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SEISMIC PERFORMANCE STATES OF PRECAST CONCRETE CLADDING CONNECTIONS

Kurt MCMULLIN[1], Yukwen WONG[2], Carter CHOI[2], Kevin CHAN[2]

SUMMARY

Experimental testing of connections used to support precast concrete cladding was conducted for seven full-scale tests. These tests represented in-plane or out-of-plane movement of a panel system, resulting in loading that the connection is intended to resist or not intended.

The objectives were to determine the true force-deflection relationships of the connections allowing for accurate pushover analysis. Using pushover analysis, various performance states were determined.

Specimens were built in the laboratory and tested under monotonic loading. Force and deflection data were monitored using both manual and automated data acquisition systems. Push-pull connections and lateral seismic connections were tested. Data was reduced to engineering terms and stiffness and the displacement for various FEMA-273 Performance States was determined.

For 25-mm Push-pull connections loaded in axial tension or compression, the performance was very ductile, with deformations over 150 mm achieved without loss of strength. However, this deformation resulted from severe bending of the supporting plate and showed very low stiffness when compared to that expected for such connections. When these connections were loaded in bending, they also failed in a ductile mode. No slip was seen between the unwelded plate washers and the supporting bracket. Significant stiffness was provided when the connecting rods were short (150 mm).

The Lateral Seismic connection also failed in a ductile mode when loaded in-plane, the direction of its' intended use. However, when loaded out-of-plane, this connection showed significant stiffness when compared to the Push-pull connection with which it is partnered. As a result, the Lateral Seismic connection failed with limited deformation (20 mm), resulting in the connection being unable to resist any in-plane motion occurring in the panel.

[1] Associate Professor, San Jose State University
[2] Graduate Student Research Assistant, San Jose State University

INTRODUCTION

Research Need.

Recent advances in earthquake engineering design have defined different performance levels for the behavior of building components to allow for life-cycle cost estimates. FEMA-273 has defined performance thresholds for building cladding systems as connection yielding, panel cracking and connection fracture (Federal Emergency Management Agency, 1997). These are listed in Table 1. In addition, corresponding performance levels for the building's primary structural system were also defined by FEMA-273 and assigned allowable transient drift ratios.

As shown in Figure 1, precast concrete panels covering the beams and columns is one common method of a building cladding system. These panels are connected to the structure using a determinant system of connections, meaning that the loss of a single connection will lead to a possibly life-threatening situation. Cladding systems are designed to allow for lateral movement of the floor slab without damaging the panels. However, at the corner of the building, incompatible movement of the panels on different sides of the building can lead to damage, even in modern structures, as has been reported after recent earthquakes (Filiatrault et al, 2001; Seike and Sakamoto, 1997).

Panel Movement.

Cladding panels may move during an earthquake in any direction in 3-D space. In-plane movement is defined as being parallel to the flat face of the panel while out-of-plane movement is orthogonal to the face of the panel. In Figure 1a, in-plane movement of the corner panel is parallel to grid 1. For end panels, the flat face is parallel to the structural framing of the building that supports the panel. (It is uncommon to have panels connecting to two faces of the building structural system, particularly in seismic areas.)

Specific components of the connections are designed to resist these movements. For this paper, a 3-D local coordinate system (U1, U2, U3) is used for each connection. Figure 2 shows one method of building the Lateral Seismic Connection. In-plane vertical movement, U3, is resisted by the bearing connections, the same that resist the gravity load of the panel. A horizontal plate is intended to resist the shear developed by in-plane horizontal movement, U2. Push-pull connections assist in resisting in-plane movement by resisting the moment developed by the eccentricity of the horizontal shear and the supports. The push-pull connections resist out-of-plane movement, U1, of the panel and at the location of the LSC, an additional push-pull connection is built as one component of the entire LSC connection.

EXPERIMENTAL TESTING OF PANEL CONNECTIONS

Test Specimens and Protocols.

Monotonic experimental testing of full-scale individual connections of cladding panels was completed prior to the analytical study. Two different connections were considered: push-pull connections made with one-inch coil rod and in-plane LSC made with steel angle and plate. Table 2 lists the test matrix for the testing.

The push-pull connections were tested for both their resistance to panel in-plane motion (bending of the rods) and out-of-plane motion (tension or compression of the rods). Failure modes considered were flexural bending of the rods, slip of the rods in the enlarged holes of the supporting beam, tension of the rods, buckling of the rods, bending of the plate that connects the push-pull connection to the column, fracture of the weld between the support plate and the steel column, and fracture of the weld between the coil nut and the panel embed plate.

Testing of the in-plane seismic connection was conducted for both in-plane and out-of-plane movement of the panel. In-plane movement considered failure modes of yielding of the steel plate or angle and fracture of the weld connecting the two. Out-of-plane movement of the panel considered bending of the plate or tube, fracture of the weld from the plate to the embedded angle, fracture of the weld from the column to the plate, fracture of the weld from the tube to the supporting plate, and fracture of the weld between the supporting plate and the column flange. It should be noted that the intent of the design engineer is that plate bending is not expected to resist out-of-plane movement of the panel.

Test Results

The primary objective of the experimental testing was to determine the force-deflection relationship for the connections that would be used for the connection elements in the nonlinear model. Specimens were loaded in one of the three local coordinate directions (U1, U2, U3) and the corresponding deflection in the same direction was measured. Figures 3 and 4 are photographs of the types of damage observed during the testing.

Figure 5 shows the results from the experiments as well as for the assumed mechanics-based model. For the experimental results of the Lateral Seismic Connection, the results of two tests were combined for the figure ((U1-ELSC2 and U1-ELSC3) because of the significant strength and stiffness that the plate provided to resist out-of-plane movement. For axial load of the push-pull connection, the mechanics-based model was determined from traditional mechanics assuming a 19-mm steel rod being stretched according to elastic-plastic material properties. The rod was assumed to be 150 mm long with 240 MPa steel. The rod was expected to be linear until yield, to then deform without increasing stress to a ductility of 30. At this point strain-hardening would occur in tension members and buckling to zero-strength would occur in compression members. Strain-hardening under tension would continue to an ultimate strength of 340 MPa and a ductility of 60, followed by zero-stiffness elongation to an ultimate ductility of 90 when the rod was expected to fracture.

In addition, various damage criteria were defined as listed in Table 3. For each type of connection damage, a connection performance limit was assigned by comparing the expected result of the damage to a definition of damage type in FEMA-273. FEMA-273 is limited in defining what types of damage might be acceptable in the various performance levels, but as an initial start into performance based design it is at least a common national standard. During the testing and after reviewing the test data, displacement thresholds where the specimen reached each damage criteria were determined. Figure 6 shows the displacement thresholds for various damage criteria and the associated Connection Performance Level. The push-pull connection showed high ductility during testing, particularly due to plastic bending of the supporting plate when the connection was loaded in the U1 direction.

The lateral seismic connection, Figure 6b, had weld fracture as the ultimate failure mode for all three tests, but usually after showing significant post-yield deflection. One finding from the testing was that the horizontal plate significantly influenced the out-of-plane resistance of the connection. This is contrary to the original design intent where this member only resists in-plane movement of the panel. Plastic deformation of the supporting tube at the push-pull connection allowed for significant deflection to occur at relatively low forces. In Figure 5, the U1 direction for the experimental LSC (U1-ELSC) shows high strength and stiffness at deflections under 10 mm. At this deflection, the weld attaching the steel plate to the column fractured, resulting in the tube resisting all out-of-plane movement at substantially less strength.

Experimental testing showed that the complexity of the geometric configurations did not allow for accurate prediction of the force-displacement relationship based upon simple structural models. A comparison between the mechanics (U1-M) and experimental (U1(T)-EPP for the push-pull connection shows that maximum strengths, initial and plastic stiffness and overall ductility were all significantly different than expected.

PUSHOVER ANALYSIS OF PANEL ASSEMBLY

Multi-Panel Model of Building Corner

A nonlinear static analytical study performed on a typical cladding system was intended to determine the ability for a cladding system to meet specific performance criteria while the primary structural system drifts laterally during an earthquake. Performance levels defined by FEMA-273 were used as criteria for this analysis (1997). A nonlinear model of a corner cladding assembly was developed, as shown in Figure 7, based upon the assumption that the concrete panel would remain essentially elastic and undamaged until long after the steel connections failed. Figure 7 shows the elastic steel beam and columns that support the panels, which were modeled as elastic shell elements. The steel beam is connected to the column with a pin joint allowing for a mechanism to develop.

The precast concrete panels are connected to the steel frame by four nonlinear link elements (connections B, C, D and E). Figure 5 shows the various force-displacement relationships used for the link elements. Vertical support (U3) was provided by stiff linear springs for all analysis runs while horizontal support (U1 and U2) varied for different analysis runs. Table 4 lists the analytical models considered. The model names depend upon the link force-displacement relationship and type of connection they represent. One option is mechanics based (M), representing a connection with strong, rigid supports. Another is experimental based (E), representing connections containing flexible components such as plates connecting the embeds to the column or beam support. For all models, Connections C, D and E were push-pull connections represented by one of these two options. Connection B was assumed to be the location of the Lateral Seismic Connection. It had the two options listed above (M and E), as well as a third option denoted O. When the model is identified as an O in the first digit, it means that the Lateral Seismic Connection has a stiff linear spring for resisting movement in the U2 direction and for movement in the U1 direction, the force-displacement relationship is the same as the one used for the push-pull connections in the same model (hence either M or E depending upon the relationship used for Connections C, D and E).

Static Pushover Analysis

The analysis was conducted using the SAP-2000 Version 8 analytical software program. Both the structural frame and concrete panels were intended to remain essentially rigid and elastic throughout the study. To allow for an isolated study of the cladding panels, the steel frame was intentionally designed to be articulated, allowing lateral movement at the top of the column with no resistance. The geometry of the frame was intended to represent one story of a multi-story building by considering midheight of the column above to midheight of the column below. The assumption was that the lateral motion of the structural system of the building would not be affected by the cladding system.

Cladding systems are built with 19 mm gaps between panels to allow for geometric variations of the panels. This gap is filled with a flexible grout during panel installation. Due to the modeling assumptions of the articulated steel frame, the model is unstable while this gap is open. Since the software program is unable to incorporate rigid body analysis, a zero-length gap element was used and the analytical model was used only for lateral behavior after sufficient drift to close the gap had occurred.

A primary factor when relating drift to damage is the relative elevation between the point of contact of the panels and the neutral axis of the floor system. If these two occur at equivalent elevations, the two panels move without potential contact. The further these two elevations separate, the less rigid body motion occurs before the gap closes. A simplified model of the gap shows that the drift to cause the gap to close, α_{RBM}, is related to the original gap opening, *gap*, and the difference in elevation, y_b, by the equation:

$$\alpha_{RBM} = gap / y_b \qquad (1)$$

The models listed in Table 4 were repeated for two different values of y_b. Values of 990 mm and 1980 mm were analyzed, thus resulting in eight different analytical models.

For each pushover analysis, the displacement of individual connections was recorded for various drift values. An example is shown in Table 5. The drift levels are those provided by FEMA-273 as the thresholds for various performance levels of special moment-resisting steel frames. (Generally, only one of the local coordinate directions is critical for an individual connection.) Each connection was assigned a Connection Performance Level from the damage thresholds shown in Figure 6. Connections achieve different performance levels depending upon the damage thresholds considered. In Figure 6b two options for damage criteria are shown. Dufring testing, weld fracture often initiated at smaller displacements than yielding of various components. However, after initiation of the weld crack, connections were still able to support increasing loads at significantly higher displacements. Hence, two sets of damage criteria were used to interpret the analytical output for the experimental link connections. In Figure 6b both U1 and U2 directions are shown considering weld fracture (WF) and without fracture (WOF). Thus Table 5 lists Connection Performance Levels for two different damage criteria. The last row of Table 5 lists the Structural Performance Level, which is determined as the poorest Performance Level that any connection achieved for that pushover model and story drift.

Table 6 lists the results of all the pushover analyses in a format comparable to Table 2-9 of FEMA-273. This table in FEMA-273 provides target design goals for performance of buildings. The analysis showed that all models could meet the recommended performance criteria for Structural Performance Level of Immediate Occupancy. This resulted from the gap being large enough to allow the 0.0070 drift to occur without closing the gap. In the Life Safety level, only one model analyzed was unable to remain Operational. This model had stiff, strong push-pull connections. In the Collapse Prevention level, all models were unable to meet the strictest performance expectations and those with stiff, strong push-pull connections were unable to meet the Life Safety and Hazard Reduced levels.

CONCLUSIONS

1. All connections tested showed ductile failure modes, with some amount of energy dissipation before final failure. Weld fracture was always the final failure mode for the Lateral Seismic Connection but for none of the Threaded Rod Push-Pull Connections.
2. Slipping of the push-pull connections was seen only in the axial tests and then only after significant plate bending had occurred. This was true even though the plate washers were not tack-welded to the support.
3. Push-pull connections were dominated by the strength and stiffness of their supports. While showing significant energy dissipation, these connections had very low stiffness to resist both intended and unintended modes of loading.
4. The Lateral Seismic Connection resulted in the plate carrying a significant amount of load when the panel moves out-of-plane, which is not intended by the original design. This resulted from the large flexibility developed by local dishing of the supporting tube. When large displacements occur, the

weld of the plate fractured, resulting in a lack of in-plane resistance. Fortunately, the analytical study showed that this failure was unlikely to occur, since the stiffness of the connection resulted in small displacements.

5. Experimental testing showed that the complexity of the geometric configurations of many cladding connections did not allow for accurate prediction of the force-displacement relationship based upon traditional structural models. Maximum strengths, initial and plastic stiffnesses and overall ductilities were all significantly different than expected.
6. For the analytical models considered, panel behavior was primarily dictated by the constitutive relationship of the push-pull connection. The experimental lateral seismic connection had high strength and stiffness, resulting in nearly identical results as if the connection had been assumed to be rigid.
7. In the majority of cases, a single connection with large displacement resulted in poor performance for the entire structure. This connection was almost always one of the push-pull connections located on the end panel away from the corner return of the end panel.
8. The vertical distance between the neutral axis of the floor system and the point of contact of two cladding panels largely influenced the building performance. As this distance increases, the lateral drift permitted before panels contact and damage initiates decreases. After contact occurs, the simple model in this study shows identical behavior regardless of the original gap between panels.

Future expansion of the study includes consideration of gravity loads in the connections prior to and during the pushover analysis. An additional damage criterion considering brittle weld fracture representing poor detailing will also be considered. Location of the LSC may also affect the outcome, in this study location B was always used as the LSC and other locations likely will result in different panel movements, possibly altering the Structural Performance Level. The studies here have been for a single bay geometry and additional geometries are to be considered. Cyclic displacement patterns representing cyclic building motion due to earthquakes will be considered to see if low-cycle fatigue failure of the connections may result in different building behaviors. Continuation of the experimental testing should allow for representation of a wider variety of cladding systems. The influence of the rigid body motion prior to panel contact was found to be critical and so additional consideration should be completed to define realistic values for this term. Recommendations about the design and detailing of individual connections will also be made.

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Notation

LSC = Lateral seismic connection, a connection detail intended to resist in-plane horizontal movement of the panel.

PPC = Push-pull connection made from low carbon steel rod with cold-formed threads, a connection detail intended to resist out-of-plane movement of the panel and the couple developed by the eccentricity of the weight of the panel.

gap = Initial distance between panels, traditionally 19 mm.

y_b = Difference in elevation between neutral axis of building floor system and point of contact between panels.

α_{RBM} = Drift required to cause panels to initially contact.

Table 1. FEMA-273 Performance Levels for Cladding Systems (1997)

Hazards Reduced N-D	Life Safety N-C	Immediate Occupancy N-B	Operational N-A
Severe damage to connections and cladding. Many panels loosened.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding but panels do not fall.	Connections yield: minor cracks (<1/16" width) or bending in cladding	

Table 2. Test Matrix for Experimental Study

Specimen	Connector		Connection Loading Direction	Panel Movement Direction	Maximum Force Resisted (kN)
U2-EPP	25 mm diameter rod	460[1]	Bending (U2 or U3)	In-Plane	9.8
U2-EPP2		150[1]			28.1
U1(T)-EPP		460[1]	Tension (U1)	Out-of-Plane	61.5
U1(C)-EPP			Compression (U1)		23.5
U2-ELSC	25 x 125 rectangular steel plate		Tension (U2)	In-Plane	307.5
U1-ELSC2			Bending (U1)	Out-of-Plane	79.0
U1-ELSC2R					73.6
U1-ELSC3	8x3 TS				162.8

Notes: 1) Rod Length, in mm.

Table 3. Damage Criteria for Connections

Damage Criteria	FEMA-273 Performance Threshold
Strength degradation.	N-C
Strength degradation of 75%.	N-D
Initial cracking of metal or weld.	N-D
Complete fracture of component and/or loss of load carrying capacity.	Not defined, unacceptable performance.

Table 4. Analytical Models

Model Name Suffix	Push-Pull Connection Element (Connections C, D and E)	Lateral Seismic Connection Element (Connection B)	Vertical Gravity Connection (Connection B and C)
OM	Mechanics	Linear Spring	Linear Spring
EM		Experimental	
OE	Experimental	Linear Spring	
EE		Experimental	

Table 5. Example Analytical Output for T-EM Model

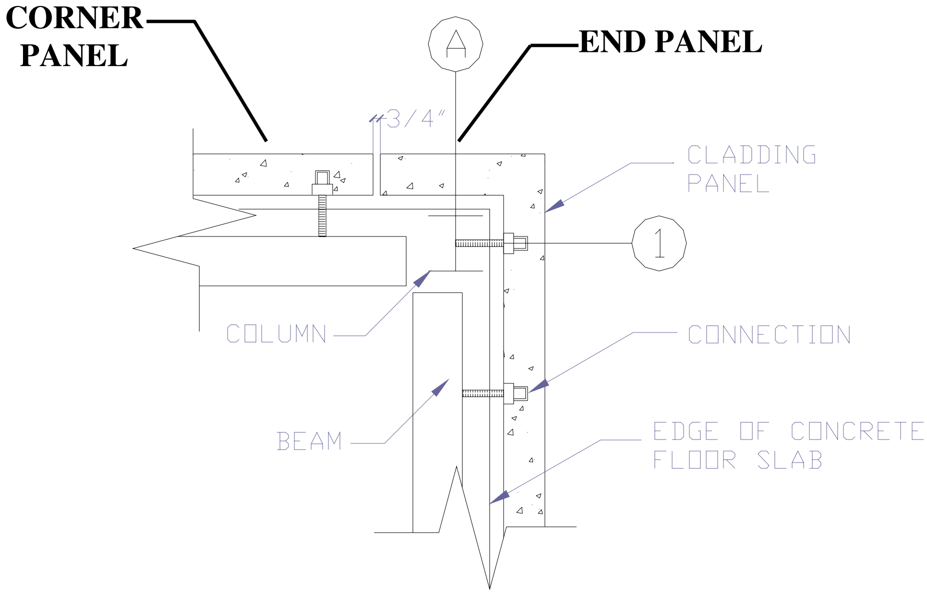
Panel	Link	Local Axis	Drift of Building								
			S1 – 0.007			S3 – 0.025			S5 – 0.05		
			Disp	Perf. Level		Disp	Perf. Level		Disp	Perf. Level	
			mm	WF	WOF	mm	WF	WOF	mm	WF	WOF
End	B*	U1	0.0	N-A	N-A	-0.2	N-A	N-A	-0.2	N-A	N-A
	C	U1	0.0	N-A	N-A	0.0	N-A	N-A	0.0	N-A	N-A
	D	U1	0.0	N-A	N-A	-0.1	N-A	N-A	-0.1	N-A	N-A
	E	U1	0.0	N-A	N-A	0.5	N-A	N-A	24.9	N-D	N-D
Corner	B*	U2	0.0	N-A	N-A	-0.6	N-A	N-A	-0.9	N-A	N-A
	C	U2	0.0	N-A	N-A	-0.4	N-A	N-A	-0.5	N-A	N-A
	D	U2	0.0	N-A	N-A	1.7	N-A	N-A	2.6	N-A	N-A
	E	U2	0.0	N-A	N-A	1.0	N-A	N-A	2.6	N-A	N-A
Middle	B*	U2	0.0	N-A	N-A	0.1	N-A	N-A	0.1	N-A	N-A
	C	U2	0.0	N-A	N-A	0.0	N-A	N-A	0.0	N-A	N-A
	D	U2	0.0	N-A	N-A	0.0	N-A	N-A	0.0	N-A	N-A
	E	U2	0.0	N-A	N-A	0.0	N-A	N-A	0.0	N-A	N-A
Building Perf. Level				N-A	N-A		N-A	N-A		N-D	N-D

* Lateral Seismic Connection
α_{RBM} = Rigid Body Motion Drift = 0.096 rad

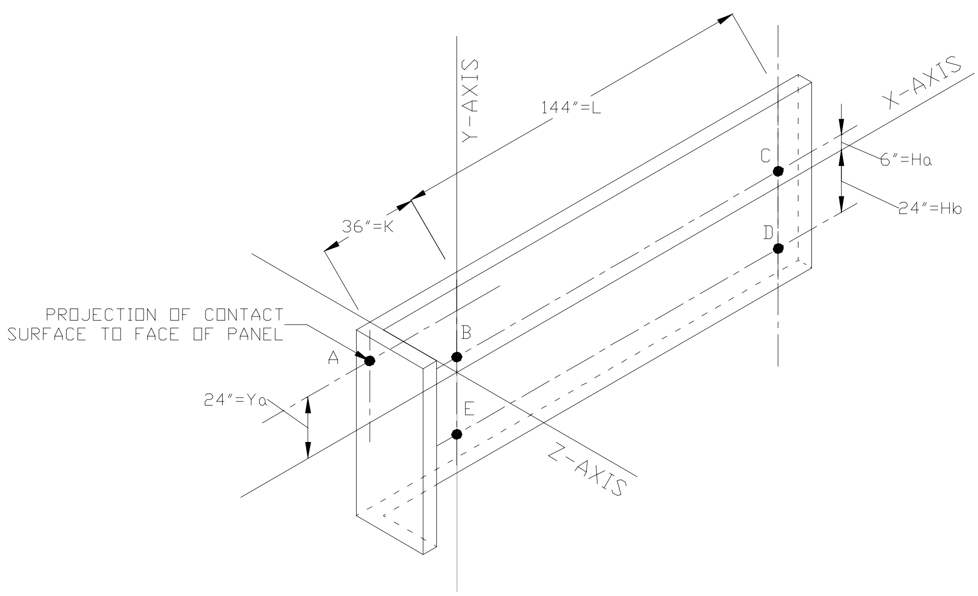
Table 6. Analytical Models That Did Not Meet Structural Performance Levels

Non-Structural Performance Levels	Structural Performance Levels/Ranges		
	S-1 Immediate Occupancy	S-3 Life Safety	S-5 Collapse Prevention
N-A Operational	None	T-0E	All
N-B Immediate Occupancy	None	T-0E	All
N-C Life Safety	None	None	T-0M T-EM-WF T-EM-WOF
N-D Hazard Reduced	None	None	T-0M

Note: Twelve different models were considered: T-0M,T-EE-WF, T-EE-WOF, T-0E, T-EM-WF, T-EM-WOF, M-0M,M-EE-WF, M-EE-WOF, M-0E, M-EM-WF, and M-EM-WOF.



a-) Plan View



b-) 3-D View

Figure 1. Precast Panel Assembly

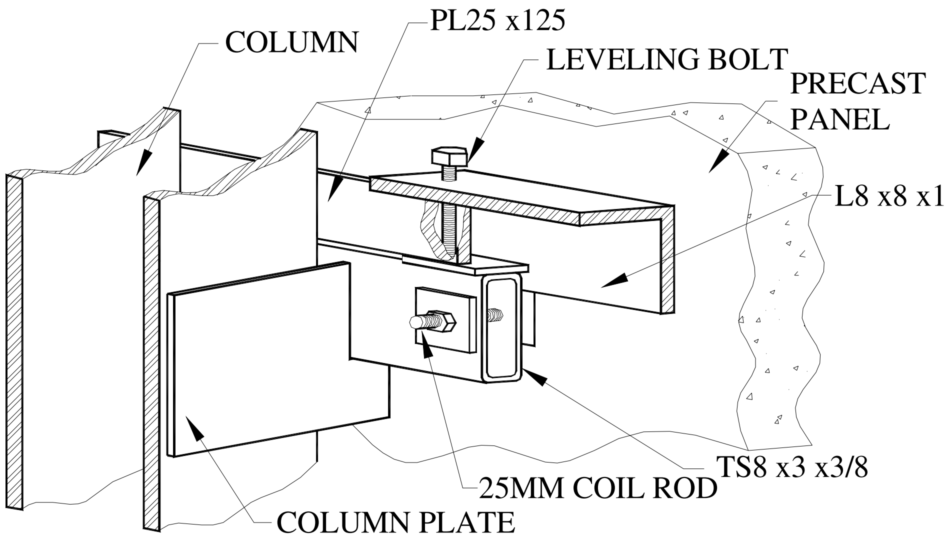


Figure 2. Connection Details

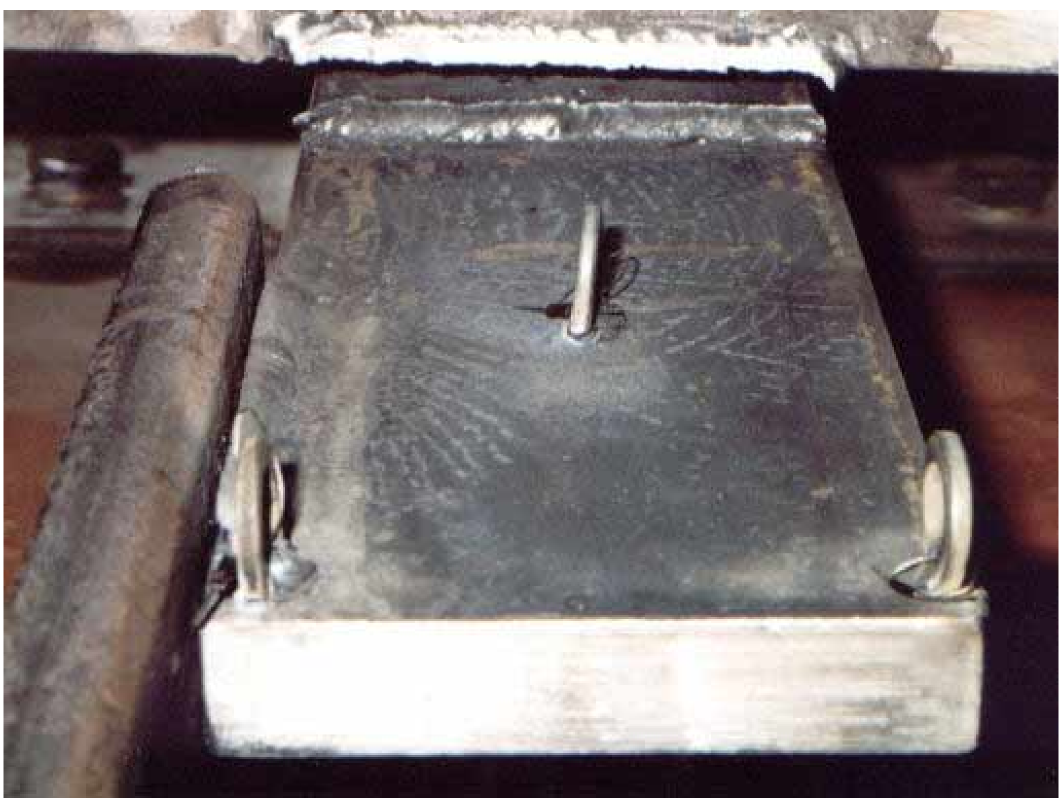


Figure 3. Fracture of Weld of U1-LSC2R.

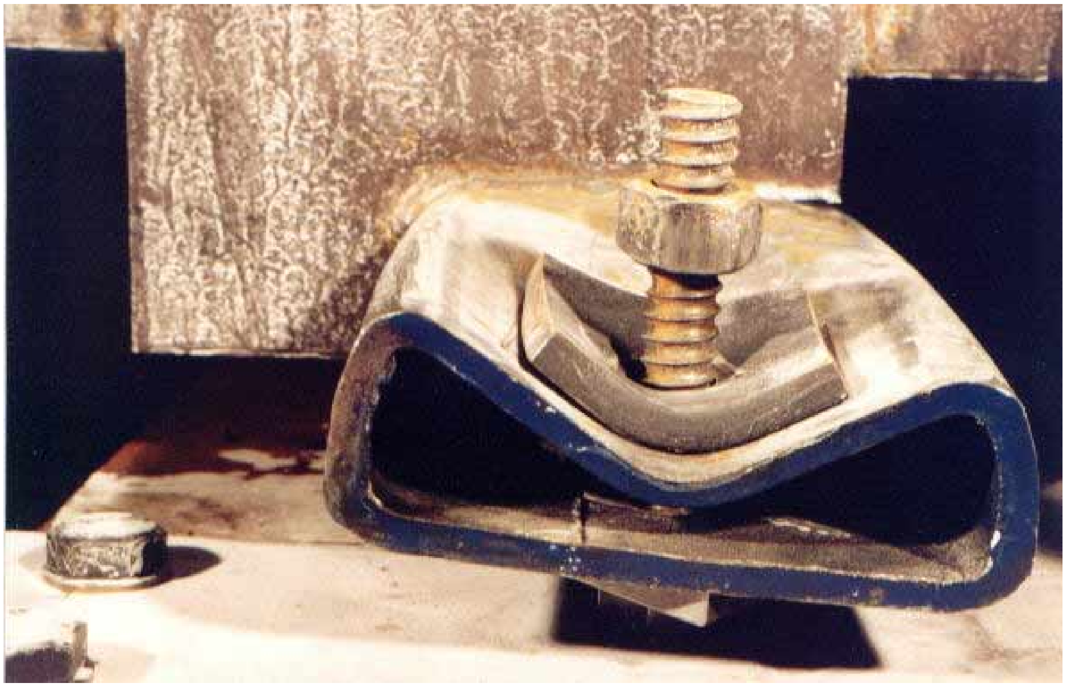


Figure 4. Dishing of U1-ELSC3

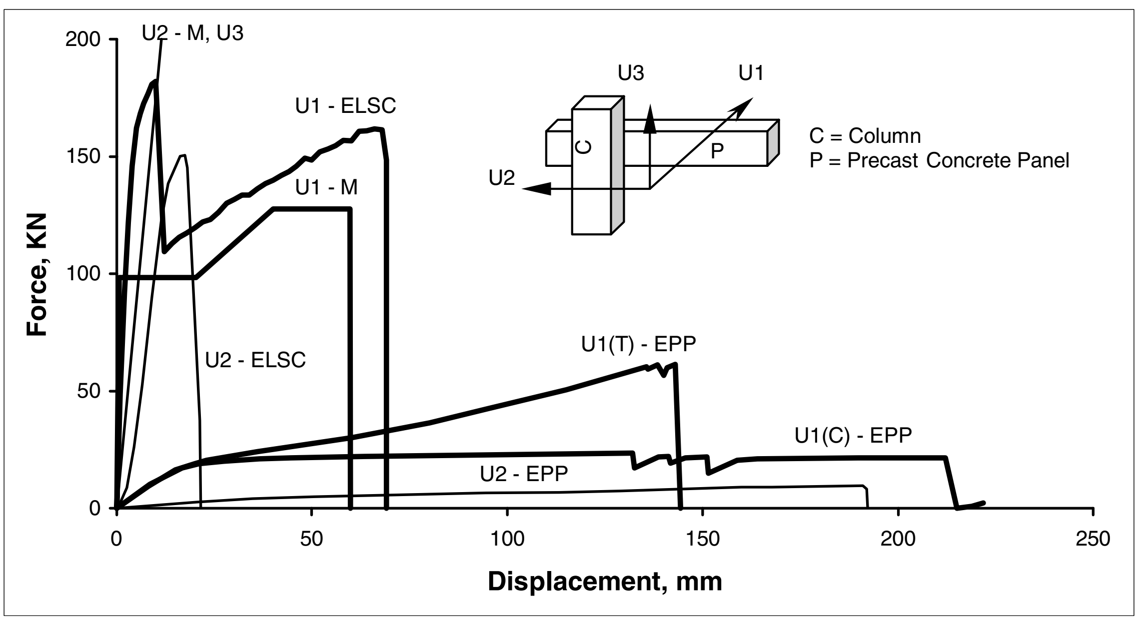
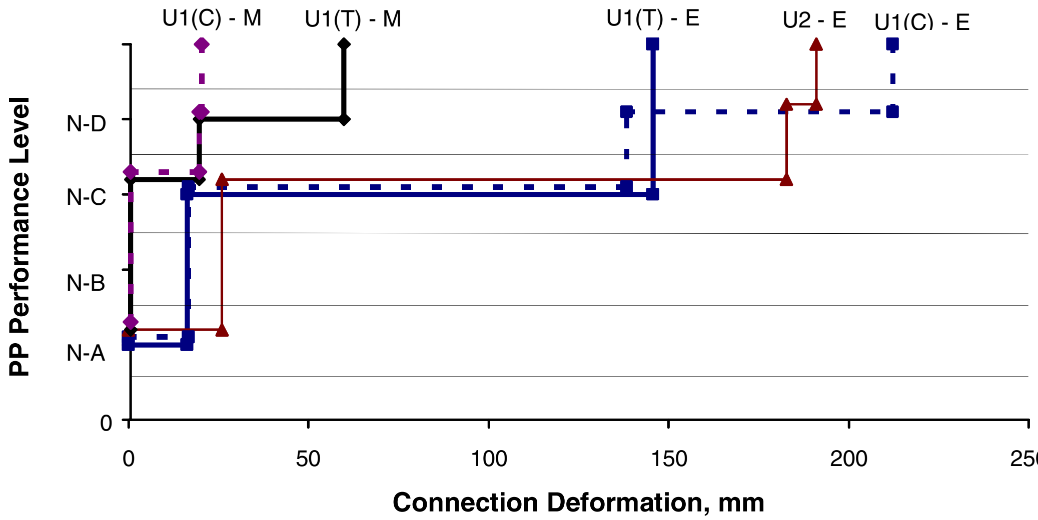
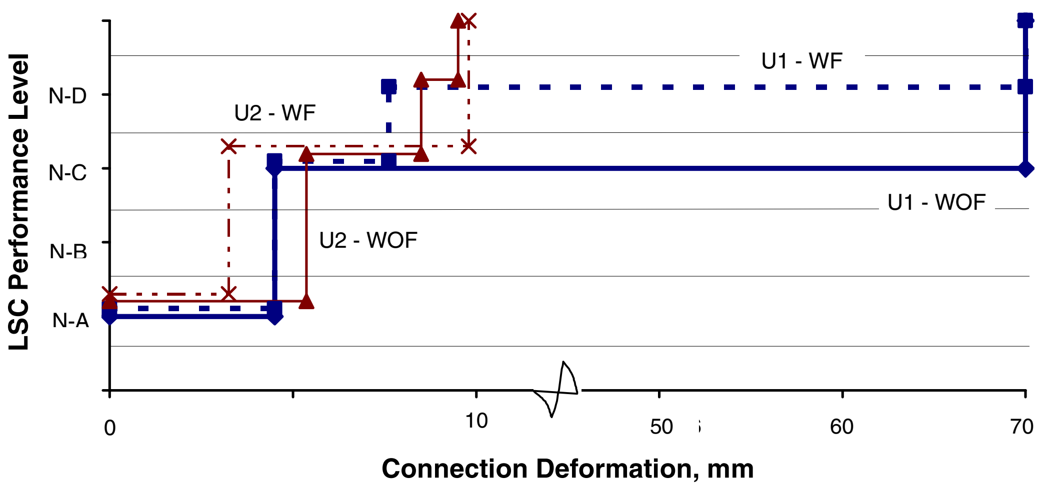


Figure 5. Connection Link Force-Deflection Relationships



a-) Push-Pull Connection Performance Level



b-)

Lateral Seismic Connection Performance Level

Figure 6. Performance Levels for Connections

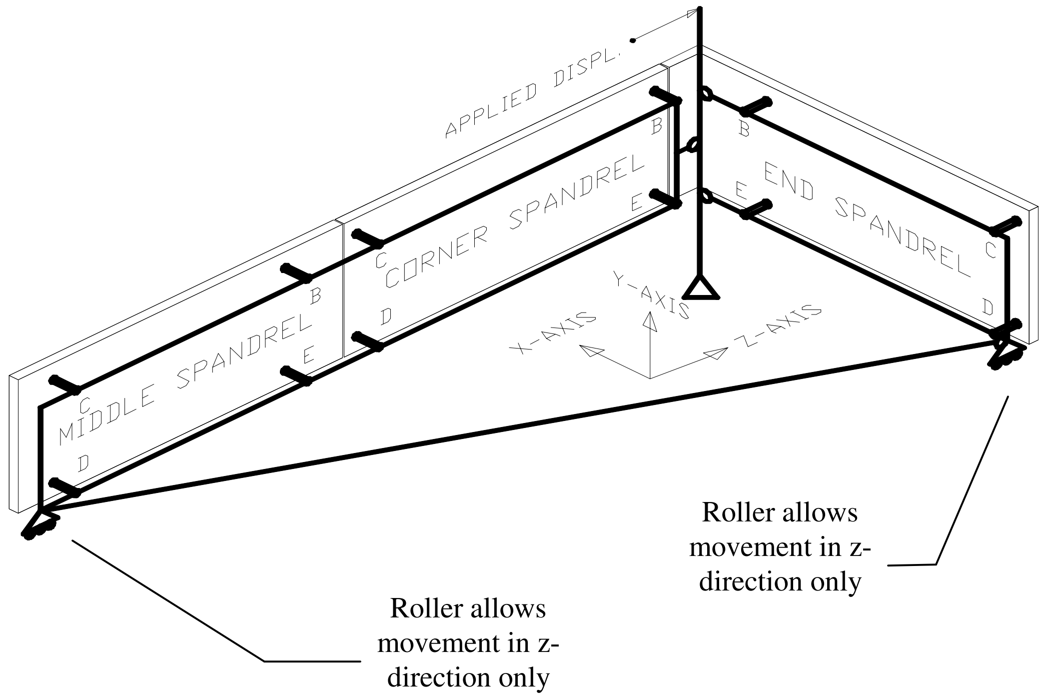


Figure 7. Nonlinear Analytical Model for Multiple Panel Model

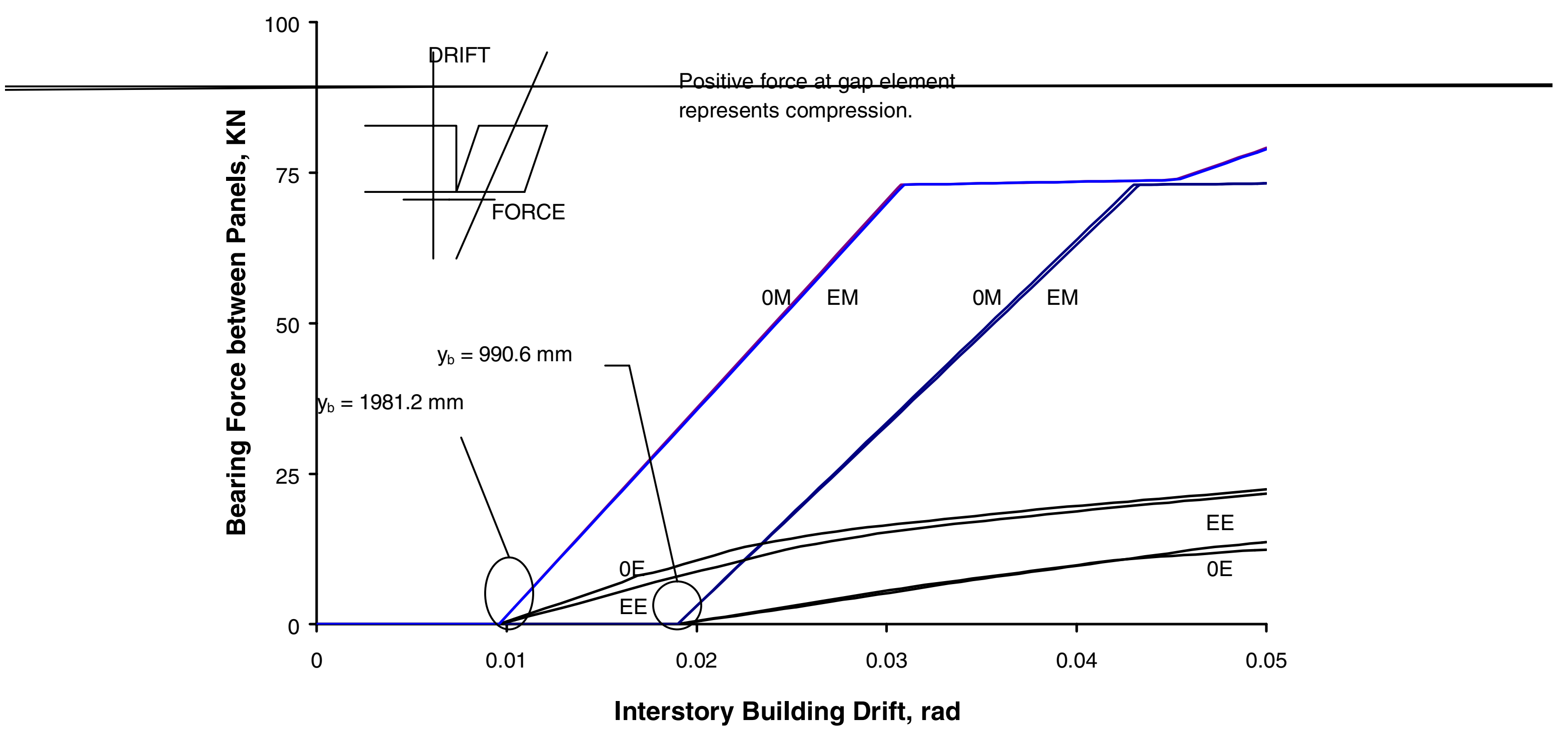


Figure 8. Pushover Curve for Multiple Panel Model