Precast Structures in Regions of High Seismicity: 1997 UBC Design Provisions

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The 1997 edition of the Uniform Building Code, for the first time, contains design provisions for precast concrete structures located in regions of high seismicity (Seismic Zones 3 and 4). This paper provides background and discussion of the new code provisions, along with a design example illustrating the provisions.

Until recently, precast concrete structures could only be built in regions of high seismicity such as Uniform Building Code (UBC) Seismic Zones 3 and 4[1] under the 1994 (and prior) UBC provision, adopted from the ACI 318 standard,[2] that would allow them "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..." [UBC 1994, Section 1921.2.1.5]. The interpretation, implementation and enforcement of this vague, qualitative requirement has, for obvious reasons, been nonuniform. The need for specific design requirements for precast structures in regions of high seismicity has been apparent for quite some time.

The very first set of specific design provisions ever developed in this country for precast concrete structures in regions of high seismicity appears in the 1994 edition of the National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions,[3] issued by the Building Seismic Safety Council (BSSC). The provisions were developed by the Concrete Subcommittee of the BSSC's 1994 Provisions Update Committee, chaired by S. K. Ghosh, with Professor Neil M. Hawkins playing a major role in the development.

Out of a desire to develop design provisions with the same scope for the Uniform Building Code, the Seismology

Committee of the Structural Engineers Association of California (SEAOC) in 1993 formed an Ad Hoc Committee on Precast Concrete, chaired by Vilas Mujumdar. From the beginning, the Ad Hoc Committee had the full participation of the concrete industry, as represented by the Precast/Prestressed Concrete Manufacturers Association of California (PCMAC) and the Portland Cement Association (PCA).

As a result of considerable efforts and deliberations stretching over nearly 2 years, the Ad Hoc Committee developed a code change that was submitted to the International Conference of Building Officials (ICBO), the issuers of the UBC, by the Division of the State Architect (DSA) of California and PCA. The code change was approved and became part of the 1997 edition of the Uniform Building Code, which was published in April 1997.

This paper discusses and provides background to the 1997 UBC provisions concerning the design of precast concrete structures in Seismic Zones 3 and 4 and illustrates the requirements with a design example. The UBC provisions themselves are reproduced, with permission from ICBO, in the Appendix to this paper.

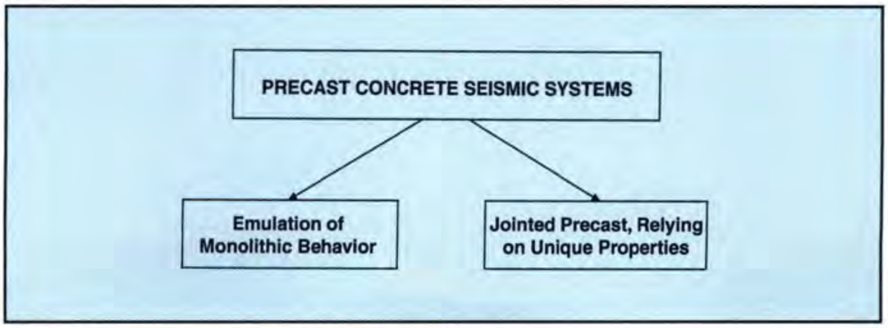


Fig. 1. Precast concrete seismic systems — Options.

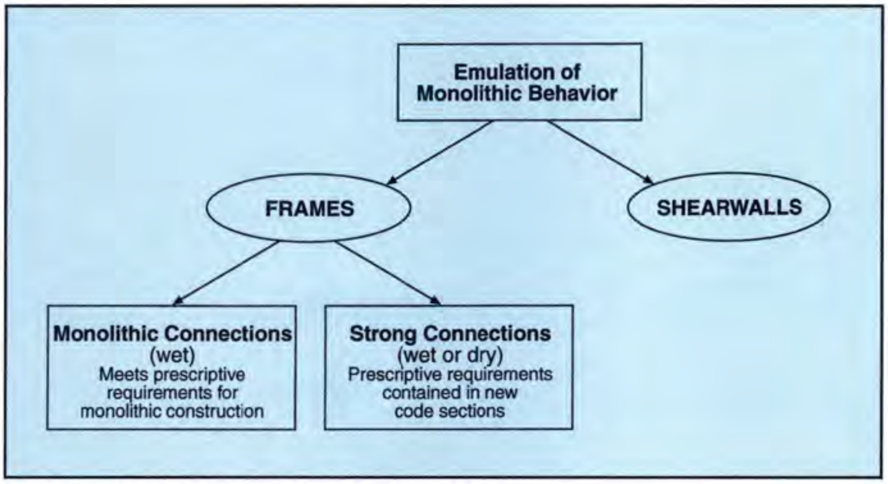


Fig. 2. Emulation of monolithic behavior — Options.

PRECAST CONCRETE LATERAL FORCE RESISTING SYSTEMS

Design Alternatives

The 1994 NEHRP Provisions present two alternatives for the design of precast concrete lateral force resisting systems (see Fig. 1). One choice is emulation of monolithic reinforced concrete construction. The other alternative is the use of the unique properties of precast concrete elements interconnected predominantly by dry connections (jointed precast). A "wet" connection uses any of the splicing methods per ACI 318-95, Section 21.2.6.1 or 21.3.2.3, to connect precast or precast and cast-in-place members, and uses cast-in-place concrete or grout to fill the splicing closure. A "dry" connection is a connection between precast or precast and cast-in-place members that does not qualify as a wet connection.

Design procedures for the second alternative (jointed precast) are included in an appendix to the chapter on concrete in the 1994 NEHRP Provisions. These procedures are intended for information and trial design only because the existing state of knowledge makes it premature to propose codifiable provisions based on information available so far.

The Ad Hoc Committee on Precast Concrete of the SEAOC Seismology Committee used the 1994 NEHRP requirements for precast concrete lateral force resisting systems as a starting point. However, the committee decided to limit their scope to frames only (excluding panel systems) and to the monolithic emulation option only, primarily due to time constraints. Jointed precast concrete is allowed only under the "undefined structural systems" provision of UBC-97, Section 1629.9.2 (1921.2.1.6*). The intent of this restriction is not to limit the development of alternative systems but to continue to rely on experimental evidence and analysis, which provide that a unique system will satisfy the intent of the code.

For emulation of the behavior of monolithic reinforced concrete construction, two alternatives are provided (see Section 1921.2.2.5 and Fig. 2): structural systems with "wet" connections (Section 1921.2.2.6) and those with "strong" connections (Section 1921.2.2.7). The different connection categories envisioned are shown in Fig. 3.

Precast structural systems with wet connections must comply with all requirements applicable to monolithic reinforced concrete construction resisting seismic forces (Section 1921.2.2.6). Prescriptive requirements are given for precast frame systems with strong connections (Section 1921.2.7). Such requirements for precast wall systems with strong connections will be developed in the future.

The 1994 NEHRP Provisions also addressed emulation of monolithic construction using ductile connections, covering both frame and panel systems, where the connections have adequate nonlinear response characteristics and it is not necessary to en-

* All section numbers refer to the 1997 edition of the Uniform Building Code, unless otherwise specified.

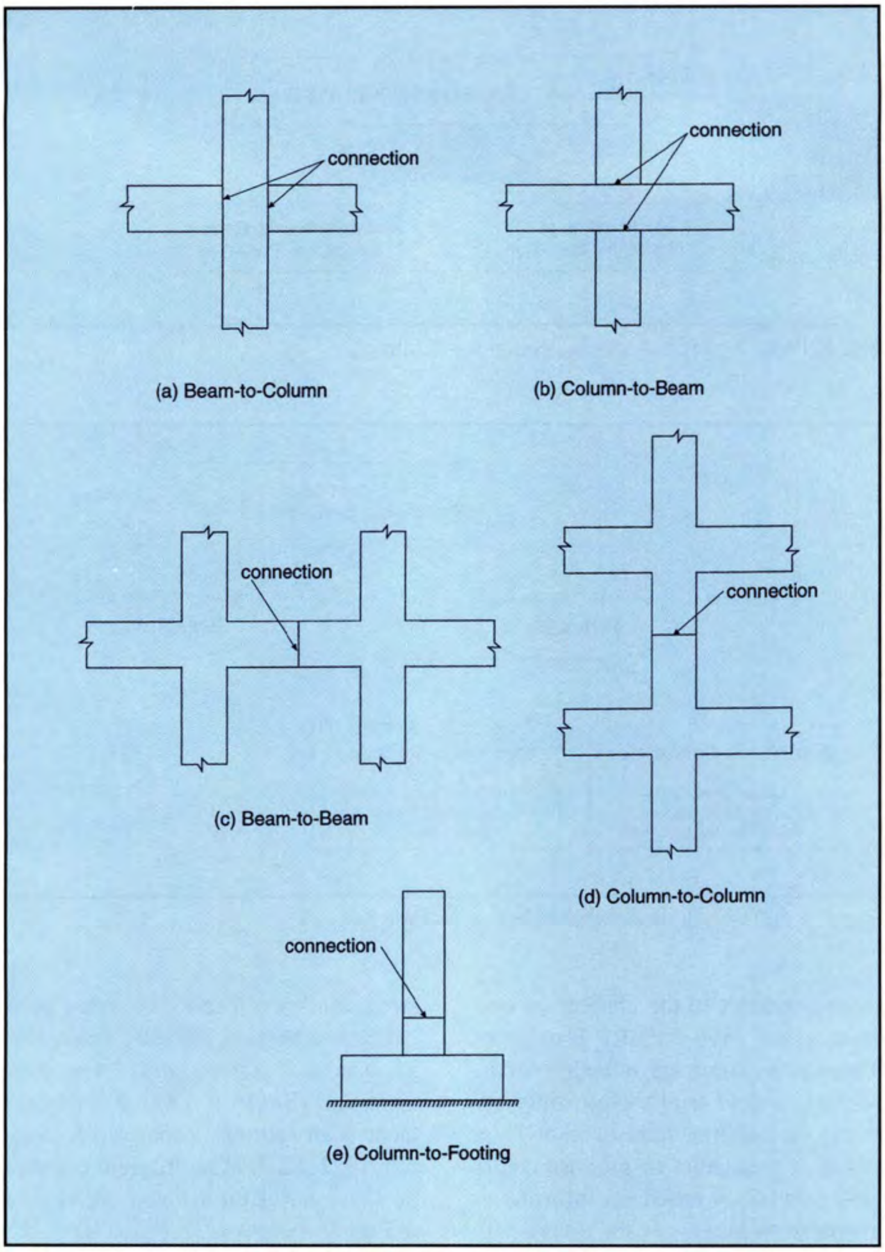


Fig. 3. Typical connection configurations.

sure plastic hinges remote from the connections. Usually, experimental verification is required to ensure that a connection has the necessary nonlinear response characteristics. The designer is required to consider the likely deformations of any proposed precast structure, vis-a-vis those of the same structure in monolithic reinforced concrete, before claiming that the precast form emulates monolithic construction. The 1997 UBC does not directly address ductile connections.

Monolithic Emulation Using Strong Connections

"Joint" is the geometric volume common to intersecting members (Section 1921.1). "Connection" is defined as an element that joins two precast members or a precast member and a cast-in-place member (see Section 1921.1 and Fig. 4). A "strong" connection is a connection that remains elastic while the designated nonlinear action regions undergo inelastic response under the "Design Basis Ground Motion" (Section 1921.1). The Design Basis Ground Motion of UBC-97 has a 90 percent probability of non-exceedance in 50 years.

For frame systems that use strong connections, considerable freedom is given to locating the nonlinear action zones (plastic hinges) along the length of a precast member. However, the nonlinear action location or the center of the region of yielding in flexure must be separated from the connection by a dis-

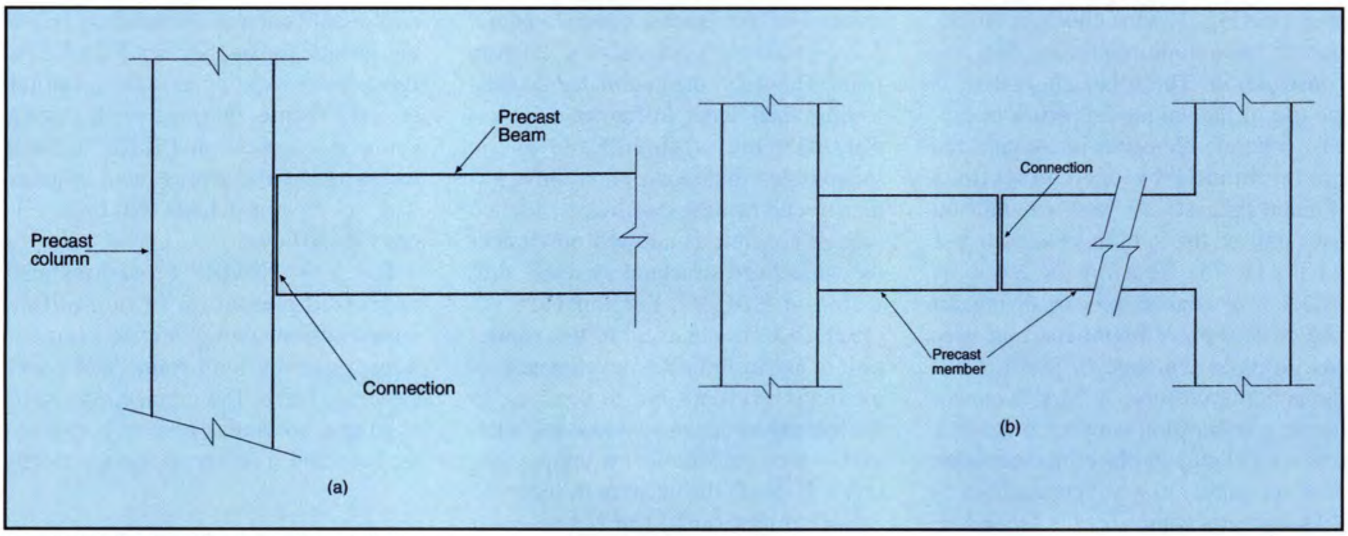


Fig. 4. Illustration of connection definition.

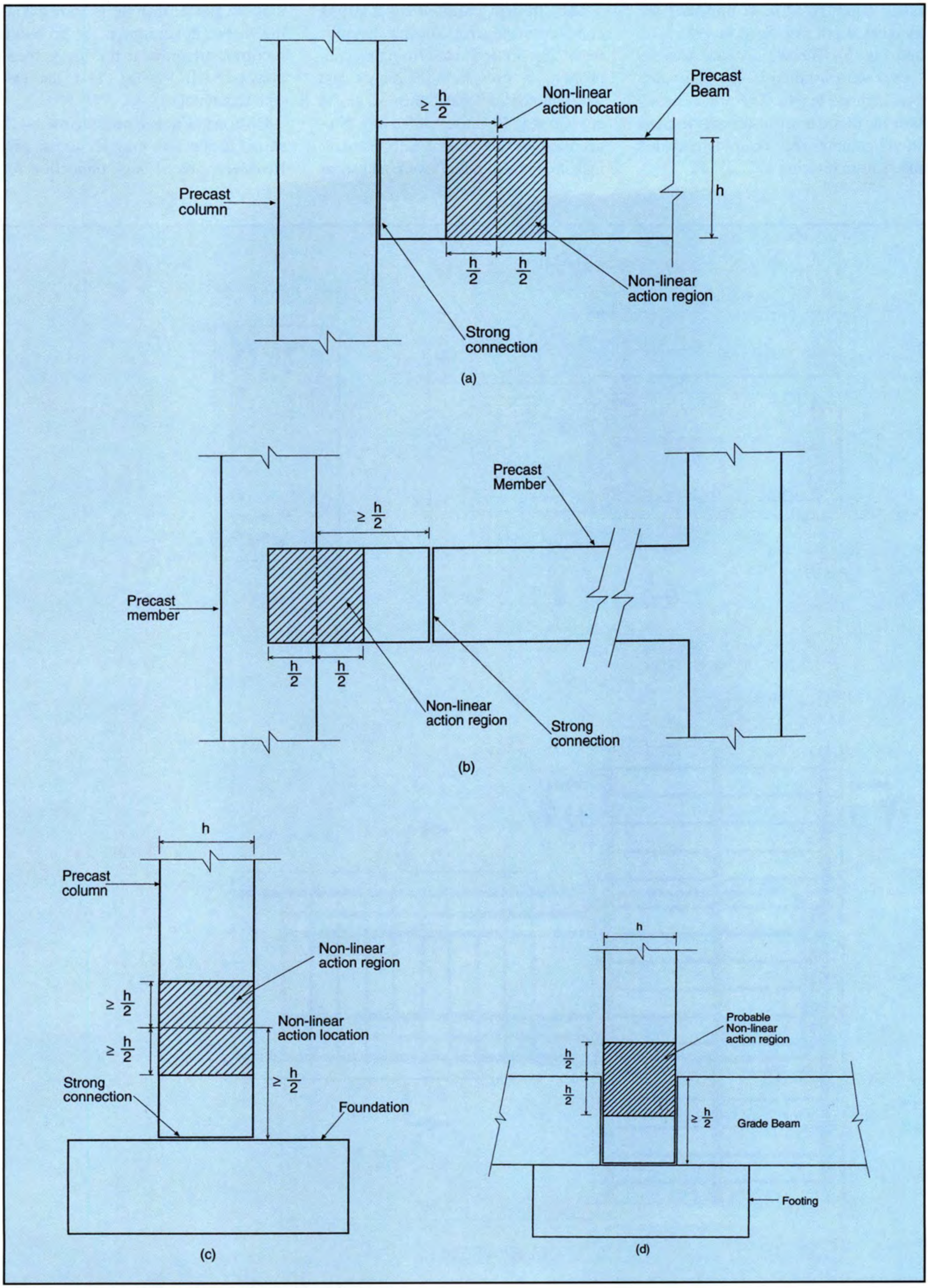


Fig. 5. Nonlinear action region and location — Examples.

tance equal to at least one-half the member depth (see Section 1921.2.7.1 and Fig. 5). The $h/2$ distance was selected with h taken as the estimated plastic hinge length. The nonlinear action locations must also provide for a strong column-weak beam deformation mechanism (Section 1921.2.7.1).

The strength required for a strong connection depends on the distance the hinges are separated from that connection, the strength of the hinges, and the nonlinear deformation mechanism envisioned. Because the strong connection must not yield or slip, its nominal strengths in both flexure and shear must be greater than those corresponding to the development of probable flexural strengths at the hinge locations [see UBC-97 Eq. (21-1) and Design Example].

Column-Face Connection — A strong connection may be wet or dry. However, any strong connection lo-

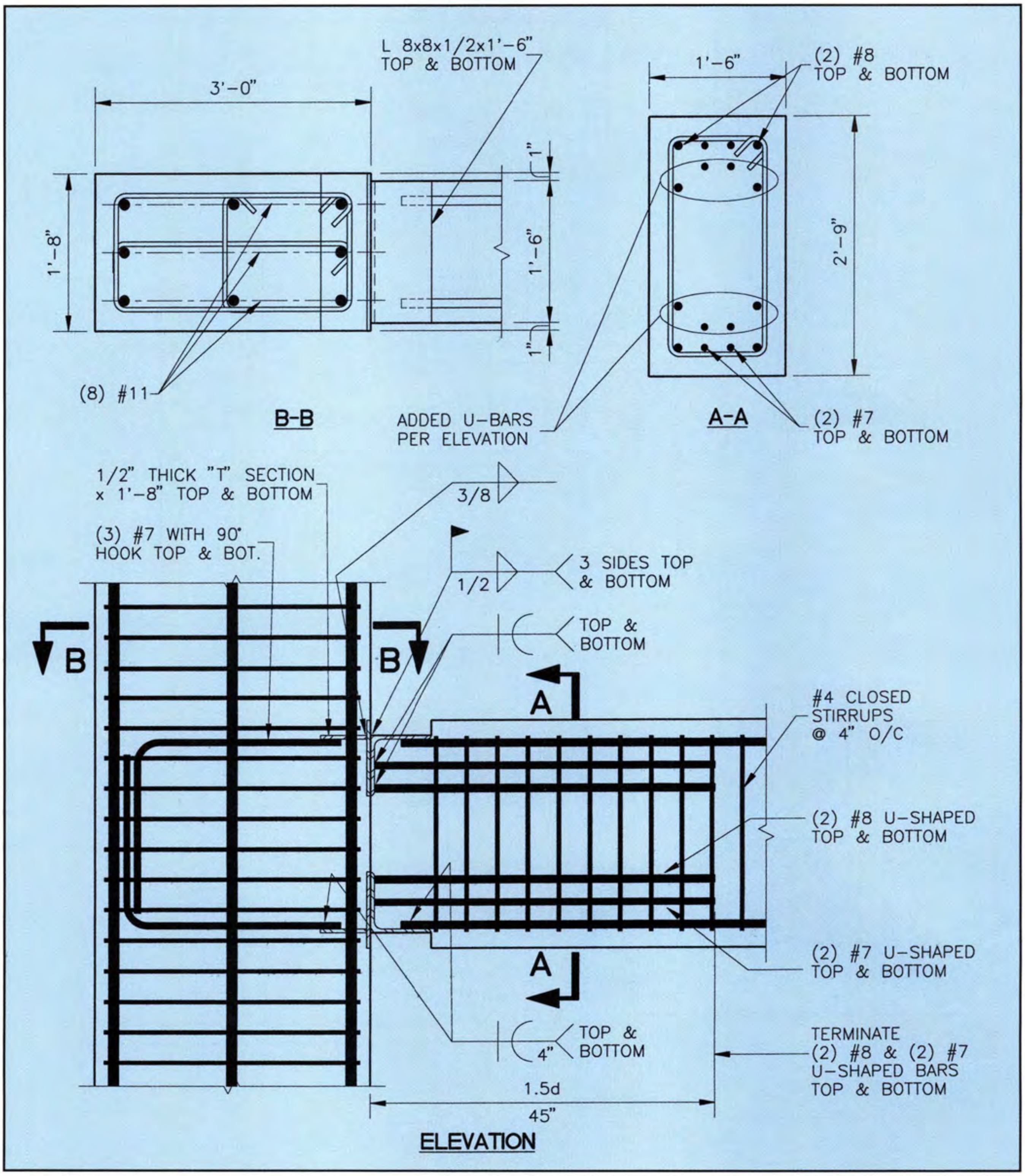


Fig. 6. Column-face dry connection (Ref. 4).

cated outside the middle half of a beam span is required to be a wet connection "unless a dry connection can be substantiated by approved cyclic test results" (Section 1921.2.7.5). Any mechanical connector located within such a column-face strong connection is required to develop in tension or compression, as required, at least 140 percent of the specified yield strength of the spliced reinforcing bars (Section 1921.2.7.5).

While column-face dry connnections have been shown to be able to develop a relocated nonlinear action region (Ochs and Ehsani;[4] French et al.[5]), the first restriction was developed to allay concern over these types of connections. The second restriction recognizes that longitudinal reinforcing bars in a column-face wet connection are likely to develop stresses significantly higher than their specified yield strengths, rendering the mechanical splices that are capable of developing only 125 percent of the specified yield strength of the spliced reinforcing bars inadequate.

The dry column-face connection[4] shown in Fig. 6 would not be allowed under Section 1921.2.7.5, unless the test results it is based upon are judged to be satisfactory by the building official, who may then approve the connection. The beam-to-beam connection shown in Fig. 7, on the other hand, could be dry.

Column-to-Column Connections — For columns above the ground floor, moments at a joint may be limited by the flexural strengths of the beams framing into that joint. However, for a strong column-weak beam deformation mechanism, dynamic inelastic analysis and studies of strong motion measurements have shown that beam end moments are not equally divided between top and bottom columns, even where those columns have equal stiffness.

Elastic analysis predicts moments as shown in Fig. 8a, while the actual situation is likely to be that shown in Fig. 8b. Accordingly, a dynamic amplification factor ψ of 1.4 is used for column-to-column connection design (see Section 1921.2.7.4 and Fig. 9). In addition, special transverse reinforcement (i.e., transverse reinforcement conforming to Sections 1921.4.4.1 through 1921.4.4.3), which is typically required only over the regions of potential plastic hinging at column ends, is required to be provided over the full column height if the axial compressive force including seismic effects exceeds $A_g f_c'/10$ (Section 1921.2.7.4).

The requirements of Section 1921.2.7.4 do not apply to column-to-column connections located within the middle third of the clear column height. However, two other restrictions then apply: (1) The design moment strength φM_n of the connection shall not be less than 0.4 times the maximum M_{pr} for the column within the story height (see Fig. 9); (2) The design shear strength φV_n of the connection shall not be less than the shear corresponding to moments equal to probable flexural

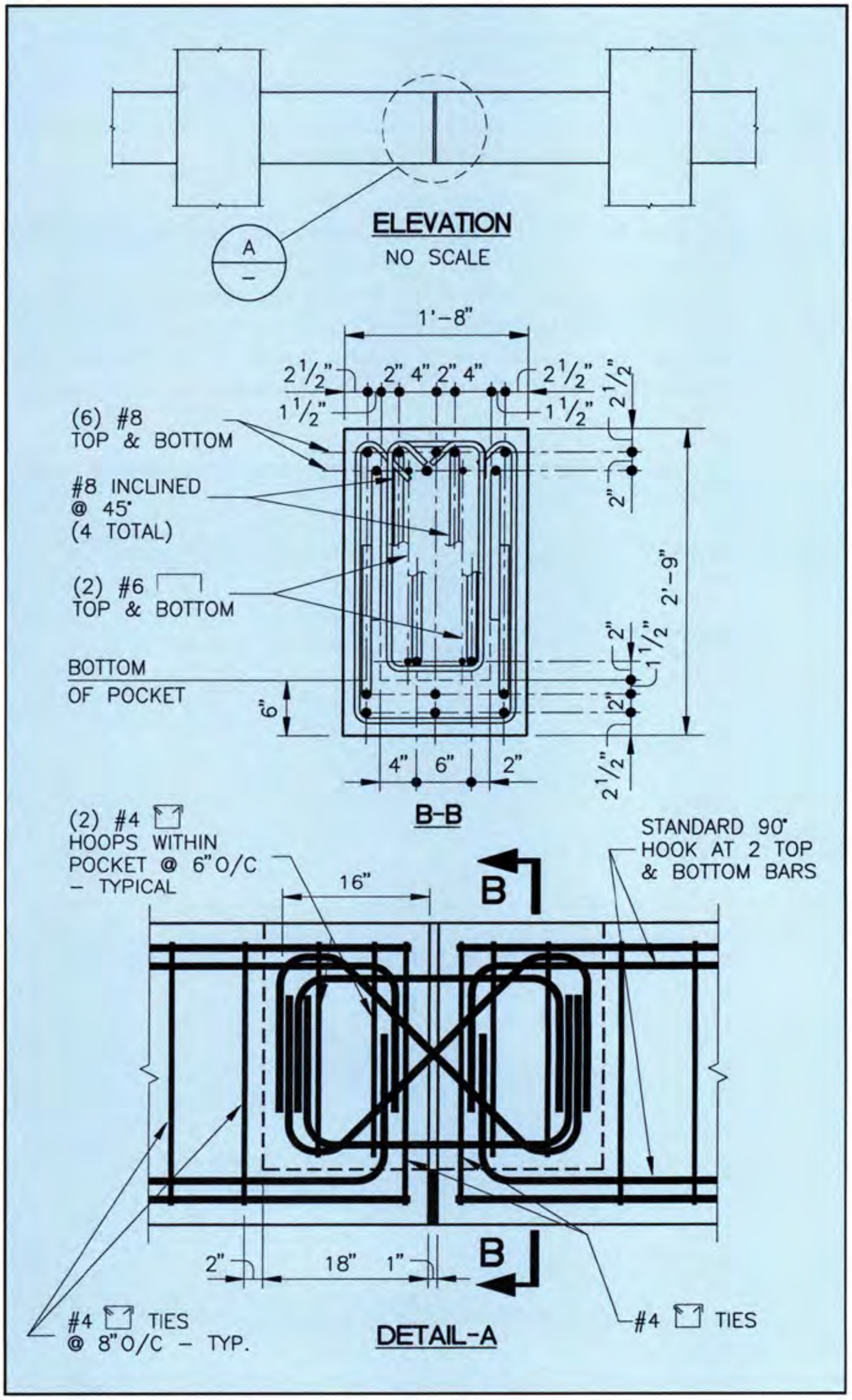


Fig. 7. Beam-to-beam wet connection near midspan.

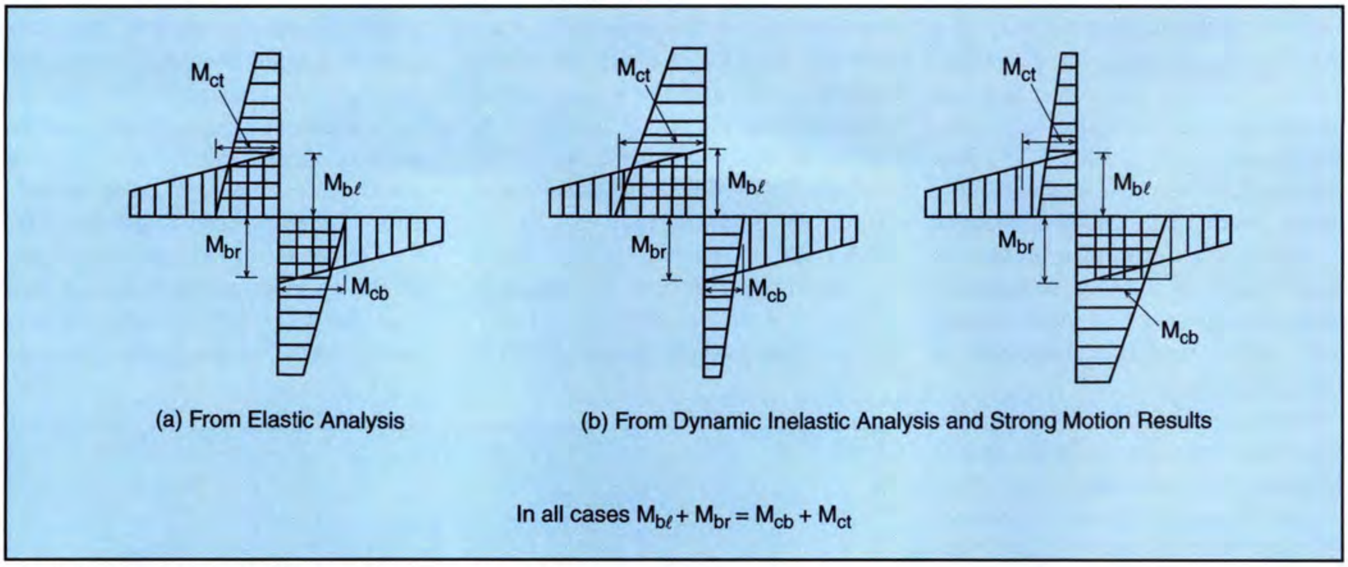


Fig. 8. Bending moments at beam-to-column connections.

strengths acting at the two ends of the column, unless the beams framing in are not strong enough to develop M_{pr} at a column end. In the latter case, the largest moment that can develop at that column end shall be used in computing shear (Section 1921.4.5.1).

Anchorage and Splices — Reinforcement in the nonlinear action (plastic hinge) regions must be fully developed outside both the strong connection region and the nonlinear action region (see Section 1921.2.7.2 and Fig. 10). Noncontinuous anchorage reinforcement of a strong connection must be fully developed between the connection and the beginning of nonlinear action region (Section 1921.2.7.2). Because the plastic hinge region may partially project into the strong connection area during a severe seismic event, the above provisions ensure that the integrity of both regions will be preserved. For example, if a wet strong connection is at the column face, the continuous reinforcement should continue past the strong connection and, if necessary, be terminated in the column.

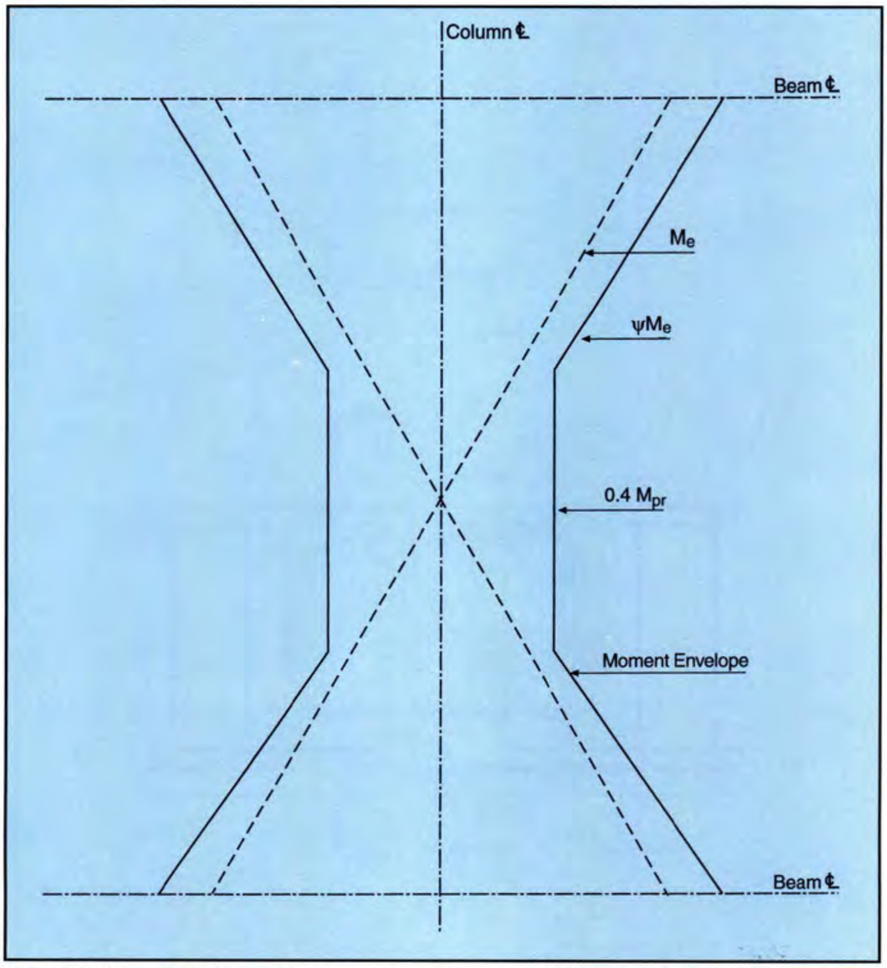


Fig. 9. Column-to-column connection — Moment envelope.

PRECAST GRAVITY SYSTEMS

When the gravity system (consisting of structural members that are not part of the lateral force resisting system) of a structure is precast rather than cast-in-place, overall structural performance equivalent or at least similar to that provided by monolithic gravity systems is sought to be ensured by two alternative provisions (Section 1921.2.1.7). The first seeks to make up for backup frame action, which is expected from monolithic gravity frames, by increasing the redundancy of the lateral force resisting system, specifically by requiring well distributed lateral force resisting elements.

A limitation on the aspect ratios of horizontal diaphragms (span-to-depth

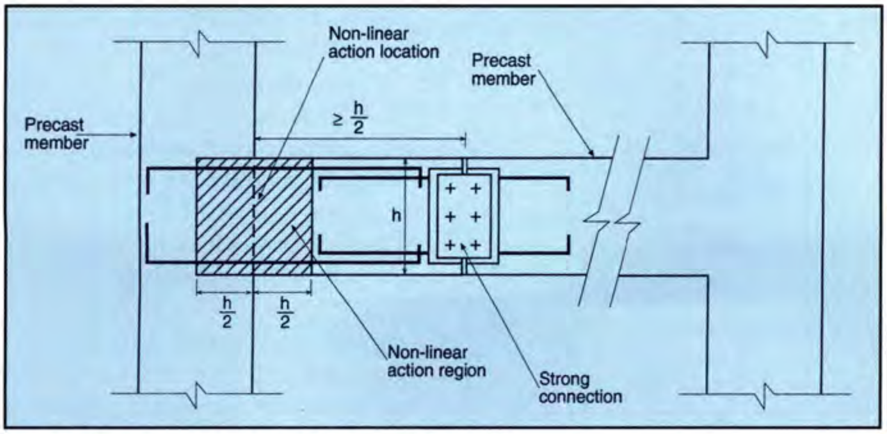


Fig. 10. Strong connection anchorage — Example.

by Δ_S, assuming fixity at the connection and a beam flexural stiffness of no more than one-half of the gross section stiffness. The displacement Δ_S is defined as the Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces (see Fig. 11). The moment M is required to be sustained through a deformation $\Delta_M = 0.7R\Delta_S$, where R is the numerical coefficient representing the inherent overstrength and global ductility capacity of lateral force resisting systems.

ratio of diaphragm or diaphragm segment ≤ 3) is imposed, thus requiring multiple lines of resistance as well as minimizing excessive diaphragm deformations under the Design Basis Ground Motion. In addition, along each line of resistance, if moment frames are utilized to carry the seismic forces, at least $[(N_b/4) + 1]$ of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the lateral force resisting system, where N_b is the total number of bays along that line at that story (see Table 1). This requirement applies to only the lower two-thirds of the stories of buildings three stories or more in height. Note that these provisions were developed independently of the redundancy provisions of Chapter 16, but with the same objectives.

Although using interior lines of resistance is very desirable, a second alternative may be considered when the diaphragm aspect ratio limitations cannot be economically met. In this case, steps are taken to ensure that the integrity of precast lateral force resisting frames or shear walls combined with precast gravity frames is maintained during severe seismic excitations.

All precast gravity frame beam-to-column connections are required to be "partially restrained." Each such connection is required to be designed to develop a strength M, where M is the moment developed at the connection locations when the frame is displaced

Table 1. Redundancy provision for precast lateral force resisting systems.

Total number of bays in frame at particular floor level	Number of bays that must be part of lateral force resisting system
2	2
3	2
4	2
5	3
6	3
7	3
8	3
9	4
10	4
11	4
12	4
13	5
14	5
15	5
16	5
17	6
18	6
19	6
20	6

The connection is permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member resists moment with the same positive or negative sign. The connection is permitted to have zero flexural stiffness up to a frame displacement of Δ_S (see Fig. 12). Because the dominance of gravity loads decreases the possibility of moment reversal due to seismic action, the slab anchorage reinforcement may be used to resist the uni-directional moment M. The mechanical properties of the anchorage reinforcement need to be carefully reviewed (see Fig.13) to ensure that the elongation corresponding to Δ_M can be safely sustained without brittle failure.

In addition to the above requirements, complete calculations for the deformation compatibility of the gravity load carrying system must be made

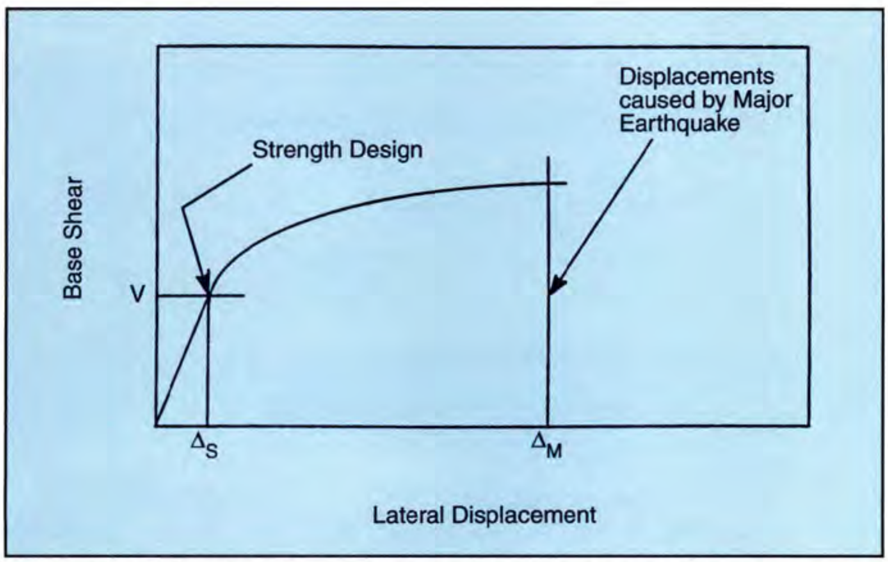


Fig. 11. Idealized load-displacement relationship of structure subject to seismic excitation.

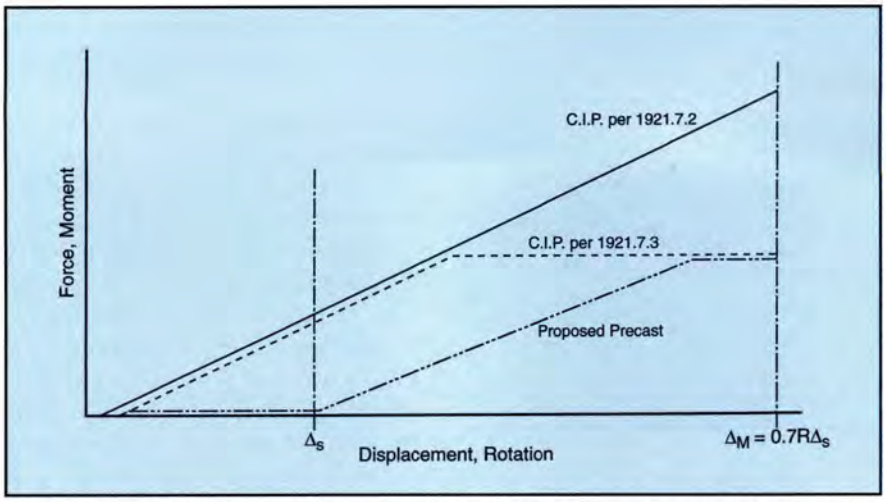


Fig. 12. Force-displacement relationship of frame members not a part of the lateral force resisting system.

in accordance with Section 1633.2.4, using cracked section stiffnesses in the lateral force resisting system and the diaphragm.

For gravity columns that are not laterally supported on all sides, such as those located at floor openings or at exterior slab boundaries, a positive connection along each unsupported direction parallel to a principal plan axis of the structure is required. The connection is required to be designed for a horizontal force equal to 4 percent of the axial load strength P_O of the column. This is to prevent any inadvertent increase in unsupported column height during strong ground motion. The above horizontal force will stabilize a column subject to a 4 percent story drift. The required detail provides a load path for the additional shears generated by a 4 percent story drift.

Bearing length is required to be 2 in. (51 mm) more than that needed for bearing strength. This provision is to prevent slippage of the horizontal members off their supports during a severe seismic event. This increase was based on an average girder depth of 50 in. (1270 mm), when the member is presumably displaced through a 4 percent angular rotation, as the frames drift laterally.

OTHER RELATED UBC PROVISIONS

Prestressed Members of Lateral Force Resisting System

Section 1921.2.5 permits prestressing tendons to be used to resist earthquake-induced flexural and axial forces in frame members, provided:

1. The average prestress f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension, does not exceed the lesser of 350 psi (2.41 MPa) or $f_c'/12$ at locations of nonlinear action (Section 1921.2.5.3). This requirement also originated with the NEHRP Provisions, except that in 1994 NEHRP, f_{pc} is required not to exceed the greater of 350 psi (2.41 MPa) or $f_c'/12$. At the time of this writing, it appears almost certain that the 1997 NEHRP Provisions will require f_{pc} not to exceed the lesser of 700 psi (4.82 MPa) or $f_c'/6$. The 1997 NEHRP is also likely to be adopted into the first (2000) edition of the International Building Code (IBC).

2. For members in which prestressing tendons are used together with mild steel reinforcement to resist earthquake-induced forces, prestressing tendons: (1) do not provide more than one-quarter of the strength for both positive and negative moments at the joint face; (2) extend through exterior joints; and (3) are anchored at the exterior face of the joint or beyond (Section 1921.2.5.4).

3. Shear strength provided by prestressing tendons is not considered in design (Section 1921.2.5.5).

There have been recent developments of alternative precast, prestressed frame systems with jointed connections (Priestley;[6] NIST[7]). However, these systems are not ad-

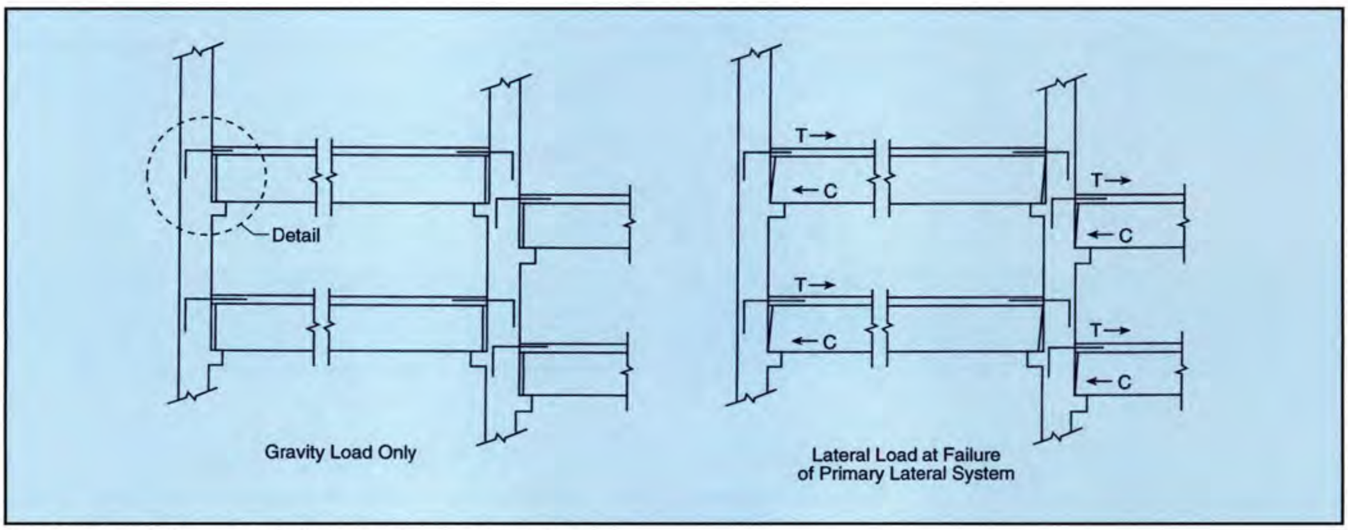


Fig. 13a. "Partially restrained" gravity frame connections.

dressed in the 1997 UBC other than through Section 1921.2.1.6 (2). The prestressing limitations in Sections 1921.2.5.3 and 4 refer to systems in which the prestressing steel in the frame is not the primary reinforcement, but is there as a result of the floor system.

Welded and Mechanical Splices

The 1994 UBC specified that reinforcement resisting earthquake-induced flexural and axial forces in frame members or in wall boundary elements were permitted to be spliced using welded splices or mechanical connectors conforming to Section 1912.4.3.3 (a full-welded splice shall develop at least 125 percent of specified yield strength, f_y, of the bar) or Section 1912.14.3.4 (a full mechanical connection shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar) "provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the center-to-center distance between splices of adjacent bars is 24 in. (610 mm) or more measured along the longitudinal axis of the member" (Section 1921.2.6.1). The staggering requirement, the entire text within quotes in the above sentence, has been dropped from 1997 UBC. However, other requirements have been added, as indicated below.

Welded splices or mechanical splices that just meet the requirements of Sections 1912.4.3.4 and 1912.4.3.5 (which are now defined as Type 1 splices) are not allowed on billet steel A615 or low-alloy A706 reinforcement within an anticipated plastic hinge region (same as the nonlinear action region), within a distance of one beam depth on either side of the plastic hinge region or within a joint, in Seismic Zones 2, 3 and 4. In those locations, in regions of moderate to high seismicity, only Type 2 splices are allowed. A Type 2 splice is a mechanical connection that develops in tension the lesser of 95 percent of the ultimate tensile strength or 160 percent of the specified yield strength f_y of the bar.

Section 1921.2.5.2 of the 1997 UBC requires that for billet steel A615 Grades 40 and 60 reinforcement, the actual yield strength based on mill tests shall not exceed the specified yield strength by more than 18,000 psi (124 MPa), and that retests shall not exceed this value by more than an additional 3000 psi (20.7 MPa).

A similar requirement is already part of the ASTM specifications for low-alloy A706 reinforcement. However, even with these safeguards in place, steel that meets the above requirements can have an actual yield strength as much as 135 percent of the specified value. Thus, a splice that is capable of developing just 125 percent of the specified yield strength of the spliced reinforcing bars may not be sufficient to ensure yielding of the reinforcement prior to splice failure. This is what prompted the definition of the new Type 2 splice and the requirement that only these splices be allowed in regions of potential plastic hinging.

Unfortunately, the splice code change could not be finalized until the

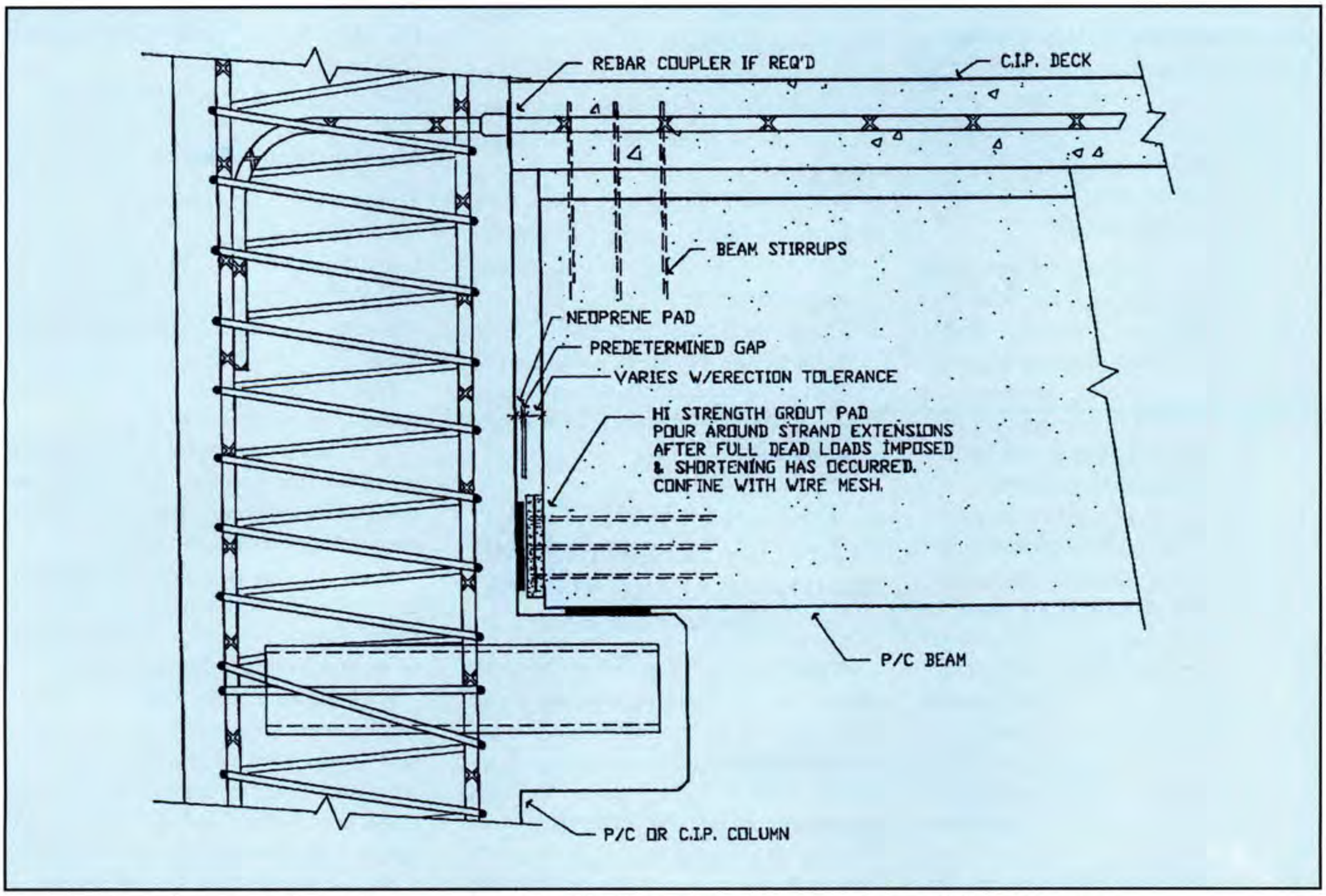


Fig. 13b. "Partially restrained" gravity frame connection detail (courtesy Culp & Tanner and PCL Construction).

last moment and, as a result, could not be properly integrated with the precast concrete code change. As indicated above, Section 1921.2.7.5 requires that any mechanical connector located within a column-face strong connection develop in tension or compression, as required, at least 140 percent of the specified yield strength of the spliced reinforcing bars. This requirement, in effect, creates a third class of splices, which is clearly unwarranted. The Portland Cement Association has initiated measures that will result in the 2000 edition of the IBC requiring mechanical connectors located within column-face strong connections to be Type 2 splices. The staggering of those mechanical connectors, then, will clearly not be required.

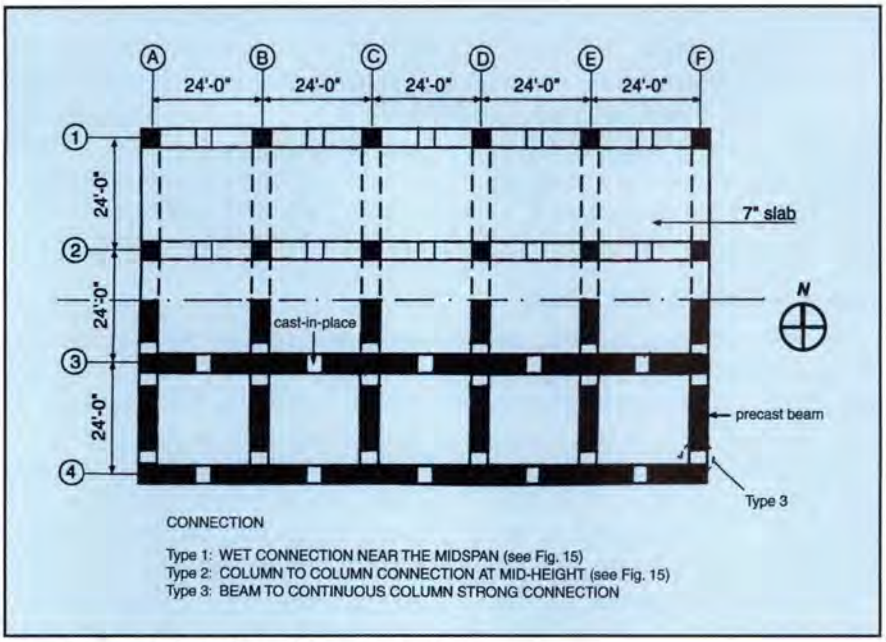


Fig. 14. Typical floor plan of example building.

CONCLUDING REMARKS

The code provisions discussed in this paper have been developed to help ensure that precast buildings are designed and detailed to resist strong ground motion. Specifically, the following design objective formed the basis of many of the provisions:

> Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as non-structural damage.[8]

A structural engineer's priority must be to accomplish this goal. The prescriptive language contained in this code change is intended to help accomplish this. However, as always, it should never be used to replace a fundamental understanding of the behavior of the building under design.

Lateral force resisting systems consisting of precast moment frames interconnected by wet or strong connections, conforming to the requirements of Sections 1921.2.2.6 or 1921.2.2.7, respectively, are considered special moment resisting frames (SMRF) of concrete and qualify for an R value of 8.5 according to Table 16-N of the 1997 UBC. Although a lower value of R for this type of lateral force resisting system was considered, the $R = 8.5$ value was agreed upon in view of the additional requirements established to improve the seismic performance of the overall precast building framing system. These requirements are given in Section 1921.2.1.7 with the intent to greatly improve the overall configuration, redundancy and restoring force characteristics of precast building framing systems.

In summary, the prescriptive and performance requirements discussed in this article focus on the following issues:

- Provide sufficient strength ($R = 8.5$).
- Limit yielding behavior to regions where the behavior is understood and controllable.
- Ensure deformation capacity in all parts of the building sufficient to withstand lateral displacements of the building subjected to a major earthquake.

DESIGN EXAMPLE OF A 12-STORY PRECAST FRAME BUILDING USING STRONG CONNECTIONS

A typical floor plan and elevation of the example building are shown in Figs. 14 and 15, respectively. The example building is the same as in Chapter 3 of Ref. 9. Figs. 14 and 15 clearly show the precast elements out of which the building is to be constructed. The relevant design data are given below.

Service Loads

Live Load = 50 psf (2.40 kPa)
Superimposed Dead Load = 42.5 psf (2.04 kPa)

Material Properties

Concrete: $f_c' = 4$ ksi (28 MPa) [6 ksi (42 MPa) for columns in the bottom six stories]
Unit Weight = 150 pcf (2400 kg/m^3)

Member Dimensions

Transverse Beams: 24 x 26 in. (610 x 660 mm)
Longitudinal Beams: 24 x 20 in. (610 x 508 mm)
Columns: 24 x 24 in. (610 x 610 mm)
Slabs: 7 in. (178 mm)

Figs. 14 and 15 depict the three connections detailed in this example:

1. A wet connection near midspan of an interior beam on the third floor level of the building. The beam is part of an interior longitudinal frame.

2. A column-to-column connection at mid-height between Levels 2 and 3 of an interior column stack that is part of an interior longitudinal frame.

3. A strong connection at the interface between a precast beam at the second floor level of the building that forms the exterior span of an exterior transverse frame, and the continuous corner column to which it is connected.

Grout sleeves for the mechanical connections are not shown in the

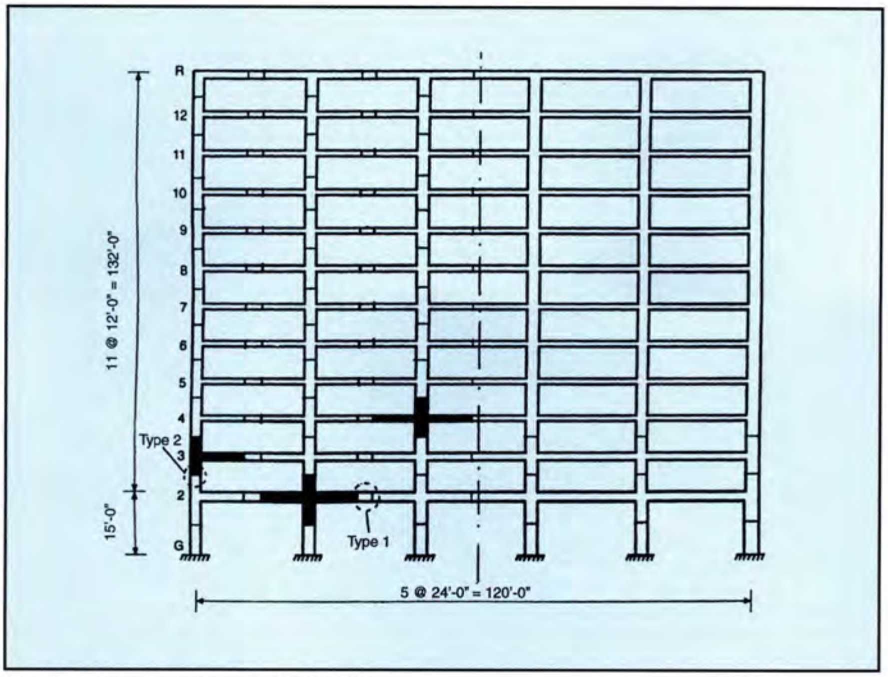


Fig. 15. Elevation of example building.

sketches of the details. The designs did not consider reinforcement for construction loads (such as lifting load). The actual construction sequence may be left up to the contractor.

Seismic Design Forces

The development of the seismic design forces on various structural components is beyond the scope of this paper. Traditional analysis methods can be used for precast frames, although care should be taken to approximate the component stiffness in a way that is appropriate for the precast components being used. For emulation design (as it is described in this paper) it is reasonable to model the beams and columns as if they were monolithic concrete.

Wet Connection Near Beam Midspan

The design bending moments for the third floor level beam that is part of an interior longitudinal frame is shown in Table 2. These design moments account for all possible load combinations. Eight #9 top bars and five #9 bottom bars are provided at all supports. The corresponding negative and positive design moment strengths are also shown in Table 2. Three of the top bars and three of the bottom bars are made continuous throughout the

Table 2. Design forces for the third floor beam forming part of an interior longitudinal frame of the building.

	M_u (ft-kips)	ϕM_n (ft-kips)
Negative	-510.8	-550.8
Positive	+311.7	+364.5

Note: 1 ft-kip = 1.356 kN-m.

spans, providing positive and negative design moment strengths of 233.1 ft-kips (316 kN-m).

At the supports, $\varphi M_n^+ = 364.5$ ft-kips > $\varphi M_n^-/2 = 275.4$ ft-kips (373 kN-m) (ok) (Section 1921.3.2.2). Also $(\varphi M_n)_{min.} = 233.1$ ft-kips > $\varphi M_n^-/4 = 137.7$ ft-kips (187 kN-m) (ok) (Section 1921.3.2.2).

Lap Splice Length — Per Section 1912.2.2:

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c+K_{tr}}{d_b}\right)}$$

$$\frac{c+K_{tr}}{d_b} \le 2.5$$

$\alpha = 1.3$ for top bars; $\alpha = 1.0$ for other bars (Section 1912.2.4)

$\beta = 1.0$ for non-epoxy-coated bars (Section 1912.2.4)

$\gamma = 1.0$ for #7 and larger bars (Section 1912.2.4)

$\lambda = 1.0$ for normal-weight concrete (Section 1912.2.4)

From Sketch 1, $c = 2.56$ in. (65 mm) $c/d_b = 2.28$, which makes it reasonable to take :

$$\frac{c+K_{tr}}{d_b} = 2.5$$

Thus:

$$l_d = \frac{3}{40}\frac{60{,}000}{\sqrt{4000}}\frac{1}{2.5}d_b = 28.46 d_b$$

$= 32$ in. (813 mm) for bottom bars

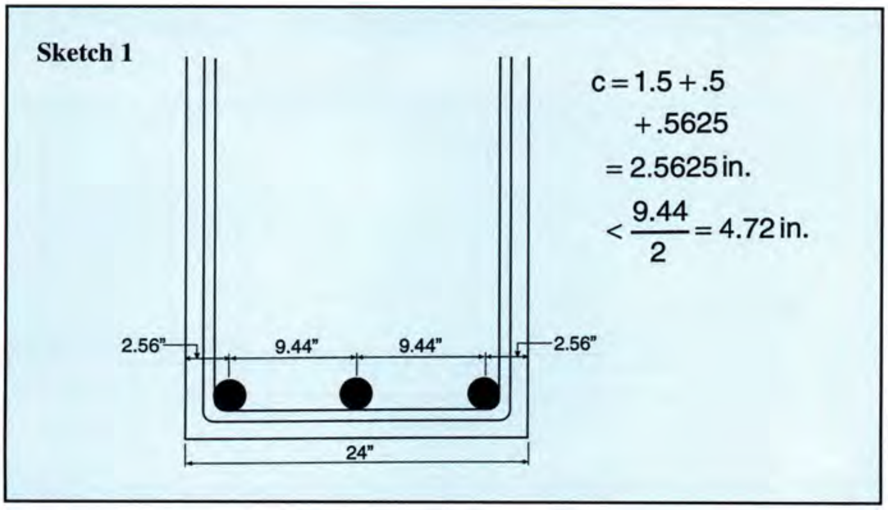
$= 1.3 \times 28.46 d_b$

$= 41.62$ in. (1057 mm) for top bars

However, it is shown later that two #9 top bars are adequate beyond 9.29 ft (2.83 m) from the faces of supports.

Thus, the top bar development length can be reduced by an excess reinforcement factor of $^2/_3$ (Section 1912.2.5), yielding an $l_d = {}^2/_3 \times 41.62 = 27.75$ in. (705 mm).

Because all three top bars and all three bottom bars are spliced at the



same location, Type B splices are to be used for both the top bars and the bottom bars. The Type B splice length = 1.3 × 32 = 41.6 in. (1057 mm) for the bottom bars. Provide the same 42 in. [3 ft 6 in. (1.07 m)] top splice lengths for both the top and the bottom bars.

Reinforcing Bar Cutoff — M_{pr}^{+} and M_{pr}^{-} are calculated with $\varphi = 1.0$ and $f_s = 75$ ksi (517 MPa), ignoring compression steel.

For five #9 bars:

$$a = \frac{A_s f_s}{0.85 f'_c b} = \frac{5(75)}{0.85(4)(24)}$$
$$= 4.6 \text{ in. (117 mm)}$$

$$M_{pr}^{+} = A_s f_s (d - a/2)$$
$$= \frac{5(75)}{12}\left(17.44 - \frac{4.6}{2}\right)$$
$$= 473.1 \text{ ft-kips (641 kN-m)}$$

since $d = 20 - 1.5$ (clear cover) $- 0.5$ (diameter of #4 stirrups) $- 0.5625$ ($^1/_2$ diameter of #9 bars) $= 17.44$ in. (443 mm).

For eight #9 bars:

$$a = \frac{8(75)}{0.85(4)(24)} = 7.35 \text{ in. (187 mm)}$$

$$M_{pr}^{-} = \frac{8(75)}{12}\left(17.44 - \frac{7.35}{2}\right)$$
$$= 688.3 \text{ ft-kips (933 kN-m)}$$

$$w_D = 0.15 \times \frac{7}{12} \times 22 \text{ (slab weight)}$$
$$+ 0.0425 \times 24 \text{ (superimposed DL)}$$
$$+ \frac{0.15 \times 24 \times 20}{144} \text{ (beam weight)}$$
$$= 3.44 \text{ kips per ft (50.2 kN/m)}$$

$0.9w_D = 0.9 \times 3.44 = 3.10$ kips per ft (45.2 kN/m) at midspan.

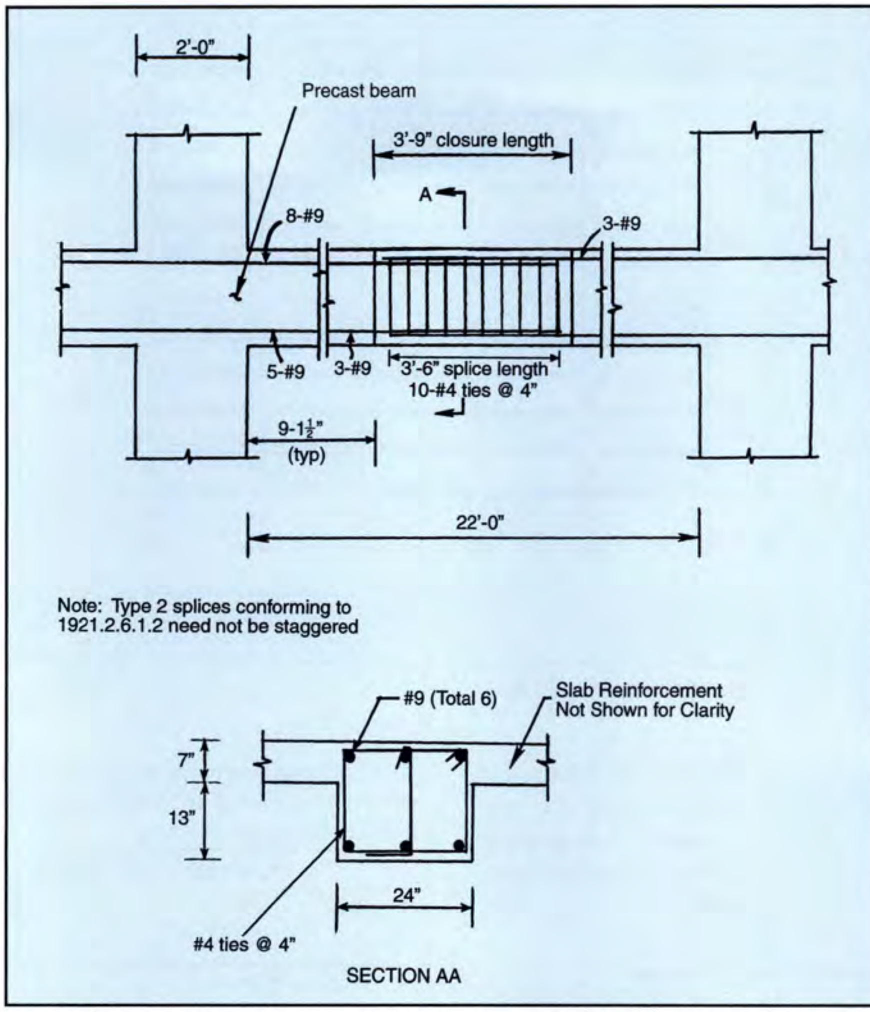


Fig. 16. Beam-to-beam wet connection near midspan of third-floor beam forming part of interior longitudinal frame.

The distance from the face of the interior support to where the moment under the loading considered equals $\varphi M_n^{-} = 233.1$ ft-kips (316 kN-m) is readily obtained by summing the moments about Section a-a:

$$\frac{x}{2}\left(\frac{3.10x}{11}\right)\left(\frac{x}{3}\right) +$$
$$688.3 - 233.1 - 71.53x = 0$$

or, $0.04697x^3 - 71.53x + 455.2 = 0$ whence, $x = 6.55$ ft (2.0 m)

Thus, five out of eight #9 bars can be terminated at a distance of 6.55 ft + d = 8.22 ft (2.51 m) from the faces of interior supports. Similar calculations can be made to show that all but three #9 top and three #9 bottom bars can be terminated short of the splice closure.

Sketch 2

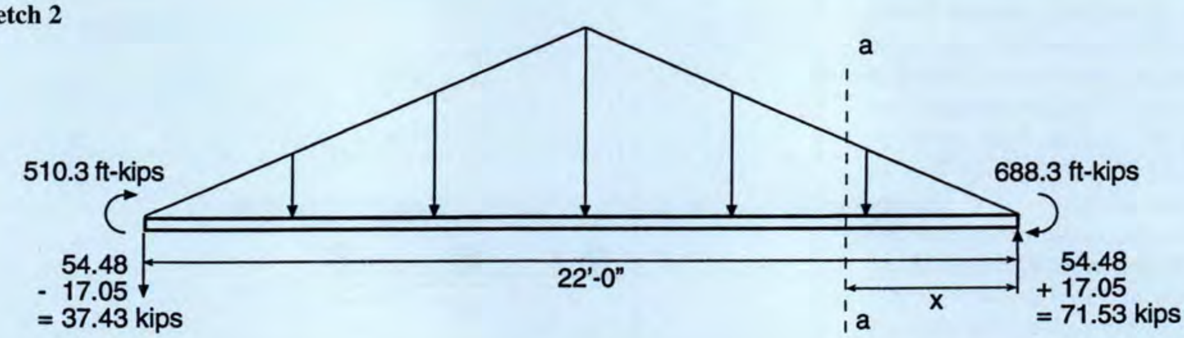


Table 3. Design forces for the interior column between the second and third floors, forming part of an interior longitudinal frame of the building.

Axial load (kips)	Moment (ft-kips)		Shear (kips)
	Top	Bottom	
1609.8	-408.3	+467.5	70.7

Note: 1 kip = 4.448 kN; 1 ft-kip = 1.356 kN-m.

The wet connection near the beam midspan is illustrated in Fig. 16.

Column-to-Column Connection at Mid-Height

The design forces for the interior column between Levels 2 and 3 (which is part of an interior longitudinal frame) are shown in Table 3.

Selection of Reinforcement — Consider twelve #10 bars.

Maximum axial load, P_u = 1609.8 kips (7156 kN) (from Table 3).

The corresponding φM_n from PCACOL program = 659.1 ft-kips > M_u = 467.5 ft-kips (634 kN-m) (ok).

It can be verified using PCACOL or other design aids that twelve #10 bars are adequate for other axial load, bending moment combinations as well.

$$\rho_g = \frac{A_{st}}{bh} = \frac{12(1.27)}{24 \times 24} = 0.0265$$

$\rho_{min} = 0.01 < \rho_g = 0.0265 < \rho_{max} = 0.06$ (ok) (Section 1921.4.3.1)

Verification of Strong Column, Weak Beam Requirement — Between the second and third floor levels, φM_n = 840.6 ft-kips (1140 kN-m), corresponding to P_u = 711.4 kips (3162 kN).

Between the third and fourth levels, φM_n = 836.1 ft-kips (1133.5 kN-m), corresponding to P_u = 655.5 kips (2916 kN).

$$\Sigma M_e = 840.6 + 836.1 = 1676.7 \text{ ft-kips } (2274 \text{ kN-m})$$

From the preceding section, for beams framing in, φM_n^- = 550.8 ft-kips and φM_n^+ = 364.5 ft-kips (747 and 494 kN-m).

$$\Sigma M_g = 550.8 + 364.5 = 915.3 \text{ ft-kips } (1241 \text{ kN-m})$$

ΣM_e = 1676.7 ft-kips > $6/5 \Sigma M_g$ = 1.2 (915.3) = 1098.4 ft-kips (1489 kN-m) (ok) (Section 1921.4.2.2)

The intent of this code provision is to prevent a story mechanism, rather than prevent local yielding in a column. The 6/5 factor is clearly insufficient to prevent column yielding if the adjacent beams both hinge. Therefore,

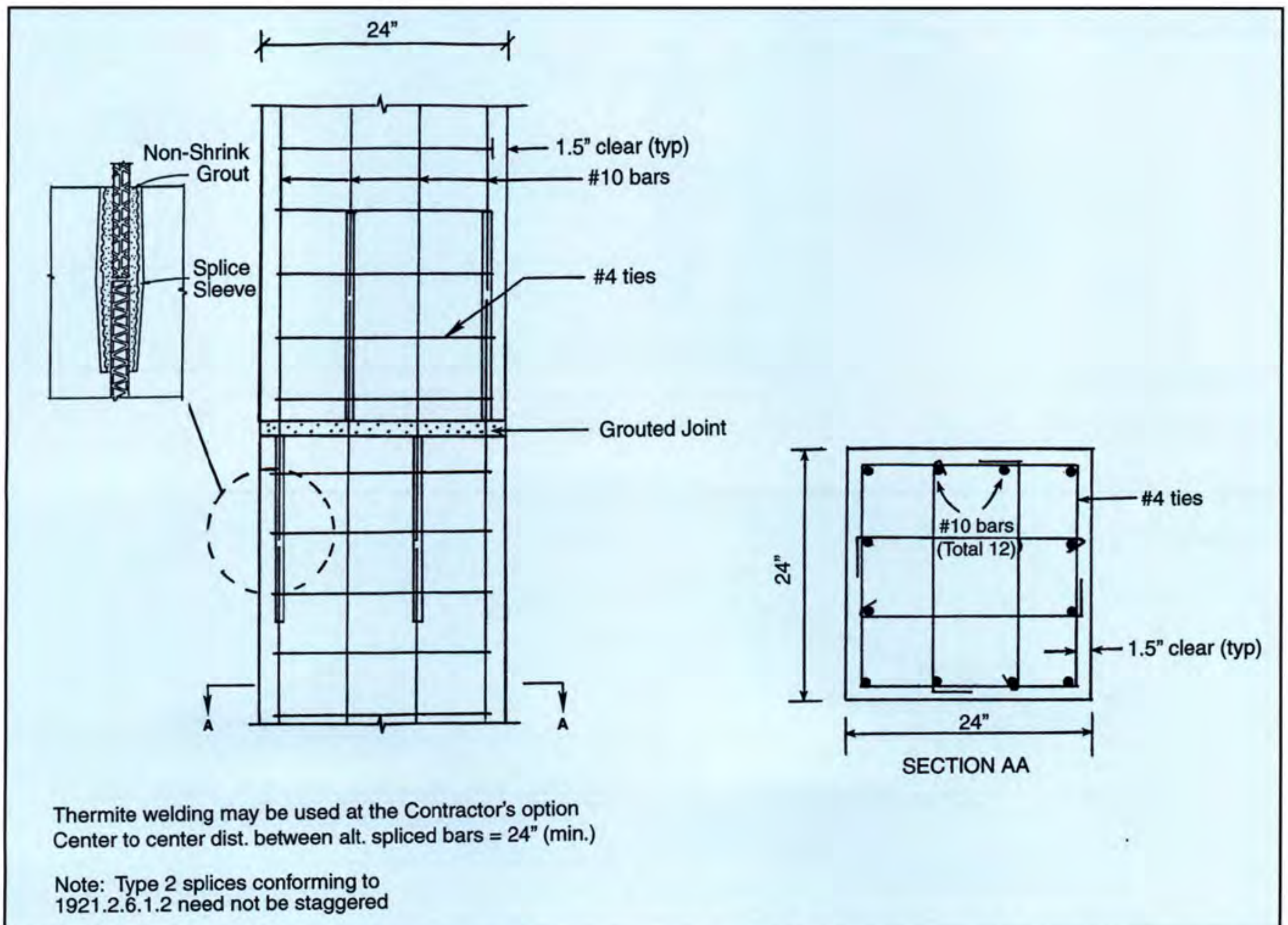


Fig. 17. Column-to-column wet connection at midheight of interior column between second and third floors, forming part of interior longitudinal frame.

confinement reinforcing is required in the potential hinge regions of a frame column. While the code describes a method for ensuring equilibrium at the joint for this calculation, the need for such accuracy is questionable given the code objective.

Minimum Connection Strength — M_{pr} for a column between the second and third floor levels, corresponding to an axial load of 711.4 kips $\approx$ 1.25 (840.6) = 1050.8 ft-kips (1425 kN-m).

Column-to-column connection must have φM_n at least equal to $0.4M_{pr}$ = 0.4 × 1050.8 = 420.3 ft-kips (570 kN-m) (Section 1921.2.7.4).

Twelve #10 bars are adequate.

Splice all twelve bars at mid-height, as shown in Fig. 17.

Column-Face Strong Connection in Beam

A strong connection is to be designed at the interface between a precast beam at the second floor level of the building that forms the exterior span of an exterior transverse frame, and the continuous corner column to which it is connected. The beam is reinforced with five #9 bars at the top and four #9 bars at the bottom at its ends. M^+_{pr} and M^-_{pr} are calculated with φ = 1.0 and f_s = 75 ksi (517 MPa), ignoring compression steel.

For four #9 bars:

$$a = \frac{A_s f_s}{0.85 f'_c b} = \frac{4(75)}{0.85(4)(24)}$$

$$= 3.68 \text{ in. } (93.5 \text{ mm})$$

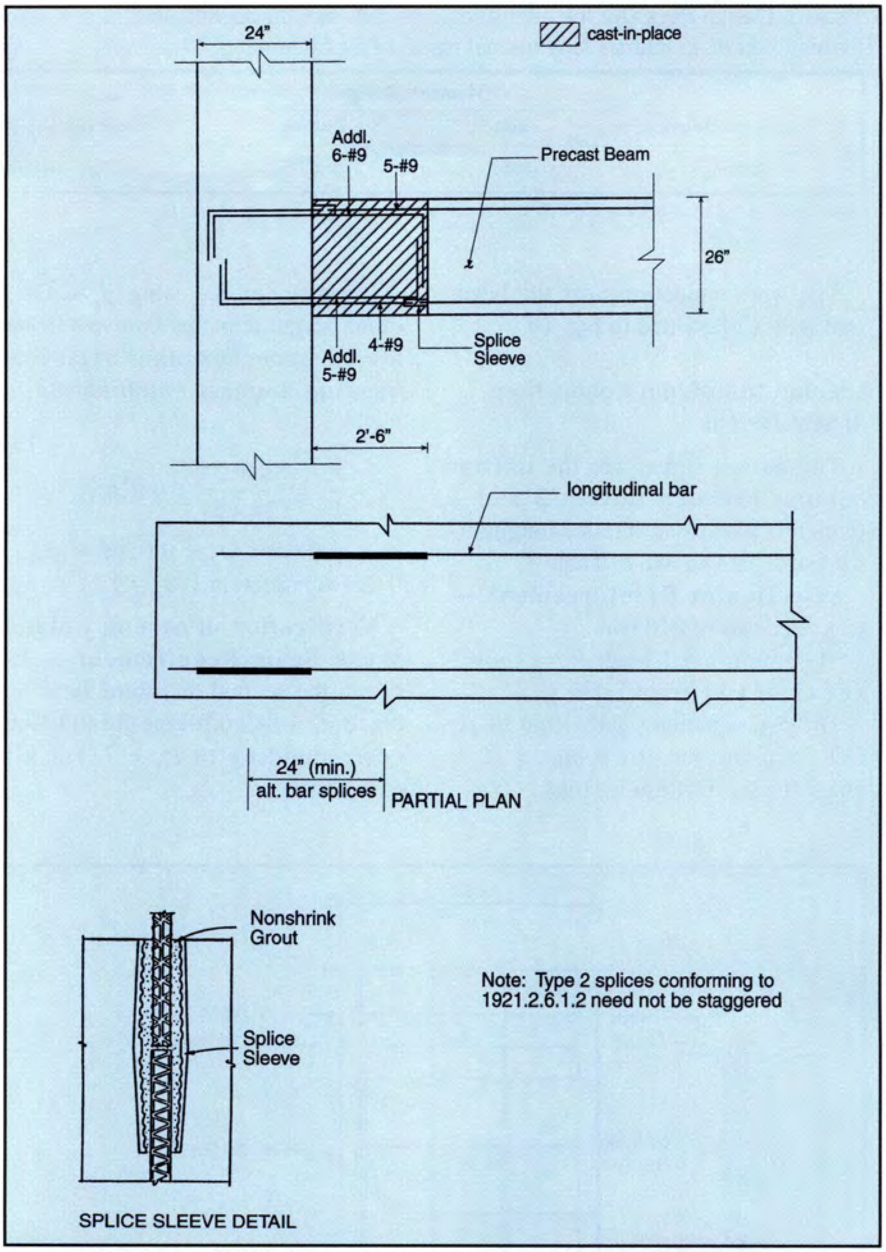
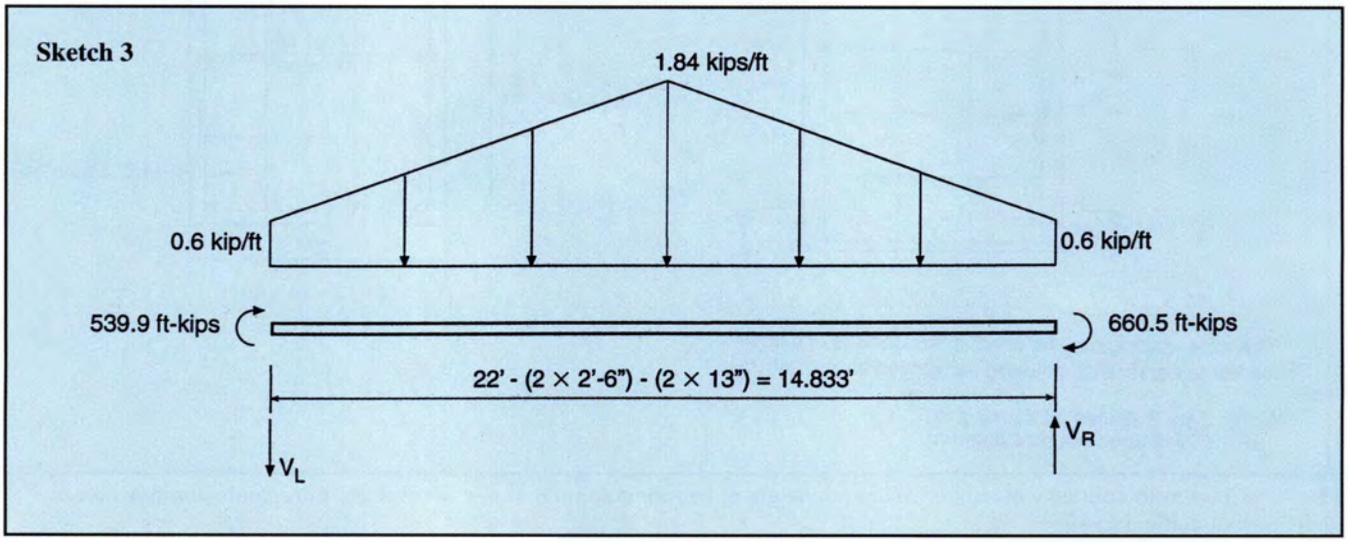


Fig. 18. Beam-to-continuous-column strong connection at exterior span of transverse frame at second floor level.



$$M_{pr}^{-} = A_s f_s (d - a/2)$$
$$= \frac{4(75)}{12}(23.44 - 1.84)$$
$$= 539.9 \text{ ft-kips (732 kN-m)}$$

since d = 26 – 1.5 (clear cover) – 0.5 (diameter of #4 stirrups) – 0.5625 ($^1/_2$ diameter of #9 bars) = 23.44 in. (7.14 mm)

For five #9 bars:

$$a = \frac{5(75)}{0.85(4)(24)} = 4.60 \text{ in. (117 mm)}$$

$$M_{pr}^{-} = \frac{5(75)}{12}(23.44 - 2.30)$$
$$= 660.5 \text{ ft-kips (896 kN-m)}$$

Assuming a 2 ft 6 in. (0.76 m) cast-in-place closure and nonlinear action location at $h/2$ from the face of the closure, the beam span between nonlinear action locations is loaded as shown in Sketch 3.

The loading on the beam is taken from Chapter 3 of Ref. 9 (see pp. 3-19 to 3-27).

Calculation of Shear Forces at Nonlinear Action Locations — Taking moments about right end of beam segment:

$$660.5 + 539.9 - \left[\frac{(1.84+0.6)}{2}\frac{14.833^2}{2}\right]$$
$$= V_L(14.833)$$

whence, V_L = 71.88 kips ≈ 71.9 kips (320 kN).

Taking moments about the left end of the beam segment:

$$539.9 + 660.5 - \left[\frac{(1.84+0.6)}{2}\frac{14.833^2}{2}\right]$$
$$= V_R(14.833)$$

whence, V_R = 89.98 kips ≈ 90 kips (400 kN).

Strength Design of Connection — From the above, design moment strengths that must be provided at the column face are:

$$\varphi M^{+}_{n\,CONNECTION} = 539.9 + 71.9\left(2.5 + \frac{13}{12}\right)$$
$$= 797.5 \text{ ft-kips (1081 kN-m)}$$

and

$$\varphi M^{-}_{n\,CONNECTION} = 660.5 + 90\left(2.5 + \frac{13}{12}\right)$$
$$= 983.0 \text{ ft-kips (1333 kN-m)}$$

Nine #9 bars at the bottom and eleven #9 bars at the top would provide φM_n^+ = 853.2 ft-kips [> 797.5 ft-kips (1081 kN-m)] and φM_n^- = 1028.7 ft-kips [> 983.0 ft-kips (1333 kN-m)].

Maximum reinforcement ratio =

$$\frac{11(1.0)}{24 \times 23.44} = 0.0196 < 0.025 \text{ (ok)}$$

Thus, the addition of six #9 bars at the top and five #9 bars at the bottom is required, as shown in Fig. 18.

Anchorage and splices:

Per Section 1921.5.4.1:

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f_c'}} = \frac{(60{,}000)(1.128)}{65\sqrt{4000}}$$
$$= 16.46 \text{ in. (418 mm)}$$

Thus, the detail shown in Fig. 18 would conform to the requirements of Section 1921.2.7.2.

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APPENDIX — EXCERPTS FROM 1997 UNIFORM BUILDING CODE PROVISIONS

(**Note:** Italicized text indicates differences between 1997 UBC and ACI 318-95.)

Chapter 19, Div. II

SECTION 1921 – REINFORCED CONCRETE STRUCTURES RESISTING FORCES INDUCED BY EARTHQUAKE MOTIONS

1921.0 NOTATIONS

$S_{e\ CONNECTION}$
= *moment, shear or axial force at connection cross section other than the nonlinear action location corresponding to probable strength at the nonlinear action location, taking gravity load effects into consideration, per Section 1921.2.7.3.*

$S_{n\ CONNECTION}$
= *nominal strength of connection cross section in flexural, shear or axial action, per Section 1921.2.7.3.*

$\Delta_m = R_w \Delta_s$*

Δ_s = *Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.*

Ψ = *Dynamic Amplification Factor from Sections 1921.2.7.3 and 1921.2.7.4.*

1921.1 DEFINITIONS

CONNECTION *is an element that joins two precast members or a precast member and a cast-in-place member.*

DRY CONNECTION *is a connection used between precast members, which does not qualify as a wet connection.*

JOINT *is the geometric volume common to intersecting members.*

NONLINEAR ACTION LOCATION *is the center of the region of yielding in flexure, shear or axial action.*

NONLINEAR ACTION REGION *is the member length over which nonlinear action takes place. It shall be taken as extending a distance of no less than h/2 on either side of the nonlinear action location.*

STRONG CONNECTION *is a connection that remains elastic, while the designated nonlinear action regions undergo inelastic response under the Design Basis Ground Motion.*

WET CONNECTION *uses any of the splicing methods, per Section 1921.2.6.1 or 1921.3.2.3, to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.*

1921.2.1.6 *Precast lateral-force-resisting systems shall satisfy either of the following criteria:*

1. Emulate the behavior of monolithic reinforced concrete construction and satisfy Section 1921.2.2.5, or

2. Rely on the unique properties of a structural system composed of interconnected precast elements and conform to Section 1629.9.2.

1921.2.1.7 *In structures having precast gravity systems, the lateral-force-resisting system shall be one of the systems listed in Table 16-N and shall be well distributed using one of the following methods:*

1. The lateral-force-resisting systems shall be spaced such that the span of the diaphragm or diaphragm segment between lateral-force-resisting systems shall be no more than three times the width of the diaphragm or diaphragm segment.

Where the lateral-force-resisting system consists of moment-resisting frames, at least $[(N_b/4) + 1]$ *of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the lateral-force-resisting system, where* N_b *is the total number of bays along that line at that story. This requirement applies to only the lower two-thirds of the stories of buildings three stories or taller.*

2. All beam-to-column connections that are not part of the lateral-force-resisting system shall be designed in accordance with the following:

Connection design force. *The connection shall be designed to develop strength M. M is the moment developed at the connection when the frame is displaced by* Δ_s *assuming fixity at the connection and a beam flexural stiffness of no more than one-half of the gross section stiffness. M shall be sustained through a deformation of* Δ_m.

Connection Characteristics. *The connection shall be permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member shall resist moment with same positive or negative sign. The connection shall be permitted to have zero flexural stiffness up to a frame displacement of* Δ_s.

In addition, complete calculations for the deformation compatibility of the gravity load carrying system shall be made in accordance with Section 1633.2.4 using cracked section stiffnesses in the lateral-force-resisting system and the diaphragm.

Where gravity columns are not provided with lateral support on all sides, a positive connection shall be provided along each unsupported direction parallel to a principal plan axis of the structure. The connection shall be designed for a horizontal force equal to 4 percent of the axial load strength (P_0) *of the column.*

The bearing length shall be 2 inches (51 mm) more than that required for bearing strength.

1921.2.2.5 *Precast structural systems using frames and emulating the behavior of monolithic reinforced concrete construction shall satisfy either Section 1921.2.2.6 or 1921.2.2.7.*

* This expression is in error. It should read $\Delta_m = 0.7R\Delta_s$.

1921.2.2.6 *Precast structural systems, utilizing wet connections, shall comply with all the applicable requirements of monolithic concrete construction for resisting seismic forces.*

1921.2.2.7 *Precast structural systems not meeting Section 1921.2.2.6 shall utilize strong connections resulting in nonlinear response away from connections. Design shall satisfy the requirements of Section 1921.2.7 in addition to all the applicable requirements of monolithic concrete construction for resisting seismic forces, except that provisions of Section 1921.3.1.2 shall apply to the segments between nonlinear action locations.*

1921.2.5 Reinforcement in members resisting earthquake induced forces.

1921.2.5.1 ***Alloy A 706 reinforcement.*** *Except as permitted in Sections 1921.2.5.2 through 1921.2.5.5,* reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary elements shall comply with *low alloy* A 706 except as allowed in Section 1921.2.5.2.

1921.2.5.3 *The average prestress f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension, shall be the lesser of 350 psi (2.41 MPa) or $f_c'/12$ at locations of nonlinear action, where prestressing tendons are used in members of frames.*

1921.2.5.4 *For members in which prestressing tendons are used together with mild reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more than one-quarter of the strength for both positive and negative moments at the joint face and shall extend through exterior joints and be anchored at the exterior face of the joint or beyond.*

1921.2.5.5 *Shear strength provided by prestressing tendons shall not be considered in design.*

1921.2.6 Welded splices and mechanically connected reinforcement.

1921.2.6.1 Reinforcement resisting earthquake-induced flexural or axial forces in frame members or in wall boundary members shall be permitted to be spliced using welded splices or mechanical connectors conforming to Section 1912.14.3.3 or 1912.14.3.4.

Splice locations in frame members shall conform to Section 1921.2.6.1.1 or 1921.2.6.1.2.

1921.2.6.1.1 Welded splices. *In Seismic Zones 2, 3 and 4, welded splices on billet steel A 615 or low alloy A 706 reinforcement shall not be used within an anticipated plastic hinge region nor within a distance of one beam depth on either side of the plastic hinge region or within a joint.*

1921.2.6.1.2 Mechanical connection splices. *Splices with mechanical connections shall be classified according to strength capacity as follows:*

Type 1 splice. *Mechanical connections meeting the requirements of Sections 1912.14.3.4 and 1912.14.3.5.*

Type 2 splice. *Mechanical connections that develop in tension the lesser of 95 percent of the ultimate tensile strength or 160 percent of specified yield strength, f_y, of the bar.*

1921.2.6.2 Welding of stirrups, ties, inserts or other similar elements to longitudinal reinforcement required by design shall not be permitted.

1921.2.7 Emulation of monolithic construction using strong connections. *Members resisting earthquake-induced forces in precast frames using strong connections shall satisfy the following:*

1921.2.7.1 Location. *Nonlinear action location shall be selected so that there is a strong column/weak beam deformation mechanism under seismic effects. The nonlinear action location shall be no closer to the near face of strong connection than h/2. For column-to-footing connections, where nonlinear action may occur at the column base to complete the mechanism, the nonlinear action location shall be no closer to the near face of the connection than h/2.*

1921.2.7.2 Anchorage and splices. *Reinforcement in the nonlinear action region shall be fully developed outside both the strong connection region and the nonlinear action region. Noncontinuous anchorage reinforcement of strong connection shall be fully developed between the connection and the beginning of nonlinear action region. Lap splices are prohibited within connections adjacent to a joint.*

1921.2.7.3 Design forces. *Design strength of strong connections shall be based on*

$$\phi S_{n\,CONNECTION} > \psi S_{e\,CONNECTION} \qquad (21\text{-}1)$$

Dynamic amplification factor ψ shall be taken as 1.0.

1921.2.7.4 Column-to-column connection. *The strength of such connections shall comply with Section 1921.2.7.3 with ψ taken as 1.4. Where column-to-column connections occur, the columns shall be provided with transverse reinforcement as specified in Sections 1921.4.4.1 through 1921.4.4.3 over their full height if the factored axial compressive force in these members, including seismic effects, exceeds $A_g f_c'/10$.*

EXCEPTION: *Where column-to-column connection is located within the middle third of the column clear height, the following shall apply: (1) The design moment strength ϕM_n of the connection shall not be less than 0.4 times the maximum M_{pr} for the column within the story height, and (2) the design shear strength ϕV_n of the connection shall not be less than that determined per Section 1921.4.5.1.*

1921.2.7.5 Column-face connection. *Any strong connection located outside the middle half of a beam span shall be a wet connection, unless a dry connection can be substantiated by approved cyclic test results. Any mechanical connector located within such a column-face strong connection shall develop in tension or compression, as required, at least 140 percent of specified yield strength, f_y, of the bar.*