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Seismic Design Considerations for Flat-Plate Construction

by J. P. Moehle

Synopsis: Design algorithms expressed in current building codes and practiced in design offices focus attention on earthquake induced lateral forces, and away from earthquake induced lateral displacements. These procedures have led to development of structural systems in which a portion of the structural frame is designed to resist the total seismic design force while a substantial remainder of the structure is proportioned assuming it resists only gravity loads. This approach is commonly applied to design of slab-column systems in regions of high seismicity. For such systems, a displacement-oriented approach has advantages. Applications of the approach are described.

Keywords: Buildings; columns (supports); earthquake-resistant structures; flat concrete plates; reinforced concrete; structural design

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INTRODUCTION

I was introduced to reinforced concrete flat-slab construction in a course taught by Professor Mete Sozen at the University of Illinois at Urbana-Champaign. The course was crafted from the historical perspectives of Talbot, and Cross, and Westergaard, and Siess, and, of course, Sozen. Their experiences were broad, and intimately known by the instructor, and over the weeks I gained from the lectures a sense of the experience for myself. Many other students, from over the years, have reminisced privately about the lasting impact of those few weeks in the classroom.

Later, as a member of Mete's research team, I had the opportunity to see Mete's research and engineering first hand. It became clear early on that Mete, though keenly analytic, preferred not to stray far from the evidence of physical investigation. When asked about his affinity for experimental research, Mete once responded (1989 EERI annual meeting) that the choice between *experimental research* and *analytical research* is as the choice between *looking* and *thinking*. How could one choose, and how could one advocate one over the other? But if a researcher has to choose a starting point, perhaps it is better to choose first to look, because by first looking one cannot help but also think later on about what was seen.

The earthquake engineer may choose to "look" in two places. The first place is the research laboratory. Though our experiments are limited in scale and scope, and polluted to varying degrees by the researcher's concept of how to model reality, they have profoundly improved the engineer's understanding of structural behavior, and have empowered builders to construct structures that perform better in earthquakes. This symposium volume and the broader technical literature contain numerous examples where experimental research has pointed out directions for improving engineering practice.

The post-earthquake setting is the second place to look for physical evidence on the seismic behavior of structures. In the post-earthquake setting the drawback of insufficient field instrumentation may be offset by an abundance of visual clues from structures and earthquakes that are real in all their details. Lessons from earthquakes therefore include aspects that may be missing in a restricted research laboratory setting.

On 17 January 1994, a moment magnitude 6.7 earthquake (the Northridge earthquake) struck southern California. Whereas several recently previous earthquakes had occurred distant from metropolitan regions[4], the Northridge earthquake provided the first modern evidence of the effects of a glancing blow on a major metropolitan region, and shed light on the possible effects of a future direct hit. The earthquake inflicted damage on construction of all materials, including wood, steel, masonry, and concrete. While the vast majority of post-1971 reinforced concrete building construction performed well by current accounts [22, 12], some concrete construction types did not fare well in several specific cases. Included in this category is perimeter moment frame building construction. This is a construction form in which the engineer designs the perimeter framing elements to resist the entire code design lateral forces, and designs the interior framing to carry only gravity loads. The combination of slab-column "gravity" framing and perimeter special moment resisting frames or perimeter walls is the subject of this paper.

DESIGN PRACTICE FOR GRAVITY FRAMING SYSTEMS

Current design practice in the United States allows the structural engineer to designate some elements of a building as lateral force resisting elements and other elements as gravity-only load resisting elements. According to this practice, the designated lateral force resisting elements must be proportioned to resist the entire design seismic force within accepted drift limits. The designated gravity load carrying system is proportioned assuming it does not contribute to seismic resistance, and must be checked to ensure that it will continue to carry the gravity loads under the design lateral deformations. This practice is common in the western United States.

In the form of construction described in the preceding paragraph, the designated lateral force resisting system may comprise structural walls or special moment resisting space frames. Where structural walls are used, the walls are commonly core walls or walls on the building perimeter. Where special moment resisting frames are used, the frame is commonly positioned around all or a portion of the perimeter. Consequently, the name "perimeter frame building" is commonly applied to this construction form.

Gravity load systems prevalently are beam-column frames, reinforced concrete slab-column frames, or post-tensioned slab-column frames. Beam-column frames may be cast in place or precast. Where precast frames are used, the diaphragm may be precast with a relatively thin topping slab. This paper focuses on reinforced concrete and post-tensioned concrete slab-column gravity systems.

Current building codes have special provisions for frame members not designated as part of the lateral force resisting system. For example, the commentary for section 21.7 of the 1989 ACI Building Code [6] states that members that are not part of the designated lateral force resisting system are not required to meet all the detailing requirements of members that are relied on to resist lateral forces, but they must be able to resist deformations above the service level and still be able to support gravity loads. According to this code, beams and columns must be analyzed to determine if they develop their strength when subjected to twice the lateral displacement under the factored lateral forces, and details are set based on this check. Other codes have other similar provisions for beam-column frames. United States codes do not contain special provisions for slab-column gravity frames in regions of high seismicity. ACI-ASCE Committee 352 [34] describes recommended practice for slab-column gravity frames.

OBSERVATIONS FROM THE 1994 NORTHRIDGE EARTHQUAKE

The 1994 Northridge earthquake caused severe damage and collapse in reinforced concrete perimeter frame buildings [22, 12]. Three main failure types are of interest in the present paper. These are:

- Failures in columns of the gravity load system. Figure 1 shows an example in which column failure appears to have contributed to collapse of the gravity load system.

- Failures in diaphragms. Figure 2 shows an example of failure in the topping slab near the connection with the lateral load resisting structural walls.

- Failures in slab-column connections. Figure 3 shows punching shear failure in a post-tensioned slab-column gravity frame with drop caps at the slab-column connection.

Although the causes of the Northridge earthquake failures have not been fully investigated, there has been credible speculation about the causes. Among the cited causes are the following:

- Engineers sometimes ignore or overlook code requirements for gravity systems.
- Commonly used methods for estimating demands on gravity systems may be inadequate.
- Code requirements for proportioning and detailing gravity systems may be inadequate.

This paper reviews recent research findings and proposes practical design methods for slab-column gravity systems in earthquake resisting buildings. It first analyzes procedures for estimating earthquake demands on gravity systems, and then reviews detailing and proportioning methods aimed at ensuring adequate member capacities.

STRUCTURAL ANALYSIS FOR BUILDINGS IN SEISMIC ZONES

For structural design it is not always necessary that the analysis accurately predict expected response. Instead, it is only necessary that the analysis efficiently produce an economical, serviceable, and safe structure. Conventional seismic design relies on simplified analyses whose results may not accurately represent actions that occur during a strong earthquake. These inaccuracies are of little consequence in design of structural systems having an abundant supply of ductility, because very ductile systems usually can be deformed beyond expected deformation levels without failure and catastrophic consequences. The same cavalier view of analysis may be inappropriate for design of gravity systems with marginal ductility and whose performance is critical to life safety. For these structural systems, within reason, the design engineer should seek analysis results that conservatively represent actual response. Otherwise, gravity systems with marginal ductility should not be permitted.

Prevalent seismic analysis methods are oriented toward determining the lateral force distributions and required strengths, and provide limited information on lateral deformations. While the virtues of strength can be argued, it is clear that strength of the lateral force resisting system does not relate simply and directly to seismic performance of the gravity load system. To understand the performance of the gravity load system, it is necessary to focus on the imposed lateral drift and the internal forces that result.

Current United States design codes rely on elastic analysis to estimate lateral drift demands. For example, section 21.7 of the ACI Building Code [6] requires the engineer to check non-lateral force resisting elements for lateral drift equal to that calculated using an elastic structural model subjected to twice the factored code forces. One reasonably might infer that the specified drift is the one expected to occur during the design earthquake. Similarly, the UBC (Uniform Building Code) [39], which governs most building construction on the west coast of the United States, specifies a design displacement equal to the product of $3/8R_w$ and the displacements calculated from the unfactored UBC design forces. (R_w is a factor by which elastic spectral forces are reduced to obtain the design lateral forces.) For conventional structural systems designed with $R_w = 8$, the ACI specified design displacement is approximately equal to the $3/8R_w$ design displacement specified by the UBC. Neither the ACI Building Code nor UBC require the elastic structural analysis to include stiffness reduction effects due to concrete cracking, foundation rotation, and diaphragm flexibility. Many designs proceed ignoring these effects.

Seemingly overwhelming evidence indicates that the displacement estimates of current U.S. design codes are unconservative. Digital computer analyses from Veletsos and Newmark [40] through Newmark and Riddel [28], Otani [29], Shimazaki and Sozen [36], Qi and Moehle [33] and others show for longer period SDOF (single-degree-of-freedom) systems that the expected peak inelastic displacement is about equal to the peak displacement calculated for the structure assuming elastic response. This result contrasts with three-eighths of this value as implied by the UBC. For shorter-period systems, the peak inelastic displacement can be expected to be at least equal to the elastic value, and for weak systems may be several times that value [36] (Figure 4). The period below which the equal displacement rule is no longer valid is approximately equal to the period at which the constant acceleration and constant velocity regions in the elastic response spectrum intersect.

The most extensive set of experimental data on inelastic displacement response of reinforced concrete physical models comes from the work of Professor Sozen and his associates at the University of Illinois. The inventory of test specimens includes moderate-scale SDOF systems and small-scale MDOF (multi-degree-of-freedom) systems. The tests are reported in a series of reports from the University of Illinois [1, 2, 5, 7, 9, 11, 15, 18, 19, 25, 26, 27, 30, 35, 42, 43].

Figure 5 is a photograph of one of the test models situated on the University of Illinois shaking table [26]. As observed by Sozen [37], the measured inelastic response has many characteristics similar to those expected for elastic response (Figure 6). This idea was later extended by Shimazaki and Sozen [36] and Bonacci [5], who noted that the peak inelastic displacements could be related to the calculated peak elastic displacements. The data in Figure 7 are for a collection of the University of Illinois test data. In this figure, calculated peak elastic displacements are obtained for damping equal to two percent of the critical value and for an initial period equal to $\sqrt{2}T_o$, where T_o = fundamental period calculated based on gross cross sections. The collected test data verify and extend the earlier findings obtained from dynamic analyses of SDOF systems. These findings indicate that, given information on ground motion and structural system properties, one can estimate maximum displacement response with simple procedures.

Of course, complicating the design analysis problem is the uncertainty regarding earthquake ground motion that will occur in the future at a particular site. In some cases, actual recorded ground motions have been seen to exceed design quantities from current codes. The Northridge earthquake provides a recent demonstration where real response spectral ordinates significantly exceed the unreduced UBC design values (Figure 8). As noted previously, the UBC allows the design to be based on a displacement equal to three-eighths of the UBC spectral displacement shown in the figure, adding to the apparent non-conservatism of the UBC.

Furthermore, to achieve a conservative estimate of expected building displacements, foundation flexibility should be included in the analysis model. This may be particularly important for buildings with few relatively slender structural walls. As illustrated in Figure 9, foundation rotation may relieve force demands on the lateral force resisting system but increase lateral interstory drift demand on the gravity system.

Diaphragm deformations should also be considered when estimating deformation demands on gravity systems. This is less likely to be an important consideration for flat-plate systems with perimeter frames than it is for precast construction or construction with few structural walls.

COLUMNS OF SLAB-COLUMN GRAVITY FRAMES

Current United States codes [6, 39] require special details for columns that are considered to be part of the lateral force resisting system. Columns of gravity frames need not satisfy these requirements if strength is not reached under the code prescribed lateral deformations. Figure 10 presents current details required by the ACI Building Code [6].

The author is not aware of any failures due to earthquakes of columns in slab-column gravity frames. However, during the Northridge earthquake several gravity columns in other types of perimeter frame buildings were severely damaged (Figure 1) and appear to have contributed to collapse. A reasonable extrapolation is that similar failures are possible in slab-column gravity frames.

To understand why failures of gravity frame columns might occur, consider the structural framing illustrated in Figure 11. The details correspond to a building in southern California [8]. The lateral force resisting system comprises a perimeter special moment-resisting frame. The interior gravity framing system comprises post-tensioned slabs with drop capitals framing into reinforced concrete columns.

Detailing of the perimeter frame columns and the interior frame columns is strikingly different (Figure 11). The perimeter frame columns have special details including well-configured, closely-spaced transverse reinforcement and lap splices restricted to regions outside locations where flexural plastic hinges may form. With these details, the perimeter frame is likely to be capable of sustained load resistance under large imposed lateral deformations.

In contrast, the columns of the gravity frame system lack special details for ductile response (Figure 11). Compared with the perimeter frame columns, the ties are poorly configured and widely spaced, and lap splices are located just above floor slabs, including just above the ground floor slab. Analysis according to ACI Building Code [6] procedures indicate that the column may fail in shear prior to developing flexural strengths at both ends.

Columns having details similar to those shown for the gravity columns of Figure 11 are known not to perform well under large lateral deformations possible during an earthquake. Wight and Sozen [41] described various problems for this type of column, including shear strength decay with increasing displacement amplitude and number of cycles. More recent studies building on the work of Wight and Sozen and others work have been reported by Lynn and Moehle [20]. Figure 12 illustrates details of column test specimens and the extremes of observed behaviors. In one of these tests, shear failure resulted in sudden loss of gravity load capacity; in others the gravity load (as high as $0.3f'_cA_g$, where f'_c = concrete compressive strength and A_g = gross section area) was sustained through deformation cycles well past the onset of shear failure. Figure 13 illustrates the trend of decreasing available shear strength with increasing displacement ductility demand for the tests (representing building columns) by Lynn and Moehle and tests (representing bridge columns) by Iwasaki [3].

The data presented in the preceding paragraph demonstrate that failures under relatively low lateral deformations are credible for gravity columns. In contrast, the perimeter columns with special ductile details are likely to be capable of significantly larger deformations. If the building in Figure 11 is subjected to earthquake ground motion, lateral displacements will be imposed on all the columns. Ignoring torsional response and diaphragm deformation, the displacements of the perimeter frame columns and the gravity frame columns will be equal. Given that both types of columns are critical to protecting life safety by sustaining vertical loads, the difference in details is difficult to rationalize.

SLAB-COLUMN CONNECTIONS OF SLAB-COLUMN GRAVITY FRAMES

Code Requirements

The ACI Building Code [6] and other United States codes do not contain special provisions for slab-column gravity frames in buildings located in regions of high seismicity. Instead, the basic non-seismic provisions for gravity load design govern all aspects of proportioning and design. Importantly, for non-prestressed slabs these provisions require that at least two of the main bottom slab bars must be effectively continuous through the column at all connections. As illustrated in Figure 14, the bottom reinforcement will act to suspend a slab in the event that a punching shear failure occurs. Top mild reinforcement is less effective in this role.

The ACI Building Code [6] contains provisions for design of reinforced concrete flat-plates in regions of moderate seismicity. Figure 15, from the

ACI Building Code Commentary [6], shows the required details. These details concentrate slab flexural reinforcement within the column strip and enforce nominal continuity of all slab reinforcement. Though not specifically required, these same details seem advisable for gravity slab-column frames in perimeter frame buildings in regions of higher seismicity.

Evaluation of Slab Performance

The ACI Building Code [6] does not contain procedures designed to evaluate expected performance of slab-column connections under the action of earthquake loads. Lacking specific seismic requirements, it is common in current design office practice to apply the conventional shear and moment transfer strength provisions of the code. According to this practice, the lateral drift as specified by the UBC [39] (that is, drift equal to the product of the calculated drift and $3/8R_w$) is imposed on an analytical model of the building, and the resulting shear and moment on the connection are compared with the corresponding design strength calculated using the eccentric shear stress model (Figure 16). If the design strength is exceeded, the connection is deemed inadequate and the design must be revised.

Figure 17 compares results of this procedure applied to three conventionally reinforced, interior slab-column connections tested in the laboratory. The three connections had different levels of gravity load applied to the slab, this load being represented in the figure by the gravity shear ratio, V_u/V_o, where V_u = the total shear due to gravity loads acting normal to the plane of the slab and V_o = the punching shear strength in the absence of moment transfer calculated according to the ACI Building Code [6]. The tests are reported by Morrison and Sozen [27] for Specimen 1, Pan and Moehle [31] for Specimen 2, and Hawkins [14] for Specimen 3. Strengths are nominal strengths calculated according to either the eccentric shear stress model of the ACI Building Code [6] or the strength corresponding to development of flexural yield lines across the full width of the slab. The stiffnesses were calculated using the effective beam width model. Only flexural deformations were considered, with column stiffness based on the gross cross section and slab stiffness determined from an effective moment of inertia equal to $\alpha\beta I_g$, where α = an effective width coefficient [32], β = a reduction factor of 1/3 to account for slab cracking [23], and I_g = moment of inertia of the gross cross section. The calculated strength for the Morrison and Sozen test is limited by the yield line strength, and strengths for the other two tests is limited by the nominal shear and moment transfer strength obtained from the eccentric shear stress model.

Two shortcomings of the calculation in relation to the experiments are noted (Figure 17). The first of these relates to calculations of the strength and the effective stiffness of the connection, both of which are required in order to obtain an estimate of the displacement at which the connection reaches its strength. Either or both of these calculated quantities may differ appreciably from the actual quantities. The second, more fundamental shortcoming is that the deformation at which the nominal strength is reached does not in any obvious way relate to the deformation capacity, which may be several times the calculated deformation.

It is apparent from Figure 17 that the deformation capacity at failure decreases as the gravity shear ratio increases. The gravity shear ratio is expressed as V_g/V_o, where V_g = shear force acting normal to the plane of the slab and V_o = the nominal punching shear strength of the slab. This trend is similar to that observed for columns with low transverse reinforcement ratio (Figure 13), and suggests that a similar model might be applied here. The model is shown conceptually in Figure 18. According to this model, lateral deformations are attributed to slab flexure only. Furthermore, the curvatures have the simple distribution illustrated here; note that the effects of gravity load moments are ignored in this assumption. For a slab reinforced with reinforcement having yield stress equal to 60 ksi, the yield rotation can be approximated by

$$\theta_y = \frac{\phi_y l}{6} \approx \frac{1}{2400}\frac{l}{h} \tag{1}$$

in which ϕ_y = yield curvature, l = center to center spacing of columns in direction of lateral loading, and h = slab thickness. A nominal rotational (or drift) ductility can therefore be defined by

$$\mu_\theta = \frac{\theta_u}{\theta_y} \tag{2}$$

in which θ_u = maximum rotational (or drift) demand (Figure 18).

Combining the nominal rotational (or drift) ductility demand expressed by Equation (2) and the relation between drift ductility demand and residual shear strength expressed by the line segments in Figure 13, one may project a hypothetical relation between rotation θ_u at failure and the direct gravity shear strength. The relation is a function of the slab l/h ratio. Figure 19 plots the projected results for 26 interior slab-column connections tested at various laboratories. (Basic test and calculation data are in Table 1.)

For the same 26 test specimens, Figure 20 plots the measured relations between drift angle at onset of punching shear failure and gravity shear ratio V_g/V_o. Although one-to-one correspondence between the analytical projections (Figure 19) and test results (Figure 20) is lacking, the trends are consistent. (Improved correspondence might be possible by including effects of gravity load moments, prestressing, and other effects in the idealization.) Slab-column connections having high gravity shear ratios are susceptible to punching shear failures at lower lateral drift ratios. Not surprisingly, connections designed as gravity-only elements tend in current practice to have high gravity shear ratios, and therefore are more prone to failure under imposed lateral deformations.

These trends are recognized in the recommendations of ACI-ASCE Committee 352 [34]. In addition to various detailing and proportioning rules, this document recommends that the gravity load shear at a connection should not exceed approximately $1.5A_{cs}\sqrt{f_c'}$, where A_{cs} = area of the slab critical section as defined by the ACI Building Code [6], and concrete strength f_c' is in psi. The purpose of this limit is to reduce the likelihood of punching shear failure at commonly expected lateral drift levels [24].

Experimentally Observed Behavior of Slab-Column Connections

Some recent experiments on reinforced concrete and post-tensioned slab-column specimens illustrate aspects of performance for slab-column connections. The reinforced concrete slab described below was designed according to current practice including some of the recommendations made earlier in this paper with regard to avoiding high gravity shears. The post-tensioned connection described below had a relatively high gravity shear ratio.

Tests [16] were carried out on a one-third scale model of a nine-panel reinforced concrete flat-plate frame to study behavior of frames designed to satisfy ACI Building Code [6] detailing requirements for frames in regions of moderate seismicity. Figure 21 depicts the geometry of the test slab. In addition to satisfying requirements of the ACI Building Code, the connections were proportioned to control the gravity shear ratio; the maximum gravity shear ratio for an interior connection was V_u/V_o = 0.3. Continuous bottom reinforcement was placed through the column cage at all connections to suspend the slab in the event that punch-through occurred. During the test, lateral deformation reversals were applied independently in each of the two principal directions in successively increasing increments while the gravity loads were maintained.

Figure 22a illustrates the measured load-displacement relation for the nine-panel test specimens for loading in NS direction prior to punching failure. The response is stable to lateral displacement drift ratio of approximately four percent of height. At this drift level, punching shear failures began at one of the interior connections and rapidly spread to the other interior and edge connections (but not the corners). Figure 22b displays the load displacement relation in the NS direction immediately before and following punching. As was visibly apparent during the test, the slab was able to sustain the vertical loading under continuing deformation reversals at four percent drift amplitude because of the catenary action provided by the bottom slab bars that were continuous through the column reinforcement cage.

Figure 23 shows geometry for a test specimen representing an interior connection of a post-tensioned flat-plate frame with strands banded in one direction and distributed in the orthogonal direction [21]. Tests were also carried out on edge and corner connections. The slabs had no bottom reinforcement through the column, and so relied on the post-tensioned steel to suspend the slab in the event of punch-through. Figure 24 shows the measured load-displacement relation for loading in the NS direction for the case where the gravity shear ratio was 0.55 (results obtained for loading in the orthogonal direction are not shown here). The specimen shows marginal lateral displacement capacity, followed by punch-through. Importantly, the post-tensioned reinforcement was adequate to suspend the slab following punching failure. Although this and other tests in the series show that the strands seem adequate for suspending the slab, some continuous bottom mild steel might still be placed in standard construction.

CONCLUSION

Design algorithms expressed in current building codes and practiced in design offices focus attention on earthquake induced lateral forces, and away from earthquake induced lateral displacements. These procedures have led to development of structural systems in which a portion of the structural frame is designed to resist the total seismic design force while a substantial remainder of the structure is proportioned assuming it resists only gravity loads. As the Northridge earthquake demonstrated, drift happens, and the engineer needs to design the entire building for it. Simple procedures, some described in this paper, can be used for this purpose.

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TABLE 1 — SLAB-COLUMN CONNECTION TEST DATA

Specimen Identifier	Reference	c_1	c_2	h	l_1	f'_c	V_g	V_g/V_o	Test Drift Ratio	Calc. Drift Ratio
Test 1	Pan and Moehle [31]	10.8	10.8	4.8	144.	4800	23.3	0.35	0.015	0.037
Test 2	"	10.8	10.8	4.8	144.	4800	23.3	0.35	0.015	0.037
Test 3	"	10.8	10.8	4.8	144.	4600	14.1	0.22	0.048	0.042
Test 4	"	10.8	10.8	4.8	144.	4600	14.1	0.22	0.032	0.042
S1	Hawkins [14]	12.0	12.0	6.0	144.	5100	28.8	0.33	0.038	0.030
S2	"	12.0	12.0	6.0	144.	3400	32.0	0.45	0.020	0.027
S3	"	12.0	12.0	6.0	144.	3200	31.2	0.45	0.020	0.026
S4	"	12.0	12.0	6.0	144.	4700	33.7	0.40	0.026	0.028
S6	Symonds [38]	12.0	12.0	6.0	144.	3400	61.0	0.86	0.011	0.014
S7	"	12.0	12.0	6.0	144.	3800	61.0	0.81	0.010	0.016
S1	Morison and Sozen [27]	12.0	12.0	3.0	72.	6600	1.4	0.03	0.047	0.039
S2	"	12.0	12.0	3.0	72.	5100	1.4	0.03	0.028	0.039
S3	"	12.0	12.0	3.0	72.	4900	1.4	0.04	0.042	0.039
S4	"	12.0	12.0	3.0	72.	5100	3.0	0.08	0.045	0.038
S5	"	12.0	12.0	3.0	72.	5100	6.4	0.17	0.048	0.035
SM0.5	Ghali [10]	12.0	12.0	6.0	72.	5300	29.0	0.31	0.060	0.015
SM1.0	"	12.0	12.0	6.0	72.	4800	29.0	0.33	0.027	0.015
SM1.5	"	12.0	12.0	6.0	72.	5800	29.0	0.30	0.027	0.016
INT	Zee [44]	5.4	5.4	2.4	72.	5400	3.6	0.21	0.033	0.042
IP2	Islam and Park [17]	9.0	9.0	3.5	108.	4600	6.4	0.18	0.050	0.044
IP3C	"	9.0	9.0	3.5	108.	4300	7.6	0.23	0.040	0.043
B7	Hanson and Hanson [13]	12.0	6.0	3.0	72.	4800	1.1	0.04	0.038	0.039
C8	"	6.0	12.0	3.0	72.	4800	1.3	0.05	0.058	0.039
1.	Martinez [21]	7.9	7.9	3.5	147.	4100	25.5	0.72	0.016	0.032
2.	"	7.9	7.9	3.5	147.	4100	23.5	0.65	0.021	0.036
3.	"	7.9	7.9	3.5	147.	4000	19.5	0.55	0.023	0.041

NOTE: All test specimens are conventionally reinforced except those reported by Martinez [21], which are of unbonded post-tensioned construction.

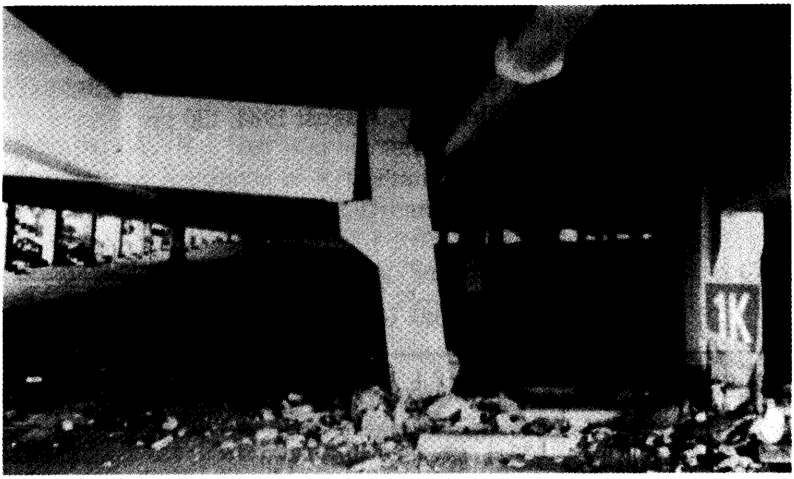


Fig. 1—Failure of gravity column in a parking structure

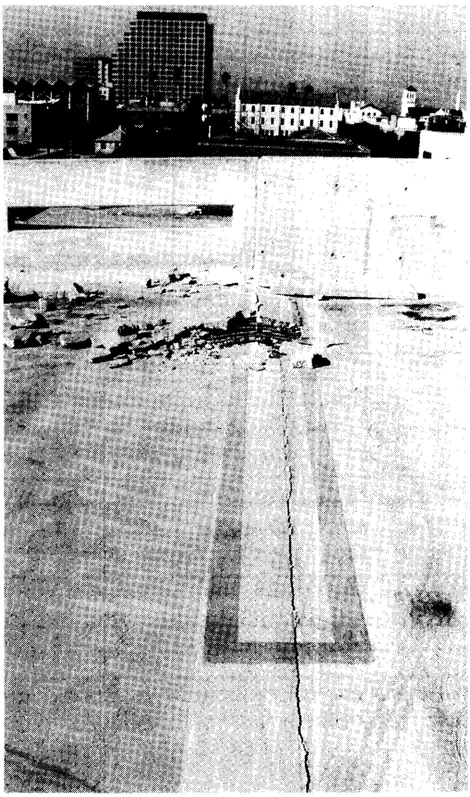


Fig. 2—Damage to topping slab in floor diaphragm

Fig. 3—Punching shear failure in post-tensioned slab-column frame

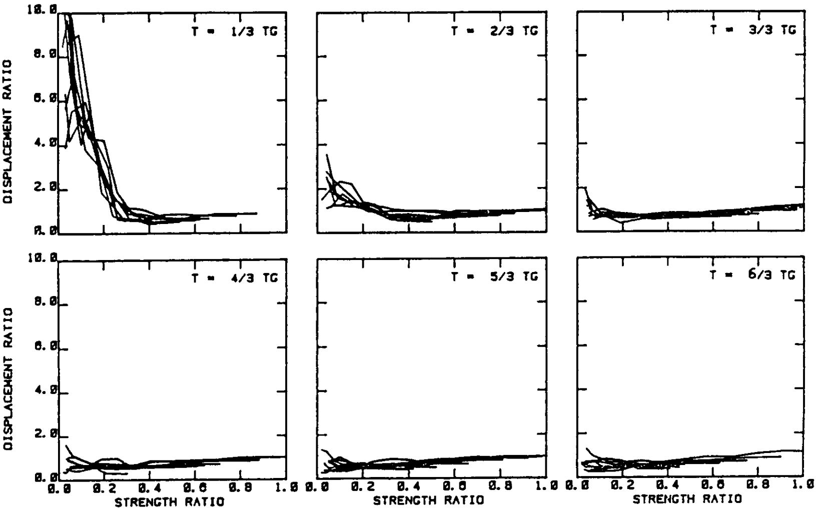


Fig. 4—Ratios of peak displacements for inelastic and elastic responses

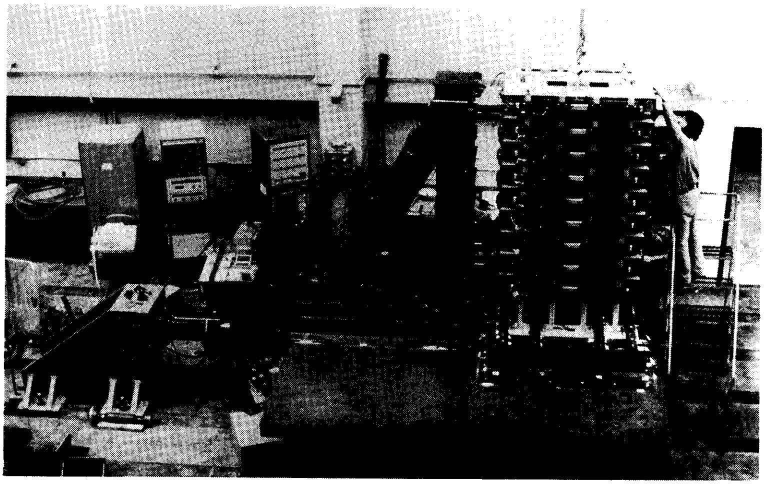


Fig. 5—One-tenth scale model of multi-story reinforced concrete test structure on a shaking table

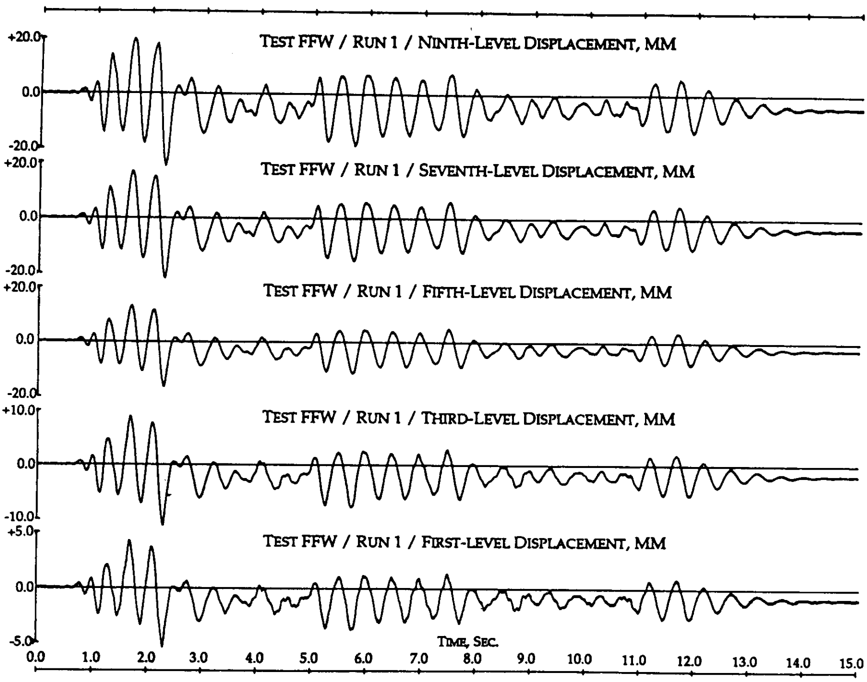


Fig. 6—Measured displacement responses for reinforced concrete test structure

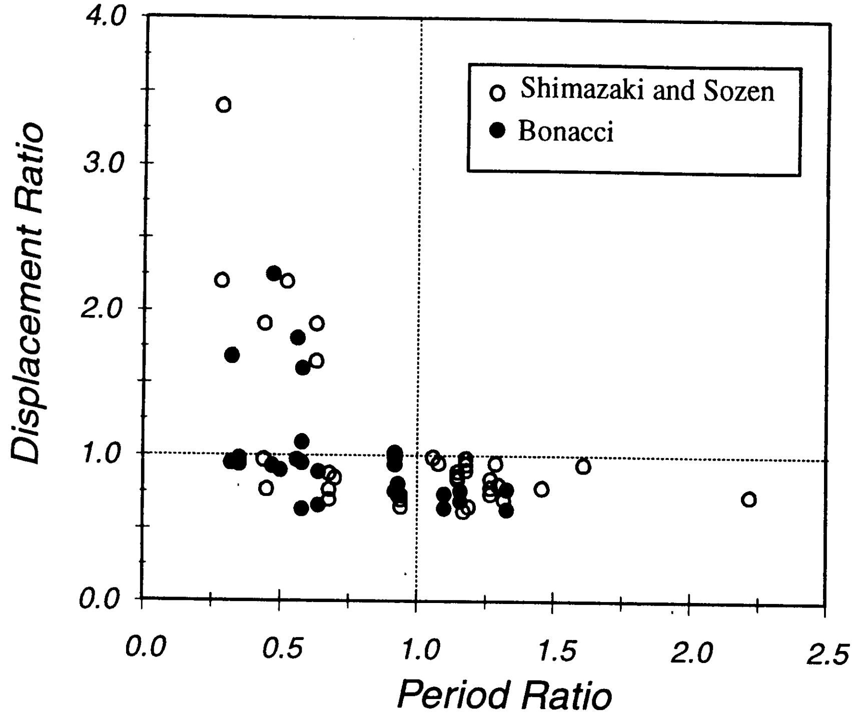


Fig. 7—Ratios of measured peak displacements to peak displacements calculated for the initial period

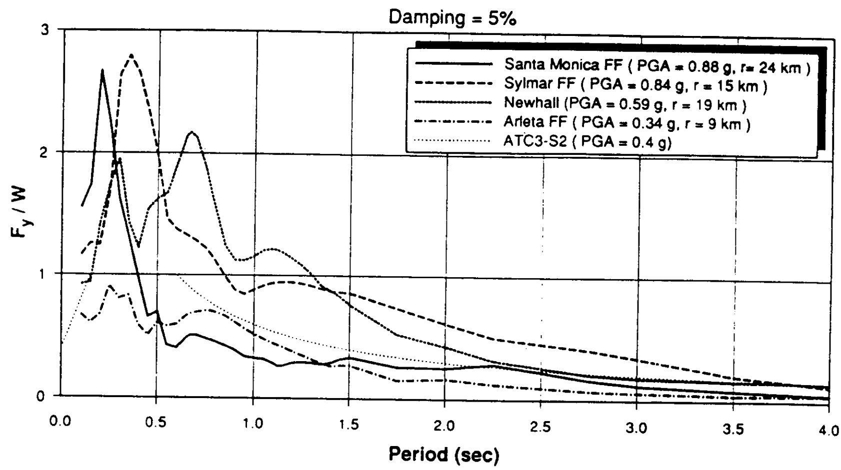


Fig. 8—Comparison of the UBC (39) design spectrum and selected linear elastic response spectra from the Northridge earthquake (12)

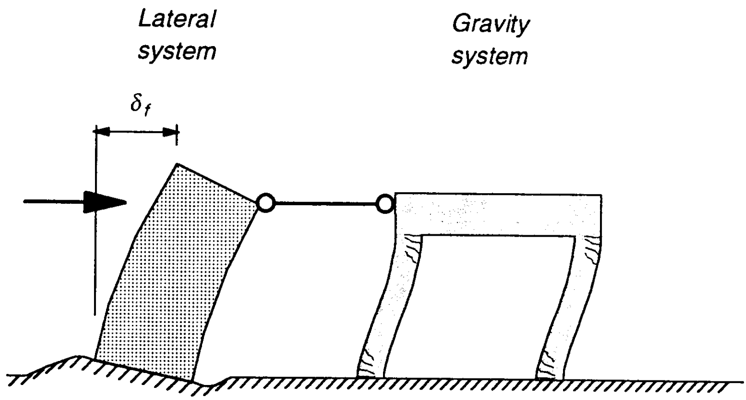


Fig. 9—Effect of foundation rotation on the deformations imposed on the gravity system

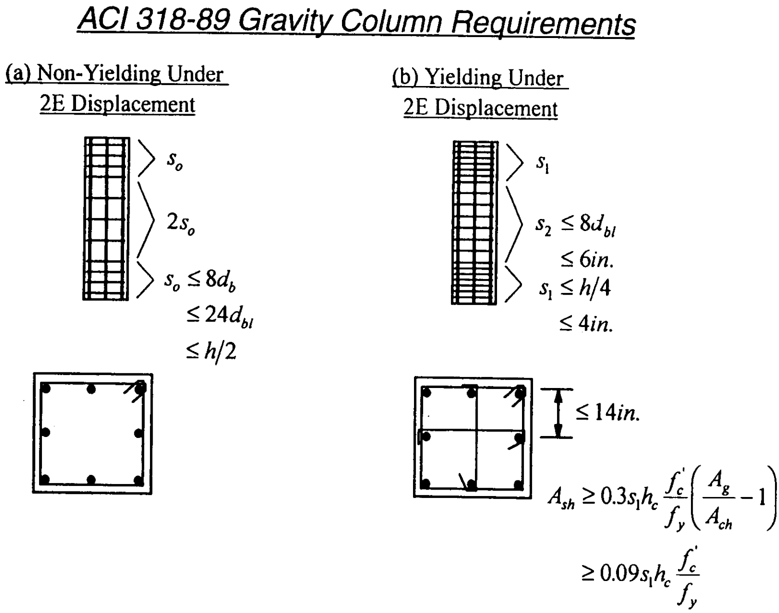
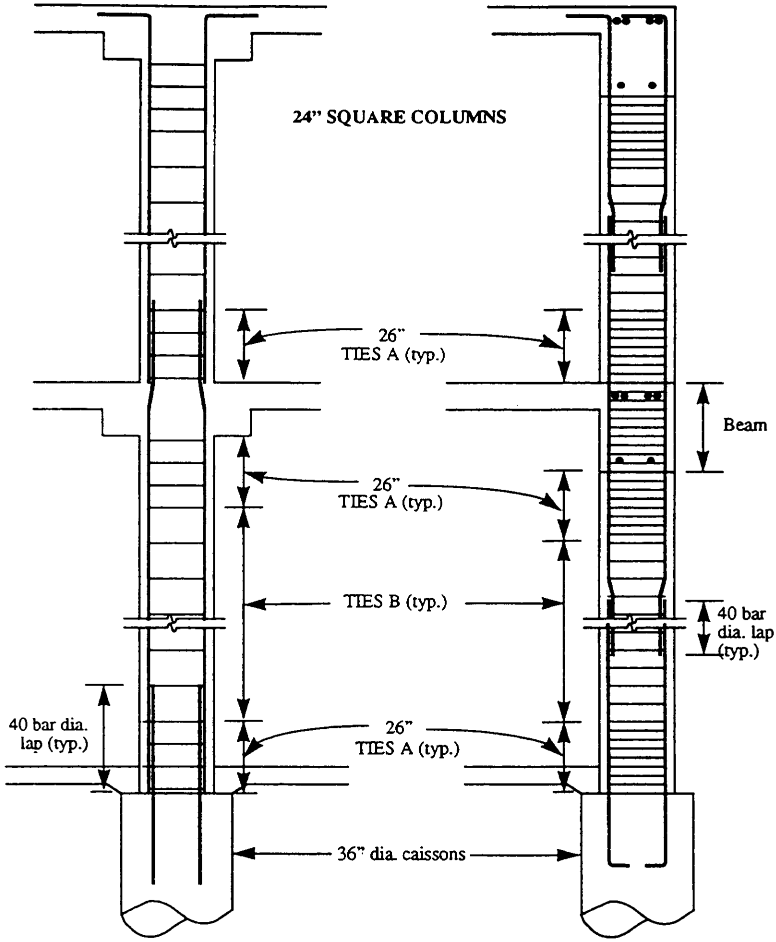


Fig. 10—Column reinforcement details required by the ACI Building Code (6)



INTERIOR COLUMN

Ties A: #3 @ 8"
Ties B: #3 @ 12"
Longitudinal Bars @ Base: 6 - #10
Longitudinal Bars @ Roof: 4 - #9

EXTERIOR COLUMN

Ties A: #4 @ 4"
Ties B: #4 @ 4" to 6"
Longitudinal Bars @ Base: 12-#11+ 8-#10
Longitudinal Bars @ Roof: 8 - #8

Fig. 11—Details of building in southern California reported by Dovitch and Wight (8)

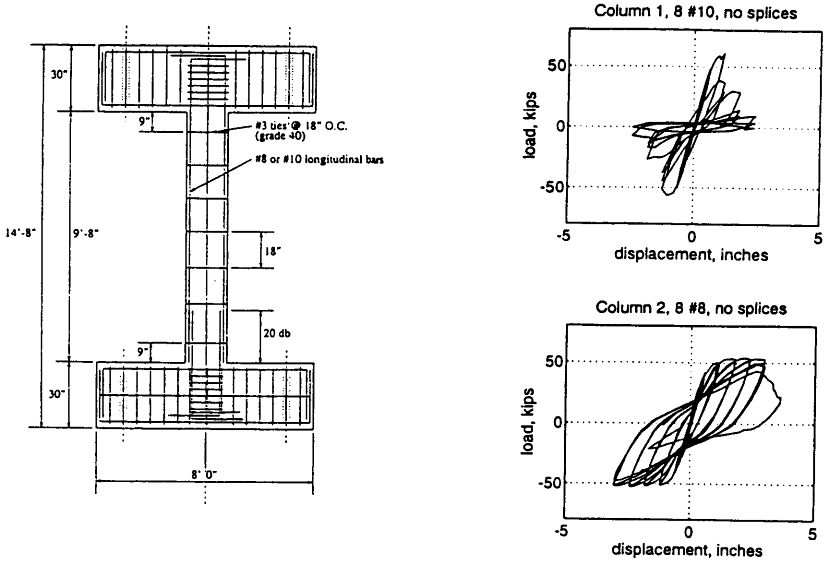


Fig. 12—Results of laboratory tests on reinforced concrete columns with details satisfying nonseismic requirements

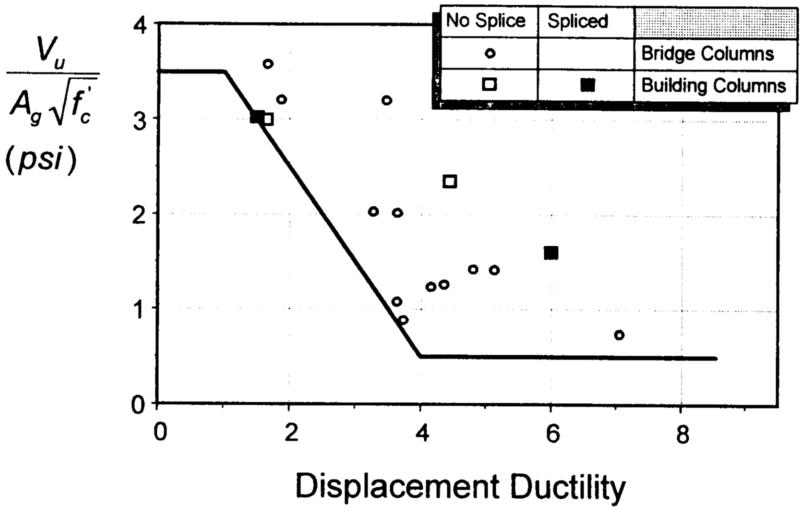


Fig. 13—Trend of decreasing shear strength with increasing imposed displacement ductility for tied columns with low amount of transverse reinforcement

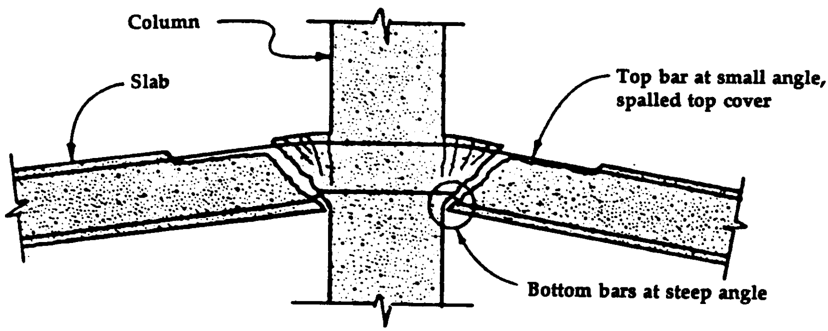


Fig. 14—Role of continuous bottom bars through the column reinforcement cage

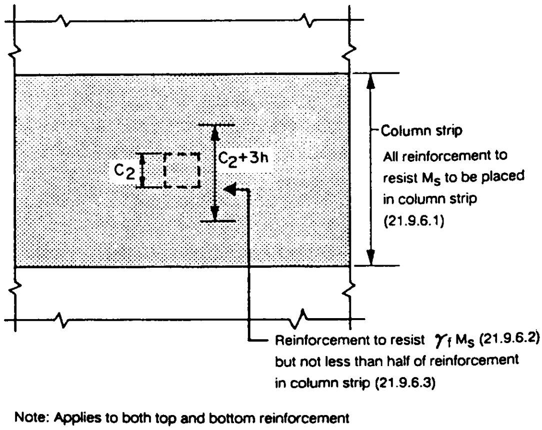
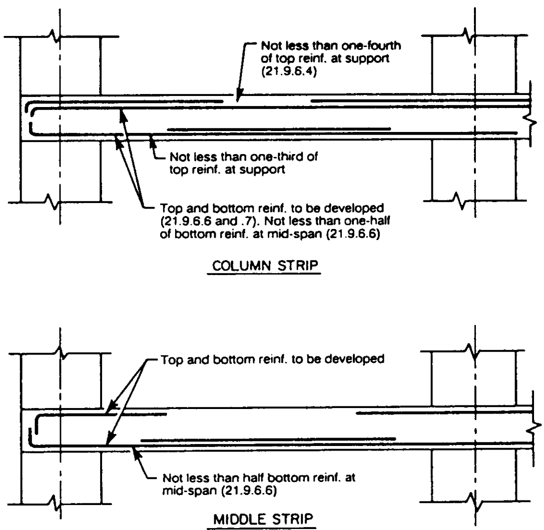


Fig. 15—Required details for slab-column connections in regions of moderate seismicity (6)



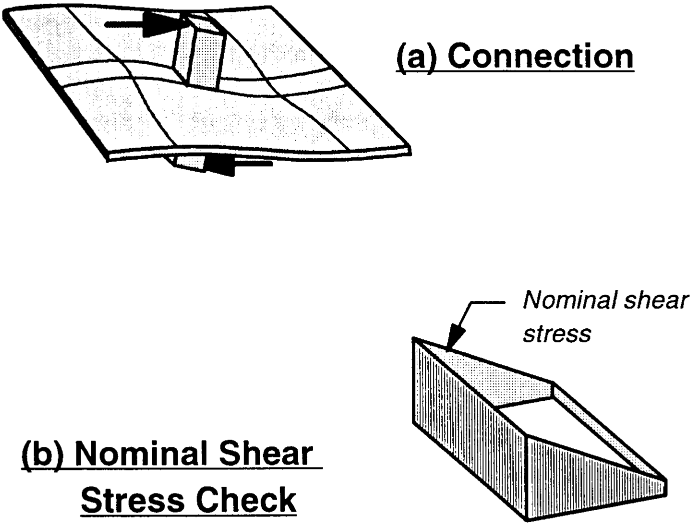


Fig. 16—Application of shear and moment transfer procedure to laterally deformed slab-column connection

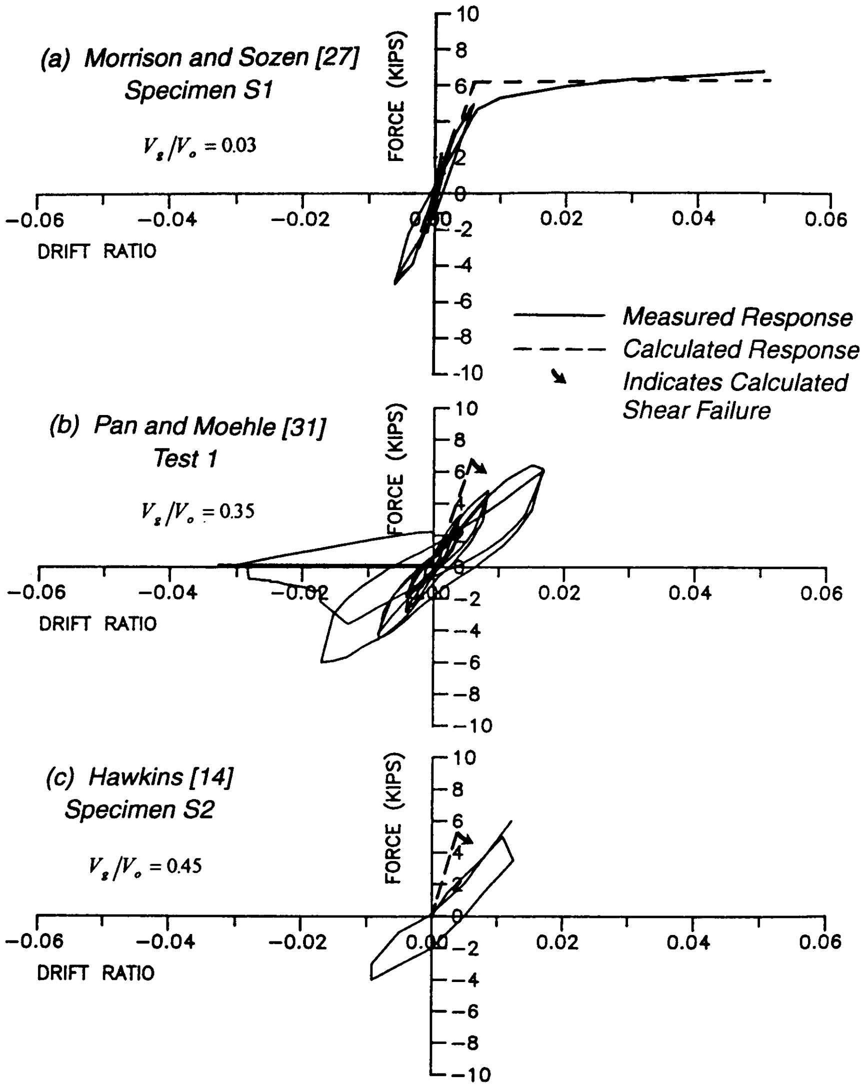


Fig. 17—Behavior of three reinforced concrete slab-column connections with varying gravity loads

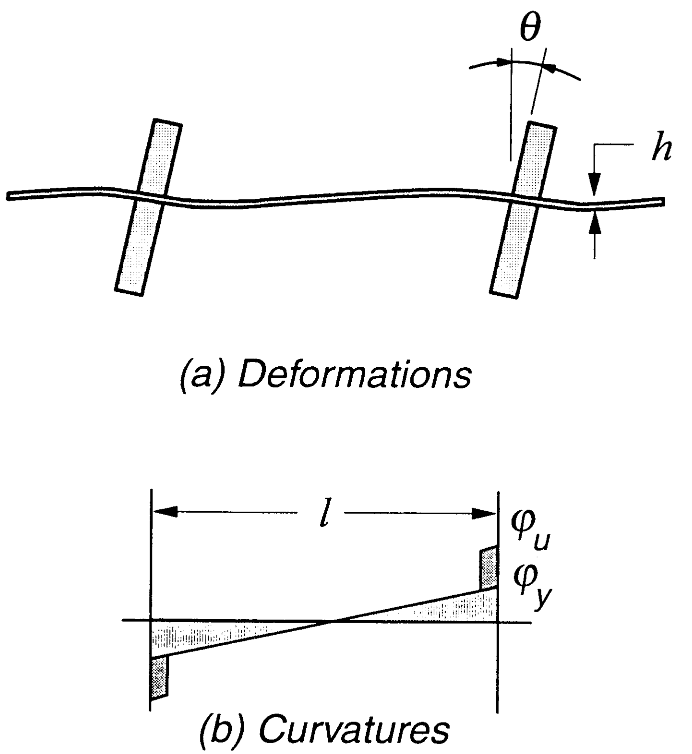


Fig. 18—Simplified model of flexural deformation in a slab

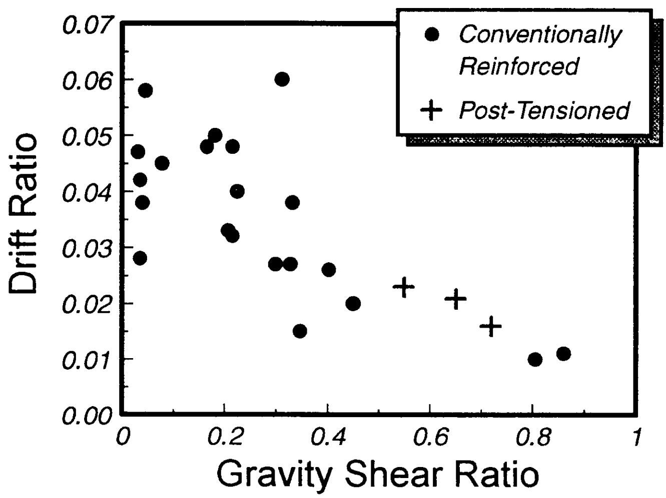


Fig. 19—Projected relation between lateral drift at failure and gravity shear ratio for 26 tests, based on model in Fig. 18 and the trend line in Fig. 13

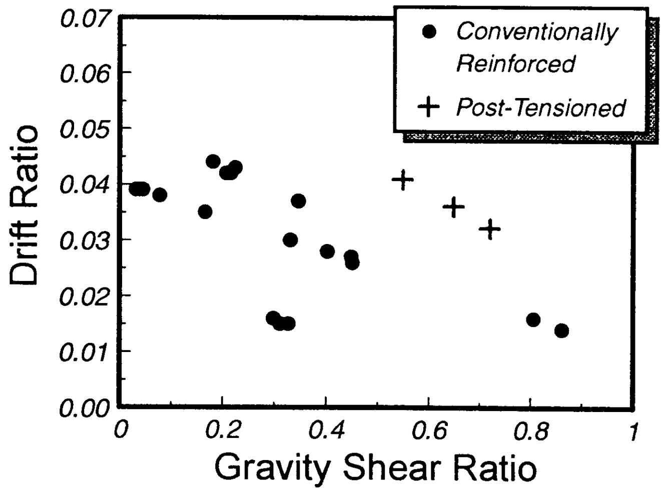


Fig. 20—Measured relation between lateral drift at failure and gravity shear ratio for 26 tests

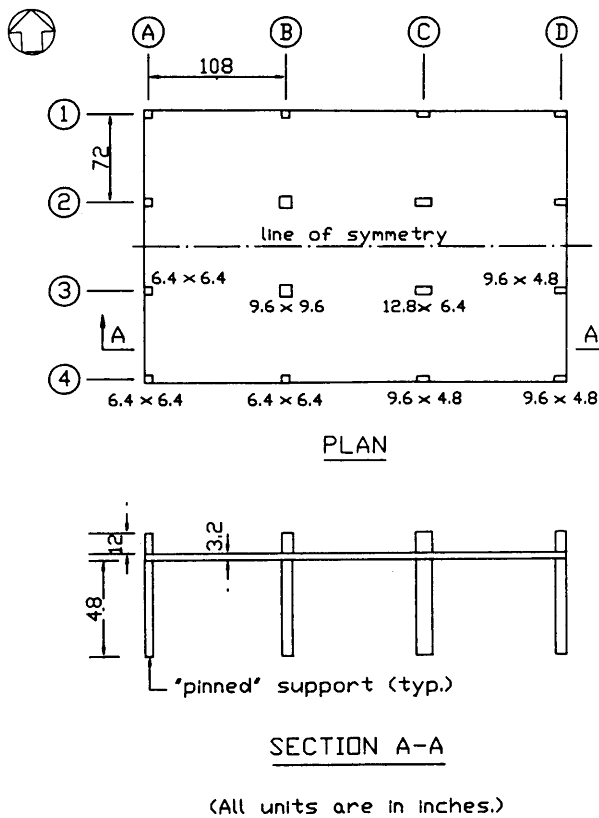
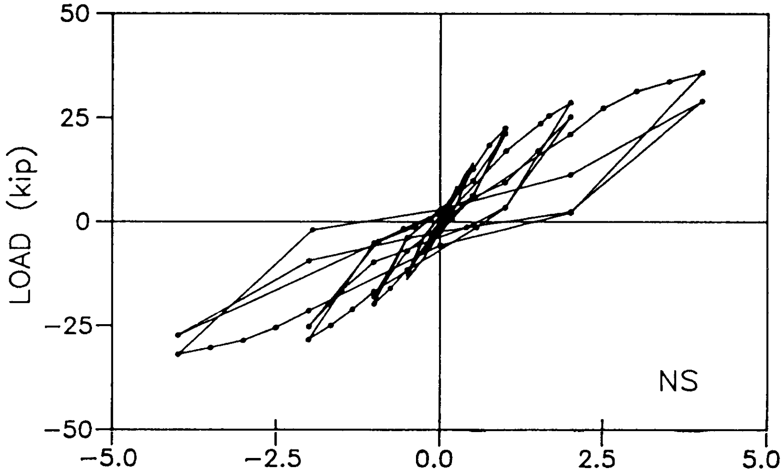
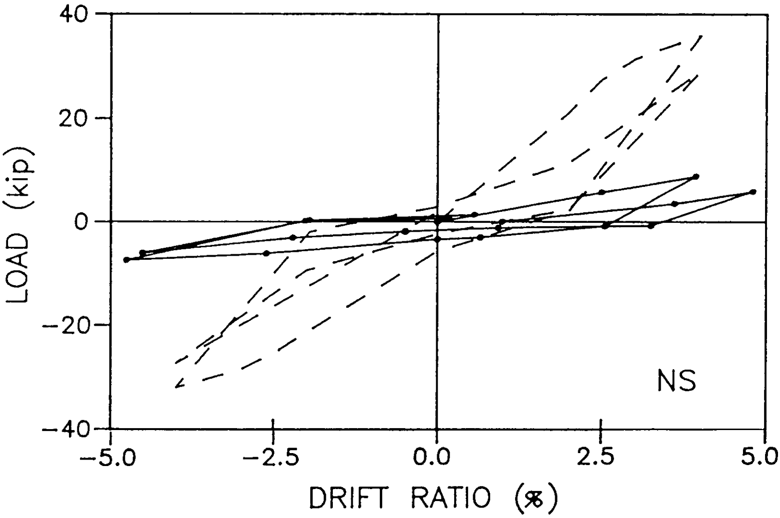


Fig. 21—Geometry of nine-panel reinforced concrete flat plate test specimen



a) Behavior prior to punching failures



b) Behavior immediately before (broken curve) and after (solid curve) punching

Fig. 22—Measured load-displacement relation for the nine-panel test specimen

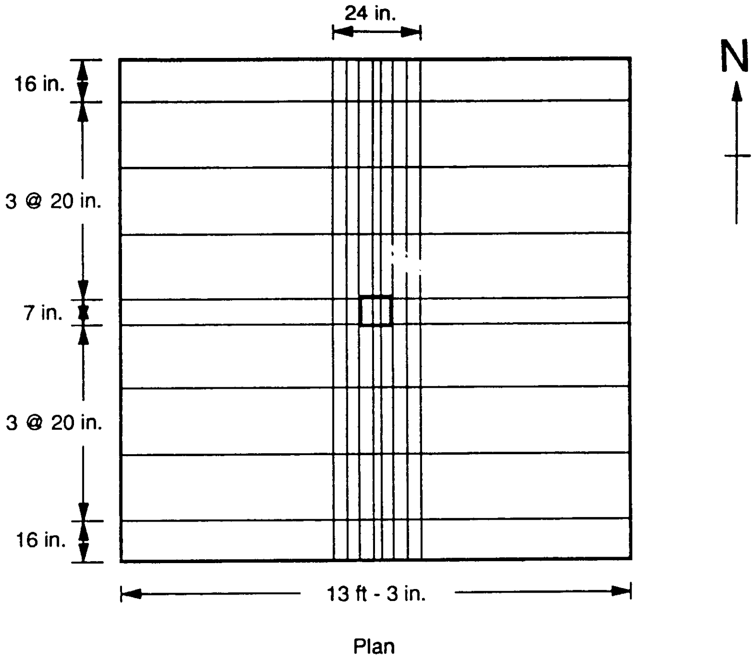


Fig. 23—Geometry of interior post-tensioned slab-column connection

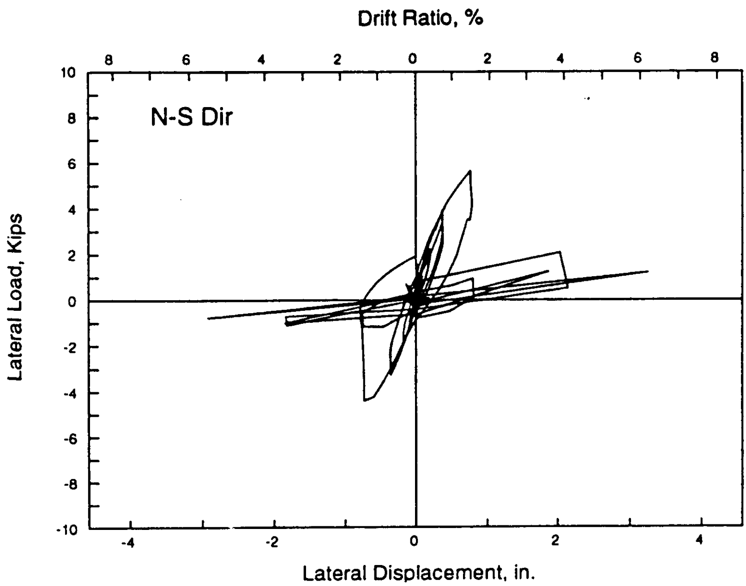


Fig. 24—Measured load-displacement relation for the interior post-tensioned slab-column connection