# Proposed Revisions to 1997 NEHRP Recommended Provisions for Seismic Regulations for Precast Concrete Structures Part 2 – Seismic-Force-Resisting Systems

by


**Neil M. Hawkins, Ph.D.**
Professor Emeritus
Department of Civil Engineering
University of Illinois at
Urbana-Champaign
Urbana, Illinois

**S. K. Ghosh, Ph.D.**
President
S. K. Ghosh Associates, Inc.
Northbrook, Illinois



This is the second in a series of three papers discussing significant modifications expected to be included in the 2000 NEHRP Provisions, dealing with the design of precast, prestressed concrete seismic-force-resisting systems. These modifications are expected to be part of the 2003 edition of the International Building Code.


In the May-June 2000 issue of the PCI JOURNAL, the history and development of the requirements of the NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions[1] for precast and prestressed concrete structures were discussed.

In this article, the specifics of the requirements of the proposed 2000 NEHRP Recommended Provisions for seismic-force-resisting frame and structural wall systems composed of precast and prestressed concrete elements are given and the basis for those provisions is documented.

The revisions discussed in this article are related to Proposal 4-37 and the associated commentary. That proposal and its commentary can be viewed on the internet at www.bssconline.org.

## ORGANIZATION OF SEISMIC-FORCE-RESISTING SYSTEM PROVISIONS

Discussed first is the organization of seismic-force-resisting system provisions.

Table 1 identifies the systems and connections covered by the proposed provisions and shows how the provisions are organized. The information concerning the systems is also presented in Fig. 1. Concrete Structure Design Requirements will appear in Chapter 9 of the 2000 NEHRP Provisions. The 2000 NEHRP Provisions will adopt ACI 318-99[2] as the basic reference document for concrete structures in Section 9.1.

The NEHRP Provisions then amend ACI 318-99 by inserting additional provisions into, or revising the existing provisions of, ACI 318-99. Bold face numerals in Table 1 and Fig. 1 starting with the number 9 identify specific provisions of the NEHRP Provisions. Bold face numerals starting with the number 21 identify specific provisions inserted into ACI 318-99. With one exception, all the seismic-force-resisting system requirements of the 2000 NEHRP Provisions, relating to precast and prestressed concrete, are placed in a new Section 21.11 added to Chapter 21 of ACI 318-99.

In ACI 318-99, the seismic risk of a region is described as low, moderate or high. Chapter 21 contains specific requirements for the design of concrete structures in regions of high and moderate seismic risk. Structures in regions of low seismic risk need only meet the requirements of Chapters 1 through 18 of ACI 318-99. In the NEHRP Provisions, the applicability of Chapter 21 requirements depends not only on the seismic risk at the location of the structure but also on the occupancy of the structure and the characteristics of the soil on which it is founded. In the NEHRP Provisions, the three considerations are combined in terms of Seismic Design Categories (SDC) which are assigned letters A through F.

ACI 318-99 recognizes SDCs A and B as being equivalent to regions of low seismic risk and needing only detailing that meets the requirements of Chapters 1 through 18. Structures assigned to SDC C are recognized as requiring detailing mandated for regions of moderate seismic risk and structures assigned to SDCs D, E and F require detailing prescribed for regions of high seismic risk. The relation between the ACI designation of seismic risk and the NEHRP Seismic Design Categories is shown in Rows 1 and 2 of Table 1.

For precast and prestressed concrete structures assigned to SDCs D, E and F, special moment frames, special structural walls, and Type Z connections must be used in seismic-force-resisting systems. For the design and detailing of special moment frames or special structural walls, the designer

Table 1. 2000 NEHRP Provisions for Precast Concrete Seismic-Force-Resisting Systems.

| **Seismicity** | **Seismic Risk Category** | | | |
|---|---|---|---|---|
| | **High** | | **Moderate** | **Low** |
| System | **Seismic Design Category** | | | |
| | F, E, and D | | C | B and A |
| Moment Frame | Emulation | Special Moment Frame Strong Connections – **21.11.5** | Intermediate Moment Frame – **21.10.7** | No Requirement |
| | | Ductile Connections Wet or Dry – **21.11.3.1** | | |
| | Non-Emulative | Interconnected with Dry Connections – **21.11.4.1** | | |
| Structural Walls | Emulation | Special Structural Wall Strong Connections N/A | As Ordinary Wall – **21.1** | No Requirement |
| | | Ductile Connections – **21.11.4.2** | | |
| | Non-Emulative | Interconnected with Dry Connections – **21.11.4.2** | | |
| Connections **21.11.6** | Connection Type Z – **21.11.6.5**<br>1. $S_{pr} = 1.4\, S_y$<br>2. Anchor for 1.3 $S_{pr}$<br>3. Strut and tie model for connection region design<br>4. Confinement if $f_c > 0.7 f'_c$<br>5. Ductility capacity > 8.0 | | Connection Type Y – **21.11.6.4**<br>1. $S_{pr} = 1.25\, S_y$<br>2. Anchor for 1.3 $S_{pr}$<br>3. Anchor directly connected to main reinforcement<br>4. Ductility capacity > 4.0 | No Requirement |

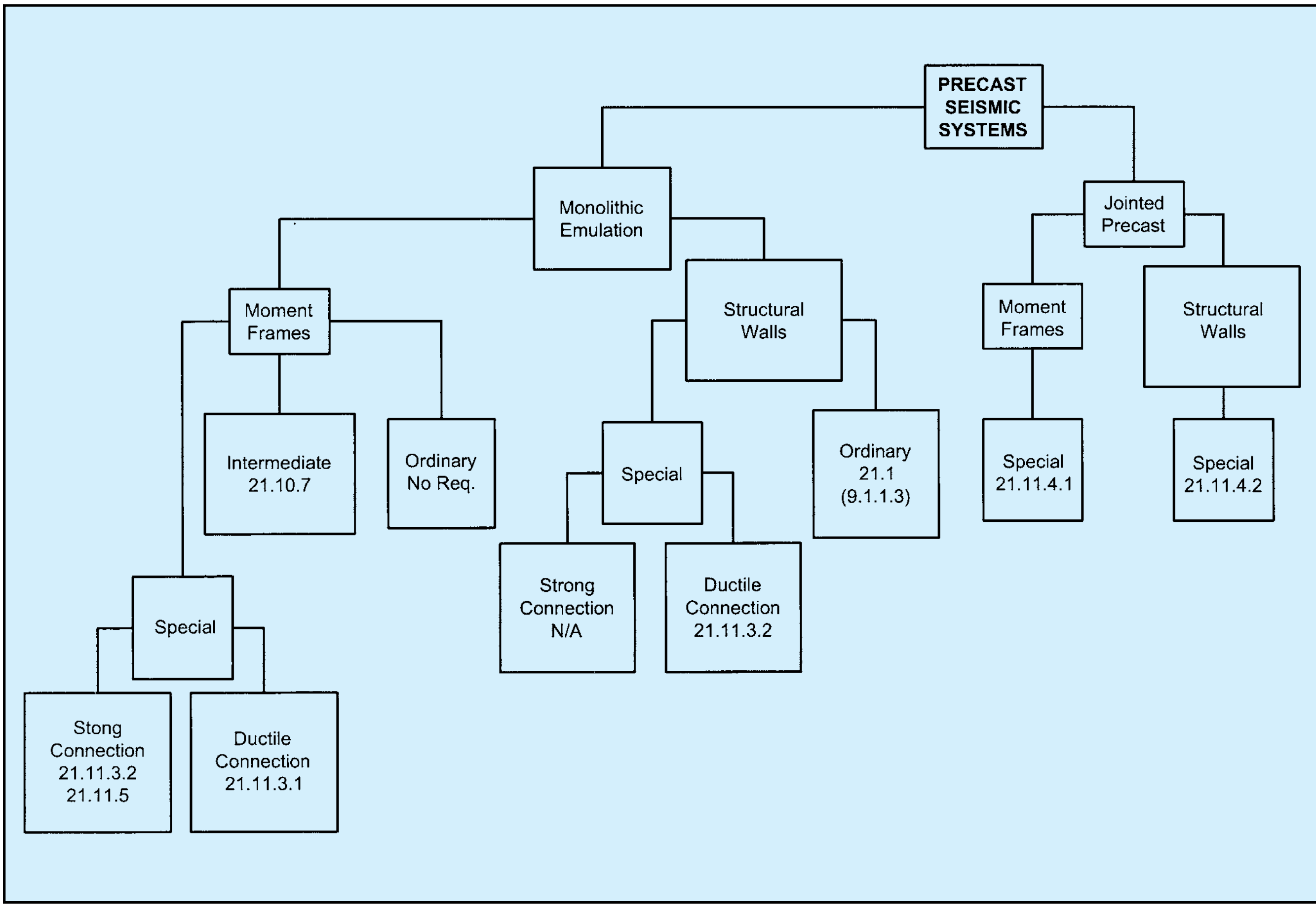



Fig. 1. NEHRP 2000 requirements concerning precast concrete seismic systems.

can use either emulation procedures or procedures that recognize the unique properties of a structure composed of interconnected precast elements (non-emulative).

For structures assigned to SDC C, moment frames made from precast elements must utilize, as a minimum, Type Y connections. However, they may also have the tougher Type Z connections if the designer so chooses. Structural walls composed from precast elements can be designed as ordinary structural walls per Chapters 1 through 18 of ACI 318-99 with the requirements of Chapter 16 superseding those of Chapter 14 and with Type Y connections, as a minimum, between elements.

## SPECIAL MOMENT FRAMES

For precast and/or prestressed concrete special moment frames, hinge locations (nonlinear action locations), must be selected so that there is a strong column/weak beam deformation mechanism under seismic effects, regardless of whether emulative or non-emulative design procedures are used.

### Emulative Design

Requirements for design of precast and prestressed concrete special moment frames, utilizing procedures and details that result in a structure with a behavior under seismic loading emulating that of cast-in-place special moment frames, were first introduced into the NEHRP Provisions in the 1994 Edition. Two design alternatives were provided and those alternatives, with minor changes, have been carried over into the 2000 NEHRP Provisions.

One procedure allows elements to be joined using ductile connections and the other allows elements to be joined using strong connections. A strong connection is designed to remain elastic while inelastic action takes place away from the connection. Because a strong connection must not yield or slip, its design strength in both flexure and shear must be greater than the bending moment and shear force, respectively, corresponding to the development of probable flexural or shear strengths at nonlinear action locations.

Ductile connections, on the other hand, have adequate nonlinear response characteristics, making it unnecessary to ensure nonlinear action locations remote from connections. Typical connection configurations are shown in Fig. 2. Additional information on the behavior and design of precast concrete structures using emulative procedures is contained in Refs. 3 through 6.

**Ductile Connections —** Where elements are joined using ductile connections, the aggregate interlock that is present at hinge locations in monolithic construction is unlikely to exist for precast concrete construction. Therefore, to prevent shear slip when the moment acting at the hinge location is at its maximum probable value of $M_{pr}$, the co-existing shear must not exceed half the sum of the nominal shear strengths, $S_{n\ Connection}$, of all the connections at the hinging section. The nominal shear strength, $V_n$, of the section where the connection is made must also not be

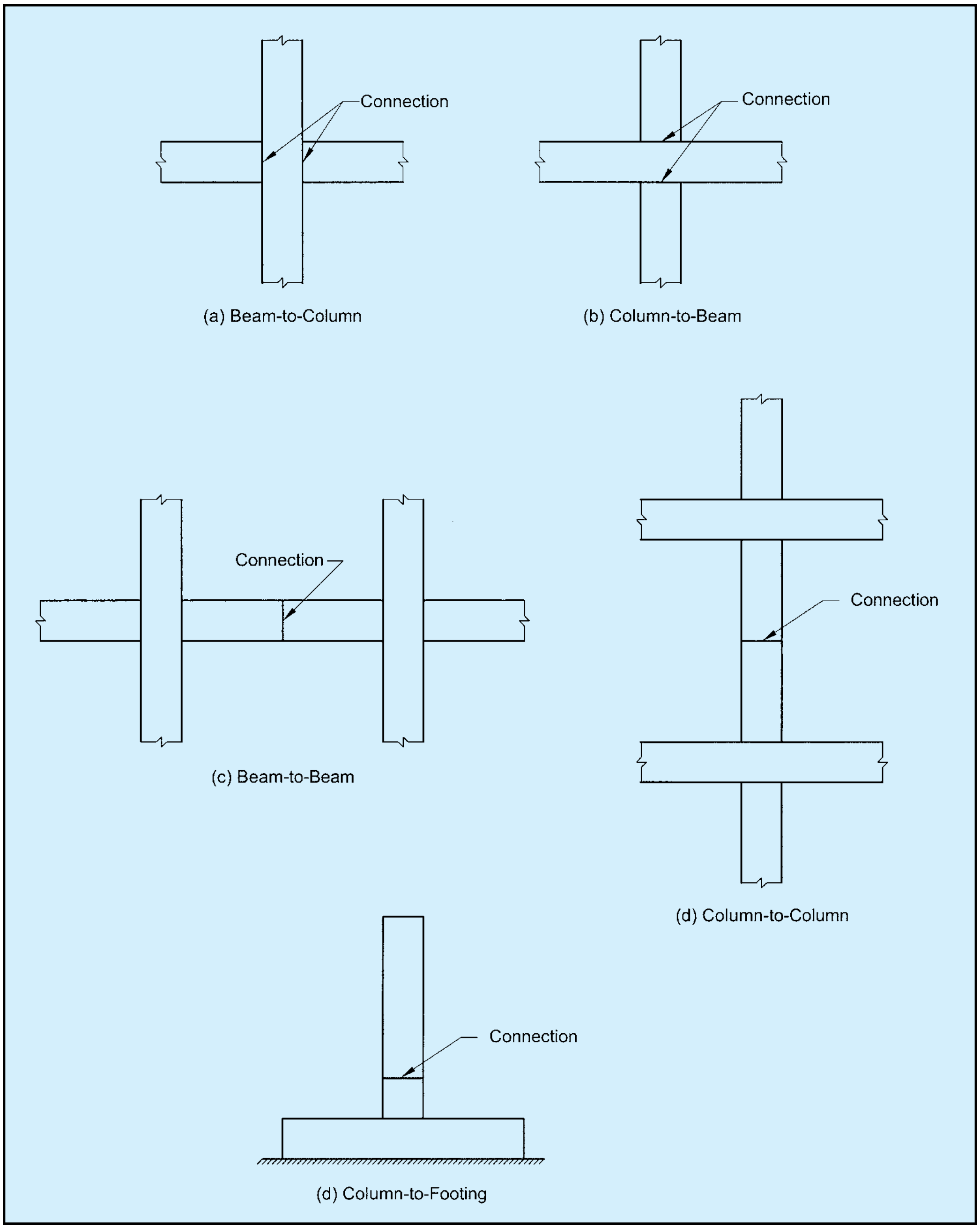



Fig. 2.
Typical precast connection configurations.

less than the shear strengths of the members immediately adjacent to the connection.

Individual connections must satisfy Type Z connection requirements. Those connections can be either "wet" or "dry." A wet connection is defined as one that uses any of the splicing methods (mechanical, welded or lap) specified in Chapter 21 of ACI 318-99 to connect precast members or precast and cast-in-place members, and uses cast-in-place concrete or grout to fill the splicing closure. One type of ductile wet connection widely used in emulative design is the "splice sleeve" connection.[7,8] Other connections with similar ductility capabilities have recently become available or are under development.

**Strong Connections —** Where strong connections are used, the nonlinear action location (center of the nonlinear action region) must be no closer to the near face of the strong connection than half the member depth. Thus, for a frame with strong connections at the beam-to-column interface, reinforcement details must result in beam hinging no closer than half the beam depth away from the column face (see Fig. 3). Any strong connection located outside of the middle half of the span of the beam must be a wet connection unless a dry connection can be justified by approved cyclic test results.

## Non-Emulative Design

Over the last decade many advances have been made in our understanding of the seismic behavior of precast/ prestressed concrete frame structures, as a result of the NIST,[9,10,11] US-PRESSS[12,13,14] and JAPAN-PRESSS research programs. Those advances have made possible the provisional standardization by ACI[15] of acceptance criteria for concrete special moment frames based on validation testing. That provisional standard, together with the

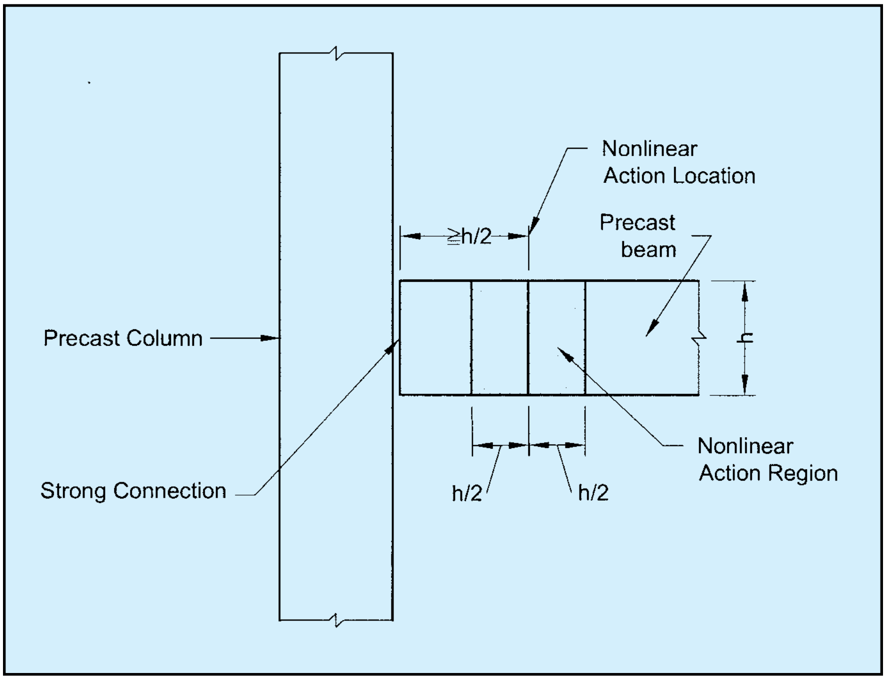



Fig. 3. Example showing nonlinear action region and location.

research advances, has made possible the development of criteria for the design of frames constructed from interconnected precast elements.

While criteria for such frames have existed in the NEHRP Provisions since 1994, the previous criteria were in an Appendix and contained penalties for the use of precast concrete elements compared to monolithic concrete elements. Those penalties are eliminated in the 2000 Provisions and the possible behavioral benefits of using precast construction are recognized.

The complexity of structural systems, configurations and details possible with the use of precast elements requires:

**1.** The selection of functional and compatible details for connections and members that are reliable and can be built with acceptable tolerances.

**2.** Experimental and analytical verification of force-deformation relationships for critical connections of the proposed seismic-force-resisting system.

**3.** Design of the building using those force-deformation relationships and recognizing the loading effects likely to be imposed by the anticipated ground motions.

Traditionally, designers have had the flexibility to widely vary connection details within prescribed code requirements. For non-emulative special moment frame design, that flexibility is sharply curtailed because experience shows that small design changes can have marked effects on the building response in an earthquake. Thus, ACI/ITG T1.1-99 requires a prior development program, including both analytical and experimental investigations of a proposed seismic-force-resisting system, before any validation testing of critical details of the generic frame is undertaken. That ACI Provisional Standard requires that:

**1.** A minimum of one module of

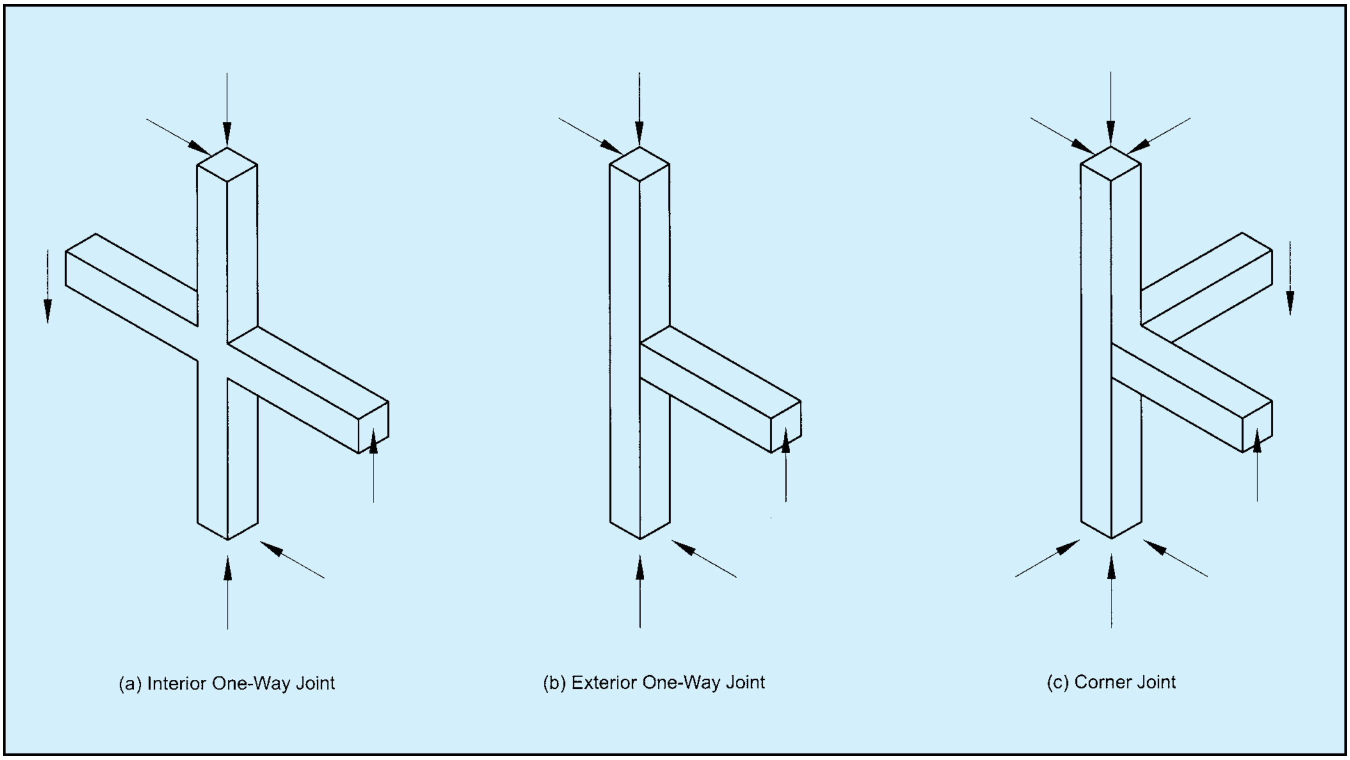



Fig. 4. Illustration of connection configuration.

each characteristic configuration of intersecting beams and columns in the generic moment frame be subject to validation testing (see Fig. 4).

**2.** That testing be conducted at a scale large enough to capture the full complexities and behavior of the materials and load transfer mechanisms in the prototype frame. Test modules must be not less than one-third scale.

**3.** The first loading cycle applied to the module be within the linear elastic response range of the module.

**4.** Test modules be subject to a sequence of displacement controlled drift cycles of increasing magnitude that are representative of the drift cycles expected under earthquake motions (see Fig. 5). Testing is to be continued until the drift ratio (see Fig. 6) equals or exceeds 0.035.

For acceptance of the generic frame, the nominal strength, $E_n$, must be developed before the drift raio exceeds the allowable story drift limitation of the governing building code (Value B in Fig. 7); and the characteristics of the third complete cycle for each test module, at a drift ratio not less than 0.035, must satisfy the following criteria:

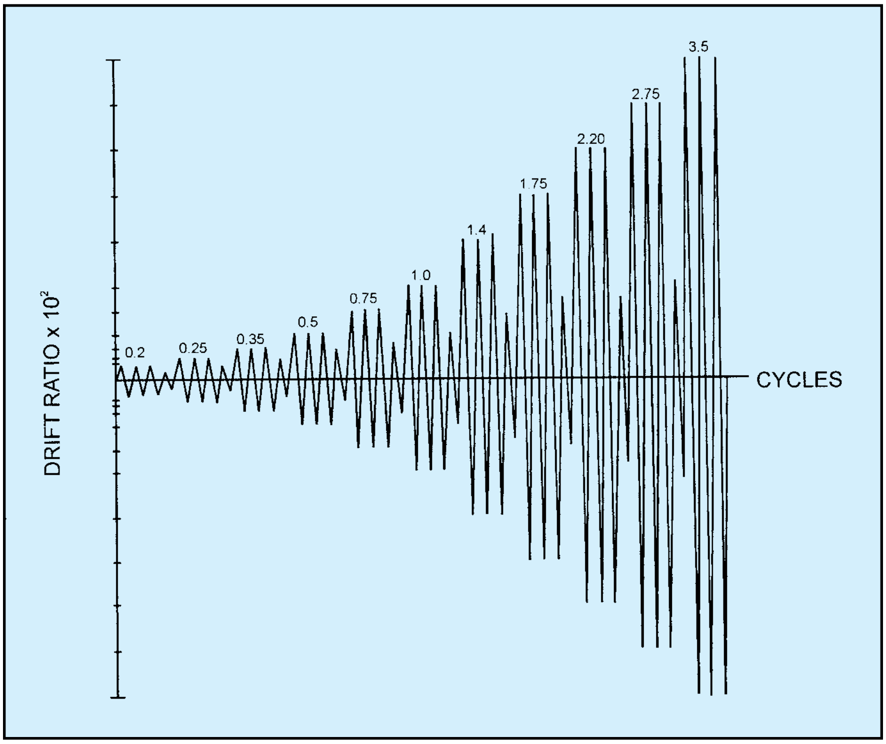



Fig. 5. Cyclic deformation history for validation testing.

**1.** The peak force for a given loading direction must be not less than 75 percent of the peak lateral load for the same loading direction (Value A in Fig. 7).

**2.** The relative energy dissipation ratio, β, must not be less than one-eighth. That ratio equals the area within the hysteretic loop divided by the areas of the circumscribing paral-

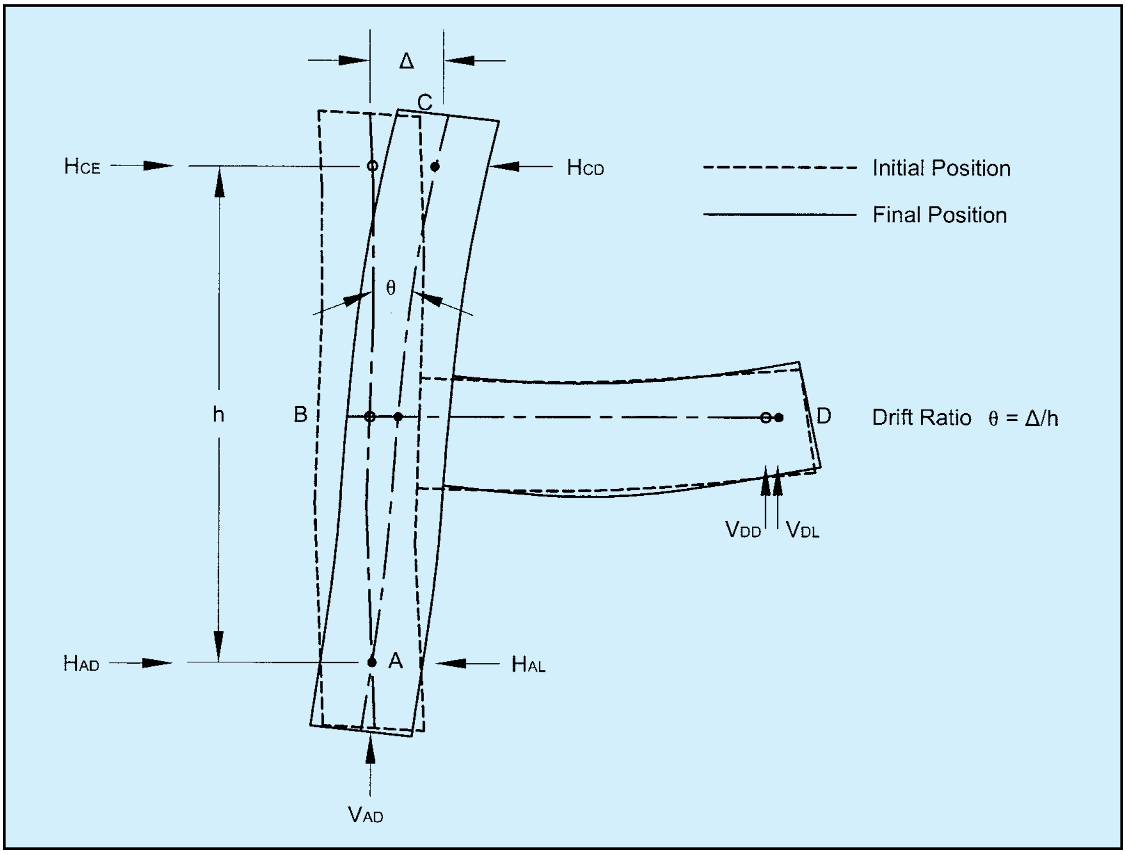



Fig. 6. Definition of drift ratio.[15]

lelograms defined by the initial stiffness for each loading direction during the first loading cycle and the peak resistance for that same direction during the third cycle to a drift ratio of 0.035 (see Fig. 8).

**3.** The secant stiffness between drift limits of -0.0035 and +0.0035 must be not less than 0.05 times the initial stiffness of the module for the first loading cycle (see Fig. 9).

The studies that lead to the specification of a limiting drift ratio of 0.035 are documented in the Commentary to ACI/ITG T1.1-99. Conventional cast-in-place special moment frames conforming to Chapter 21 of ACI 318-99 will have limiting drift ratios approaching 0.035 but they may be unable to achieve that limit on a consistent basis for the range of properties found in practice.[11] Thus, precast/prestressed special moment frames in the 2000 NEHRP Provisions are being held to a standard at least as high as that for cast-in-place special moment frames.

Some strength degradation at high cyclic-drift ratios is inevitable and the degree of degradation that can be expected will increase with increase in the limiting drift ratio. A strength degradation of 25 percent is consistent with analytical and experimental findings for a drift ratio of 0.035. For a given earthquake motion, the maximum drift experienced by a structure increases as its relative energy dissipation ratio decreases.

If the relative energy dissipation ratio is less than one-eighth, oscillations may continue for a considerable time after the earthquake and low cycle fatigue effects can result. If the stiffness is very small around zero drift ratio, the structure is prone to large displacements for small lateral force changes following a major earthquake and is, therefore, vulnerable to low cycle fatigue effects in aftershocks and moderate winds.

For precast/prestressed special moment frames, the 2000 NEHRP Provisions add three additional items to the ACI/ITG T1.1-99 criteria as follows:

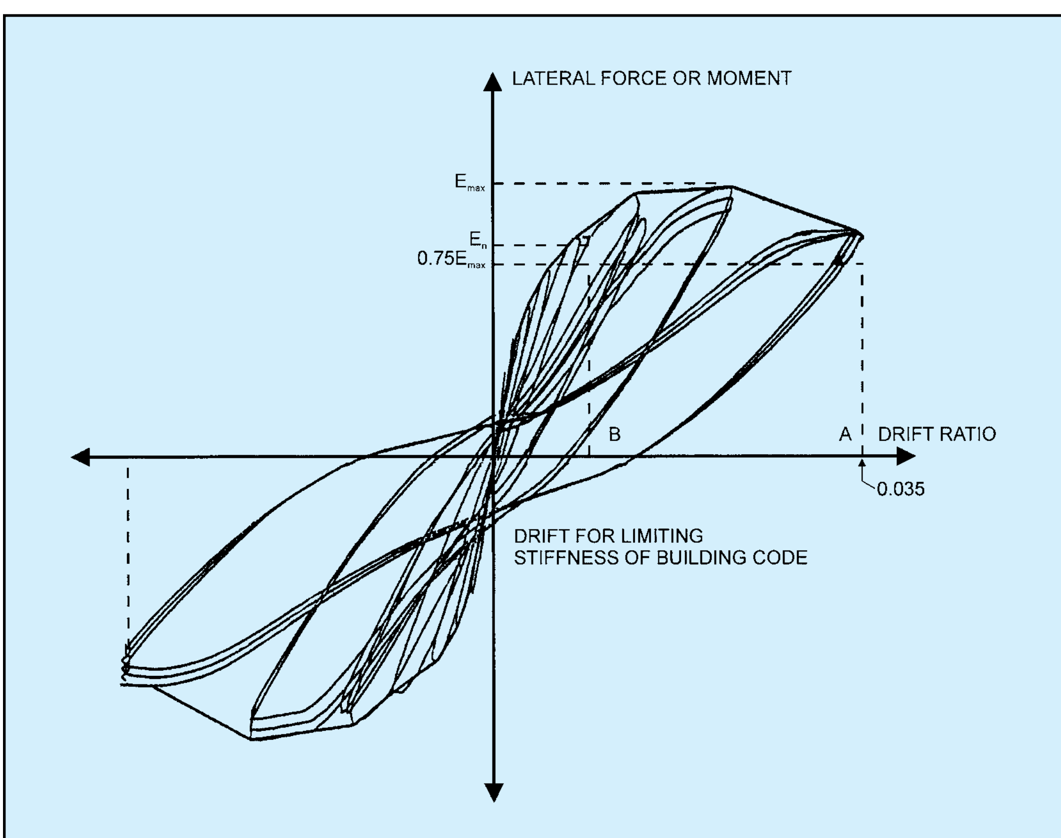
**1.** The test modules must be shown to be able to continue to carry the gravity loads that act on them in the generic frame at a 0.05 drift ratio. This requirement was considered necessary to document that the precast/prestressed frame had a toughness equivalent to that anticipated for a cast-in-place concrete frame.

**2.** Unless there was substantial experimental evidence obtained in a prior development program, the validation tests of ACI/ITG T1.1-99 must be conducted at full scale and be at least two in number for each characteristic configuration of intersecting beams and columns.

While the Commentary to T1.1-99 implies that experimental evidence should be obtained in a prior development program, the Provisional Standard does not require it. Rather, T1.1-99 requires only that, prior to the validation testing, a design procedure should have been developed for the generic frame, and that procedure used to proportion the test modules.

In the NEHRP Provisions, the number of tests required in the prior development program is not specified. However, the results for the several different frame systems studied in the PRESSS (Precast Seismic Structural Systems) program suggest that five or more tests at one-quarter scale or greater should be made in order to provide the range of experimental information needed to develop a math-

Fig. 7. Quantities used in evaluating acceptance criteria.



ematical model sufficiently accurate for purposes of analysis.

**3.** The special moment frame must be designed using the nonlinear response history analysis procedure specified in Section 5.8 of the 2000 NEHRP Provisions, using the force-deformation characteristics for the nonlinear action locations obtained from the module tests.

The 2000 NEHRP Provisions requires designs to be carried out using structural analyses conforming to one of six types. Those types are Index Force Analysis, Equivalent Lateral Force Analysis, Modal Response Analysis, Linear Response History Analysis, Nonlinear Static Analysis, and Nonlinear Response History Analysis.

The type of analysis required depends on the SDC of the structure, its height and irregularity. The typical cast-in-place special moment frame in structures with a limited degree of irregularity and not more than 17 or 18 stories in height can be analyzed with any one of the foregoing six procedures except Index Force Analysis.

By contrast, a precast special moment frame can be designed using Nonlinear Response History Analysis only. That method requires that a mathematical model be used for the structure that directly accounts for the nonlinear hysteretic behavior of the components of the structure. That model is then used to determine the response of the structure, through methods of numerical integration, to suites of ground motion compatible with the design response spectrum for the site of the structure.

Use of Nonlinear Response History Analysis is required for non-emulative precast concrete special moment frames. This is because none of the other four procedures permitted for cast-in-place frames can realistically capture the strength and deformation demands placed on the structure by the range of structural characteristics permitted by T1.1-99.

## SPECIAL STRUCTURAL WALLS

The studies that led to the development of the acceptance criteria of T1.1-99 for special moment frames also catalyzed studies that have resulted in the development of similar acceptance criteria for special structural walls.[16] The validity of those criteria for walls have been demonstrated by the results of tests in the direction of the structural walls of the PRESSS five-story building.[14]

The 2000 NEHRP Provisions require that the substantiating experimental evidence and analysis for special structural wall systems meet requirements similar to those of T1.1-99 for the design procedure used for the test modules, the scale of the modules, the testing agency, the test method and the test report. The minimum test module must be a stack of wall panels at least two stories high.

Based on the work described in Ref. 16, the test module must perform satisfactorily under cyclic loading at a limiting drift ratio that is a function of the characteristics of the wall and is given by the criterion:

$$1.0 \le \Delta / h_w \text{ (percent)} = 0.67\,[h_w / l_w] + 0.5 \le 3.0 \qquad (1)$$

where

$h_w$ = height of entire module

$l_w$ = length of entire module

Criterion 1 was derived after an examination of results from tests on 178 cast-in-place walls with aspect ratios

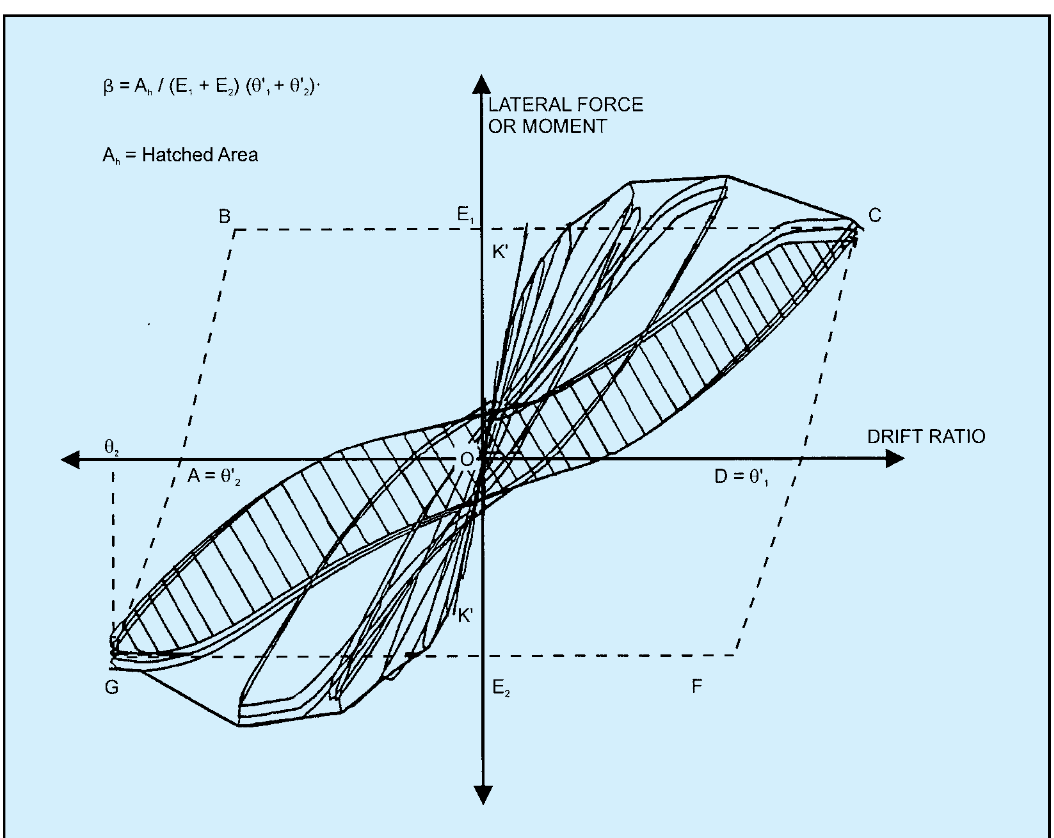



Fig. 8.
Relative energy dissipation ratio.[15]

($h_w/l_w$) between 0.25 and 3.53. Those results clearly showed, as apparent from Fig. 10(a), that the limiting drift ratio at the peak load varied linearly with the aspect ratio and varied between about 0.67 and 3.0 percent. For ductile behavior, some post-peak degradation must be expected, although the acceptable degree should be less for walls than for frames because the limiting drift ratio is less for walls.

Analysis showed the acceptable degree of degradation should be limited to 20 percent for walls as compared to 25 percent for frames. For that condition, Criterion 1 combines the drift predictions for a simple mathematical model of a wall hinging at its base and the use of a maximum displacement ductility factor of eight to assess the limiting drift. The relationship derived between displacement ductility and limiting drift for varying $h_w/l_w$ values is compared to the 0.035 limiting drift for frames in Fig. 10(b).

For the third cycle at a drift ratio equaling or exceeding the value given by Criterion 1, the 2000 NEHRP Provisions requires that the test module exhibit:

**1.** A degradation in post-peak capacity not exceeding 20 percent.

**2.** A relative energy dissipation ratio, defined in the same manner as in ACI ITG/T1.1-99 (see Fig. 8) that equals or exceeds 15 percent.

**3.** A stiffness around zero drift that equals or exceeds that required by the acceptance criteria of ACI ITG/T1.1-99 (see Fig. 9).

The basis for the slightly higher energy dissipation ratio required for walls than for frames is also documented in Ref. 16. In the five-story PRESSS building tests, a pair of vertically coupled precast panel structural wall stacks were used, with each of the stacks having centrally located unbonded post-tensioned tendons.

The results of those PRESSS tests for the shear wall direction and analytical studies of precast post-tensioned walls reported in Refs. 14 and 16 validate the appropriateness of the criteria specified in the 2000 NEHRP Provisions for structural walls constructed from precast panels.

## CONNECTIONS

Dry connections for seismic-force-resisting systems are classified into two types, namely, Type Y and Type Z. At nonlinear action locations, displacements both in the direction of action of the connection, and transverse to it, must be controlled. For example, if a sliding shear connection is to be provided between two precast concrete members, then there must also be a tie between the two members to prevent the sliding surfaces from separating.

Type Y connections must be able to develop, for the flexure, shear, or axial load, or combinations of those quantities expected to act on the connection, a probable strength, $S_{pr}$, determined using a $\phi$ value of unity, that is not less than 125 percent of the yield strength of the connection. In essence, the connection must be able to strain harden.

Under cyclic loading the connection must be able to develop a displacement, at $S_{pr}$, that is at least 4.0 times its displacement at yield. The anchorage of the connection into the precast member on either side of a joint must be designed to develop in tension 1.3

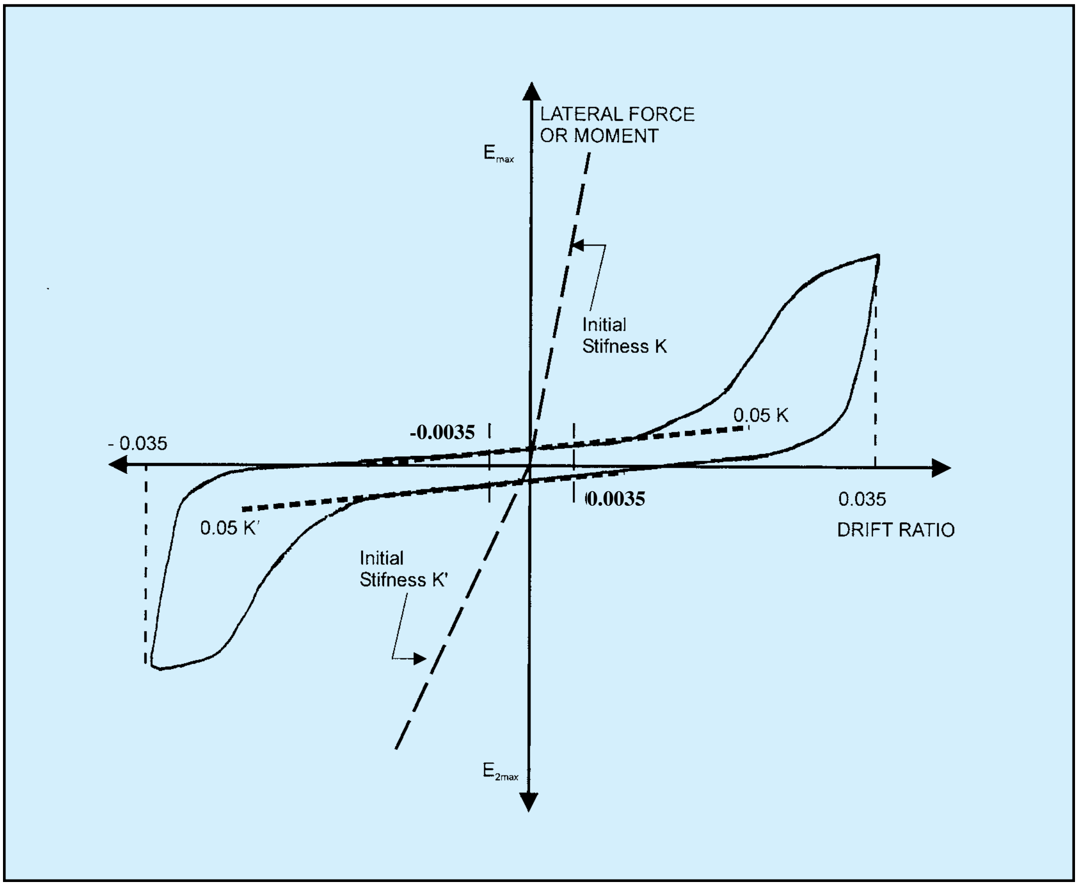



Fig. 9. Unacceptable hysteretic behavior.[15]

times $S_{pr}$, and be connected directly by a Type 2 splice to the principal reinforcement of the precast or cast-in-place element.

For Type Z connections, $S_{pr}$ must be not less than 140 percent of the yield strength of the connection, and under cyclic loading the connection must be able to develop a displacement at $S_{pr}$ that is at least eight times its displacement at yield. The anchorage for the connection must also meet in both tension and compression all the requirements for Type Y connections. Equilibrium based plasticity models (strut-and-tie models), as described in 18.13.5 of ACI 318-99, are to be used for the design of the connection region.

Confinement reinforcement in the form of closed hoops or spirals with a yield force not less than 0.05 times the compressive force and with a spacing not greater than 3 in. (76 mm) must be provided around the anchorage where the local compressive stress at $S_{pr}$ exceeds $0.7 f'_c$ . The connection region is defined in the same manner as "anchorage zone" in Section 2.1 of ACI 318-99.

The testing of connections and the evaluation of results must be made in accordance with the principles of ACI ITG/T1.1-99. Appropriate procedures for testing connections will be described in more detail in the next issue of the PCI JOURNAL in the discussion of the design provisions for untopped diaphragms contained in the Appendix to Chapter 9 of the 2000 NEHRP Provisions. Connections at nonlinear action locations in modules of frames and structural walls used for validation testing are deemed to satisfy the provisions for connections if the results for the test module satisfy the acceptance criteria for frames or structural walls, as appropriate.

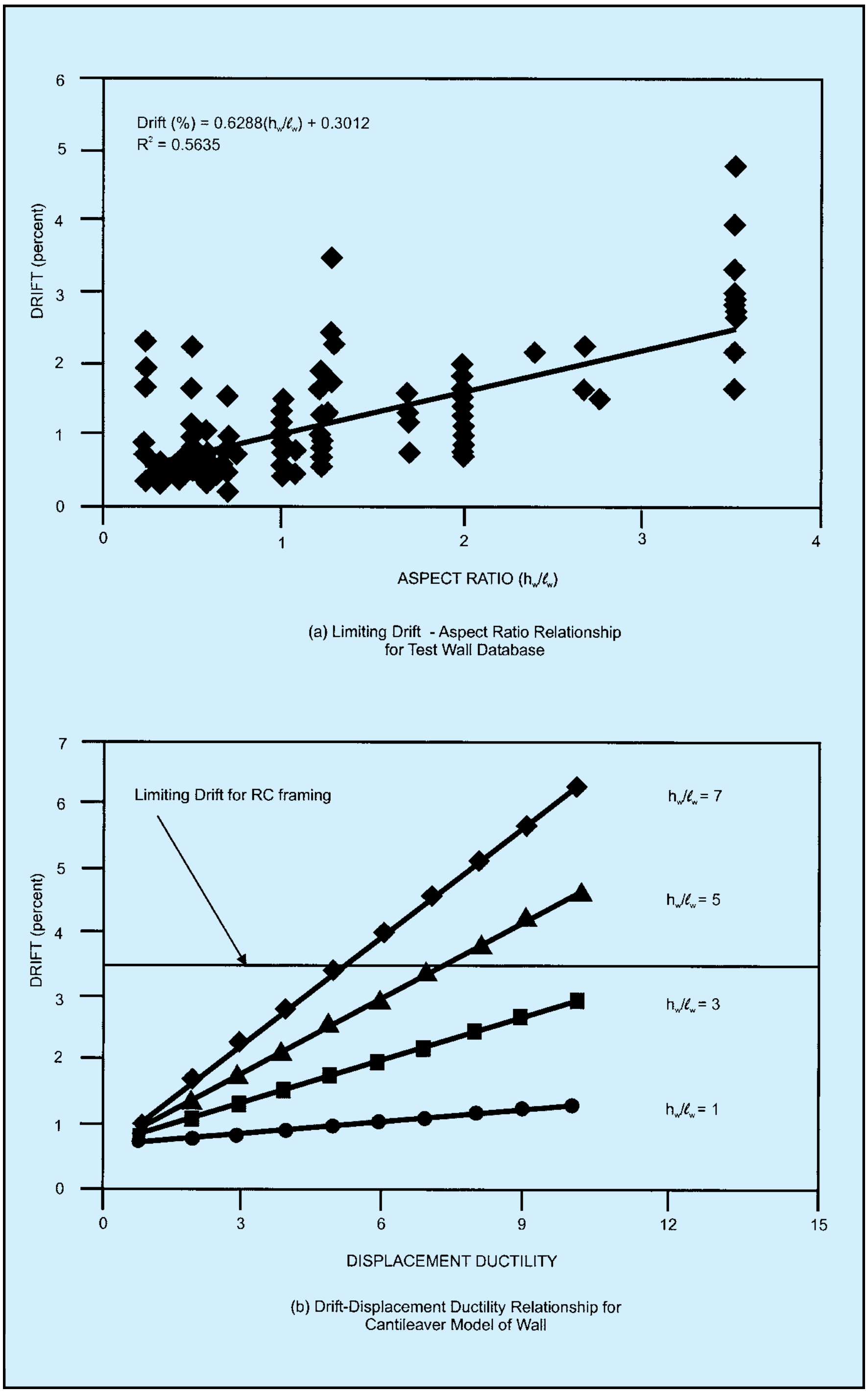



Fig. 10. Basis for Criterion 1 for special structural walls.

## CONCLUDING REMARKS

The 2000 NEHRP Provisions are expected to include expanded seismic design provisions for precast, prestressed concrete lateral-force-resisting systems. Emulative design, where parity with the seismic performance of cast-in-place reinforced concrete structures is sought, and non-emulative design, where the unique properties of precast concrete construction are sought to be taken advantage of, are both permitted to be used. Emulative design procedures are prescribed for special moments frames as well as for special structural walls (suitable for use in Seismic Design Categories D, E and F).

Under the special moment frame provisions, designers may utilize strong connections which remain elastic as inelastic action takes place away from those connections, or they may utilize ductile connections in which seismic energy dissipation is allowed. Only the ductile connection option is available under the special structural wall provisions. Non-emulative design procedures are also prescribed for special moment frames as well as for special structural walls.

## ACKNOWLEDGMENT

The original proposal leading to the reported changes from the 1997 to the 2000 Edition of the NEHRP Provisions was prepared by a PCI Fast Team consisting of Ned Cleland, Thomas D'Arcy, Robert Fleischman, S.K. Ghosh, Neil Hawkins, Phillip Iverson, Michael Oliva and Richard Sause. The contribution of the members of this team is gratefully acknowledged.

## REFERENCES

1. Federal Emergency Management Agency, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, 1997 Edition, FEMA 302, February 1998; and Commentary, FEMA 303, Washington D.C., February 1998.
2. ACI Committee 318, "Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)," American Concrete Institute, Farmington Hills, MI, June 1999.
3. Restrepo, J. I., Park, R., and Buchanan, A. H., "Tests on Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames of Buildings," PCI JOURNAL, V. 40, No. 4, July-August 1995, pp. 44-61.
4. Restrepo, J. I., Park, R., and Buchanan, A. H., "Design of Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames," PCI JOURNAL , V. 40, No. 5, September-October 1995, pp.68-77.
5. Yoshioka, K., and Sekine, M., "Experimental Study of Prefabricated Beam-Column Subassemblages," ACI SP-123, *Design of Beam-Column Joints for Seismic Resistance*, American Concrete Institute, Detroit, MI, 1991, pp. 465-492.
6. Kurose, Y., Nagami, K., and Saito, Y., " Beam-Column Joints in Precast Concrete Construction in Japan," ACI SP-123, *Design of Beam-Column Joints for Seismic Resistance*, American Concrete Institute, Detroit, MI, 1991, pp. 493-514.
7. Yee, A. A., "Design Considerations for Precast Prestressed Concrete Building Structures in Seismic Areas," PCI JOURNAL, V. 36, No. 3, May-June 1991, pp. 40-55.
8. Warnes, C. E., "Precast Concrete Connection Details for All Seismic Zones," *Concrete International*, V. 14, No. 11, November 1992, pp. 36-44.
9. Cheok, G. S., and Lew, H. S., "Performance of Precast Concrete Beam-to-Column Connections Subject to Cyclic Loading," PCI JOURNAL, V. 36, No.4, July-August 1991, pp. 56-67.
10. Cheok, G. S., Stone, W. C., and Kunnath, S. K., "Seismic Response of Precast Concrete Frames with Hybrid Connections, *ACI Structural Journal*, V. 95, No. 5, May 1998, pp. 527-539.
11. Stanton, J. F., Stone, W. C., and Cheok, G. S., "A Hybrid Reinforced Precast Frame for Seismic Regions," PCI JOURNAL, V. 42, No. 2, March-April 1997, pp. 20-32.
12. Priestley, M. J. N., "Overview of PRESSS Research Program," PCI JOURNAL, V. 36, No. 4, July-August 1991, pp. 50-57.
13. Priestley, M. J. N., "The PRESSS Program — Current Status and Proposed Plans for Phase III," PCI JOURNAL, V. 41, No. 2, March-April 1996, pp. 22-40.
14. Priestley, M. J. N., Sritharan, S., Conley, J. R., and Pampanin, S., "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building," PCI JOURNAL, V. 44, No. 6, November-December 1999, pp. 42-67.
15. ACI Innovation Task Group 1 and Collaborators, "Acceptance Criteria for Moment Frames Based on Structural Testing," American Concrete Institute, Farmington Hills, MI, 1999.
16. Seo, S-Y., Lee, L-H., and Hawkins, N. M., "The Limiting Drift and Energy Dissipation Ratio for Shear Walls Based on Structural Testing, *Journal of the Korean Concrete Institute*, V. 10, No. 6, December 1998, pp. 273-311.