Untopped Precast Concrete Diaphragms in High-Seismic Applications

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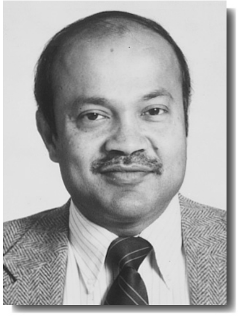


This article indicates how code-complying untopped precast concrete diaphragms may be designed for buildings assigned to high Seismic Design Categories (D and above) by the International Building Code or the NFPA 5000 Building Code.

During the last decade, major changes have been made to the seismic design provisions in the model codes of the United States. These changes have largely come from the National Earthquake Hazards Reduction Program (NEHRP) Provisions,[1] sponsored by the Federal Emergency Management Agency (FEMA) and developed through the Building Seismic Safety Council (BSSC).

The NEHRP Provisions, first published in 1985, have been updated every three years since then. The 1997 NEHRP Provisions formed the basis of seismic design provisions in the first edition (2000) of the International Building Code (IBC).[2] The 2000 NEHRP Provisions form the basis of the seismic design provisions of the 2003 IBC as well as the just-published first edition (2003) of the NFPA Building Code.[3]

A number of features of the 1997 (and subsequent) NEHRP Provisions have had a profound impact on seismic design by the above model codes:

First, the design-basis earthquake is no longer an earthquake with a 10 percent probability of non-exceedance in 50 years (or an earthquake with a return period of approximately 500 years). The design earthquake of the 1997 (and subsequent) NEHRP Provisions is two-thirds of the Maximum Considered Earthquake (MCE), which for coastal California is the largest earthquake that can be delivered by the known seismic sources. Elsewhere in the country, the Maximum Considered Earthquake is an earthquake with a 2 percent probability of non-exceedance in 50 years (or an earthquake with a return period of approximately 2500 years).

Second, the ground motion parameters input in seismic design are now spectral accelerations at periods of 0.2 second (S_s) and 1 second (S_1), corresponding to the MCE, on soft rock typical of the western United States (Site Class B).

Third, the ground motion parameters S_s and S_1 are modified by a short-period site coefficient, S_s, and a long-period site coefficient, S_1, respectively, both of which are functions not only of the Site Class (soil characteristics), but also of seismicity at the site of the structure (F_a is a function of S_s, while F_v is a function of S_1).

Fourth, and perhaps most importantly, the level of detailing required for a structure (as well as height limits on structural systems, whether dynamic analysis is required as the basis of design, and other decisions) is determined by the Seismic Design Category (SDC) to which the structure is assigned. The SDC combines the occupancy of the structure, the seismicity at the site of the structure (short-period seismicity represented by S_s as well as long-period seismicity represented by S_1), and also the soil characteristics (or Site Class) at the site of the structure.

Seismic Design Categories A, B require ordinary detailing in compliance with the applicable requirements of Chapters 1 through 18 of ACI 318-99 or -02,[4] as is required by ACI 318 for regions of low seismicity (Uniform Building Code or UBC Seismic Zones 0 and 1). SDC C requires intermediate detailing which necessitates ordinary detailing plus compliance with the applicable requirements of Section 21.10 of ACI 318-99 or Sections 21.12 and 21.13 of ACI 318-02, as is required by ACI 318 for regions of moderate seismicity (UBC Seismic Zone 2).

SDC D, E and F require special detailing in compliance with the applicable requirements of Chapters 1 through 17 of ACI 318-99 or -02, plus Sections 21.2 through 21.8 of ACI 318-99 or Sections 21.2 through 21.10 of ACI 318-02, as is required by ACI 318 for regions of high seismicity (UBC Seismic Zones 3, 4).

Because of the soil-dependence of the Seismic Design Category, moderate- or high-seismic-zone detailing is no longer a problem confined to the West Coast and certain other parts of the country. Particularly for structures founded on softer soils, moderate- or even high-seismic-zone detailing may now be required for buildings designed for such unlikely places as Cincinnati, Ohio, Atlanta, Georgia, and Charleston, South Carolina.

SEISMIC DESIGN PROVISIONS FOR PRECAST CONCRETE STRUCTURES

The ACI 318 standard, through its 1999 edition, did not contain precast-specific provisions in Chapter 21 (Special Provisions for Seismic Design). Precast moment frames in regions of moderate to high seismicity and precast shear walls in regions of high seismicity could be designed under the provisions of ACI 318 only using the equivalency provision of Section 21.2 which requires a precast concrete structure to be equivalent to a comparable monolithic concrete structure in terms of strength as well as toughness (an inclusive term for quantities related to deformation as well as energy dissipation).

Specific seismic design provisions for precast concrete structures in regions of moderate to high seismicity were first introduced in the 1994 NEHRP Provisions by way of amendments to Chapter 21 of ACI 318-89 (Revised 1992). The provisions, with modifications, were adopted into the 1997 UBC[5] by way of amendments to ACI 318-95. The 1997 NEHRP Provisions as well as the 2000 IBC contains essentially the same seismic design provisions for precast concrete structures as the 1997 UBC. The 1997 NEHRP Provisions makes amendments to Chapter 21 of ACI 318-95, while the 2000 IBC amends Chapter 21 of ACI 318-99.

The 2000 NEHRP Provisions contains the most comprehensive design requirements for precast concrete structures in regions of moderate to high seismicity. Included are design provisions for precast concrete structures that emulate cast-in-place construction (emulative design) and those that do not (non-emulative design or jointed precast). Emulative design of special moment frames may be done using strong connections that remain elastic while inelastic displacements in the design-basis earthquake take place at locations remote from the connections.

Alternatively, such a design may be accomplished using ductile connections that have adequate nonlinear response characteristics, so that earthquake energy may be dissipated in the connections themselves. Emulative design of special shear walls may be done using ductile connections. Non-emulative design of special moment frames as well as special shear walls is also permitted. Non-emulative design of special moment frames requires compliance with ACI Standard T1.1.[6] Such design of special shear walls requires compliance with portions of T1.1 and other criteria that are modeled after or are modified versions of those in T1.1.

The 2002 edition of ACI 318 for the first time has specific seismic design provisions for precast concrete structures in regions of moderate to high seismicity. The NEHRP 2000 precast provisions were used as a starting point. Emulative design requirements for an intermediate precast shear wall for use in regions of moderate seismicity were added. Non-emulative design requirements for special shear walls were dropped. Other modifications were made that are not particularly relevant to discussion in this article.

UNTOPPED DIAPHRAGMS FOR BUILDINGS IN HIGH SEISMIC DESIGN CATEGORIES

For the 2000 NEHRP Provisions, design requirements were also developed for untopped diaphragms for use in buildings assigned to high Seismic Design Categories (D and above). To gain consensus, compromises were made that included very conservative diaphragm design forces intended to ensure that the diaphragms would remain elastic during the design seismic event.

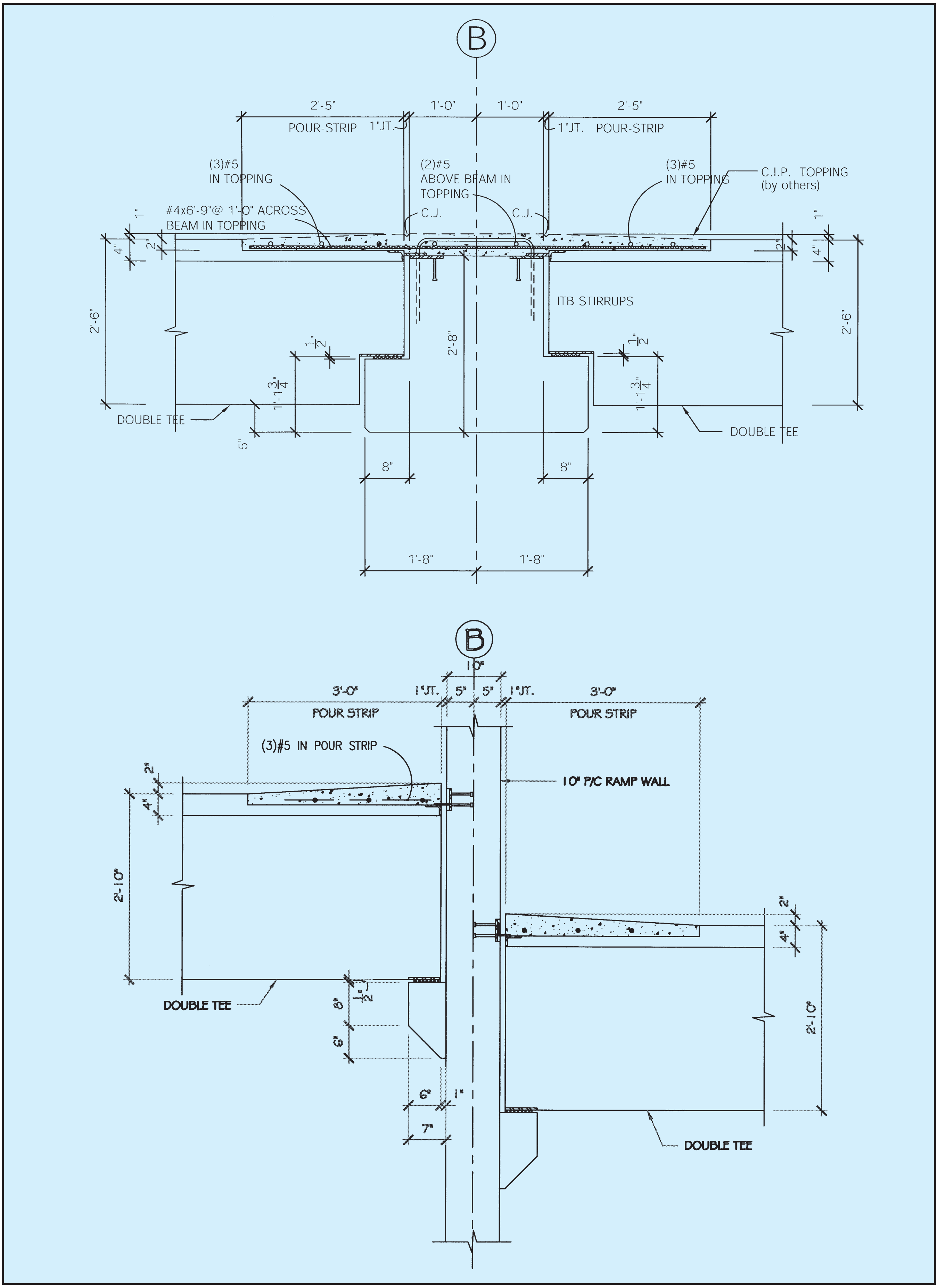


Fig. 1. Cast-in-place pour strips as part of untopped precast diaphragms.

It may be argued that the same force levels should be used in the design of all diaphragms (including topped precast and cast-in-place diaphragms) that are supposed to remain elastic through the design seismic event. The Provisions Update Committee of the BSSC chose not to include the untopped diaphragm requirements in the main body of the 2000 NEHRP Provisions; they chose, instead, to place them in an Appendix to the Concrete chapter, for trial design and comments.

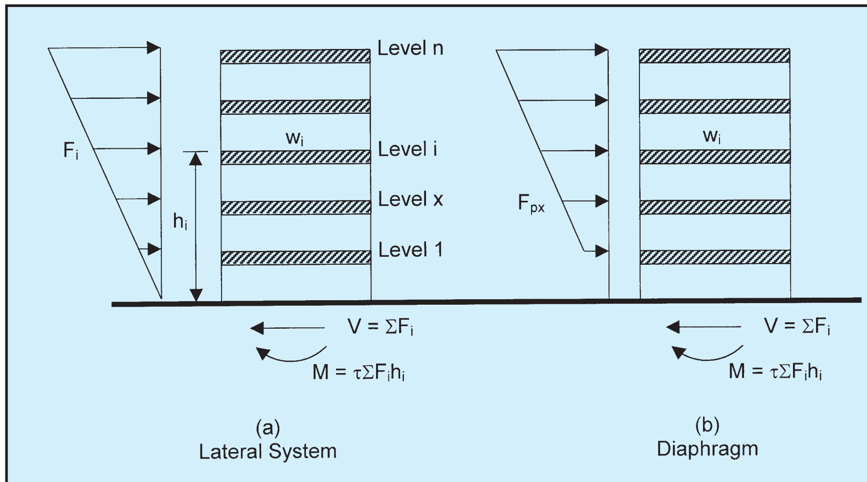
The diaphragm design provisions of ACI 318 underwent significant changes in the 1999 edition and have remained unchanged since then. Section 21.9 of ACI 318-02 contains design requirements for two types of precast diaphragms. The cast-in-place composite topping slab diaphragm transmits lateral forces to vertical elements of the lateral-force-resisting system through composite action of precast double tees or hollow-core slabs and a cast-in-place topping slab.

Note that in a cast-in-place noncomposite topping slab diaphragm, the topping acting alone as the diaphragm transmits lateral forces to vertical elements of the lateral-force-resisting system. Both types of diaphragms obviously require a cast-in-place topping. Although there is no specific provision that prohibits the use of an untopped (or pretopped) diaphragm, such a design does not comply with the requirements of Section 21.9.

In order to use untopped precast double tees or hollow-core slabs as the structural diaphragm in a high Seismic Design Category design, the authors believe it to be necessary to apply ACI 318-02 Section 21.2.1.5:

"A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter."

This provision has been part of Chapter 21 of ACI 318 for several editions and, as mentioned, has been the primary basis used to qualify precast wall and frame systems before prescriptive provisions were developed to describe in detail how these systems should be designed to meet the requirement of equivalency. In the absence of a consensus for specific prescriptive provisions, it is necessary to demonstrate that an untopped precast diaphragm satisfies this equivalency requirement.



$$F_i = \frac{w_i h_i^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad F_{px} = \left(\frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} \right) w_{px}$$

k = 1 for T ≤ 0.5 sec.
= (T+1.5)/2 for 0.5 sec. < T < 2.5 sec.
= 2 for T ≥ 2.5 sec.

Fig. 2. Equivalent lateral forces for (a) lateral-force-resisting system, (b) diaphragm.

In our view, there are four key parts to the requirements: (1) experimental evidence; (2) analysis; (3) equal to or exceeding; and (4) comparable monolithic reinforced concrete structure. To show compliance, it is probably easiest to treat these in reverse order.

Comparable Monolithic Reinforced Concrete System

The comparable monolithic reinforced concrete system is the cast-in-place topping slab diaphragm of ACI 318-02 Section 21.9.3. This is a cast-in-place topping slab proportioned and detailed to act alone as the diaphragm, resisting the entire diaphragm design forces. The use of cast-in-place pour strips at the ends of the untopped precast elements, designed for the tension and compression chord forces, essentially provides this cast-in-place topping system for that part of the untopped diaphragm (see Fig. 1).

In Section 21.9.7.2, the ACI Code requires that the entire shear in the diaphragm be carried by the reinforcing steel [Eq. (21-11)] and that no reliance be placed on any shear strength contributed by the concrete. The shear transfer for the untopped system must be comparable to that provided by the shear reinforcement designed by Eq. (21-11).

At this point, we are not aware of specific tests that have been made to demonstrate the equivalent strength and ductility of mechanically welded connections to replace continuous chord reinforcing. This might be done in the future, but for now it would appear that pour strips with continuous reinforcement for the chords should be used for high seismic design.

Equivalent Performance

The "equal to or exceeding" provision is generally addressed by a consensus in the industry that precast con-

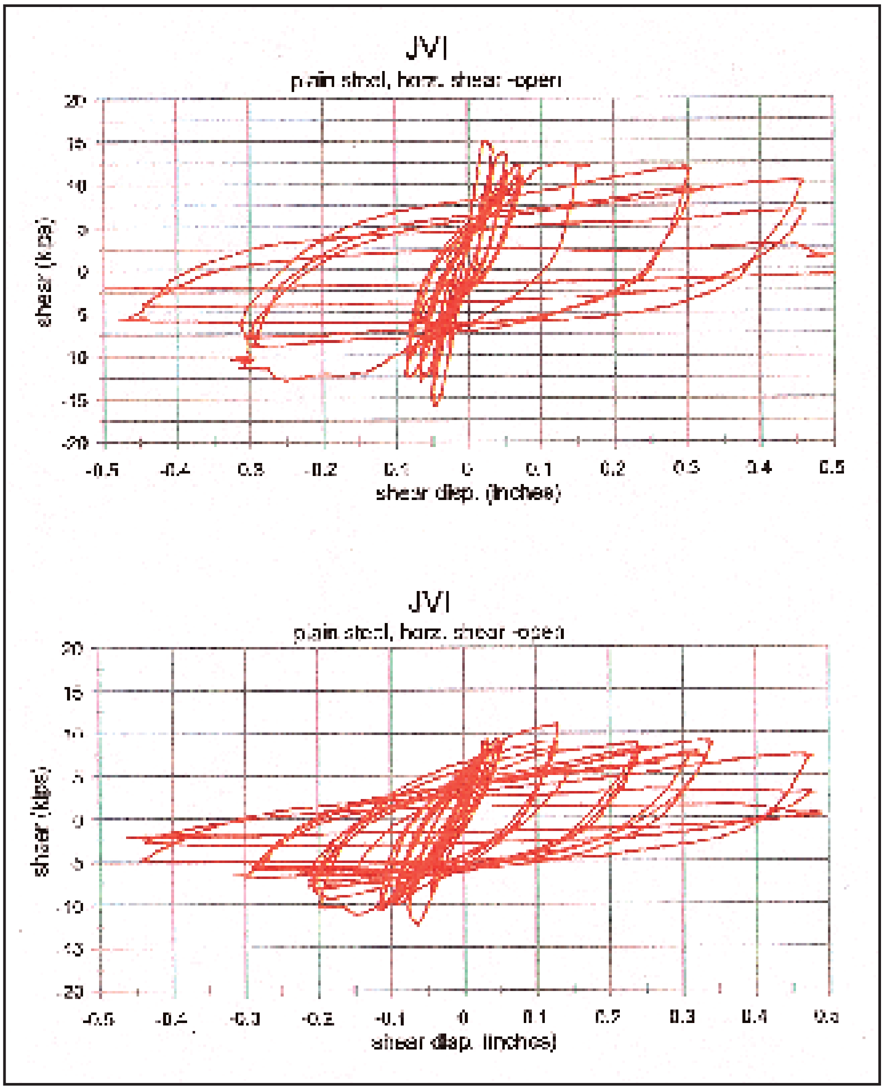


Fig. 3. Shear-to-shear displacement relationships of plain carbon steel connectors under cyclic horizontal shear with joint opening. Courtesy: JVI Inc., Lincolnwood, Illinois.

crete diaphragms should be designed to remain essentially elastic through the design seismic event. It is important to understand that the model code requirements for diaphragm design are not consistent with this implied intent of the code. This apparent anomaly deserves more discussion than can be provided here.

The code values for response modification and displacement amplification factors are based on the assumption that it is the vertical elements of the lateral-force-resisting system that will yield and dissipate energy. If the yielding occurs in the diaphragm, the design load path may be compromised and the inelastic characteristics of the vertical elements of the lateral-force-resisting system may not be mobilized. The code-prescribed design forces, however, are at a level that is comparable with the reduced forces used for the design of the vertical lateral-force-resisting system (see Fig. 2) and could result in yielding in the diaphragm in the design-basis earthquake. We recognize this deficiency and recommend that the code-prescribed diaphragm design forces be amplified by a factor for diaphragm design purposes, to prevent inelastic diaphragm response in the design-basis earthquake.

There has been a significant disagreement on what this factor should be. Some suggest that it should be the system overstrength factor, Ω_o, assigned to each seismic-force-resisting structural system defined in the IBC. These values range from $2^1/_2$ to 3 for concrete systems. Others believe that Ω_o multiplied by the redundancy factor, ρ, should be used as the multiplier. This, in fact, is required by the untopped diaphragm appendix of the 2000 NEHRP Provisions. The redundancy factor varies from 1 to $1^1/_2$, so that the multiplier could be as high as $4^1/_2$.

Research on precast diaphragms after the Northridge earthquake, however, suggests that a value of about 2 is sufficient as long as the design for the most severely loaded floor is applied to every floor to protect the lower floors from higher mode effects.[7] Where the diaphragm span is not excessive, the use of a multiplier of 2 or higher on code-prescribed diaphragm design forces will ensure that vertical elements of the lateral-force-resisting systems will yield before the diaphragm.

Analysis

The question of equal or greater toughness leads to the third part, the analysis. With the design of precast diaphragms, much of the analysis follows a horizontal plate girder analogy that is not unique to precast concrete. With precast jointed systems, however, it has been recognized by Nakaki[8] that the strain related to deformation is concentrated at the joints.

This realization is recognized empirically by ACI 318-02 Section 21.9.5.1, where it is requires that wires in welded wire fabric in topping slabs be spaced at not less than 10 in. (250 mm) on center. The idea is to provide a longer length between wire anchorages so that the strain from joint opening is spread over a longer distance to avoid fracture in the wires as they stretch. Nakaki suggests that the joint spread be checked by analysis and compared to the wire strain capacity in topped systems and to the connection strain capacity in untopped systems. If necessary, the chord reinforcement may need to be increased to control this joint opening, beyond the calculated requirements for chord strength.

To some degree, this extra analysis may be avoided if the welded shear connections used in place of the wire fabric across the joint are shown to have sufficient deformation capacity. They must sustain their design strength through the maximum anticipated joint opening to demonstrate sufficient toughness of the system.

Experimental Evidence

Tests have been carried out by Oliva[9] on many prototype flange connectors to determine their strain capacity at the University of Wisconsin in Madison. Many of the common plant-fabricated connectors designed with reinforcing bars butt-welded to the backs of plates failed to show sustained capacity or strain tolerance under reversed cyclic loading. A couple of commercial flange connections, including JVI's Vector connector that was designed specifically to have improved strain capacity, showed that they have sustained shear capacity even with a $^1/_4$ in. (6 mm) or more joint opening and under reversed cyclic loading (see Fig. 3). The test results for their connector are available from JVI.

This addresses the fourth part of meeting the requirements of Section 21.2.1.5: experimental evidence. With the selection of tested welded connections as the replacement for the steel reinforcing to provide the shear strength required in Section 21.9.7.2, where the deformation reflected as joint opening is analyzed and controlled, it is possible to demonstrate equivalency of the untopped system.

We have made calculations for common diaphragm conditions with reasonable spans and aspect ratios that show that these connections did not need additional chord reinforcement for protection against joint strains. This might not be true if the diaphragm spans get long or the span-to-depth ratio gets large. Therefore, the diaphragm probably needs to be checked if the aspect ratio is larger than 3.

CONCLUDING REMARKS

It is suggested in this article that it is possible to design untopped precast concrete diaphragms for buildings assigned to high Seismic Design Categories (D and above) under the equivalency clause of Section 21.2.1.5 of ACI 318-02. The paper outlines how such equivalency is to be achieved.

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