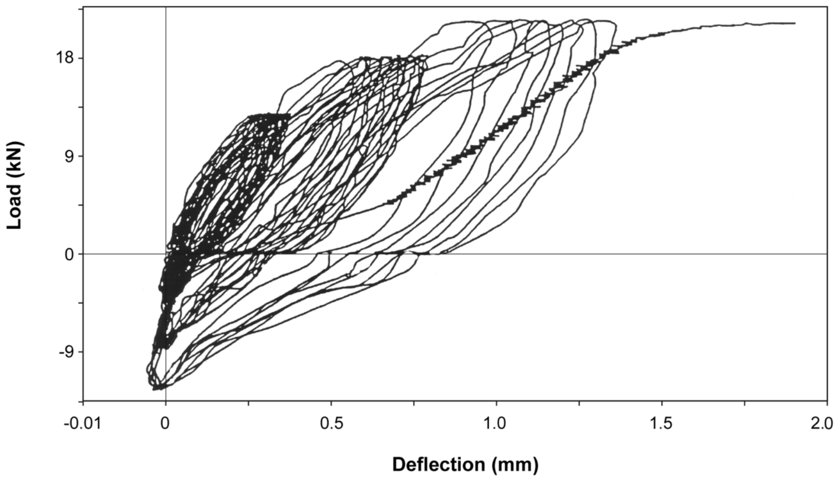

Fig. 7 Complete load deflection curve for a typical C162 tension splice joint test

level of confidence as in Dolan's research of nailed and bolted joints), but this did not occur.

C132 test: Four joints survived the cyclic tests and were ramped to failure following the cycles. Four joints failed during the cycles, all in the third stage of the load regime. Comparison of C132 with the static ramp load shows overwhelming evidence of degradation (*p*-value = 0.000). Since the first two stages of this test were less severe than the equivalent stages of C16, which showed no degradation, we can conclude that most of the strength degradation occurred because of the third and final stage of the loading. Comparison of C132 and C162 revealed no degradation (*p*-value = 0.218; Table 3), and results suggest that the first and second (1.33 vs 1.6) stages of the tests have little impact on the degradation of the joints. We suspect that most of the strength degradation occurs during the third stage (or the major event). As we try to determine an adequate DOL factor, this test comparison suggests that the use of 1.6 is appropriate. By using 1.6, we gain an increase in design capacity without sacrificing confidence in the joint's ability to withstand a cyclic event. We make no sacrifice in confidence because C132 showed the same degradation.

C8 test: Only one joint survived the cyclic tests and was ramped to failure following the cycles. Eight joints failed during the cycles. We chose this test because a DOL factor of 1.8 fell between 1.6, the current DOL factor for seismic/wind and the highest value tested that showed no sign of strength degradation, and 2.0, the lowest value that showed strength degradation. Since 2.0 showed degradation and 1.6 did not, testing 1.8 seemed like a reasonable step. The comparison between C8 and the static ramp load suggests possible degradation (*p*-value = 0.051). To adequately evaluate C8, we would need to test a larger sample set. There is no evidence that C8 is statistically different from C6 (*p*-value = 0.219; Table 3). Although C8 and C6 are statistically the same population, C6 shows no strength degradation compared to the static, while C8 possibly shows some. We would need to test more samples to determine if the DOL factor could be higher than 1.6 (e.g., 1.8).

In summary, little strength degradation appeared in the joints during the first and second stages of the loading function. Most of the degradation appeared to occur during the third and final stage of the loading function (major event).

Table 4 Heel joint test summary

Test	Strength[a] (kN)	Stiffness[a] (10^5 N/mm)	Moisture content (%)	Specific gravity
Static	25.6 (10, 6%)	0.18 (9, 14%)	12	0.48
C16	24.7 (10, 6%)	0.09 (8, 65%)	12	0.47
C162	23.7 (10, 17%)	0.17 (4, 72%)	12	0.47
C18	25.1 (10, 9%)	0.18 (8, 29%)	12	0.48

[a]Values in parentheses are numbers of observations and coefficients of variation, respectively.

3.2 Heel joints

Considering the results from the tension splice joint tests, we tested heel joints under only three cyclic loading conditions (C16, C162, and C18), because we did not expect other cyclic loading results to provide useful information. Table 4 shows the average strength, stiffness, moisture content, and specific gravity for heel joints under static and three types of cyclic loading conditions.

3.2.1 Static test

All joints failed in tooth withdrawal. The strength, stiffness, and load deflection curves were similar to data already reported in the literature (Gupta and Gebremedhin 1990, Kent *et al*. 1997, Redlinger 1998). A typical load deflection curve is shown in Fig. 8.

3.2.2 Cyclic tests

C16 test: All 10 of these joints survived the cyclic tests and were ramped to failure following the cycles. The results of this test indicated no strength loss in comparison with the static tests (p-value = 0.340).

C162 test: Four joints survived the cyclic portion of the test. The C162 results do not indicate any strength degradation in comparison with the static test (p-value > 0.05), indicating that a DOL factor

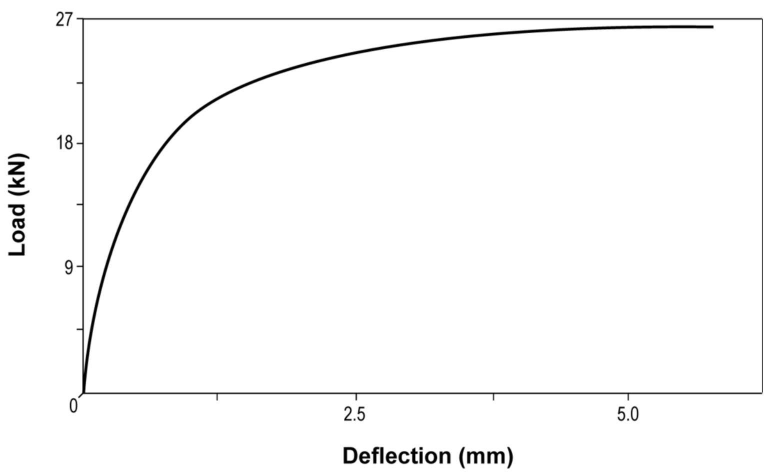

Fig. 8 Load deflection curve for a typical static heel joint test

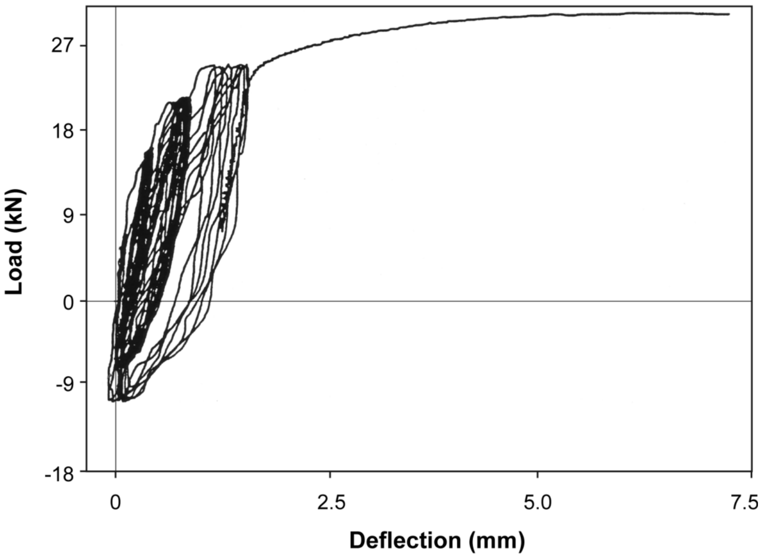

Fig. 9 Complete load deflection curve for a typical C162 heel joint test

of 1.6 is adequate for the heel joint design. Fig. 9 shows a load deflection plot. Unlike the situation for tension splice joints, the maximum load in the third stage was still below the strength of the joint and might have been the reason there was no strength degradation.

C18 test: Eight joints survived the cyclic tests and were ramped to failure following the cycles. Strength did not degrade in comparison with the static strength (p = 0.589).

4. Conclusions

We tested 70 tension splice joints and 40 heel joints under several cyclic loading regimes of varying magnitude and number of cycles. We used a *t*-test to compare the strengths of all the joints. Duration of load results from both tension splice joint and heel joint tests show that the current duration of load factor of 1.6 for earthquake loading is adequate for these joints. A DOL factor of 1.6 is appropriate for tension splice joints because most of the degradation occurred during the last stage (2.0 times the design load), regardless of the magnitude of the load used in the second stage. Results for heel joint tests also support a DOL factor of 1.6. None of the testing cases showed any strength degradation, suggesting that a DOL factor of 1.6 is adequate.

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