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EXPERIMENTAL STUDY OF JOINT CONNECTIONS IN PRECAST CONCRETE WALLS

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Abstract

A research project is in progress, under the auspices of the U.S.-Japan coordinated PREcast Seismic Structural Systems (PRESSS) program, to investigate the seismic performance of joints between panels in precast concrete shear walls. This study is a component of a larger PRESSS research project at the University of Nebraska-Lincoln which seeks to study the behavior of a six-story precast concrete office building exposed to moderate seismic risk (Tadros et. al., 1993; Tadros et. al. 1994). Although the focus of the larger study is on structural systems for regions of moderate seismicity, precast shear wall connections for a wider range of seismic applications are targeted here.

Introduction

The objective of the present study is to develop, by means of carefully calibrated experiments, accurate behavioral models and design rules for connections in precast shear walls. The connections under study are intended to improve seismic resistance of precast shear walls by enhancing the transfer of lateral forces between connected panels, and by increasing overall system toughness through energy dissipation. To this end, an experimental effort encompassing cyclic, lateral load tests of twelve 2/3 scale specimens of precast shear wall connections is underway at the National Institute of Standards and Technology.

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A compendium of twelve connection details for precast shear walls, six for vertical joints and six for horizontal joints, have been selected for study. The selected details were either modified from existing precast construction practice in the U.S.A., adapted from previously proposed details that have not been used, or developed as part of this study. Criteria used in the selection process include potential for improved behavior, ease of erection, and overall cost. This paper summarizes the design and expected behavior of the six vertical joint connections.

Specimen Design and Test Setup

The vertical joint specimens are models of localized regions in the precast shear wall which contain the vertical joint connections (Fig. 1). Vertical interface shear force demand for the specimens was inferred from the structural analysis results that were used to design the six-story precast office building in the University of Nebraska-Lincoln. Lateral loads were determined assuming that the building is exposed to UBC seismic zone 2B (International, 1991). Details of that building design, which comprises separate vertical and lateral load-resisting systems, are discussed elsewhere (Tadros et. al., 1993; Tadros et. al. 1994). The precast shear walls (Fig. 1) resist a significant portion of the design lateral loads for the entire building, and they serve as prototypes for the connection details and test specimens in this study.

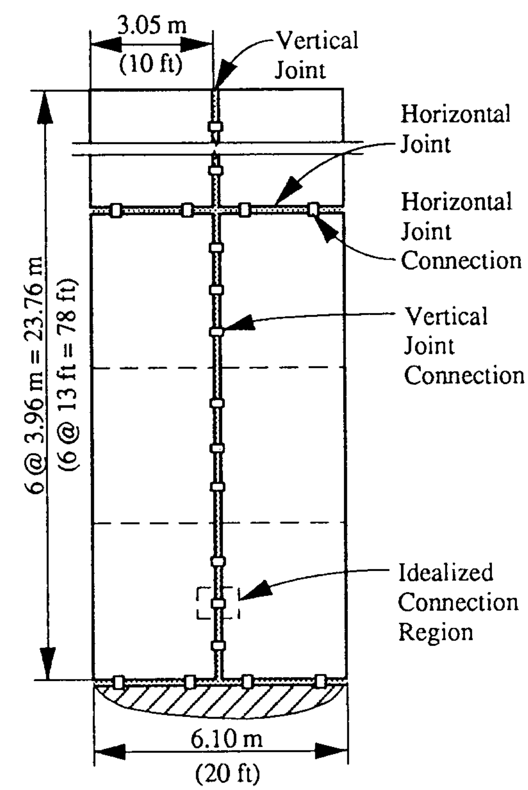


Fig. 1 Prototype Precast Concrete Shear Wall

The prototype shear walls are part of the stairwells in the six-story precast office building, and they comprise four panels which are stacked two wide and two high, as shown in Fig. 1. Each panel is three stories tall and 3.05 m long. Connections are provided along horizontal and vertical joints between the panels, and the vertical joint connections are designed to transfer vertical interface shear stresses between adjacent panels.

Shear force demand for vertical joint connections was determined assuming that the panels remain elastic during a design-magnitude earthquake. The connection elements which span the joint, on the other hand, were proportioned to yield, or otherwise mobilize energy dissipation, during such an event. The shear wall was assumed to be a vertical cantilever beam, and elastic analysis procedures for shear stresses in beams were used to determine first-story vertical shear force given the design base shear force. A target shear force equal to 98 kN (22 kips) was determined for vertical joint connections in this manner, after consideration was given to the 1:2/3 length scale, as well as the difference in model and prototype panel thicknesses. In calculating this force, it was assumed that three vertical joint connections are provided per story.

Specimen configuration and test setup for the vertical joint connection experiments are shown in Fig. 2. The connection and localized region of the panels are cast with monolithic top/base girders that serve to attach the specimens to the testing apparatus. These girders also serve to apply the shear force to the panel with a uniform stress distribution. The vertical joint has been rotated to a horizontal position, and a load/displacement history which captures the important features of seismic loading is applied to the top panel.

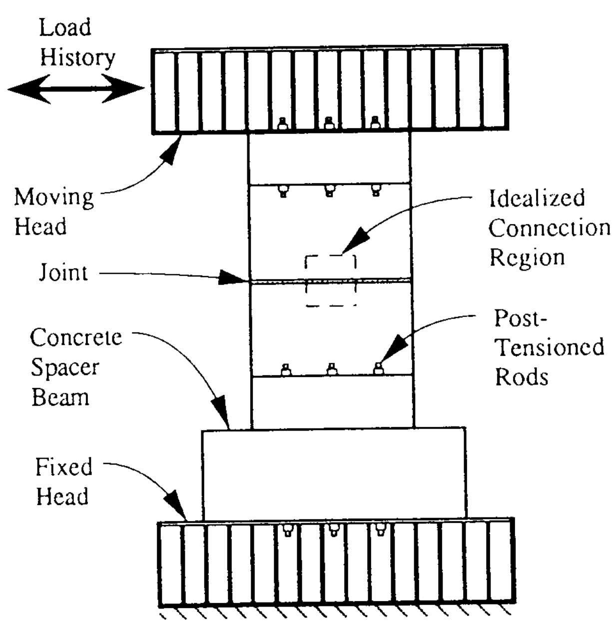
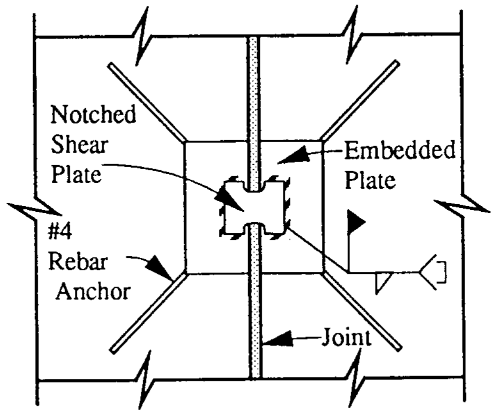


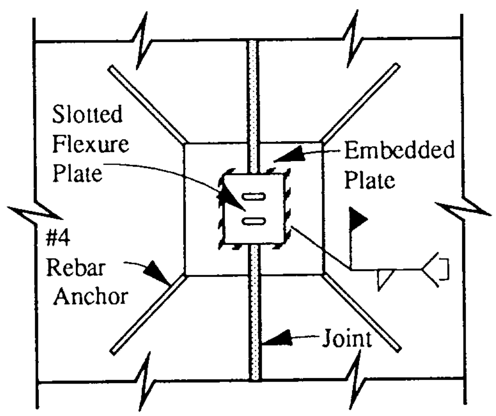
Fig. 2 Test Specimen for Vertical Joint Connections

Development of Connections

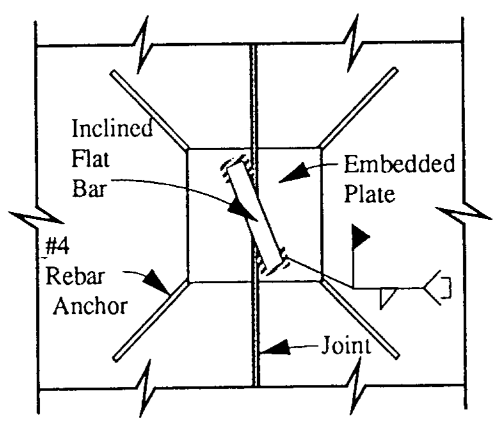
The six vertical joint connections have been subdivided into two groups. The first group (Fig. 3) includes those connections which are joined using field welding only, whereas connections in the second group (Fig. 4) rely on bolting, or a combination of bolting and welding. In addition, the connections in the second group are easily replaceable in the event of a damaging earthquake. This last feature is a direct outcome of bolting as the method of joining, and it increases the overall value of the connection for seismic applications.



a) Notched Shear Plate (NSP)



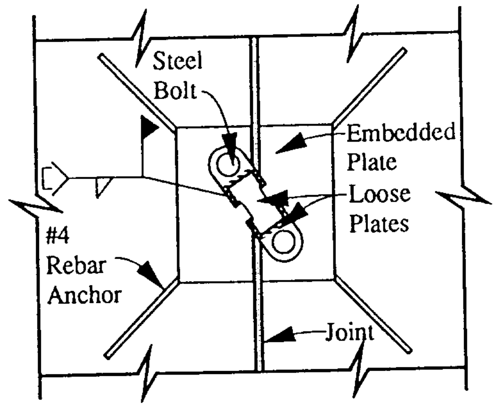
b) Slotted Flexure Plate (SFP)



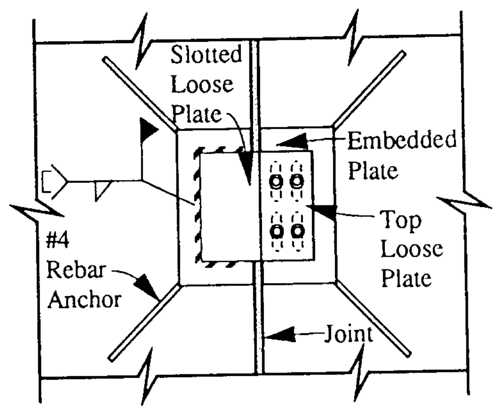
c) Inclined Flat Bar (IFB)

Fig. 3 Welded Details for Vertical Joint Connections

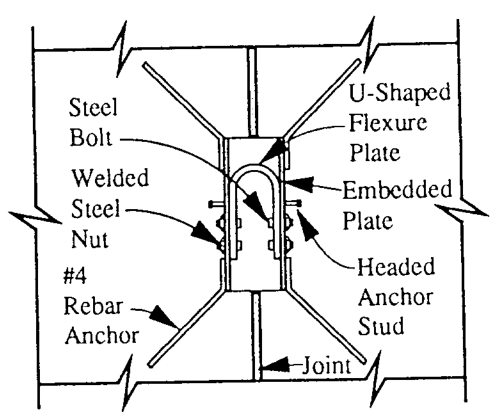
Panel reinforcement is identical for all six vertical joint specimens, as are the anchor plates in the welded connection details (Fig. 3). For the first two bolted connections (Fig. 4a and 4b), the anchor plates have been modified slightly, while the anchor plate in the last bolted detail (Fig. 4c) has been reoriented so that it is normal to the panel. To allow for the use of bolts, steel hex nuts are welded to the reverse side of the anchor plates in the bolted connections.



a) Pinned Tension Strut (PTS)



b) Brass Friction Device (BFD)



c) U-Shaped Flexure Plate (UFP)

Fig. 4 Bolted Details for Vertical Joint Connections

In all six connection details, the connector plates are treated as the weak link, while the welds, bolts and anchorages are proportioned to resist, without yielding or failing, the stresses associated with the connector plates attaining their strength. For each anchor plate, four 13-mm (0.5-in.) diameter headed anchor studs have been selected to resist the vertical force associated with connector plate strength, while the #4 (13-mm, 0.5-in. diameter) reinforcing bar anchors were proportioned such that the horizontal component of force resists the torque generated by the maximum vertical force in the connection. It is noted that the anchor studs, being on the reverse side of the anchor plate, are not shown in Figs. 3 and 4.

The welded details (Fig. 3) are modifications of one of the most common connections in precast practice in the U.S.A. for vertical joints. However, the shape and/or orientation of the 9.5-mm (0.375-in.) thick rectangular loose plate, which is normally welded to the anchor plates in a horizontal position, have been modified. In the first welded connection (Fig. 3a), the loose plate has been notched, and dimensions have been selected so that the 70-mm (2.75-in) notched depth of the plate resists in direct shear the targeted connection force at first yielding. The enlarged ends of the plate enable the use of sufficiently large fillet welds so that weld design strength exceeds the strength of the loose plate.

In the second welded detail, the 133-mm (5.25-in) total depth of the three slender beams between slots in the loose plate have been proportioned such that they resist in flexure the targeted connection force at first yielding. The third welded detail is an attempt to load the 44-mm (1.75-in.) wide loose plate axially, for which purpose it has been rotated so that it makes a 20-degree angle with the vertical joint. This connection has been tested in monotonic tension, and it has been found to be as good as, if not better than, shear plates (Stanton et. al., 1986).

The Pinned Tension Strut (Fig. 4a) is an attempt to improve the Inclined Flat Bar detail (Fig. 3c). There is evidence indicating that sizable flexural stresses are induced in this bar during loading, and that these stresses limit the amount of plastic deformation before failure (Stanton et. al., 1986). The flat bar is replaced by three loose plates, of which two are bolted. The third plate, which is notched, is welded to the other two and has an effective width of 51 mm (2 in.). The notches serve to control the location of yielding and plastic deformation in the 9.5-mm (0.375-in.) thick plate, as well as to limit plate strength so that it does not exceed weld strength. The bolted ends eliminate the potential for flexural stresses, and the three-plate arrangement provides sufficient tolerance to accommodate the usual variation in actual vertical joint width. To accommodate the plate width, the strut intersects the vertical joint at a 35-degree angle.

The Brass Friction Device (Fig. 4b) is an adaptation of the Slotted Bolted Connection developed by Popov (Grigorian et. al. 1992). Key elements of this connection are the 3-mm (0.125-in.) thick half-hard cartridge brass plates placed between the anchor plate, the slotted plate, and the top loose plate. Brass plates provide a reliable friction force [5], and the required 38-kN (8.6-kip) normal force is generated by steel disc springs placed below the bolt heads.

Inelastic flexural deformations, which are mobilized by the rolling action of the u-shaped plate as it bends and unbends, is the source of resistance and hysteretic energy dissipation in the last bolted connection (Fig. 4c). This device was developed by Kelly as an energy dissipating connector for precast panels (Kelly et. al. 1972), and it is used in the present study with essentially no change. The central portion of a 16-mm (0.625-in.) thick plate that is 127 mm (5 in.) wide is bent into a semicircle with an inside radius of 43 mm (1.69 in.). Tolerance to accommodate variable-thickness joints can be provided by fabricating the u-shaped plate such that it fits in the pocket even when the vertical joint width is equal to zero. Shim plates can be used as needed to properly install the u-shaped plate.

Conclusion

Table 1 summarizes the expected response of the six vertical joint connections. Initial stiffnesses were calculated assuming elastic behavior, and the other response parameters were estimated assuming that the connections respond exclusively in the mode identified in Table 1. In calculating yield force and ultimate strength, the steel plates were assumed to have yield strength F_y and strength F_u equal to 248 N/mm^2 (36 ksi) and 400 N/mm^2 (58 ksi), respectively, and maximum displacements are based on an assumed uniaxial tensile strain at necking ε_u equal to 0.20 and a shear strain γ_u equal to $\varepsilon_u/\sqrt{3}$. Because large-amplitude load reversals are likely to affect behavior, maximum displacement and energy dissipation, which were calculated assuming monotonic loading, are reported as normalized indices for the purpose of comparison.

Table 1. Estimated response of vertical joint connection details.

	Vertical Joint Connection Detail					
Parameter*	NSP	SFP	IFB	PTS	BFD	UFP
Mode	Shear	Flexure	Axial	Axial	Friction	Flexure
Initial Stiffness	1,300 (7,440)	1,290 (7,380)	455 (2,600)	588 (3,360)	2,580 (14,750)	43 (240)
Yield Force	99.1 (22.3)	98.0 (22.0)	98.7 (22.2)	98.4 (22.1)	102 (23.0)	39.1 (8.8)
Ultimate Strength	160 (35.9)	237 (53.3)	159 (35.8)	158 (35.6)	120 (23.0)	63 (8.8)
Disp. Index†	1.0	2.1	6.1	1.8	7.8	8.1
Energy Index†	1.0	3.1	6.1	1.7	5.0	3.2

*Units are kN (kips) for force and kN/mm (kip/in.) for stiffness.
†Relative to response of Notched Shear Plate.

With large initial elastic stiffnesses, the Notched Shear Plate and Slotted Flexure Plate connections are well-suited for strong coupling of shear wall panels (i.e. monolithic behavior), but low deformation and energy dissipation capacities limit the use of these connections to regions of low-to-moderate seismic risk. In developing the Pinned Tension Strut detail as an improvement of the Inclined Flat Bar detail, the volume of steel available to absorb energy by plastic deformation has been reduced considerably, however, the potential for compression buckling has been reduced as well. Due to the buckling potential of the Inclined Flat Bar and the limited toughness of the Pinned Tension Strut, these connections should be restricted to moderate seismic demands.

The Brass Friction Device and the U-Shaped Flexure Plate connections are better suited for cases in which large excursions beyond yield are likely, particularly the former which can be detailed to provide both ample toughness and strength. Due to shallow connector depth, in relation to that of other connector plates, the U-Shaped Flexure Plate connections possesses smaller stiffness and strength. Unlike the other five connection details, it was not possible to proportion the u-shaped plate to resist the target vertical shear force and still retain realistic dimensions. This feature, as well as the small stiffness to horizontal in-plane forces in the wall, make this connection ideal for control joints which also serve as dampers.

Acknowledgements

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