

Appendix to Chapter 9

UNTOPPED PRECAST DIAPHRAGMS

PREFACE: Reinforced concrete diaphragms constructed using untopped precast concrete elements are permitted in the text of the *Provisions* for Seismic Design Categories A, B, and C but not for Categories D, E, and F. For the latter, the precast elements must be topped and the topping designed as the diaphragm. For resisting seismic forces, a composite topping slab cast in place on precast concrete elements must have a thickness of not less than 2 in. (51 mm) and a topping slab not relying on composite action with the precast elements must have a thickness of not less than 2-1/2 in. (64 mm).

There are two principal reasons why a framework for the design of untopped diaphragms for Seismic Design Categories D, E, and F may be desirable. One relates to the performance of topping slab diaphragms in recent earthquakes and the other to durability considerations. The 1997 *Provisions* incorporated ACI 318-95 for which the provisions for topping slab diaphragms on precast elements were essentially the same as those in ACI 318-89. In the 1994 Northridge earthquake, performance was poor for structures where demands on the topping slab diaphragms on precast elements were maximized and the structures had been designed using ACI 318-89. The topping cracked along the edges of the precast elements and the welded wire reinforcement crossing those cracks fractured. The diaphragms became the equivalent of an untopped diaphragm with the connections between precast concrete elements, the connectors, and the chords not detailed for that condition. Another problem found with topping slab diaphragms was that the chords often utilized large diameter bars, grouped closely together at the topping slab edge. Under severe loading, these unconfined chord bars lost bond with the concrete and thus lost the ability to transfer seismic forces.

ACI 318-99 was significantly revised for structural diaphragms to add new detailing provisions in response to the poor performance of some cast-in-place composite topping slab diaphragms during the 1994 Northridge earthquake. New code and commentary sections 21.7 and R21.7 were added to Chapter 21. Cast-in-place composite topping slabs and cast-in-place topping slab diaphragms were permitted by ACI 318-99, but no mention was made of untopped precast diaphragms. The diaphragm provisions of ACI 318-99 were carried over unchanged into ACI 318-02 and placed in Sec. 21.9 rather than Sec. 21.7, where they had been in ACI 318-99.

The evidence from the recently completed PRESSS 5-story building test (M. J. NM. Priestley, D. Sritharan, J. R. Conley, and S. Pampanin, "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building," *PCI Journal*, Vol. 44, No. 6, November-December 1999), from Italian and English tests (K. S. Elliott, G. Davies, and W. Omar, "Experimental Hollow-cored Slabs Used as Horizontal Floor Diaphragms," *The Structural Engineer*, Vol. 70, No. 10, May 1992, pp. 175-187; M. Menegotto, "Seismic Diaphragm Behavior of Untopped Hollow-Core Floors," *Proceedings, FIP Congress*, Washington, D. C., May 1994), and from the 1999 Turkey earthquake is that such diaphragms can perform satisfactorily if they are properly detailed and if they and their connections remain elastic under the force levels the diaphragms experience. However, further additions to the ACI 318-02 requirements are needed if such performance is to be achieved. In particular, the diaphragm design forces and detailing requirements for ductility of connections (as a second line of defense) require revision.

These *Provisions* incorporate ACI 318-02, which recognizes that for topping slab diaphragms a controlling condition is the in-plane shear in concrete along the edges of the precast elements. Ductility is provided by requiring that the topping slab reinforcement crossing those edges be spaced at not less than 10 inches on center. While those requirements are based on the best

available engineering judgment and evidence, they have not as yet been proven to provide adequate safety either by laboratory testing or field performance. Due to the dimensions of the precast element relative to the thickness of the topping slab, it may well be prudent to have seismic provisions for diaphragms incorporating precast elements controlled by untopped diaphragm considerations and to have those provisions modified for topped diaphragms. Further, in geographic areas where corrosive environments are a significant concern, the construction of untopped diaphragms using “pre-topped” precast elements rather than topped elements, can be desirable.

This appendix provides a compilation of current engineering judgment on a framework for seismic provisions for untopped diaphragms. That framework does not, however, adequately address all the concerns needed for its incorporation into the text of the *Provisions*. This appendix proposes that a diaphragm composed of untopped elements be designed to remain elastic, and that the connectors be designed for limited ductility, in the event that design forces are exceeded during earthquake response and some inelastic action occurs where the demands on the diaphragm are maximized. By contrast, for all other systems assigned to Seismic Design Category D, E, or F, the philosophy of the *Provisions* is to require significant ductility. For the approach of this appendix, critical issues are how best to define:

The design forces for the diaphragm so that they are large enough to result in essentially elastic behavior when the demands on the diaphragm are maximized, or whether that criterion is even achievable;

1. The relation between the response of the diaphragm, its dimensions, and the ductility demands on the connectors;
2. The ductility changes that occur for connectors under various combinations of in-plane and out-of-plane shear forces, and tensile and compressive forces;
3. The boundary conditions necessary for testing and for application of the loading for the validation testing of connectors; and
4. The constraints on connector performance imposed by their size relative to the size of the diaphragm elements.

The use of this appendix as a framework for laboratory testing, analyses of the performance of diaphragms in past earthquakes, analytical studies, and trial designs is encouraged. Users should also consult the *Commentary* for guidance and references. Please direct all feedback on this appendix and its commentary to the BSSC.

In this appendix, the untopped precast diaphragm is designed to remain elastic by requiring that its design forces be based on Eq. 4.6-2, and be not less than a minimum value dependent upon the seismic response coefficient, with both values multiplied by the overstrength and redundancy factors associated with the seismic-force-resisting system. In addition, the connections are required to be able to perform in a ductile manner in the unlikely event that the diaphragm is forced to deform inelastically.

A9.1 GENERAL

A9.1.1 Scope. This appendix provides guidelines for the design of diaphragms using untopped precast concrete elements for Seismic Design Categories D, E, and F.

A9.1.2 References

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| ACI 318 | <i>Building Code Requirements for Structural Concrete</i> , American Concrete Institute, 2002 |
| ACI T1.1-01 | <i>Acceptance Criteria for Moment Frames Based on Structural Testing</i> , American Concrete Institute, 2001. |

ATC-24 *Guidelines for Seismic Testing of Components of Steel Structures*, Applied Technology Council, 1992.

A9.1.3 Definitions

Boundary elements: See Sec. 2.1.3.

Chord: See Sec. 12.1.3.

Collector: See Sec. 4.1.3.

Component: See Sec. 1.1.4.

Design strength: See Sec. 4.1.3.

Diaphragm: See Sec. 4.1.3.

Drag strut: See Sec. 4.1.3.

Nominal strength: See Sec. 4.1.3.

Quality assurance plan: See Sec. 2.1.3.

Required strength: See Sec. 4.1.3.

Seismic Design Category: See Sec. 1.1.4.

Seismic-force-resisting system: See Sec. 1.1.4.

Structure: See Sec. 1.1.4.

Untopped precast diaphragm: A diaphragm consisting of precast concrete components that does not have a structural topping meeting the requirements of these *Provisions*.

A9.1.4 Notation.

C_S See Sec. 5.1.3.

F_{px} See Sec. 4.1.4.

w_{px} See Sec. 4.1.4.

ρ See Sec. 4.1.4.

ϕ See Sec. 5.1.3.

Ω_0 See Sec. 4.1.4.

A9.2 DESIGN REQUIREMENTS

Untopped precast floor or roof diaphragms in Seismic Design Category D, E, or F shall satisfy the requirements of this section.

A9.2.1 Configuration. Untopped diaphragms shall not be permitted in structures with plan irregularity Type 4 as defined in Table 4.3-2. For diaphragms in structures having plan irregularities Type 1a, 1b, 2, or 5 as defined in Table 4.3-2, the analysis required by Sec. A9.2.2 shall explicitly include the effect of such irregularities as required by Sec. 4.6.

A9.2.2 Diaphragm demand. Rational elastic models shall be used to determine the in-plane shear, tension, and compression forces acting on connections that cross joints. For any given joint, the connections shall resist the total shear and total moment acting on the joint assuming an elastic distribution of stresses.

The diaphragm design force shall be taken as the lesser of the following two criteria:

1. $\rho\Omega_0$ times the F_{px} value calculated from Eq. 4.6-2, but not less than $\rho\Omega_0 C_S w_{px}$; or
2. A shear force corresponding to 1.25 times that corresponding to yielding of the seismic-force-

resisting system, calculated using ϕ value(s) equal to unity.

In item 1 above, the overstrength factor, Ω_o , shall be that for the seismic-force-resisting system as specified in Table 4.3-1, the redundancy factor, ρ , shall be as specified in *Provisions* Sec. 4.3.3, and the seismic response coefficient, C_s , shall be as determined in accordance with *Provisions* Sec. 5.2.1.1.

A9.2.3 Mechanical connections. Mechanical connections shall have design strength, for the body of the connector, greater than the factored forces determined in accordance with Sec. A9.2.2.

Mechanical connections used at joints shall be shown by analysis and testing, under reversed cyclic loading, to develop adequate capacity in shear, tension, and compression (or a combination of these effects) to resist the demands calculated in accordance with Sec. A9.2.2. Testing of connections and evaluation of results shall be made in accordance with the principles specified in ACI T1.1 and ATC-24.

When subjected to the specified loading, connections shall develop ductility ratios equal to or greater than 2.0. The behavior of connection embedments shall be governed by steel yielding and not by fracture of concrete or welds.

Connections shall be designed using the strength reduction factors, ϕ , specified in ACI 318. Where the ϕ factor is modified by Sec. 9.3.4 of ACI 318, the modified value shall be used for the diaphragm connections.

Where the design relies on friction in grouted joints for shear transfer across the joints, shear friction resistance shall be provided by mechanical connectors or reinforcement.

A9.2.4 Cast-in-place strips. Cast-in place strips shall be permitted in the end or edge regions of precast components as chords or collectors. These strips shall meet the requirements for topping slab diaphragms. The reinforcement in the strips shall comply with Sec. 21.9.8.2 and 21.9.8.3 of ACI 318.

A9.2.5 Deformation compatibility. In satisfying the compatibility requirement of Sec. 4.5.3, the additional deformation that results from the diaphragm flexibility shall be considered. The assumed flexural and shear stiffness properties of the elements that are part of the seismic-force-resisting system shall not exceed one-half of the gross-section properties, unless confirmed by a rational, cracked-section analysis.

A9.2.6 Beam connections. Ties to supporting members and bearing lengths shall satisfy the requirements for design force and geometry characteristics specified for the connections in Sec. 21.11.4 of ACI 318.

A9.2.7 Quality assurance. Diaphragms shall have a quality assurance plan in accordance with Sec. 2.2.1 of these *Provisions*.