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
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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 0.3\(f_c A_y\), where \(f_c\) = concrete compressive strength and \(A_y\) = 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 $3/8b$) 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, $V_g/V_p$, where $V_g$ = the total shear due to gravity loads acting normal to the plane of the slab and $V_p$ = 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 $\alpha\beta I_p$, where $\alpha$ = an effective width coefficient [32], $\beta$ = a reduction factor of 1/3 to account for slab cracking [23], and $I_p$ = 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 $\frac{V_J}{V_o}$ where $V_J = \text{shear force acting normal to the plane of the slab}$ and $V_o = \text{the nominal punching shear strength of the slab}$. 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

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

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

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

in which $\theta_u = \text{maximum rotational (or drift) demand}$ (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 $\theta_u$ 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 \( V_f/V_o \). 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 \( 1.5A_s\sqrt{f_c} \), where \( A_s \) = 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 shear. 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 \( V_f/V_o = 0.3 \). 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 punching failure. 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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REFERENCES


6. "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318R-89)," reported by ACI Committee 318, American Concrete Institute, Detroit, Michigan, 1989.


29. Otani, S., "Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis," *Journal of the Faculty of Engineering (B)*, University of Tokyo, Tokyo, Japan, Vol. 36, No. 2, pp. 125-159.


**TABLE 1 — SLAB-COLUMN CONNECTION TEST DATA**

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<th>$l_1$</th>
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**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

**ACI 318-89 Gravity Column Requirements**

(a) Non-Yielding Under 2E Displacement

- \( s_o \)
- \( 2s_o \)
- \( s_o \leq 8d_b \)
- \( \leq 24d_{pl} \)
- \( \leq h/2 \)

(b) Yielding Under 2E Displacement

- \( s_i \)
- \( s_i \leq 8d_{pl} \)
- \( \leq 6\text{in.} \)
- \( s_i \leq h/4 \)
- \( \leq 4\text{in.} \)

\[
A_{sh} \geq 0.3s_i h_t \frac{f_e}{f_y} \left( \frac{A_g}{A_{ch}} - 1 \right) \\
\geq 0.09s_i h_t \frac{f_e}{f_y}
\]

Fig. 10—Column reinforcement details required by the ACI Building Code (6)
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
All reinforcement to resist $M_2$ to be placed in column strip (21.9.6.1)

Reinforcement to resist $\gamma_1 M_2$ (21.9.6.2) but not less than half of reinforcement in column strip (21.9.6.3)

Note: Applies to both top and bottom reinforcement

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
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
Load-Deformation Relationship of High-Strength Reinforced Concrete Beams

by S. Otani, S. Nagai, and H. Aoyama

Synopsis: Force-deformation relationship of high-strength reinforced concrete beam members observed in the laboratory test was idealized by a trilinear relation for use in a nonlinear earthquake response analysis. Methods to evaluate the relationship were examined and the reliability of the methods were discussed with respect to the observed relations. Calculated initial stiffness is shown to significantly underestimate the observed value; a large coefficient of variation was attributed to accidental and shrinkage cracking in the specimen prior to the test. A similar large coefficient of variation was observed in the evaluation of cracking moment. Yield and ultimate moments could be favorably estimated by the theory. An empirical formula was proposed to evaluate yield deformation. An importance of controlling the elastic modulus of concrete in construction is emphasized if a structure is expected to behave as designed during an earthquake.

Keywords: Beams (supports); cracking (fracturing); deformation; high-strength concretes; modulus of elasticity; nonlinear response; reinforced concrete; ultimate moments; yield point
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INTRODUCTION

The demand for high-rise reinforced concrete (RC) buildings has accelerated the research on the use of high strength materials. Different characteristics of high-strength RC members are anticipated. The production of high strength materials requires special quality control. Therefore, the Ministry of Construction, Japanese Government, organized a national project for the "Development of Advanced Reinforced Concrete Buildings Using High-strength Concrete and Reinforcement (New RC Project)" from 1988 to 1993. The material strength ranged from 30 to 120 MPa for the concrete and from 400 to 1,200 MPa for the longitudinal reinforcement. High-strength lateral reinforcement of yield stresses ranging from 600 to 1,300 MPa is commonly used in Japan. Methods to evaluate the force-deformation relationship of RC beams were examined.

Member end moment-rotation relation of an RC member varies with damage associated with the intensity and distribution of moment along the member and loading history. A nonlinear earthquake response analysis of a building requires two types of mathematical modeling: (a) modeling of the distribution of stiffness (damage) along a member and (b) modeling of the force-deformation relation under stress reversals. A common and simple model to idealize the stiffness distribution along a member is to assume all inelastic deformation to concentrate at the
member ends, ignoring distributed cracks along the member; the member-end rotation of the model is evaluated for a given member end moment assuming a linear moment distribution along the member with the inflection point at the mid-span. A hysteresis model works on the moment-rotation relation under monotonically increasing loading. An RC beam is designed to behave dominantly in flexure, being prevented from failure in less ductile modes. Therefore, the moment-rotation relation of an RC member was assumed to change its stiffness at flexural cracking of concrete and tensile yielding of longitudinal reinforcement at the critical section.

DATABASE OF RC BEAM TESTS

Test data of beams, yielding in flexure before failure, were searched from literature (Ref. 1-22) between 1982 and 1992 in Proceedings of Japan Concrete Institute (JCI), Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan (AIJ), Annual Reports of the New RC Project, and reports of research institutes of Japanese construction companies. The specimens must satisfy the following conditions; (a) rectangular cross section, (b) same amount of top and bottom longitudinal reinforcement, (c) width wider than 150 mm, and (d) overall depth deeper than 225 mm. Among 105 beam specimens obtained, the concrete strength ranged from 26 to 98 MPa; the shear span-to-depth ratio from 1.0 to 3.6; the tensile reinforcement ratio from 0.44 to 2.70 percent; the yield strength of longitudinal reinforcement from 261 to 976 MPa.

The specimens were tested under two types of loading methods (Fig. 1) simulating the conditions during earthquake excitation; i.e., (a) Type-A test (31 specimens): statically indeterminate beams with two stiff end stubs subjected to lateral displacement at the two ends maintaining the end stubs in parallel during loading, and (b) Type B test (74 specimens): simply supported beams, normally with two loading stubs, subjected to two-point loading causing the point of inflection at the center of the middle span. In Type A test, moment distribution is not known due to statical indeterminacy; linear moment distribution is assumed with an inflection point at the mid-span. In Type B test, the damage within the test span tends to concentrate at an end during the test. Simply supported specimens subjected to mid-span loading were not selected for the study.

The load-deformation relation curves under reversed cyclic loading were obtained from the generous researchers, and were digitized at the
University of Tokyo for the sequence of parts where the load exceeded the maximum load of the previous loading cycles. The error of digitization is less than 0.3 percent, on the average 0.2 percent, of the full scale.

IDEALIZATION OF MOMENT-ROTATION RELATION

The member-end moment was obtained from shear force measured in the test span by assuming the inflection point at the mid-span. The member-end rotation of Type A specimens was calculated from the measured relative lateral displacement divided by the clear span, but the member end rotation of Type B specimens was used as the researchers reported. The observed moment-rotation curve was idealized into a trilinear relationship (Fig. 2).

Although many specimens failed in shear or bond-splitting modes after flexural yielding, the initial stiffness may not be affected by the failure modes. Therefore, all 105 specimens were used in the study of initial stiffness and flexural cracking moment. On the other hand, the yield deflection is increased by the damage associated with the failure modes. Therefore, those specimens failing in shear or bond-splitting modes within a deflection equal to three times flexural yielding deflection were excluded from the examination of yield and ultimate points; 38 specimens were used to study the yield deflection.

The initial stiffness in a test was defined as a secant slope at a load equal to one-half of the reported cracking load. The cracking load was not reported in five specimens, hence the cracking moment was evaluated by assuming the tensile strength of concrete to be $0.56\sqrt{f_{c}}$ (Ref. 23). Note that the cracking load is normally reported at a loading step when cracking is detected for the first time; i.e., the reported cracking load is normally higher than the actual cracking load. Therefore, the cracking point was determined from the shape of force-deformation relation by the method described in the following paragraph.

The stiffness of reinforced concrete section changes drastically at the yielding of tensile reinforcement. If tensile reinforcement is placed in double layers in the section, the stiffness changes at the yielding of the outer layer reinforcement and then of the inner layer reinforcement. In order to select a single yield point, the yield point and cracking moment were defined such that the energy stored at the ultimate deformation should be the same for the test and the model making the absolute difference in the energy to be minimum (Fig. 2).
Resonance at the ultimate point was taken as the observed maximum response. The determination of an ultimate deformation is an important but possible issue; the ultimate deformation is not a unique value but highly dependent on the progress of concrete deterioration dictated by loading history and failure modes. Small stiffness after yielding will not change appreciably by the choice of an ultimate deformation. Therefore, the ultimate deformation was selected to be an arbitrary deformation at a deformation ductility factor of four.

An iterative procedure was used to define the yield point and cracking moment for the established initial stiffness and ultimate point; i.e., (a) a trial yield displacement was assumed, (b) an ultimate displacement was selected at four times yield deformation, (c) post-yield stiffness was determined by connecting the ultimate point and a point on the observed curve at 2.5 times yield deformation, and (d) the cracking moment and yield deformation were determined for equal absorbed energy at the ultimate deformation and minimum absolute difference.

**INITIAL STIFFNESS**

The methods to evaluate stiffness parameters and their reliability with respect to the observed values are discussed. The initial elastic stiffness $K_E$ was evaluated by the elastic theory of a lineal member considering flexural and shear deformation:

$$\frac{1}{\psi} = \frac{1}{K_f} + \frac{1}{K_s}$$

(Eq. 1)

where $K_f$: flexural stiffness ($= 6 E_c I_e / L$), $K_s$: shear stiffness ($= G_c A / (2 \kappa)$), $E_c$: elastic modulus of concrete, $I_e$: moment of inertia of uncracked transformed section, $L$: member length, $G_c$: shear modulus of concrete, $A$: cross sectional area, $\kappa$: shape factor for shear deformation ($= 1.2$). The shear modulus of concrete was estimated from the elastic modulus $E_c$ and assumed Poisson's ratio of 0.20. Elastic modulus $E_s$ of steel was assumed to be 206 GPa.

The initial stiffness was calculated using the observed elastic modulus of concrete and the clear span. The observed initial stiffness (Fig. 3) was notably lower and, on the average, 0.53 times that calculated with a large coefficient of variation ($= 0.51$) for 73 specimens with reported concrete elastic moduli. The large coefficient of variation and discrepancy between the test and calculation was probably attributed to (a) technical difficulty in measuring accurate initial stiffness in the test and (b)
formation of accidental and shrinkage cracks prior to the test. In a real structure, flexural cracks under gravity loading, shrinkage cracks, cracks after medium intensity earthquake excitation may exist, and the initial stiffness for the analysis is difficult to estimate. However, the initial stiffness and cracking force level of a single-degree-of-freedom system do not influence the maximum response amplitude as long as the attained response ductility factor reached more than 4 (Ref. 24). The response in this range is more sensitive to the secant stiffness and the resistance at yielding rather than the initial stiffness.

Furthermore, the actual elastic modulus in a structure, as built, is not known at the time of structural design although the initial stiffness of a member is directly dependent on the modulus. The elastic modulus of concrete is normally difficult to control in construction. Therefore, the initial stiffness of a structure can be significantly different from the value assumed by a structural engineer.

An empirical expression was proposed for the elastic modulus $E_c$ (Ref. 25), taking into account compressive strength and density of concrete, type of coarse aggregates and mineral admixture;

$$E_c = k_1 \times k_2 \times 3.35 \times 10^4 \times (\sigma_{u} / 60)^{0.3} \times (\gamma / 2.4)^2 \text{ (MPa)} \quad (Eq. 2)$$

in which $k_1$: factor representing type of coarse aggregates, $k_2$: factor representing kind of mineral admixture, $\sigma_{u}$: observed concrete strength (MPa), $\gamma$: unit density of concrete (ton/m$^3$). The factor $k_1$ is 0.95 for crushed quartzite, crushed andesite, basalt and clayslate aggregates, 1.0 for other coarse aggregates, and 1.2 for crushed limestone and calcined bauxite aggregates. Factor $k_2$ is 0.95 for silica fume powder of blast furnace slag and fly ash fume, 1.00 for concrete with mineral admixture or with other mineral admixture, 1.10 for fly ash. Ninety-five percent of test data are shown to fall within 20 percent of the empirical expression (Fig. 4). The modulus should be controlled in construction within an acceptable range from the value specified by the structural engineer.

**CRACKING MOMENT**

Cracking moment $M_{cr}$ is calculated on the basis of the observed splitting tensile strength $\sigma_t$ of concrete and the section modulus $Z_e$ of the uncracked transformed section. The ratio of the reported to the calculated cracking moments is compared in Fig. 5 for 59 specimens; the average ratio was unexpectedly good at 1.03, with a significantly large
coefficient of variation of 0.50.

Cracking tensile strength $\sigma_{cr}$ of concrete was determined by dividing the cracking moment of the trilinear idealization of the moment-rotation relation by the section modulus $Z_e$ of uncracked transformed section. The cracking tensile strength and compressive strength of concrete are compared in Fig. 6 for 68 specimens. A wide scatter of data can be observed, but a tendency is observed for the cracking tensile strength to increase with the compressive strength. Following empirical relation was derived:

$$\sigma_{cr} = 1.26 \times \sigma_{\mu}^{0.45}$$  \hspace{1cm} (Eq. 3)

The cracking tensile strength should not be used as the tensile strength of concrete, but is intended to evaluate a moment level at which the initial elastic stiffness of the trilinear idealization changes in the member-end moment-rotation relationship.

**YIELD MOMENT AND ROTATION**

Yield moment at the critical section was calculated for the yielding at an imaginary centroid of tensile reinforcement. The amount of tensile reinforcement is normally limited well below the balanced tensile reinforcement ratio; hence, the stress-strain relation of the concrete was assumed to remain linearly elastic when the tensile reinforcement first yielded under bending. In addition, the following assumptions were made in calculating yield moment; i.e., (a) plane section remained plane after deformation, and (b) the concrete in tension did not resist tensile stresses. The elastic modulus of concrete was determined by Eq. 2 with $k_1=k_2=1.0$ and $\gamma = 2.4 \text{ ton/m}^3$, while the compressive strength of concrete and the yield stress of reinforcement were obtained from the reported material tests. In 2 specimens out of 38, the calculated stress at the extreme compressive fiber exceeded the compressive strength of concrete; these specimens were removed from the study.

The calculated yield moment and the estimated yield moment of the trilinear idealization are compared in Fig. 7 for 36 specimens. The average ratio of the estimated to the calculated yield moment is 1.12 with a coefficient of variation of 0.078; only one estimated yield moment was smaller (0.94) than the calculated value. The yield moment at which the stiffness of an RC member changes drastically may be calculated conservatively by the flexural analysis.
Member end rotation at flexural yielding was calculated using cracked transformed section for flexural deformation \( (\theta_f = M_p L / 6E_c I_{cr}) \), elastic stiffness for shear deformation \( (\theta_s = 2k \cdot M_y / G A_L) \) and pullout deformation \( \theta_{slip} \) of longitudinal reinforcement from the anchorage. The pull-out deformation \( \delta_{slip} \) was calculated by the following expression (Ref. 26):

\[
\delta_{slip} = \varepsilon_y (2 + 3,500 \varepsilon_y) \cdot d_b / (\sigma_y / 20)^{2/3}
\]  

(Eq. 4)

where, \( \varepsilon_y \): yield strain of longitudinal reinforcement, \( d_b \): diameter of longitudinal reinforcement, \( \sigma_y \): concrete strength (MPa). The center of rotation at the critical section was assumed to be at the centroid of compressive reinforcement; i.e., \( \theta_{slip} = \delta_{slip} / (d - d_c) \), where \( d \): effective depth and \( d_c \): depth to the centroid of compressive reinforcement.

The calculated yield rotation and the estimated yield rotation of the trilinear idealization are compared in Fig. 8.a for 36 specimens. The estimated rotation was, on the average, 2.54 times larger than the calculated rotation with a coefficient of variation of 0.22. Calculated yield rotation significantly underestimates the estimated rotation. This discrepancy is attributable to the additional rotation caused by shear cracking and the error in evaluating the pull-out deformation of longitudinal reinforcement from the anchorage.

An empirical expression was derived for the yield deformation by assuming the yield rotation at a member end consists of the rotations from flexural deformation, \( \theta_f \), shear deformation, \( \theta_s \), and deformation, \( \theta_{slip} \) due to the pull-out of longitudinal reinforcement from the anchorage zone. A regression analysis with respect to the estimated yield rotation was carried out to determine coefficients:

\[
\theta_y = 1.15 \theta_f + 12.6 \theta_s + 3.89 \theta_{slip}
\]  

(Eq. 5)

The calculated yield deflection and the observed yield rotation of the trilinear idealization are compared in Fig. 8.b for 36 specimens. A coefficient of variation of the ratio was 0.20 with a mean of 1.0.

**ULTIMATE MOMENT**

Flexural strength of beam section is not sensitive to the shape of stress-strain relationship nor the compressive strength of concrete because the
neutral axis depth is so small that the distance between the resultant compressive and tensile forces cannot change appreciably within the section. The ultimate moment was calculated using the plasticity theory suggested by Eberhard and Sozen (Ref. 27), in which the flexural mechanism was assumed to form by the yielding of tensile reinforcement followed by the compressive failure of concrete. Instead of strain compatibility, equilibrium conditions for axial force and bending moment of section were used based on the lower bound theorem. The maximum bending resistance was sought by satisfying the yield criteria of the materials and is given by

\[ M_u = k \cdot \sigma_c \cdot b \cdot d_c^2 + a_t \cdot \sigma_y (d - d_c) \]  
(Eq. 6)

in which, \( \sigma_c \): compressive strength of concrete, \( b \): width of section, \( d_c \): distance from the extreme compressive fiber to the centroid of compressive reinforcement, \( a_t \): area of tensile reinforcement, \( \sigma_y \): yield stress of tensile reinforcement, \( d \): effective depth of section. Values of \( k \) are \( 3/8 \) and \( 1/2 \) for triangular and rectangular stress distribution of compressive concrete with maximum stress of \( \sigma_c \), respectively. The stress in the compressive reinforcement must be checked not to exceed the yielding stress.

The ratio of the observed to the calculated ultimate moments is compared with respect to compressive strength of concrete in Fig. 9 for 38 specimens; the average ratio is 1.15 with a coefficient of variation of 0.074 for the triangular concrete stress block, and the average of 1.13 with a coefficient of variation of 0.069 for the rectangular stress block. The shape of a stress block shape should be carefully selected in the evaluation of ultimate moment of a column.

CONCLUDING REMARKS

Force-deformation relationship of high-strength reinforced concrete beam members observed in the laboratory test was idealized by a trilinear relation for use in a nonlinear earthquake response analysis. Methods to evaluate the relationship were examined and the reliability of the methods were discussed with respect to the observed relations.

Calculated initial stiffness is shown to significantly overestimate the observed value; a large coefficient of variation was attributed to accidental and shrinkage cracking in the specimen prior to the test. Similar large coefficient of variation was observed in the evaluation of
cracking moment. Yield and ultimate moments could be favorably estimated by the theory. An empirical formula was developed to evaluate yield deformation.

For a structure to behave, in an elastic range, as the structural engineer envisions in the design, the elastic modulus of concrete must be carefully and duly controlled.

ACKNOWLEDGMENT

The writers express their deepest gratitude to the authors of papers (Refs. 1-22) for generously providing the valuable test data. This research was originated as a part of Structures Committee work of the New RC project, and further extended after the project. Figure 4 is generously provided by Dr. T. Noguchi, research associate, Department of Architecture, University of Tokyo.

REFERENCES


23. Sugano, S., "Experimental study on restoring force characteristics of reinforced concrete members (in Japanese)," a thesis submitted to the University of Tokyo for a partial fulfillment of the requirements for doctor of engineering degree, University of Tokyo, December 1970.


a) Type A test

b) Type B test

Fig. 1—Loading methods in laboratory

Fig. 2—Idealization of observed moment-rotation relation
Fig. 3—Reliability of calculated initial stiffness

Compressive Strength of Concrete (MPa)

Observed to Calculated Initial Stiffness

mean = 0.53
cov = 0.51

Fig. 4—Elastic modulus and strength of concrete (Noguchi)

Compressive Strength of Concrete (MPa)

Observed to Calculated Modulus of Elasticity
Fig. 5—Reported to calculated cracking moment

Fig. 6—Cracking tensile strength and compressive strength of concrete
Fig. 7—Calculated and observed yield moment

Fig. 8—Calculated and observed yield rotation
Fig. 8 cont.—Calculated and observed yield rotation

Fig. 9—Observed and calculated ultimate moment (triangular stress block)
An Expert System for Reinforced Concrete Structural Damage Quantification

by P. Gülkan and A. Yakut

Synopsis: Objective evaluation of structural damage in buildings which have been subjected to strong ground motions is an undertaking where expert knowledge and the ability to process correlated but fuzzy information in a consistent way must be blended. Often, in the immediate aftermath of earthquakes field data is collected by survey teams whose expertise is variable. The use of knowledge-based systems capable of reaching an unequivocal decision on the damage state of a given building on the basis of queries arranged in a consistent hierarchical order would remove human subjectivity. This paper describes the internal design of an expert system called EPEDA which is used as a tool for making a numerical ranking of damage in reinforced concrete buildings. Damage to individual elements is quantified on the basis of severity, relative member importance and number of affected elements. Factors contributory in nature to the damage are summed with this score, as are scores expressing the overall system vulnerability. The final score is expressed as a number ranging from zero to 100. An example case is worked out to illustrate how the system works.

Keywords: Buildings; damage; earthquakes; expert system; reinforced concrete
Polat Güulkan is a professor of civil engineering at Middle East Technical University (METU), Ankara, Turkey. His current research interests include development of analytical tools for description of seismic damage in reinforced concrete structures, vulnerability assessment of a structural systems, and seismic hazard.

Ahmet Yakut is a doctoral candidate and research assistant at METU where he also received his previous degrees. His current research involves the use of neural networks, knowledge-based systems and fuzzy set theory in designing an automated environment for rational damage assessment.

INTRODUCTION

General Remarks

Attributes of structural damage have been established fairly well for individual components or for relatively simple subassemblies on the basis of information collected from laboratory tests. This information forms the major component of the knowledge pool to which reference is made for statements of what is termed as damage perceived to exist on other, possibly similar, structural entities. The task of establishing the degree of damage which a particular building has suffered during the course of having been shaken by a strong ground motion is a far more arduous undertaking. This is not only because the number of components is more numerous but because one does not have the hindsight benefit of experimental measurements or analytical calculations from which unequivocal statements can be made concerning the level of destructive cycles of loading into the post-yield range. Often, cracking in critical vertical members appears to be less severe than it really is due to the superposed dead load, or may even be invisible during a rapid inspection visit. Material properties or reinforcement placement details are not known, and construction defects exacerbate the task of assigning a number or a definite index to the damage level of a given building. Yet, quantification of the damage state would be a very desirable step because this would permit a more rational procedure for calibrating building code requirements to be established, and enable engineers to develop reliable vulnerability functions or damage loss estimate curves.

Government compensation schemes in Turkey require that every building be assigned a descriptive adjective in the aftermath of a natural disaster
such as an earthquake. This assignment is accomplished on the basis of a damage assessment form which is frequently filled by technical-level personnel who are typically overworked or inexperienced. The form is obsolete, and asks an incomplete set of questions from which the eventual verdict is derived. The work described in this study has as one of its immediate objectives the task of updating and expanding this form, but it also explores how tools derived from neural networks and fuzzy information processing can be potentially utilized to work behind the scenes in facilitating this work. The lessons learned from destructive earthquakes are well documented. In order to maximize the utility of the knowledge learned from destructive earthquakes, an expert system is developed for presenting the performance evaluation and behavior of buildings stressed or damaged during earthquakes.

**Previous Studies**

Due to the importance and universal appeal of the problem, many studies using knowledge-based approaches have been carried out to evaluate the damage states of buildings which have experienced earthquakes.

Shwe and Adeli (1) proposed an approach for presenting the performance evaluation and behavior of buildings stressed or damaged during earthquakes using expert system paradigm and computer graphics. They developed a prototype knowledge-based expert system, EXQUAKE2, for earthquake damage evaluation and knowledge of performance of a common class of buildings in California earthquakes. The knowledge base of EXQUAKE2 contains lessons learned from earthquakes. They used the EXSYS professional expert system shell. They concluded that collecting and presenting the knowledge of structural damage and behavior during earthquakes in a single expert system with a user-friendly electronic access would have a significant effect on the dissemination of this information.

Another study, carried out by Pagnoni, et al. (2), produced an expert system, AMADEUS, which is a prototype of a knowledge-based system for on-site assistance to the nonspecialist in emergency condition assessment of buildings damaged by earthquakes. They developed in the Pc-plus expert system development tool. It provides a detailed guide to the survey and evaluation of seismic damage to masonry construction. They have integrated a data base with the system for the automatic storage of the information collected during the inspections. In conclusion, they stated, AMADEUS, providing a detailed guide to the survey and evaluation of seismic damage of buildings, promises to contribute to the improvement of quality, uniformity, and efficiency in the usability assessment process, and suggests that knowledge-based systems can be effectively used in the
surveying and diagnostic tasks often encountered in civil engineering.

Park and Ang, and Park et al. (3,4) provided a formal method for evaluating structural damage of reinforced concrete buildings under earthquake ground motions. They investigated the statistics of the parameters affecting damage, and calibrated the measures of damage from empirical data. The damage index developed in their studies has been used in much subsequent work.

A recently held IABSE (International Association for Bridge and Structural Engineering) colloquium on knowledge-based systems in civil engineering has devoted considerable interest to instruments for damage or state determination. Not surprisingly many of these papers are concerned with a fair assessment of damage to structures of different types in China where (we assume) the government, as that in Turkey, wishes to play the role of the unerring global insurer in dispensing compensation to property owners. Shepherd and Haynes (5) have described an expert system applicable to one and two story buildings, representative of the background and methodology of experienced engineers familiar with the evaluation of existing unreinforced masonry buildings in accordance with the Los Angeles Building Code. The EXSYS shell program was used for the study. Their system needs building properties to be input. Then it establishes Code lateral forces and stresses in the main resisting elements including walls, piers, diaphragms, chords and various structural connections. They checked the expert system by application to three prototype buildings, and obtained excellent correlation with traditional assessments. Hsu and Yeh (6), and Brancaleoni et al. (7) explored ways in which artificial inference engines could be devised to assist in the detection of defects in "weighing" the damage of earthquake-stricken buildings. These researchers have reached the conclusion that it may be efficient to develop intelligence aided decision making systems in the earthquake damage prediction domain.

Two ATC reports, ATC-21 (8) and ATC-22 (9), deal with the assessment of seismic hazard in the existing building stock. The first attempts to score the hazard in buildings under (pre-earthquake) inspection, depending on structural characteristics, and is used in conjunction with a data collection form. ATC-22 is a technical manual for utilizing the methodology in ATC-21.

Williams and Sexsmith (10) have reviewed seismic damage indices for concrete structures, with particular reference to their use in retrofit decision making. They note that most damage indices are cumulative in nature which reflects the dependence of damage on both the amplitude and number of cycles of loading. Global damage indices may be calculated by taking a weighted average of the local indices throughout a structure, or
by comparing how modal properties of structures change during damaging ground motions. They suggest that, whereas most damage indices are based on flexural modes of failure there is a need to verify the ability of the indices to represent shear damage.

**DOMAIN OF THE PROBLEM**

**Post-Earthquake Damage Assessment**

When a strong earthquake strikes a populated area, a large number of buildings may suffer damages of various degrees, occasionally leading to the total collapse of some of them. Building officials and damage inspection teams are then faced with chaotic and confusing circumstances during which they have to make quick and reliable judgments in assessing the damage degree, the safety, and the usability of these buildings. This operation is referred to as Emergency Post Earthquake Damage Assessment (EPEDA). It typically consists of a quick reconnaissance of the buildings in the area to determine whether they can still assume the functions they had been designed for, without a substantial reduction in the safety conditions that existed before the earthquake.

The primary purpose of the emergency damage inspection is to save human lives and prevent injuries by identifying buildings that have been weakened by the earthquake and are therefore threatened by subsequent aftershocks. The other important objective of this operation is to avoid unnecessary waste of resources and additional human suffering by identifying habitable and easily repairable buildings, and hence reduce the number of homeless people and the economic cost of the disaster.

Unfortunately, after an earthquake, the demand on building experts often exceeds by far their availability. In many instances, unexperienced engineers and poorly, if at all, trained technicians are assigned to this difficult task without specific criteria about what to do and how to decide (2). In many instances, or in countries like Turkey where a strong paternalistic tradition exists in government relief following disasters, this assessment may subsequently be extrapolated to form the basis for compensatory payments to individuals who have suffered property losses.

The operation of damage assessment is generally done in the following way: the building inspection team fills out a one-page form consisting of a series of questions covering general information on the type of structure, its location, and the state of damage of selected components of the building. To date, these questionnaires have been designed as tools for uniform gathering of data. There is no intention to guide the inspector in the reasoning about the situation he or she is confronted with, nor is there
a well-defined attempt to assist him or her in the evaluation and decision-making process. For instance, the assessment of the degree of damage to the structure requires from inspectors a quantitative assessment which most often is described in vague and imprecise language, or is beyond their technical capabilities.

A portable, interactive, rule-based system for assisting inexperienced engineers or technicians during the emergency condition assessment would be a good answer to the problem of expertise-transfer to which we have made implicit reference earlier. Such a system would encode the methodology followed by experts in the field and make it available to other professionals. Ideal though it sounds, this level of assessment would fail to satisfy some very important requirements. First, a rule-based approach follows rigidly prescribed conditions, or aggregates of conditions formulated by interlocking IF-THEN-ELSE branches, but there is always some uncertainty in every judgment entered on a form. This uncertainty must be accounted for by recognizing that state descriptions are fuzzy information (11). Second, the process possesses no learning ability, so that repetitive information must be entered each time even though this is not strictly needed in situations when identical or similar patterns are in existence. These deficiencies should be removed for increased reliability of the assessment.

An all-seeing system for post-earthquake damage assessment is difficult to design, and can be the source of many legal entanglements if not used properly. The work described in this paper was initiated with the modest objective of updating the format of the current damage data collection form in use in Turkey. Its scope has now been enlarged to include also the desirable features to which we have referred in the preceding paragraph.

In this paper we will focus only on reinforced concrete building type structures, although the methodology is general enough to be calibrated for other structural types. We have attached the popularly preferred acronym of EPEDA to our methodology.

We stress that the methodology described in this article is concerned exclusively with structural damage, which may be less costly in terms of dollar loss than architectural damage and damage to building contents and components (12).

An Overview of EPEDA

EPEDA is an advisory system for the condition assessment of reinforced concrete buildings hit by an earthquake. Its purpose is to assist the engineer during the emergency inspection following an earthquake by providing a rational and uniform set of guidelines for condition assessment.
Based on the inspector's observations, EPEDA helps him or her make quick and accurate decisions regarding the severity of the damage and the habitability state of the building. In this process, it should be clear that EPEDA is not designed to replace the inspector but to guide him or her through the reasoning process to ensure that the engineer's approach to the problem is correct. The system also provides the inspector with the specialized knowledge required in particular situations and suggests a final decision with respect to the habitability status of the building under inspection. The knowledge base upon which the damage state for the entire building is established is the "most heavily damaged floor level," typically the ground floor. This principle is of course not entirely devoid of a paradoxical element. Another item representing a potentially complicating circumstance is when even the most heavily damaged floor level is too large for the canvassing team to inspect fully, typically due to time pressures. This difficulty is avoided by noting that the scores are normalized with respect to the number of elements falling into a given damage category, so that even if data representing a part of that floor is entered into the knowledge base, the final score will not be biased provided the data characterizes the remainder of that floor.

The visible part of EPEDA consists of a form with blanks to be filled, and a booklet which assists the inspector in this task. The form must fulfill several objectives, and our efforts to keep it to the shortest possible length notwithstanding, is six pages long. The questions in the form are answered with the aid of icons, drawn in more detailed form in the booklet, and supported by verbal explanations which are based on structural theory and empirical knowledge. These fall into four major categories:

1. Administrative (ownership, address, casualties if any, etc.),
2. General (geometrical/architectural characteristics, structural irregularities, spans, etc.),
3. Load Resisting Mechanism Characteristics (type of framing, wall-frame, or box, type of floor system, whether poured in place or prefabricated, filler wall properties, inferred material strengths, foundation system, apparent workmanship quality, etc.),
4. Damage Attributes (permanent displacements, extent and severity of column, wall, beam or slab damage, etc.). The form is designed to work also in conjunction with a renewable relational data base, with queries arranged to form a reasoning basis.
The Knowledge in EPEDA

The methodology developed and encoded in EPEDA is based on a notion of Global Damage State, which is a qualitative measure of the safety of the building under inspection. The value of the Global Damage State directly dictates the decision to be taken regarding the usability of the structure. If it is severe, then the building should be evacuated, and eventually condemned; if it is slight then the building is declared as safe and may be inhabited; if it is in a state of moderate damage then the building requires repair and therefore can be occupied only after repairs have been done. (These three states follow from currently accepted legal descriptions, and do not necessarily agree with practice in other countries.) Any of the outcomes may apply only to the whole building and not to only a part of it. The damage state associated with the building is the result of a consistent reasoning involving three principal elements: the geotechnical observation of the immediate vicinity of the building and the foundation system, the state of the structural system, and the additional hazard, if any, represented by its deformed configuration. The global damage state is determined by calculating a Total Damage Score (TDS), which consists of contributory damage from five different damage sub-scores. The first component is termed as the Damage Exacerbating Score (DES) which includes the general architectural features of the building and which reflects its structural vulnerability. For example, the presence of mezzanines or penthouses, plan or elevation irregularities, being situated as the end unit of a row of buildings in an urban block, omission of adequate clearances from adjoining buildings, or having floors at different levels from such buildings, spans longer than 7 m, poor supporting soils, inferior materials or workmanship each brings DESs of prescribed magnitudes. The upper limit sum for all DESs is 5.

The System Damage Score (SDS) reflects the structural damage experienced by the structural members, and is calculated by the reasoning described in Figure 1. The inputs to this score are governed by the severity, extent (expressed as element damage score), and relative importance of the members in question. The ranking of element-level severity falls into the following categories:

<table>
<thead>
<tr>
<th>Category</th>
<th>Multiplier</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage</td>
<td>0</td>
</tr>
<tr>
<td>Slight damage</td>
<td>1</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>2</td>
</tr>
<tr>
<td>Heavy damage</td>
<td>4</td>
</tr>
</tbody>
</table>
Extent reflects how widely spread the damage of all descriptions is in a given class of members, and varies between 0 (none affected) and 1 (all affected to some degree). Relative Element Importance (REI) differentiates between structural members serving different functions, particularly against lateral forces. Beams, for example, are assigned a relative importance of 1, while columns are assigned 2, and structural walls 6. The Element Damage Score (EDS) is calculated as:

$$EDS = \frac{REI \times (\text{# slight damage} + 2 \times \text{# moderate damage} + 4 \times \text{# heavy damage})}{\text{Total # elements}}$$

The booklet contains lengthy, explanatory discussions of what visual status in a given type of member signifies what damage level. These discussions are supported by sketches, photos and some theoretical background material assuming the person doing the inspection is familiar with elementary structural theory. It would be just as false to expect a total novice to enter the required information in a totally correct way as to claim that damage descriptions are perfectly stable parameters with no scatter about some best estimate value. Even the apparent ductility demand fails to serve the function of a perfectly stable parameter which is uniquely linked to apparent damage.

The SDS is then calculated as the sum of the EDSs, normalized with respect to the number of elements contributing to that score in accordance with their relative importance. The remaining three sub-scores adding to TDS are the permanent lateral displacement at the most severely distorted floor (PLDS, range 0-10), visible excessive foundation settlement (EFSS, range 0-3), and the visible damage to the stairs and the roof (RSDS, range 0-2). The permanent story displacement at the most severely distorted floor (usually the ground floor) is accorded a heavy score because it gives an indication of the deformation history for all of the elements there. The maximum sum of these three additive sub-scores to TDS plus DES is 20. Inasmuch as the total score for a completely severely damaged building should be 100, 80 percent of SDS is considered in determining TDS.

With TDS figured out, the verbal expressions for the damage state are calibrated as follows:

- $0 < TDS < 5$ No damage
- $6 < TDS < 15$ Slight damage
- $16 < TDS < 43$ Moderate damage
- $44 < TDS < 100$ Severe damage/collapse
System Architecture

EPEDA is a rule-based system. Its knowledge base is composed of 64 rules. These rules are arranged so that the hierarchical structure of Figure 2 is reflected. Most of the rules are in the form of mathematical equations because the final goal of the system is reached when the Total Damage Score for the structure is determined.

The damage levels for each structural resisting member are described through a series of external programs implemented in Q-BASIC. For this reason EPEDA calls 20 external programs. When the user starts an EPEDA run, he faces directly these external programs. Each external program is called for only a single variable, which means that the user is prompted to input a number for each appearance of an external program.

The knowledge base uses three structures to control and organize the information: Parameters, Rules and Frames. Parameters are specific facts or pieces of information that can hold one or more values. They are organized in sets and belong to frames. Rules embody the codified knowledge. Their action is to modify values of parameters depending on the data gathered. They also are organized in sets belonging to frames. Frames are used to group parameters and rules related to a specific subproblem, and are organized in a hierarchical manner. A conclusion that is reached by the system is called "goal." The final goal of EPEDA is the estimation of damage level. It can take the following actions depending on the value of TDS:

- **IF** No Damage **THEN** building is habitable
- **IF** Slight Damage **THEN** building habitable through provisions
- **IF** Moderate Damage **THEN** building must be repaired and thus is temporarily not habitable
- **IF** Severe Damage **THEN** building must be evacuated
- **ELSE** Collapsed

Each frame is responsible for the evaluation of a sub-goal that counts toward the achievement of the final goal.

The way the system goes about its task is illustrated in Figure 3. To reach the final decision, it needs to quantify the various damage states. Its first sub-goal is the geotechnical damage, determined from the ground deformation description, which is itself a sub-goal of a lower frame, determined from the visible soil profile and soil conditions (expressed in terms of visible settlement). A high geotechnical damage score leads to the evacuation of building with no need for further consideration. In the
other cases, the system evaluates the structural hazard. Two sub-goals directly affect the evaluation of the structural damage score: the structural damage level and the vulnerability of load carrying mechanisms. The system also evaluates complementary damage determined from the ratio of relative story drift to the story height. After the evaluation of this sub-goals, and with the information gathered in the process, the system makes its estimation of damage level and usability conditions.

**Expert System Development Environment**

EPEDA has been implemented on an IBM compatible PC using the EXSYS Professional Expert System Shell (13). EXSYS Professional is a microcomputer-based expert system development package implemented in C. The external programs of EPEDA have been developed in Q-Basic. An external program can be called from the EXSYS Professional in several ways:

1. It may be called at the beginning of execution
2. It may be associated with a qualifier or a variable
3. It may be called through the use of RUN command within rules

This shell does not contain any rules itself, but is designed to enable the user to create his own expert system by entering the rules which will be processed and run by the EXSYS program. The user prompts the computer to help solve the problem by entering the individual IF-THEN-ELSE rules explaining the steps involved in the decision-making process. The rules are a collection of English sentences and/or mathematical equations which can be easily read by anyone familiar with the problem domain. A rule is made up of a series of IF conditions and list of THEN and ELSE statements reflecting the probability of a particular choice being an appropriate solution to the problem. If the software determines that all of the IF conditions in a rule are true, it adds the rule's THEN conditions to what it knows to be true. If any of the IF conditions are false, the ELSE conditions are added to what is known. The computer determines what additional information is needed and how best to get the information. If possible, the program will derive information from other rules rather than asking the user for information. This ability to derive information allows the program to combine many small pieces of knowledge to arrive at logical conclusions about complex problems. EXSYS derives information by backward chaining unless commanded to use the forms of forward chaining available in the program.
AN EXAMPLE DIALOGUE WITH EPEDA

In order to illustrate the use of EPEDA, an example building which had been damaged during the Erzincan earthquake of 13 March 1992 (14) will be taken. The building, currently in use, is a law enforcement facility, and has since been retrofitted with the addition of shear walls. It has a rather regular structural layout and framing system. It had no shear walls when the earthquake occurred. It has 4 storeys one of which is the basement. The plan and elevation of the building are given in Figure 4.

Damage State of Building

The information about the condition of the building has been obtained from a report filed for this building by inspectors during emergency post earthquake damage survey. (The example is necessarily one where impressions of other assessors expressed in different terms must be utilized because EPEDA did not then exist. Learning to use EPEDA must be done in the hands-on style, but this cannot be done without visiting the site and taking stock of the damage visually.) We would stress that the data base is thus actually foreign to EPEDA, but still provides sufficient information for our purposes here. The structural skeleton of the building resisted the earthquake generated ground motions fairly adequately, fulfilling code expectations. A few columns and beams experienced severe damage, exhibiting fractured ends, exposed reinforcement bars, and widespread cracking. A somewhat larger number of columns and beams were judged to have had slight or moderate damage. On the other hand, almost all of the infill walls were severely damaged. The knowledge about the damage experienced by elements at the first story, the most severely damaged story, is summarized in Table 1.

Application to the Building

The following are the reconstructed answers to the questions posed by EPEDA:

-> The first set of questions are answered using data in Table 1.
-> The staircase had no damage.
-> There was slight damage in the roof.
-> Relative permanent story drift \((d/H)\) = 0.0016.
-> Workmanship quality was good.
-> Material quality was good.
-> Type of soil was firm clay/firm sand. There were no ground deformations.
-> Visual inspection of concrete quality was possible. Its quality
Concrete strength was measured with an impact hammer. It varied between 15-22 MPa.

The longest span did not exceed 6 m.

No expansion joint existed within the framing system.

Split floor levels did not exist.

The building stands free of any adjoining buildings.

Structural irregularity in elevation does not exist.

Plan irregularity exists.

A penthouse does not exist.

"Short" columns exist.

Maximum foundation settlement is less than 0.2 m.

No excessive ground deformation in the vicinity was visible.

No part of the building had collapsed.

Note that qualifications of workmanship or materials are based on local traditions and judgment which may not apply in a different environment. Damage state of the building judged by the initial inspectors was: moderate damage, should be repaired before use. The significant part of the information put out by EPEDA is reproduced below:

Damage Exacerbating Score: 1.75
Permanent Lateral Displacement Score: 2
System Damage Score: 28.98
Roof and Staircase Damage Score: 0
Excessive Foundation Settlement Score: 0
Total Damage Score: 27

This leads to the conclusion of moderate damage, should be repaired before use. This and the previous judgment are both in agreement with our own visual observations.

CONCLUSIONS

EPEDA is a prototype expert system developed primarily for reinforced concrete monolithic frame plus shear wall systems. It has been extended to other structural systems already simply by identifying damage mechanisms endemic to those types. The knowledge implemented here is not definite, because in order to be more reliable much more information should be implemented to the knowledge base. Its current capability is enough to estimate the damage level of any R/C building which has experienced an earthquake and has therefore been inspected for that purpose.
It is important to stress that the system assists the inspector by suggesting conclusions about the damage level of building and its usability decision. Integration of a database with the system will help the emergency management authorities in expediting the processing of the inspection data. In its current form EPEDA reasons in a very rigid manner, and does not learn from past experience. These deficiencies will be removed as it matures.

In conclusion, EPEDA provides a detailed guide to evaluate the condition and seismic damage state of buildings. This enables a uniform, non-subjective and efficient damage assessment procedure to be implemented. Our final comments are about the EXSYS Shell: although EXSYS has many features, some of them could not be used due to incomplete program files. The principal disadvantage of EXSYS that it has no meta rules option. The version used in this study does not perform the forward chaining process. Another source of irritation was the fact that, it asks questions about the THEN parts of the rules without checking about the fulfilment of the IF conditions.

REFERENCES


Acknowledgments

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### TABLE 1 — DAMAGE STATISTICS FOR THE EXAMPLE BUILDING

<table>
<thead>
<tr>
<th>Member</th>
<th>Total Number</th>
<th>Undamaged</th>
<th>Slightly damaged</th>
<th>Moderately damaged</th>
<th>Severely damaged</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>28</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Beams</td>
<td>53</td>
<td>20</td>
<td>22</td>
<td>8</td>
<td>3</td>
</tr>
<tr>
<td>Shear walls</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Joints</td>
<td>28</td>
<td>20</td>
<td>8</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Infill walls</td>
<td>30</td>
<td>0</td>
<td>5</td>
<td>10</td>
<td>15</td>
</tr>
</tbody>
</table>

**SEVERITY (MEMBER DAMAGE STATE)**

**EXTENT (EDS)**

**RELATIVE MEMBER IMPORTANCE (REI)**

**MEMBER STRUCTURAL DAMAGE (SDS)**

Fig. 1—Reasoning of SDS
Fig. 2—Hierarchical organization
Fig. 3—Dependency network of EPEDA
Fig. 4—Example building plan and elevation
Condition and Reliability Assessment of Constructed Facilities

by A. E. Aktan and D. N. Farhey

Synopsis: Nondestructive and destructive dynamic field testing and structural identification studies on actual constructed facilities are presented. The specimens discussed here include a 27-story RC flat-slab building, an RC-slab bridge, two 80-year-old steel-truss bridges, and three RC-slab on steel-girder bridges of various ages. The seismic vulnerability of the mid-rise building was evaluated and the test bridges were rated by code procedures as well as by field-calibrated comprehensive 3-D FE models developed by structural identification. Experimentally measured and analytically simulated modal flexibilities of the bridges were correlated with deflections obtained under proof-load-level truck-load tests. The rating factors obtained by field-calibrated models exceeded the corresponding operating rating factors by two and a half to four times for all of the test bridges. These studies revealed our capabilities for evaluating vulnerability or reliability of different classes of facilities. The bridge rating efforts helped to identify and conceptualize a number of unresolved important issues that influence bridge rating and management. Serviceability aspects that emerged as critical were studied through the relative contributions of different mechanisms to bridge deflections.

Keywords: Bridges (structures); deterioration; dynamic tests; nondestructive tests; structural analysis; structures; tests
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Daniel N. Farhey is a post-doctoral visiting scholar at the University of Cincinnati and research associate at the Infrastructure Institute. He received his DSc from the Technion–Israel Institute of Technology in 1991. His research interests include experimental testing, reinforced concrete structures, earthquake engineering, and structural identification.

INTRODUCTION

During the past decade, the engineering community has come to accept that many public facilities have not performed adequately either due to natural hazards or premature aging and deterioration while in service. Meanwhile, societal expectations of facility performance have increased. For example, it would be desirable to expect from a public facility a service life of 100 years, while many bridges continue to require extensive rehabilitation in only 20-40 years. Furthermore, it is realized that many regions, where seismic events were not considered as a design criterion, may be exposed to considerable seismic risk given a design life of 100 years. Therefore, civil engineers now confront the problem of assessing the reliability (or vulnerability) of existing constructed facilities and making management decisions regarding maintenance, upgrade, renewal, deconstruction, etc. As a result of this new awareness, many federal agencies are encouraging research related to innovative concepts and strategies such as "performance-based design," "lifetime cost," "maintenance at the onset of deterioration," and "reliability-based maintenance management". This research is in conjunction with technological advances such as "nondestructive experimental condition assessment" and "renewal engineering using smart materials," which are needed to improve the technical aspects of the state-of-the-practice in infrastructure preservation.

Most researchers concur that being able to establish the condition of an actual constructed facility, with possible defects, damage, and deterioration is the single most important challenge obstructing effective and reliable infrastructure management (1). Condition assessment should be objective, quantitative, comprehensive, and should lead to an accurate assessment
of the state-of-health in terms of facility reliability. Condition assessment should also provide an adequate basis for the design of any necessary corrective action. Twenty five years ago, as the senior writer was advised by Mete, the objectives of structural engineering analysis and design were to acquire a constructed facility with desirable performance. It was thought that limitations in our in-depth understanding of actual loading and response mechanisms did not matter as long as constructed facilities performed desirably. It is now realized that the actual real performance of constructed facilities can no longer be ignored in engineering practice, research, and teaching. The remaining issue is the capability of accurately observing, measuring, conceptualizing, and modeling actual constructed facility behavior.

**OBJECTIVES AND SCOPE**

Engineering operations, which are not yet codified or based on a sufficient knowledge-base, first require clear and realistic definitions and implementable tools. Some of these operations related to infrastructure preservation are: Condition assessment, damage assessment, field testing, strength evaluation, NDE, seismic vulnerability, rating, reliability, and maintenance, renewal, or deconstruction design and verification.

The authors explored the above operations through nondestructive and destructive testing of a number of actual constructed facilities. A methodology for condition assessment and reliability evaluation (in the context of seismic vulnerability and/or live-load rating) has been developed with the related definitions and tools and demonstrated through examples. The starting point was the knowledge-base in earthquake engineering (2), in conjunction with concepts and tools transferred from mechanical, aerospace, materials, and electrical engineering.

The first objective of the article is to present the proposed methodology for condition assessment and reliability evaluation, with definitions for related terms, and to summarize applications to a building and several bridges.

The second objective is to summarize the main points learned about loading and response mechanisms of actual constructed facilities, with emphasis on how some lesser understood mechanisms may be governing condition assessment and reliability evaluation. In particular, the relationship between defect, damage, or deterioration vs. facility behavior at advanced limit-states, which could only have been observed by damage-level tests of prototype facilities, is discussed.
DEFINITION OF TERMS

State Parameters

Definition of state parameters of bridges and means to quantify them in terms of objective measurable indices should precede the definition of condition assessment. For example, the state parameters of bridges may include initial strains and stresses, mechanical, chemical, and metallurgical characteristics of the materials at the microscopic and coupon levels, local (element-level) and global (substructure and system levels) initial forces, distortions, and displacements, stiffness/flexibility, inertia, damping, frequency, mode shape properties, etc.

Performance Parameters

In addition, performance parameters of bridges and their relationship with the state parameters should be established. For example, some of the performance parameters of bridges would describe aesthetics, functionality, serviceability, toughness (resistance to defects, deterioration, and damage), safety in terms of damage and failure mechanisms, their corresponding capacities and modes, attributes related to ease of inspection and maintenance, and lifecycle cost.

Furthermore, the phenomena of Defect, Deterioration, and Damage (DDD) should be defined. It is important to note that the influence of DDD on state parameters and performance has very complex relationships not clearly understood.

Defects

Defects are known or unknown errors in conceptual and detail designs, fabrication, construction, and maintenance. For example, collapse of the Silver Bridge over the Ohio River in 1967, which led to the National Bridge Inspection program, has been attributed to a defect in conceptual design which permitted the catastrophic failure of the complete structure following a local eye-bar failure at the upper chord. An example of detail design defects would be those revealed by the Cypress elevated freeway failures during the 1989 Loma Prieta earthquake. The failures along more than a one mile stretch of the freeway were attributed to numerous detailing defects in the numerous types of bends, in conjunction with the
amplifications in seismic demands due to soil conditions. Each type of bend was profoundly and differently influenced by the detailing defects, which were not detectable from the blue-prints. Prof. Bertero questioned whether one could have identified the defects and estimated the performance before the earthquake, even if the blueprints accurately showed the exact details. It follows that many unknown defects, which may appear innocuous even to experienced engineers, may be adversely influencing the serviceability and damageability performances of typical bridges.

Deterioration

*Deterioration* refers to long-term and gradual changes in the state parameters at the local, regional, and global levels due to common causative mechanisms during service. If *deterioration* is not mitigated, it may eventually accumulate into *damage*. Examples to that are environmental attack mechanisms such as freeze-thaw in conjunction with alkali-silica reaction in concrete, or moisture in conjunction with electro-chemical mechanisms leading to corrosion of steel. While deterioration is unavoidable, many deterioration mechanisms may be mitigated by effective maintenance. In many cases *deterioration* is triggered by and compounded by design and construction *defects*. For example, it is well known that cracking in concrete bridge decks accelerate many deterioration mechanisms and deck replacement has become the most significant component of bridge rehabilitation costs. Concrete deck cracking, on the other hand, occurs due to many mechanisms related to the design, placement, and curing of the concrete as a material, the flexibility characteristics of the steel-grid support structure, traffic and environmental loading, and environmental attack mechanisms. Clear and complete understanding of such deterioration mechanisms, and their compounding design, construction, and maintenance defects will provide significant long-term financial rewards.

Damage

*Damage* is broadly defined as clearly measurable changes in the state parameters, affecting performance. Various more specific definitions of damage have been offered in the engineering literature at the material, element, or structural levels. Visible degradation of structural elements, reduction in the structural stiffness, strength, or energy dissipation properties, and changes in the structural state properties compared to a baseline state have been suggested.
Following the research conducted in the early 1980's to explore health-monitoring of offshore platforms (3) and based on their experiences during the past decade, the authors adopted the following definition for bridge damage: A measurable anomalous change in the incremental local flexibility of a critical region. For example, if \( f_{ij} \) and \( f'_{ij} \) are the diagonal flexibility coefficients of two neighboring nodes in a critical region, temporal or event-induced anomalous changes in the difference \( (f_{ij} - f'_{ij}) \) indicate damage.

Past experimental experience has shown that the modal space alone does not adequately represent the structural status. Frequency changes are not sufficient to locate the damage and mode shapes are modestly affected when the damage is not severe. Therefore, it was necessary to use flexibility space quantities as damage indices. Alternative derivatives requiring reduced effort, such as only one column of \( f_{ij} \), or variation of curvature \( \frac{d^2 f_{ij}}{dxdy} \), or other representative induction formulations may also be used as damage evaluation indices. More research is needed on a facility-specific, type-specific, or prototype-specific manner to focus on more customized parameters for damage detection.

**Condition Assessment**

*Condition assessment* is defined as measuring and evaluating the state properties of a constructed facility and relating these to the performance parameters. Condition assessment includes identification of any defects, deterioration, and damage. Condition assessment should first focus on the global assessment of the state of health, i.e., whether DDD exist or not. Once this is evaluated, the DDD have to be located as for their exact place. Then, focus on regional or localized problems to pinpoint the structural category of the DDD. Subsequently, engineered quantification of the DDD is required. The information and products (such as analytical characterizations) which result from these steps should facilitate an accurate evaluation of their impact on structural reliability and rational management decisions. Currently, condition assessment of bridges are conducted in the context of visual inspections, during when subjective condition indices ranging between 0-9 are assigned to different components. A global condition index between 1-5 is then assigned to the complete facility.

If we accept the above objective definition for condition assessment, we would for example, measure the flexibility of a bridge with spatial and temporal resolutions, and define condition indices in terms of bridge flexibility in addition to spatial and temporal derivatives of flexibility. We would also capture visual images of the critical regions of the structure and digitally correlate measured indices to visual signs of deterioration or
damage for redundancy and calibration of the operation. Reverse-CAD technologies may assist geometric analytical modeling for evaluation. These operations require experimental technologies for measuring flexibility reliably and practically, and photogrammetric image-processing techniques for visual documentation. While all of these technologies already exist or are emerging in different fields, their adaptation to highways still require a spectrum of fundamental and applied research with technology-transfer.

Facility Reliability or the State-of-Health

Facility reliability or the state-of-health is defined as the probability of satisfactory facility performance given the results of condition assessment, expected remaining life of the facility and the projected performance demands and facility capacity supplies over its lifetime. System reliability is reliability of the system in terms of component reliabilities and their interactions while vulnerability is 1 – reliability. Presently, it is not possible to express reliability or vulnerability in a complete probabilistic frame even for isolated constructed facilities. We substitute quasi-deterministic Capacity/Demand indices at the member or structural level. Reliability evaluation becomes especially complicated if serviceability and lifecycle cost issues are incorporated in defining acceptable performance. Currently, most engineers consider reliability purely from a safety viewpoint. On the other hand, reliability of certain types of facilities within a system and therefore of the complete system may in fact be governed by the serviceability and lifetime cost rather than safety.

When we consider reliability evaluation in the context of current operations of highway engineers, bridge rating constitutes a good example. We currently conduct rating to determine the safety of a bridge in terms of its live load capacity either for operation (permit loading) or inventory purposes (posting). For the former, we typically use idealized analytical models and nominal values of the geometry and material parameters without accounting for inspection results. If inspection results indicate a need for posting, rating is carried out with an effort to incorporate the effects of deterioration and damage on capacity. The results of inventory rating are incorporated in management. The two fundamental problems which affect the reliability of this process are the subjective nature of assessing condition during visual inspections, and a lack of knowledge and proven analytical modeling techniques for incorporating all of the critical mechanisms and parameters related to the loading, responses, and capacities. The performance and reliability of bridges and pavements can be related with traffic flow in two possible time scales: 1) Short term for providing a better understanding of the real-time dynamics of the system, and 2) long-term evaluation for addressing trends in bridge or pavement safety or serviceability.
Management

Management is the decision-making process for selecting and prioritizing the necessary operations to maintain the reliability of a facility within acceptable limits. Management may be viewed as an optimization problem, where the objective function would be to achieve acceptable system performance at minimum lifetime cost. The constraints of the problem would naturally incorporate the economic, social, and political impacts of the lack of performance of a component, and the effect of component reliability on the system reliability. Ensuring a desirable outcome from a management decision would depend on the amount and accuracy of information regarding the condition and reliabilities of individual components, with the realism in defining the objective function and the constraints of the problem. Moreover, facility reliability is affected by different types of corrective action for restoring component reliabilities. Examples are, selection of different maintenance (repair, retrofit, or rehabilitation) operations, their timing, and partial or complete renewal decisions. Facility reliability should be objectively verified and quantified for effective management. It is further clear that separately managing different components of a system without incorporating their interactions cannot ensure cost-effective management. Currently, while separate management systems are being developed for bridges, pavements, and traffic, there is no known effort for integrating the management of the complete highway transportation system.

METHODOLOGY FOR CONDITION ASSESSMENT AND RELIABILITY EVALUATION

The methodology is based on the structural identification concept proposed by Liu and Yao (4). The main steps are: 1) A-priori geometric modeling; 2) modal testing to establish flexibility and global condition; 3) calibration and completeness check of the analytical model; 4) additional experiments for determining local defects or material characterization as called-for from Step 2, to measure local response mechanisms for the completeness of the model as called-for from Step 3, and to verify the accuracy and completeness of the field-calibrated model; and, 5) applications based on the field-calibrated model: Linear and nonlinear sensitivity analyses for reliability evaluation, management decisions, retrofit design, etc.

Applications of the methodology to a building and several bridges have been extensively published (5-22), demonstrating each of the above steps. A brief overview is provided here to emphasize the behavior aspects of the tested facilities, which could not have been conceptualized without an application of the methodology described above.
EXAMPLE APPLICATIONS

Twenty-Seven-Story Building

Overview – Sander Tower, a 27-story reinforced concrete (RC) flat-slab building with a central coupled-core system (Fig. 1) was used as a test specimen to explore techniques for accurately evaluating the seismic vulnerability of existing building construction types east of the Rocky Mountains that have not yet experienced damaging earthquakes. Structural identification of a mixed microscopic/element-level ETABS model was accomplished by closed-loop modal testing and measuring the lateral flexibility. A 3500-lb-mass linear inertia-mass exciter was developed for modal testing of large facilities.

Structural identification revealed a number of less-understood response mechanisms of mid-rise buildings in the region with shallow spread-footings, such as the kinematics at the ground level, effective slab-coupling of two cores, and in-plane diaphragm stiffness (11, 16).

The implosion of the building provided a further opportunity (Fig. 2). The event was filmed with high-speed cameras and image processing was used to understand the sequence of failure and the kinematics of collapse. Valuable lessons were learned regarding the limitations in the applicability of current seismic vulnerability evaluation guidelines east of the Rocky Mountains, some of the policy issues in seismic risk mitigation in the Central U.S., and how these problems distinctly differ from the corresponding problems in the Western U.S.

Important Lessons and Effect of Defect – The difficulty in interpreting the Capacity/Demand ratios, which are obtained for evaluating vulnerability, was an important lesson. Many elements and slab-column connections of the test building had Capacity/Demand ratios of less than one under the maximum credible event. However, realizing that the maximum credible event corresponded to a 2500 year return, and that the vulnerability evaluation procedure was beset by many sources of uncertainty, even experienced engineers would have difficulty in interpreting the results of Capacity/Demand ratios and in reaching a definitive conclusion regarding vulnerability. In the absence of codes, other regulation, or public awareness, it is nearly impossible to make a convincing argument for seismic upgrading, unless an engineer is convinced that there is a high likelihood of collapse under an expected design event and that in turn the owner can be convinced.
Image-processing studies and simulations following the implosion revealed that removing the support of only two of the columns in the building adjacent to the cores could have been sufficient to precipitate a complete collapse. Moreover, the debris revealed defect due to a combination of an unusual fracture-based brittle failure of stratified lightweight concrete at the interfaces of the slabs and vertical core and column elements, with inadequate detailing at these connections (Fig. 3 (11)). Since the fracture plane reveals a much smaller punching-shear capacity, the seismic response and behavior may be poorer than implied by analysis. However, the magnitude and rate of implosion-induced forces differ considerably from a seismic event. Therefore, without additional experiments on flat slab-column connections with stratified concrete, it is not possible to definitively sound an alarm with reasonable confidence. These questions cannot be fully answered by simplistic scaled-model tests in the laboratory. Therefore, full-scale system-level tests under realistic loading rates are required.

RC-Slab Bridge

Overview – The reliability evaluation methodology described above was applied to a decommissioned three-span RC slab bridge (Fig. 4) as the structure was subjected to shake-down loading to failure. The incrementally increasing static truck-loading was simulated by the use of rock-anchors and a closed-loop loading system (Fig. 5). The service, ultimate, and failure behavior was extensively measured, documented, and archived (7, 13). Nonlinear system identification permitted a study of the state-of-the-art in Nonlinear Finite Element Analysis (NLFEA) procedures (10). The research also provided a demonstration and verification for the damage detection by an evaluation of modal flexibility (12).

Important Lessons and Effect of Deterioration – The concrete at the shoulders of the 40-year old bridge had extensive deterioration due to freeze-thaw cracking and alkali-silica attack, to the extent that cores could not be taken. The appearance of this concrete, as well as a rating factor ([Live Load Carrying Capacity]/[Demand]) of slightly less than one obtained by simplistic analyses following the AASHTO Manual 1983 (23), had led to the decommissioning of this bridge. FE analyses indicated a rating factor of five without considering any load and strength reduction factors. The bridge actually resisted an equivalent of 22 rating trucks, corresponding to an actual rating factor of 10 considering the appropriate load and capacity reduction factors.

The main mechanism which enhanced the load capacity of the bridge was identified as the compressive membrane forces which developed due to the restraint provided by the shear-key at the interface of the slab and the
abutment. Under service and damage limit states, the slab distributed the applied load such that there were no significant demands on the shoulder concrete. However, when the applied load reached a level of 20 trucks, the slab rotations at the abutment shifted the load path to the deteriorated shoulder concrete, leading to a brittle shear failure (Fig. 6). The failure initiated as diagonal tension failure at the edge of the slab, moving as a dynamic front until the nearly circular path observed in Fig. 7 was completed. It was deduced that while the deteriorated shoulder concrete did not influence the service and damage limit state responses of the bridge, it governed the failure limit state behavior and the bridge capacity. Neither the failure mode nor the actual capacity could be predicted by the nonlinear FE analyses, although these were based on the identified structural and material characteristics. Following the failure and a careful analysis of the measured kinematics of slab response at the abutment, it was possible to simulate the measured global and local responses up to failure, although the failure mode could not be simulated since it is not possible to simulate a dynamic failure front occurring under a static load applied in load control with available analytical, numerical, and computational capabilities.

Steel-Truss Bridges

Overview — A series of nondestructive and destructive tests with accompanying analytical studies were conducted on two 80-year-old steel through-truss bridges (Fig. 8), to serve as additional demonstrations of the condition assessment and reliability evaluation method. Both bridges, which had riveted built-up elements rigidly connected by gusset plates, were instrumented with over 150 transducers to capture all the important global and local responses. The nondestructive tests included diagnostic truck-load tests and modal testing with a servo-control 3500-lb-mass linear actuator. Following retrofit at some critical connections, destructive tests were carried out by loading through a full-width load distribution frame and using hydraulic actuators reacting against rock-anchors, (Fig. 9).

The principal reason for steel-truss bridge collapses is typically failure due to fatigue, aging, or deterioration at the critical locations of non-redundant designs. The research permitted: 1) To evaluate whether there should be a public safety concern due to many pre-A7 steel truss bridges that remain from the early 1900's; 2) to explore effective upgrading methods; 3) to explore the applicability of the modal-test-based condition assessment method; 4) to discuss issues related to the definition and conceptualization of structural damage; and, 5) to explore issues in real-life implementation of structural control by utilizing one of the test bridges.
Important Lessons and Effect of Damage – The first bridge, with 150-ft-span Pratt trusses, yielded under an equivalent of 12 HS-20-44 trucks and failed under 18 HS-20-44 legal trucks due to rupturing of the lower chord at the pin-support connection. This bridge was posted to one-third of a truck. The bridge exhibited extensive deformation capacity in spite of a number of deep rust-pockets in the yielding elements (Fig. 10). The failure occurred when the rollers, locked due to extensive rusting, suddenly began to slip, leading to a surge of tension in the lower chord. The sudden surge was adequate to rupture the deteriorated gusset plates at the chord-pin connection (14). At intermittent points during the damage limit-state response, modal tests were carried out and modal flexibility was measured. Modal flexibility revealed location and extent of damage, and proved to be a reliable tool for the condition assessment of steel bridges.

The second bridge, with 250-ft-span Camelback trusses, resisted an equivalent of 20 trucks and could not be loaded further because the capacity of the loading system was reached. Two elements of the bridge yielded under a load corresponding to 18 trucks. However, following yielding, modal tests indicated that the flexibility decreased rather than increased. This was traced to the fact that the lower chord was being laterally constrained by the abutment backwalls. The global stiffness of this bridge increased following extension after yielding of some bridge elements as the roller travel was not permitted by the backwalls. This observation indicated that the definition of damage as a loss of global structural stiffness is too simplistic for most real-life structures (15, 16).

The 250-ft-span bridge also served as a test specimen to explore issues in real-life implementation of active control (Fig. 11). An electromagnetic linear active-mass driver system of 4450 N (1000 lbf) output was used to successfully control different types and levels of excitation generated by a servohydraulic linear mass exciter of 6675 N (1500 lbf) over 1-20 Hz bandwidth. The algorithm of the active vibration control was based on the Adaptive Modal Filter (AMF) method (19). A number of issues related to control system reliability and component failures were identified, which could only have been observed in a real-life implementation.

CONCLUSIONS

Service-life deterioration and aging effects have been quantified on 2-to-80-year-old bridges. Functional-aging phenomena, such as rollers locked due to extensive rusting and material aging on shoulders of bridge decks, have been observed to significantly impact the failure load and behavior of bridges. Important structure-specific mechanisms have been identified for the service, yield, incipient failure, and ultimate load levels by the
nondestructive and destructive, dynamic and static tests and the accompanying analytical studies.

This stage of the research reveals the feasibility and reliability of condition assessment and reliability evaluation by structural identification. The proposed methodology, tested on a variety of constructed facilities, has been proven to measure their condition in terms of objective indices.

Following system identification studies, the analytical model calibrated based on field measurements provides an invaluable practical tool for long-lasting use along the lifecycle of the structure. Being "field-calibrated", this model realistically quantifies the state-of-health and contributions of the different mechanisms participating in the actual resistance, rendering a uniform distribution of safety. In fact, it enables the determination of the actual facility-specific limit states of the structure and an explicit check of varying capacity and performance at all the critical limit states, including serviceability, fatigue, stability, deterioration, and collapse. Furthermore, this tool provides objective and reliable rating for design and management of effective maintenance, repair, and rehabilitation.

Actually, condition indices assigned by visual inspection influence critical bridge management decisions concerning repairs, posting, rehabilitation, and replacement. In this research, the current bridge rating procedures have been shown as conservative from a strength viewpoint while not being successful in assessing or facilitating desirable service load behavior. It is now emerging that bridge evaluation should include an objective quantitative assessment of serviceability in addition to strength and safety.

REFERENCES


Fig. 1—Twenty-seven story reinforced concrete flat-slab building (20)

Fig. 2—Twenty-seven story reinforced concrete flat-slab building during implosion

Fig. 3a—Twenty-seven story reinforced concrete flat-slab building after implosion (11)
Fig. 3b—Critical perimeter under punching shear (11)

Fig. 4—Reinforced concrete-slab bridge specimen

Fig. 5—Loading setup of reinforced concrete-slab bridge
Fig. 6—Failure of reinforced concrete-slab bridge

Fig. 7—Change in load path due to deteriorated shoulder

Fig. 8—Steel-truss bridge specimens
Fig. 9—Setup of loading system for steel-truss bridge

Fig. 10—Rust-pocket in truss member
Fig. 11—Active vibration control of steel-truss camelback bridge
Confinement of Rectangular Reinforced Concrete Bridge Columns and Pier Walls

by M. S. Saiidi, N. Wehbe, S. Acharya, and D. Sanders

Synopsis: This article presents a review of (1) previous experimental studies on the earthquake response of square reinforced concrete columns and a discussion of their applicability to bridge columns in areas of moderate and high seismic risk, (2) confinement steel design for rectangular columns based on different codes and methods and an example column to compare these codes, and (3) two concrete confinement models in relationship to their application in estimating a range of displacement ductility for square columns, rectangular columns, and pier walls. The results of part (1) showed that previous tests on square columns are mostly under relatively large axial stresses which represent the state of building columns and that the data are aimed at generally areas of high seismic risk. Part (2) showed a considerable variation among different codes and methods in terms of the amount of lateral reinforcement and the parameters considered in design. The results of part (3) indicated that measured displacement ductilities were generally within a range calculated using the two confinement models selected in this study.

Keywords: Bridges (structures); columns (supports); confined concrete; ductility; earthquakes; reinforced concrete; walls
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INTRODUCTION

Rectangular bridge columns are commonly used in the piers of many reinforced concrete bridge substructures throughout the world. A survey of state departments of transportation in the United States in mid 1980's indicated that nearly 30 percent of highway bridges with two or more spans incorporate rectangular bridge columns (1). The response of rectangular columns during strong earthquakes depends on the earthquake demand and reinforcement details and the degree of confinement of concrete. To assess the ability of bridge columns to accommodate the necessary drift without collapsing, methods have been developed which are primarily based on tests on circular columns which utilize spirals for confinement (2, 3, 4). Data on the cyclic response of square columns have been somewhat limited (5,6). More scarce are data on the earthquake response of rectangular reinforced concrete columns.

Bridge columns differ from building columns because they are larger than building columns and they carry smaller axial loads than building columns do. While it can be argued that the size effect is not necessarily critical
in attempting to apply building column data to bridge columns, the difference in the level of axial load is important and should not be neglected. Another observation regarding the provisions for design of confinement steel in bridge columns is the fact that design guidelines have been developed for the most severe earthquake effects. There is a lack of accepted design methods for areas of moderate seismicity in which the ductility demand may be lower.

The purpose of this paper is to review the available methods for confinement steel design in rectangular reinforced concrete bridge columns including those subjected to moderate earthquakes. Because the seismic response of bridge pier walls in the weak direction is similar to that of columns, a discussion of confinement steel requirements and ductility capacity of pier walls is also included. Excluded in the paper are shear steel requirements which may control the design of lateral steel when confinement requirements are relatively low.

RESEARCH SIGNIFICANCE

Many reinforced concrete highway bridge columns in the United States are rectangular. Recent strong earthquakes have emphasized the need for these columns to be ductile and strong and have revealed the importance of confinement reinforcement. The amount of available information on the seismic performance of rectangular reinforced concrete bridge columns is very limited. In this paper the existing experimental data are discussed. An overview of different design code methods for confinement steel design is presented, and the applicability of different confinement models to rectangular bridge columns is evaluated. In the absence of experimental data, these confinement models may be used to obtain a range of ductility capacity that might be expected from rectangular bridge columns during strong earthquakes.

PREVIOUS RESEARCH ON THE CYCLIC RESPONSE OF SQUARE COLUMNS

Because of the apparent similarity of the confinement provided by square ties to that of the rectangular ties, and due to a lack of experimental data on the earthquake response of rectangular columns, a review of previous cyclic load tests of square columns with rectilinear ties and cross ties was conducted (5, 7-11). Figure 1 shows a summary of the peak displacement ductilities in terms of the axial load ratio, $P_u/(f'_c A_g)$ where
P, f'_c, and A_g are the applied axial load, concrete compressive strength, and the gross cross sectional area of the member. The numbers in the figure refers to the names of the first authors of the publications from which the data were obtained. The ductilities are the maximum values attained by specimen failure or by terminating the test. Table 1 presents the main data from each reference. It can be seen that the material properties are generally in the range of the values used in bridge columns, and the dimensions may be thought of as being one-fourth to one-half scaled representation of prototype bridge columns. The longitudinal steel ratios in the specimens tested by Ozcebe and Sheikh are somewhat larger than what is usually used in bridge columns.

The axial load ratio for bridge columns is typically less than 0.3. It can be noted in Fig. 1 that for moderate ductility demand of the order of 2 to 4, and for columns with a relatively small axial load ratios, very little data are available on specimens with rectilinear transverse steel. This is particularly true in light of the fact that the data points marked by "4" are for columns with a longitudinal steel ratio which is higher than that of bridge columns. Some available test data on square columns (27-29) were excluded from the summary presented in Table 1. The excluded data are those of test specimens having relatively small shear spans (shear span-to-depth ratios = 1.67 to 2.33), and thus, are not representative of bridge columns.

CONFINEMENT STEEL REQUIREMENTS FOR RECTANGULAR COLUMNS

In the plastic hinge region of reinforced concrete members sufficient transverse reinforcement is needed to confine and prevent the disintegration of concrete and to prevent the buckling of the longitudinal steel. Different methods are available for design of confinement reinforcement. A critical review of the ACI code provisions (12) was presented in Ref. (25) for building columns. A summary of the ACI and other methods for the plastic hinge region of rectangular columns is presented in this section. Note that the required steel areas need to be satisfied for each orthogonal principal direction of the column section.

American Concrete Institute (ACI)

The American Concrete Institute provisions consider a structural member to be a column if the axial load index, P_u/(f'_c A_g), exceeds 0.1 (12).
Parameters $P_u$, $f'_c$, and $A_g$ are the factored axial load, concrete compressive strength, and the gross cross sectional area of the member. Bridge columns typically meet this requirement. The minimum total cross sectional area of rectangular hoops and cross ties is the greater of

$$A_{sh} = 0.3 s h_c \frac{f'_c}{f_{yh}} \left[ \left( \frac{A_g}{A_{ch}} \right) - 1 \right]$$

(1)

and

$$A_{sh} = 0.09 s h_c \frac{f'_c}{f_{yh}}$$

(2)

where

- $s$ = spacing of transverse reinforcement along the axis of the member.
- $h_c$ = cross-sectional dimension of column core measured center-to-center of confining reinforcement.
- $A_g$ = gross area of section.
- $A_{ch}$ = cross-section area of a structural member measured out-to-out of transverse reinforcement.
- $f_{yh}$ = specified yield strength of transverse reinforcement.

The general purpose of the requirements is to improve ductility of concrete. Equations (1) and (2) intend to provide the same degree of confinement as that in spiral columns. Considering that ACI provisions are generally for building design, the applicability of the requirements to bridge columns is not addressed in the code. The spacing of lateral reinforcement is limited to the smaller of 100 mm (4 in.) or one-quarter of the minimum member dimension.

American Association of State Highway and Transportation Officials (AASHTO)

The current provisions of AASHTO (13) are adopted from those of ACI. The lateral steel area is based on Eq. 1, but Eq. 2 has a coefficient of 0.12 instead of 0.09. The maximum spacing limits in AASHTO are the same as those in ACI.
California Department of Transportation (Caltrans)

The Caltrans provisions specify Eq. 1 as one of two expressions for the minimum area of transverse reinforcement (14). The other equation is:

\[
A_{sh} = 0.12 s_t h_c \frac{f_{c'}}{f_y} \left( 0.5 - 1.25 \frac{P_o}{f_c A_g} \right)
\]

(3)

where

\[ s_t = \text{spacing of transverse reinforcement along the axis of the member.} \]

\[ P_o = \text{design axial load due to gravity and earthquake loads.} \]

Equation 3 is adopted from the New Zealand Code (16) and reflects test results which have shown that confinement requirement should be a function of the level of axial force. A minimum spacing of 50 mm (2 in.) is specified for the transverse steel. The maximum spacing limit is the smallest of one-fifth of the column section dimension, 200 mm (8 in.), and six times the longitudinal bar diameter. The last limit is to prevent buckling of the column bars. These limits apply regardless of whether the lateral reinforcement is controlled by shear or confinement.

Paulay and Priestley

Paulay and Priestley (15) observe the dependency of confinement steel requirement on the level of axial force by recommending the following equation for the reinforcement area:

\[
A_{sh} = k s h_c \frac{f_{c'}}{f_{y h}} \frac{A_g}{A_c} \left( \frac{P_u}{f_c A_g} - 0.08 \right)
\]

(4)

where \( k = 0.35 \) for a required curvature ductility \( \mu_\phi = 20 \) and \( k = 0.25 \) when \( \mu_\phi = 10 \). Other values may be found by interpolation or extrapolation.

In addition to the level of axial load, this equation depends on the expected curvature ductility demand. The flexibility provided by including the ductility demand makes the expression useful for not only bridge columns which experience large drifts but also those which are in areas
of moderate seismicity where the ductility demand may be lower. For low values of the axial load index, \( P_u/(f'_c A_g) \), the confinement requirements become relatively small and shear will control the design. The maximum spacing of the confinement bars is limited to the smallest of one-third of the minimum column dimension, six times the longitudinal bar diameter, and 180 mm (7 in.).

**New Zealand Code**

The New Zealand code specifies the larger of steel area from two expressions both of which are functions of the axial load (16). The total area of transverse bars in each direction is the larger of

\[
A_{oh} = 0.3 s h_c \left( \frac{A_g}{A_c} - 1 \right) f'_c \left( 0.5 \cdot 1.25 \frac{P_e}{\phi f'_c A_g} \right)
\]

and the steel area from Eq. 3. These equations are similar to the AASHTO requirement except that they are modified by the factor which reflects the effect of axial load. The vertical spacing of the transverse steel is limited to the smallest of six longitudinal bar diameter, one-fifth of the minimum dimension of the column section, and 200 mm (8 in.).

**Commission of the European Communities**

A draft document for the seismic design of bridges was prepared in March 1994 by the Commission of the European Community (17). In this document, bridge piers with an axial load index of 0.08 are required to be detailed to provide concrete confinement in the plastic hinge region. A "mechanical" reinforcement ratio is defined for each direction of the column as follows:

\[
\omega_{wd} = \rho_w \frac{f_{yd}}{f_{cd}}
\]

where

\[
\begin{align*}
\rho_w &= \text{transverse reinforcement ratio.} \\
A_{sw} &= \text{total area of hoops or ties in the one direction of}
\end{align*}
\]
confined. 

\[ b = \quad \text{dimension of the concrete core perpendicular to the direction of confinement under consideration, measured to the outside of the perimeter hoop.} \]

\[ f_{cd} = \quad \text{dependable concrete compressive strength} \]

\[ = (0.85f'_{cll})/\gamma_c \quad \text{(with } \gamma_c = 1.5) \]

\[ f_{yd} = \quad \text{design strength of longitudinal reinforcement.} \]

The reinforcement ratio is determined from

\[ \omega_{wd} \geq 1.30(0.15 + 0.01\mu_c)\frac{A_c}{A_{cc}}(\eta_k - 0.08) \geq 0.08 \quad (7) \]

where

\[ \mu_c = \quad \text{required curvature ductility} \]

\[ A_c = \quad \text{gross concrete area of the section} \]

\[ A_{cc} = \quad \text{confined (core) concrete area of the section} \]

\[ \eta_k = \quad \text{normalized axial load} \]

\[ = N/(A_{cc}f_{ck}) \]

\[ N_C = \quad \text{axial load} \]

\[ f_{ck} = \quad \text{characteristic (specified) concrete strength} \]

The minimum amount of transverse ties is specified as

\[ \frac{A_t}{s} = \sum A_s \frac{f_{ys}}{1.6f_{yt}} \quad (8) \]

where

\[ A_t = \quad \text{area of one leg tie, mm}^2. \]

\[ s = \quad \text{distance between tie legs, mm.} \]

\[ \sum A_s = \quad \text{sum of the areas of the longitudinal bars restrained by the tie, mm}^2. \]

\[ f_{yt} = \quad \text{yield strength of tie.} \]

\[ f_{ys} = \quad \text{yield strength of the longitudinal reinforcement.} \]

The vertical spacing of the transverse steel is limited to the smaller of six longitudinal bar diameter and one-fifth of the minimum dimension of the column core section.
Under a contract from the California Department of Transportation, a project is currently in progress to revise and improve the Caltrans seismic design guidelines for highway bridges (18). A draft document has been prepared (19). However, the current provisions for confinement steel in rectangular reinforced concrete columns have been maintained as of this writing.

**EXAMPLE BRIDGE COLUMN**

To compare the confinement steel designed based on different guidelines, a representative rectangular bridge columns was designed. The cross section of the column excluding the transverse reinforcement is shown in Fig. 2. The longitudinal steel ratio in this column is approximately two percent. The concrete compressive strength was assumed at 27.6 MPa (4,000 psi) and the steel yield stress was assumed at 414 MPa (60,000 psi). The confinement steel ratio was determined as a function of the axial load ratio using different guidelines. Because the Caltrans provisions are similar to those of the Eurocode and the New Zealand code, only the Caltrans requirements are presented. Included in the analysis were the ACI and AASHTO minimum transverse steel requirements for gravity loads only.

Figure 3 shows the results. The confinement steel ratios based on Paulay and Priestley’s method are for curvature ductilities, $\mu_v$, ranging from 5 to 26. For low values of axial load, shear and not confinement would control the amount of lateral reinforcement. The ACI minimum tie requirement for areas of low seismicity is also shown. In bridges for which seismic detailing of columns is not currently considered, the transverse steel ratio is represented by the line showing "ACI-NON-SEISMIC" in Fig. 3. Future provisions for some of these bridges which are located in areas of moderate seismicity could perhaps require reinforcement ratios which may be in between the gravity load design and the seismic design requirements. Note that the ACI and AASHTO requirements are independent of the level of axial load, whereas Caltrans results depend on the axial load level although to a lesser degree than the results based on Paulay and Priestley. The latter suggest that the ACI results may be unconservative when the curvature ductility demand exceeds 20 and the axial load ratio exceeds 0.3.
REQUIREMENTS FOR PIER WALLS IN THE WEAK DIRECTION

The design of pier walls in the weak direction is based on the methods used for columns, and the provisions described for columns in the previous section apply. Additional requirements used in several codes is the minimum cross tie and transverse steel ratios each of which is to be at least 0.25 percent (12 to 14). The ACI provisions allow for a reduction of this limit to 0.20 if the design shear stress does not exceed 0.083 $\sqrt{f'_c}$ MPa ($\sqrt{f'_c}$ psi) (in which $f'_c =$ concrete compressive strength). Both ACI and AASHTO codes specify a maximum spacing of 460 mm (18 in.). The Caltrans provisions distinguish between cross ties and horizontal steel (lateral ties) parallel to the long direction of the wall section. The maximum spacing of 200 mm (8 in.) is specified for lateral ties in the plastic hinge area. The maximum spacing of cross ties in the plastic hinge zone is 300 mm (12 in.) in the vertical direction and 150 mm (6 in.) in the horizontal direction.

ESTIMATES OF DISPLACEMENT DUCTILITY CAPACITY

To calculate the failure displacement, a representative constitutive model for concrete is needed. The primary features of the stress-strain relationship for concrete are the peak stress and the failure strain. Other parameters such as the shape of the stress-strain relationship do not generally affect the outcome. The peak stress and the failure strain are both sensitive to the amount and distribution of the confinement steel. Many models have been developed for the constitutive relationship of confined concrete, the majority of which are based on compression loading of test specimens (3, 5, 20 to 22). A comprehensive discussion of these models is beyond the available space for this article but may be found elsewhere (21, 23).

Based on a review of different models and their applicability to rectangular hoops, two confinement relationships, the modified Kent and Park (5) and the other by Mander, et. al. (3), were selected for this study. Compared to the test data on square columns described in previous sections, these models appear to provide the upper and lower bound estimates of displacement ductilities. These models are briefly described herein.
Modified Kent and Park (5)

In this model, strength and ductility of core concrete are enhanced by the confinement provided by transverse hoops. The model consists of an ascending parabolic branch and a descending straight branch. The concrete strength, $Kf_c'$, is reached at a strain of $0.002K$. The stress-strain relationship is

(i) For $\varepsilon_c \leq 0.002K$

$$f_c = Kf_c' \left[ \frac{2\varepsilon_c}{0.002K} - \left( \frac{\varepsilon_c}{0.002K} \right)^2 \right]$$  \hspace{1cm} (9)

(ii) For $\varepsilon_c > 0.002K$

$$f_c = Kf_c' \left[ 1 - Z_m(\varepsilon_c - 0.002K) \right] \geq 0.2Kf_c'$$  \hspace{1cm} (10)

where:

$$K = 1 + \frac{0.5}{\rho_s f_y h}$$  \hspace{1cm} (11)

$$Z_m = \frac{0.5}{\frac{3 \cdot 0.26f_c'}{145f_c' - 1000} + \frac{3}{4} \rho_s \sqrt{\frac{h''}{s_h}} - 0.002K} \hspace{1cm} \text{(MPa)} \hspace{1cm} (12)$$

$\rho_s$ is the lateral steel volumetric ratio measured to the outside of hoops and $f_c'$ is the unconfined concrete compressive strength.

Mander, et al. (3)

In this model, the strength of confined concrete, $f_{cc}'$, is related to the confining pressure, $f_y'$, developed at the yield of lateral reinforcement. For circular sections, or square sections of equal confining steel in both directions, the confined concrete strength is
\[ f_{cc}' = f' \left( -1.254 \cdot 2.254 \sqrt{1 + \frac{7.94 f_{l}'}{f_{c}'}} - \frac{2 f_{l}'}{f_{c}'} \right) \] (13)

For rectangular sections, \( f_{lx}' \) and \( f_{ly}' \), in the \( x \) and \( y \) directions, can be found as follows

\[ f_{lx}' = K_a \rho_x f_{yh} \] (14)

\[ f_{ly}' = K_a \rho_y f_{yh} \] (15)

where \( K_a \) is 0.75 for rectangular columns and 0.6 for walls.

The strain, \( \varepsilon_{cc} \), at maximum stress is given by

\[ \varepsilon_{cc} = 0.002 \left[ 1 \cdot 5 \left( \frac{f_{cc}'}{f'} - 1 \right) \right] \] (16)

and the ultimate compression strain is given by

\[ \varepsilon_{cu} = 0.004 \cdot \frac{1.4 \rho_s f_{yh} \varepsilon_{sm}}{f_{cc}'} \] (17)

where \( \rho_s \) is the volumetric ratio of confining steel.

**Displacement Ductilities for Square Test Specimens**

The displacement ductility of a selected number of the square test specimens for which test data are available was calculated based on the confined concrete models described in the previous sections (5, 8-11). The columns selected for the analysis were those with relatively low axial load index which would be representative of axial load index in bridge columns. Measured concrete and steel properties were used when they were reported. Otherwise, the specified values were used. The "yield" displacement was calculated by including the rigid body displacement of the columns due to bond slip rotation. The bond strength was assumed to be 9.5\( \sqrt{f_{c}/d_b} \) (psi) [20\( \sqrt{f_{c}/d_b} \) (MPa)]. The ultimate curvature was based on the extreme compression fiber strain in concrete reaching the ultimate strain. The plastic hinge length was assumed to be that recommended
by Paulay and Priestley (15). The expression in this reference is based on test on reinforced concrete columns, and it implicitly includes the effect of bond slip.

The results are presented in Table 2. Note that the transverse steel ratios, the yield stresses, and the longitudinal steel ratios are also listed. The specimens are identified by the name of the first authors. In the tests, a specimen was considered to have failed when the lateral load capacity dropped by 15 to 20 percent of the peak load. A "" in the measured column indicates that the specimen did not necessarily fail at that ductility level. The "" for the last specimen indicates that the ductility of 6 was accomplished only for one cycle followed by a drastic strength degradation. The first specimen shown for Ozcebe was reinforced only with perimeter tie bars which is not representative of the current practice. As expected, the ductility capacity of this specimen was very low. Implied in the analytical model is a reasonably well distributed confinement reinforcement. Perimeter ties alone do not sufficiently confine the concrete which is away from the corner bars. The second specimen shown for Atalay was fitted with perimeter ties at a spacing of 125 mm (5 inches). The failure of this specimen was initiated by buckling of longitudinal bars which explains the low measured ductility level. A comparison of the measured and calculated ductilities shows that the measured values were generally within the limits predicted by Mander et al. and the modified Kent and Park. The modified Kent and Park model led to upper bound estimates of ductilities. The results suggest that Mander's model would provide a reasonable and a lower bound estimate of the displacement ductilities.

**Displacement Ductilities for Rectangular Test Specimens**

Four nearly one-half scale rectangular column specimens will be tested at the University of Nevada, Reno, during the next four months. These columns are 0.6 m by 0.38 m (2 ft. by 1.25 ft.) and the height from the top of the footing to the horizontal load point is 2 m (6.67 ft.). The specified concrete compressive strength is 27.6 MPa (4 ksi) and the steel yield stress is 414 MPa (60 ksi). The test variables are the axial load index and the transverse steel ratio listed in Table 3. The transverse steel ratios are 40 and 60 percent of those recommended by AASHTO. Figure 4 shows the details of specimens A1 and A2. The specimens will be subjected to cyclic loads in the strong direction. Estimates of displacement ductilities for these columns are shown in Table 3. The plastic hinge length was that from Paulay and Priestley (15). It can be seen that for specimens with an axial load index of 0.1 (A1 and B1) a
displacement ductility in the range of 5 to 9 is expected. The calculated ductility range for columns with a 0.25 axial load index is 3 to 6.

Displacement Ductilities for Wall Test Specimens

The available data on weak direction testing of bridge pier walls are limited. A recent study (24) on half-scaled reinforced concrete pier walls has provided some data which were used in this study. Six 0.25-m (10 in.) thick walls with a height to the loading point of 3 m (10 ft.) were tested under a constant axial load and cyclic lateral loads in the weak direction. Two of the walls did not incorporate any cross ties and were not included in this study. The other four are listed in Table 4. Letters "L" and "H" indicate low and high vertical steel, respectively. Walls designated with a "P" had cross ties only over the lower 0.5 m (20 in.), while those with a "U" were reinforced with cross ties over the entire height. The specified concrete compressive strength was 27.6 MPa (4 ksi) and the steel yield stress was 414 MPa (60 ksi). In the absence of an established expression for the length of the plastic hinge for walls loaded in the weak direction, the plastic hinge length was assumed to be equal to the wall thickness. Bond slip deformations were included in the calculation of yield displacement using the same bond strength as that for column bars. The range of calculated displacement ductilities and the measured values are shown in Table 4. It can be seen that the measured values were within the range of calculated ductilities. In walls with a high vertical steel ratio the result based on the Mander, et. al. model matched the test data very closely.

SUMMARY AND CONCLUSIONS

Nearly one-third of bridges in the United States have rectangular reinforced concrete columns. The response of rectangular columns under earthquake loads has not been studied in any significant detail. Rather, current codes have relied on test data which are primarily on circular columns to develop design guidelines. The confinement of concrete by rectilinear hoops and cross ties is known to play a major role in the seismic performance of reinforced concrete columns. It is known that the mechanism of confinement provided by non-circular ties is different than that of circular ties. This paper presents a summary review of the available data on square columns as they are more relevant to rectangular columns than circular columns are. It is shown that the majority of the data are for building type columns and that data are scarce for columns
in areas of moderate seismic risk. The paper also presents the confinement reinforcement design provisions of various codes. A comparison of these methods is presented and it is shown that there can be a considerable disagreement among different codes.

In the absence of test data, analytical models may be used to provide an estimate of the ductility capacity of rectangular columns. A review of the available confinement models showed that the lower and upper bound estimates of displacement ductility may be generally obtained by using models by Mander and the modified Kent and Park, respectively. It is shown that data from the cyclic tests of square columns and pier walls tested in the weak direction generally fall within the range calculated based on these models. This could indicate that the confinement models are appropriate for rectangular bridge columns and the weak direction response of the walls.

ACKNOWLEDGMENTS

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REFERENCES


12- American Concrete Institute Committee 318, "Building Code Requirements for Reinforced Concrete and Commentary," American Concrete Institute, Detroit, Michigan, 1992.


27- "A List of Experimental Results on Deformation Ability of Reinforced Concrete Columns under Large Deflections (No. 3)," (in Japanese), Building Research Institute, Ministry of Construction, Japan, 1978.


### Table 1 — Summary of Data from Previous Cyclic Load-Tests on Square Columns

<table>
<thead>
<tr>
<th></th>
<th>$f'_{c}$, Mpa (Ksi)</th>
<th>$f'_{y}$, Mpa (Ksi)</th>
<th>Cross-sections, mm (inches)</th>
<th>Transverse $\rho_v$, %</th>
<th>Longitudinal $\rho_v$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Azizinamini</td>
<td>41.4 (6.0)</td>
<td>414 (60)</td>
<td>455 x 455 (18 x 18)</td>
<td>2.35, 1.29</td>
<td>1.95</td>
</tr>
<tr>
<td>Sheikh</td>
<td>27.6 (4.0)</td>
<td>414 (60)</td>
<td>305 x 305 (12 x 12)</td>
<td>1.68, 3.06</td>
<td>1.30</td>
</tr>
<tr>
<td>Priestley</td>
<td>21.4 - 41.4 (3.1 - 6.0)</td>
<td>300 - 375 (43.5 - 54.4)</td>
<td>550 x 550 (21.7 x 21.7)</td>
<td>1.5, 2.0</td>
<td>2.3</td>
</tr>
<tr>
<td>Oszcebe</td>
<td>32 - 39 (4.6 - 5.7)</td>
<td>425 - 470 (61 - 68)</td>
<td>350 x 350 (13.8 x 13.8)</td>
<td>1.69, 2.54, 1.95</td>
<td>3.27</td>
</tr>
<tr>
<td>Park</td>
<td>21.4 - 41.4 (3.1 - 6.0)</td>
<td>300 - 375 (43.5 - 54.4)</td>
<td>550 x 550 (21.7 x 21.7)</td>
<td>1.5, 2.0</td>
<td>2.3, 3.5</td>
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<tr>
<td>Soesianawati</td>
<td>40 - 46.5 (5.8 - 6.7)</td>
<td>255 - 364 (37 - 53)</td>
<td>400 x 400 (15.7 x 15.7)</td>
<td>0.53, 0.74</td>
<td>0.79, 1.13</td>
</tr>
<tr>
<td>Atalay</td>
<td>29.1 - 33.3 (4.2 - 4.8)</td>
<td>363 - 429 (52.6 - 62.2)</td>
<td>305 x 305 (12 x 12)</td>
<td>0.89, 1.49</td>
<td>1.7</td>
</tr>
<tr>
<td>Specimen</td>
<td>$p_1$ %</td>
<td>$f_{yx}$ Mpa (Ksi)</td>
<td>$p_0$ %</td>
<td>Axial Load % of $f_y A_o$</td>
<td>$\mu_\Delta^{(1)}$</td>
</tr>
<tr>
<td>--------------</td>
<td>--------</td>
<td>-------------------</td>
<td>--------</td>
<td>---------------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>Azizi</td>
<td>2.35</td>
<td>414 (60)</td>
<td>1.95</td>
<td>20</td>
<td>6.6</td>
</tr>
<tr>
<td></td>
<td>1.29</td>
<td>414 (60)</td>
<td>1.95</td>
<td>30</td>
<td>3.7</td>
</tr>
<tr>
<td>Priestley</td>
<td>1.5</td>
<td>297 (43)</td>
<td>1.79</td>
<td>28</td>
<td>5.9</td>
</tr>
<tr>
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<td>3.5</td>
<td>297 (43)</td>
<td>1.79</td>
<td>60</td>
<td>4.1</td>
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<tr>
<td>Ozcebe</td>
<td>1.69</td>
<td>470 (63.5)</td>
<td>3.32</td>
<td>15</td>
<td>8.6</td>
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<tr>
<td></td>
<td>1.95</td>
<td>425 (61.6)</td>
<td>3.32</td>
<td>15</td>
<td>8.7</td>
</tr>
<tr>
<td>Park</td>
<td>2.3</td>
<td>316 (45.8)</td>
<td>1.79</td>
<td>21</td>
<td>3.2</td>
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<tr>
<td></td>
<td>2.0</td>
<td>297 (43.1)</td>
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<td>2.5</td>
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<tr>
<td>Soesianawati</td>
<td>-0.84</td>
<td>364 (53)</td>
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<td>0.90</td>
<td>364 (53)</td>
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<td>363 (52.6)</td>
<td>1.7</td>
<td>9</td>
<td>11.6</td>
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<td></td>
<td>0.89</td>
<td>392 (56.8)</td>
<td>1.7</td>
<td>17</td>
<td>4.7</td>
</tr>
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(1) Mander et al
(2) Modified Kent and Park
(3) Measured
### TABLE 3 — LOWER AND UPPER BOUND ESTIMATES OF DUCTILITIES FOR RECTANGULAR COLUMNS

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\rho_1$ %</th>
<th>$\rho_2$ %</th>
<th>Axial Load % of $f'_{c}A_g$</th>
<th>$\mu_{\Delta}^{(1)}$</th>
<th>$\mu_{\Delta}^{(2)}$</th>
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<tr>
<td>A1</td>
<td>0.75</td>
<td>2.2</td>
<td>10</td>
<td>5.5</td>
<td>6.8</td>
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<tr>
<td>A2</td>
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<td>2.2</td>
<td>25</td>
<td>3.6</td>
<td>4.3</td>
</tr>
<tr>
<td>B1</td>
<td>0.92</td>
<td>2.2</td>
<td>10</td>
<td>6.2</td>
<td>8.5</td>
</tr>
<tr>
<td>B2</td>
<td>0.92</td>
<td>2.2</td>
<td>25</td>
<td>3.9</td>
<td>5.4</td>
</tr>
</tbody>
</table>

(1) Mander et al  
(2) Modified Kent and Park

### TABLE 4 — LOWER AND UPPER BOUND ESTIMATES OF DUCTILITIES FOR PIER WALLS

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\rho_1$ %</th>
<th>$\rho_2$ %</th>
<th>Axial Load % of $f'_{c}A_g$</th>
<th>$\mu_{\Delta}^{(1)}$</th>
<th>$\mu_{\Delta}^{(2)}$</th>
<th>$\mu_{\Delta}^{(3)}$</th>
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<td>LP</td>
<td>0.45</td>
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</tr>
<tr>
<td>LU</td>
<td>0.37</td>
<td>1.3</td>
<td>5.2</td>
<td>5.1</td>
<td>8.3</td>
<td>5.6</td>
</tr>
<tr>
<td>HU</td>
<td>0.37</td>
<td>2.3</td>
<td>5.9</td>
<td>3.9</td>
<td>6.3</td>
<td>4.0</td>
</tr>
</tbody>
</table>

(1) Mander et al  
(2) Modified Kent and Park  
(3) Measured
Fig. 1—Available test data for cyclic response of square columns

Fig. 2—Cross section of example column
Fig. 3—Confinement steel requirements for different design methods
Fig. 4—Details of rectangular column specimens
Displacement-Based Assessment of Reinforced Concrete Frames in Earthquakes

by J. F. Bonacci and J. K. Wight

Synopsis: Members of earthquake-resisting reinforced concrete frames—such as beams, columns, joints and anchorages—are designed on the basis of force demands. Detailing requirements are established from collected experimental observations of measures which are most effective in maximizing overall cyclic toughness of frame assemblies. In this paper, a displacement-based approach to evaluating detailing requirements for frame elements is presented. Expressions are derived for the participation of beams, columns, joints and anchorages in overall story drift. Simple element models are presented for beam-column joints and anchorages, and guidelines for conventional sectional analysis of beams and columns are given. With an assessment of the local demand in each element type and mechanical models of element behavior, it is demonstrated that member variables normally considered as part of detailing can be accounted for in a quantitative supply-versus-demand fashion. A case study is made for an example in the ACI Committee 352 Recommendations to illustrate how a displacement-based frame evaluation is carried out and to provide a reference point for comparison with an existing design approach for beam-column joints.

Keywords: Anchorage (structural); beams (supports); columns (supports); deformation; earthquake-resistant structures; joints (junctions); reinforced concrete; shear properties; structural design
John F. Bonacci is Associate Professor of Civil Engineering at the University of Toronto. He is a member of ACI Committees 352 (Monolithic Joints and Connections), 408 (Bond and Development of Reinforcement), 442 (Lateral Forces), and 445 (Shear and Torsion).

James K. Wight, FACI, is Professor of Civil Engineering at the University of Michigan, Ann Arbor. He chairs the ACI Technical Activities Committee and is a member of ACI Committees 318 (Building Code), 352, 368 (Earthquake Resisting Concrete Structural Elements and Systems), and 445.

INTRODUCTION

A displacement-based approach to earthquake-resistant design is one that endeavors to quantify what happens when the lateral-load strength of a structure is overcome and it begins to soften. In this light, displacement-based design approaches attempt to make a more direct assessment than a strength-based method of whether the softened structure stiffness and toughness are adequate. A principal motivation for developing this kind of approach is that it would be more sensitive to performance questions, whether they arise from a post-quake evaluation or as part of a special design request by the developer.

The extent of prescriptions that can be made from a displacement-based design assessment depends on how detailed the description is of the deformed structure. So, while it is probable that several notions exist about what displacement-based design entails, any such approach begins with an estimate of the displacement response of the structure to a credible earthquake threat. With the evaluation taken only this far, the displacement profile furnishes a basis to judge whether the system is sufficiently stiff to protect it from what experience has shown to be dangerous distortion levels. If interstory displacements can be decomposed into element contributions, then a quantitative evaluation of toughness is possible, provided that the mechanics of element behaviors (i.e. deformation patterns and corresponding material strains) are well-understood. An extreme view of a displacement-based approach to design would be that it makes traditional strength-based prescriptions unnecessary. The objectives of such a stand-alone displacement-based approach should be to provide sufficient stiffness for the specified performance target (anywhere from prevention of collapse to protection of sensitive nonstructural elements) and to detail members for toughness consistent with the design drift level translated
to local deformation demands. (In this latter aspect, strong-column/weak-beam proportioning, or other similar principles that guide provision of relative rather than absolute strength, might still be of vital importance.) Having taken on the structural performance so directly, it can be argued that the overall lateral-load strength is moot; which is not to say that, as a by-product of such a design approach, it would be necessarily any lower than what is currently prescribed in building codes. (For short-period structures shaken by relatively longer period motion, displacement response is more sensitive to strength. In such cases, the extreme view characterized above is obviously not valid.)

Examples of displacement-based design for several RC element types have appeared in literature (1-4). The objective of these has been to transform certain aspects of detailing (proportioning and reinforcement decisions guided by empirical rules of good practice) to a more quantitative form of design (making sure supply meets demand). Most of these recent works have addressed cases for which the critical member response involves flexure in combination with axial effects. For isolated slender walls or cantilever bridge piers, the translation of overall structural displacement to local member demands (curvature, in this case), and then to material strains (applying the familiar plane-sections idealization and sectional equilibrium), is simple. But, with demand carried to this level of the structure, it becomes possible to quantify several important aspects of member response. Reinforcement tensile strain values will indicate the likelihood of fracture. Transverse reinforcement can be evaluated, and possibly altered, on the basis of the confinement it provides to meet the concrete compression demand as well as for the bracing it provides against bar buckling that may result from the steel compression demand (both of these evaluations being dependent on additional suitable mechanical models).

Such seemingly robust displacement-based member design procedures have not been put forward for cases in which there are several members contributing to the total story drift response and when the actions of one or more of the member types are not governed by flexure--two conditions which are anything but rare in conventional RC framing systems. In this paper the concept of displacement-based member evaluation will be applied to RC frames, with particular emphasis on decomposition of story drift into member contributions and on the use of a mechanical model for beam-column joints as a necessary part of the overall design process.
Procedures for displacement-based evaluation and design of RC frames can range from rather simple (100% of story drift assigned to beam flexural hinges) to quite complex (dynamic, nonlinear finite-element modeling). To be most useful in practice, the ratio of the amount and accuracy of information provided to the amount of labor and sophistication required should be maximized. The extent of idealizations (to be discussed in the following paragraphs) made in this study fell between the two example bounds and purposely as close as possible to the simple end. Added complexity is justified by the additional benefit of design information for columns, joints and anchorages.

The principal objective of this work was to establish a procedure for estimating the contributions of beams, columns, joints and anchorages to the total story drift. Even for static, linear elastic response of just the flexural elements of a frame, statical indeterminacy makes it inconvenient to evaluate story distortions on a calculation pad or in a spreadsheet. For this reason, the object of analysis was chosen to be a typical interior segment consisting of a joint with adjacent beams and columns removed at their inflection points (Fig. 1). Having thus avoided the obstacle of statical indeterminacy, it was no longer worthwhile to attempt tracking the shift of inflection points that would accompany frame softening. In this light, inflection points were assumed at midspan of beam and column members (Fig. 1), which, while admittedly unrealistic, has the advantage of being comparable with most other experimental and design models--for example, (5) and references thereof.

**ELEMENT CONTRIBUTIONS TO STORY DISPLACEMENT**

Interstory drift for the frame assembly (Fig. 1) is assumed here to be the sum of contributions from four sources: (a) beam bending (for moments at the joint face either below or above the yield point), (b) beam-end rotation at the face of the joint due to bar elongation along the anchorage zone (usually referred to as "slip"), (c) column bending (assuming no yield), and (d) joint shear deformation (assuming no yield of joint reinforcement). Expressions for each of these contributions will be presented in the following subsections.

Considerations of post-yield conditions for columns and joints are purposely excluded here as a result of design objectives rather than for lack of any particular modeling capability. As part of the traditional, strength-based approach to design of earthquake-resisting frames, members are proportioned, reinforced and detailed in such a way that
overall softening results from flexural hinging concentrated at the beam ends near the face of the joint. Of all the various sources of frame flexibility, beams have proven to be the easiest to detail for tough cyclic response. This fact transcends the specifics of a design methodology. The potential for acquisition of more potent performance indicators from a displacement-based evaluation should not lead the engineer into taking risks that run counter to the weight of experimental evidence.

Even in frames designed for strong-column/weak-beam response, some column yielding can occur in stories above the base as a result of increasing beam flexibility with hinging and higher-mode inertial effects. Given that the analytical assembly is isolated and statically determinate, no attempt was made to account for these effects. Existing design recommendations (5) specify margins that should be maintained between the sums of beam and column flexural capacities at joints in order to minimize the chances and consequences of isolated column yielding.

Numerous well-instrumented experimental studies (6,7,8) have shown that joint deformation increases abruptly as hoops yield, often leading to significant softening of the overall frame assembly with cycling. With hoops yielding, restraint to joint volume growth is greatly diminished leaving the concrete susceptible to low-cycle fatigue. For this reason, it is suggested that the main criterion for displacement-based design of joints should be the prevention of hoop yield. Given this criterion, it is further assumed that monotonic envelopes of material response are sufficient for representing joint behavior.

Because bar inelasticity along the anchorage zone is unavoidable when beam flexural hinging develops, bar yield penetration must be accounted for in calculating end rotation due to slip. No explicit consideration is given to the possibility of low-cycle fatigue in anchorages, as it is expected that existing design recommendations (5) would be followed.

Other potential contributions to frame displacements that are not considered here are column axial effects, and shear deformation in the spans of beams and columns. Of these, the most significant is likely to be shear distortion over the hinge region of beams, which is not included as part of conventional sectional analysis. Hinge shear softening can occur because of the net average tensile strain associated with post-yield bending. The tendency for beam growth will be restrained, to an unknown extent, by the in-plane stiffness of floor slabs, which are known to exhibit drift-dependent participation in beam action (9,10,11). Leaving this contribution out of the total is likely to lead to underestimation of total beam flexibility and consequent overestimates of deformation demand in the other elements for a given
level of story drift. Lateral displacement from column axial effects, which will be significant only in slender frames, is more a result of overall frame flexural action than of interstory shear distortions. So, while it should be considered as part of the total displacement of the complete frame (for evaluating P-delta effects or building separations), it need not be for displacement-based evaluation of story members. Even when column axial flexibility is not considered in the story evaluation, the value of axial load should be, as it may have a significant effect on the lateral flexibility of the column as well as on the shear flexibility of the joint.

Refer to the Notation section at the end of this paper for descriptions of terms appearing in the following subsections.

**Beam Contribution**

From Fig. 2a, it is apparent that story displacement from beam bending is:

\[ \Delta_b = \theta_b H = \frac{2 t_b H}{L} \]  

(1)

for which \( t_b \) is the tangential deviation at the inflection point, which is determined by weighted summation of the general-case curvature distribution shown in Fig. 2b. For this purpose, moment-curvature response computed from a standard sectional analysis routine can be idealized as bilinear, with a breakpoint at yield and constant post-yield rigidity established from a fit to the computed envelope (Fig. 2c). Considering the convenient reduction to a statically determinate analysis model and the assumption of midspan inflection points, the added complexity from rigorous consideration of unsymmetrical resistances for positive and negative moment cannot be justified. Instead, it is suggested that yield moments and curvatures, as well as post-yield rigidities, be averaged for the positive and negative bending cases. For the idealized curvature distribution shown in Fig. 2b, it is noted that the length of the hinge region is determined by the ratio \( M_{mb}/M_{vb} \) and that the corresponding value of peak curvature \( \phi_{mb} \) depends on the idealized post-yield rigidity \( E_{lpb} \). The resulting expression:

\[ \theta_b = \frac{\phi_{yb} L_n^2}{12 L} \left[ 1 + \frac{M_{yb}}{M_{mb}} + \frac{\phi_{mb}}{\phi_{yb}} \left( 2 - \frac{M_{yb}}{M_{mb}} - \frac{M_{yb}^2}{M_{mb}^2} \right) \right] \]  

(2)

is modified by substitutions to express \( \phi_{yb} \) in terms of the lateral force \( V \) (Fig. 1) and to replace the ratio \( M_{mb}/M_{yb} \) with the term \( P+1 \) (where \( P = (\mu_p-1) E_{lpb}/E_{lob} \) and \( \mu_p \) is the curvature ductility ratio), simplified, and
then multiplied by H to yield the final expression:

\[
\Delta_b = \frac{V H^2 L_n}{24 E I_{ob} (1+P)^3} \left( \frac{L_n}{L} \right)^2 \left( P^2 (1+2\mu_\phi) + 3P(1+\mu_\phi) + 2 \right)
\]  

(3)

From the statics of Fig. 1, the lateral force \( V \) can be related to the end-moments developed in the beams by:

\[
V = \frac{2 M_{mb}}{H (1 - \frac{h_c}{L})} = \frac{2 M_{yb}}{H (1 - \frac{h_c}{L})}
\]  

(4)

**Anchorage Contribution**

The anchorage contribution to story drift for the frame component being analyzed as shown in Fig. 3a, can be written as:

\[
\Delta_a = \theta_{\text{slip}} \left( 1 - \frac{h_c}{L} \right) H = \frac{\text{slip}}{\text{arm}} \left( 1 - \frac{h_c}{L} \right) H
\]  

(5)

Bar elongation occurring over the anchorage length has been labelled "slip", though the term also connotes relative motion between the terminal point of a bar and the surrounding concrete, which can result when the anchorage is inadequate. As bars are normally continuous through interior joints of earthquake-resisting frames, anchorage demands from the critical bending section are unlikely to penetrate to bar terminations unless bond is very poor. In this formulation, it is assumed that there is no free-end component to the total slip contributing to concentrated rotation at the joint face.

The most convenient modeling of anchorage response results from assuming constant average bond stress, \( u \), so that bar stress decays linearly from the peak demand to zero over a length that can be determined from equilibrium (Fig. 3b). In reality, bond stress is not constant over the anchorage length because it depends on the relative displacement that has occurred between the bar and surrounding concrete, which in turn depends on the accumulation of reinforcement strain starting from the point of zero bar stress. But it has been shown that the effects of reinforcement flexibility are simply to lower the apparent average bond strength that should be assumed with respect to the peak attainable local value (4). In this work, it is assumed that average bond strength is given as a multiple of \( \sqrt{f'_c} \). Given these assumptions, the length required for bar stress to decay to zero is:

Reinforcement strain over the anchorage length is shown in Fig. 3c for
the general case of beam bar strain demand in excess of yield. Steel stress-strain is modeled as bilinear, with the post-yield portion given by a best-fit line starting at the yield point. Integration of this strain distribution over the length \( L_a \) provides an expression for total slip as a function of strain demand at the face of the joint:

\[
slip = \frac{d_b E_s \epsilon_{sy}^2}{8u} \left[ 1 + \frac{E_{sh}}{E_s} \left( \frac{\epsilon_{sm}^2}{\epsilon_{sy}^2} - 1 \right) \right]
\]  

for which the ratio \( \epsilon_{sm}/\epsilon_{sy} \) can be related to \( \mu_p \) using the results of beam sectional analysis. The basic shape of the strain-slip curve from Eq. (7) agrees with those determined from nonlinear finite element analyses (4) and pullout tests (12) of yielding bars. Obviously, the absolute magnitudes of slip computed with Eq. (7) will be very sensitive to the assumed value of bond strength.

Slip was converted to beam end rotation by dividing it by an assumed effective arm of \( 0.7h_b \), which represents the distance between top and bottom reinforcement layers. Combining Eqs. (5) and (7), the final expression anchorage contribution is:

\[
\Delta_a = \frac{d_b E_s \epsilon_{sy}^2}{5.6u} \left( \frac{H}{h_b} \right) \left( 1 - \frac{h_c}{L} \right) \left[ 1 + \frac{E_{sh}}{E_s} \left( \frac{\epsilon_{sm}^2}{\epsilon_{sy}^2} - 1 \right) \right]
\]

**Column Contribution**

Given the assumed midspan inflection points, column distortion reduces to the "shear-building" value (Fig. 4), accounting for the clear span only and using the yield point to define an average flexural rigidity:

\[
\Delta_c = \frac{V (H-h_b)^3}{12E I_c}
\]

for which \( E I_c = M_y/\phi_y \), as determined from sectional analysis including axial loading.
Joint Contribution

Joint shear deformation will contribute to story lateral displacement in the manner shown in Fig. 5. Provided the shear strain \( \gamma_j \) is known, the geometry of deformation is such that the resulting story displacement is:

\[
\Delta_j = \gamma_j H \left( 1 - \frac{h_b}{H} - \frac{h_c}{L} \right),
\]

(10)

Joint shear strain \( \gamma_j \) can be determined from beam moments as:

\[
\gamma_j = \frac{v_j}{G_j} = \frac{2M_b}{G_j b_c h_c h_b} = \frac{VH}{G_j b_c h_c h_b} \left( 1 - \frac{h_c}{L} \right),
\]

(11)

which expresses that joint shear \( v_j \) is what must develop to produce the steep (relative to member spans) moment gradients that are demanded by beams and columns between faces of the joint region. Though beam moments will limit the joint shear demand in strong-column/weak-beam frames, the joint shear stress can be determined as the sum of either column or beam end moments divided by the joint volume. As it is unlikely that any portion of the member intersection region will be inert with regard to joint deformation, the joint volume is taken as the product of gross column area times beam depth.

Modeling requirements for joints inferred from the displacement-based approach outlined thus far would include: (a) the ability to monitor joint reinforcement strain as a function of joint participation in story drift; (b) provision of an effective shear modulus for the reinforced joint up to the point of hoop yield [Eq. (10)]; and (c) sensitivity to joint design variables in the event that the displacement-based evaluation reveals a potential shortcoming in joint performance. These modeling requirements are in every way parallel to those demanded for displacement-based evaluation of flexural members. Detailed evaluation of flexural members is only possible if they are analyzed using appropriate kinematic idealizations (plane sections remain plane), sectional equilibrium, and representative concrete and steel stress-strain relationships (with the option of accounting for confinement effects in concrete).

For the purposes of joint evaluations in this study, a derivative (no hoop yield, no horizontal normal stress) of an extensive set of published modeling prescriptions (13) was used. Sample computations for applying the formulation in both the pre- and post-hoop-yield cases were presented in the appendix to Reference 14. The procedure for determining shear stress \( v_{j,y} \) and effective modulus \( G_j \) at the point of hoop yield requires iteration for fine convergence. Iteration is essential
if the principal concrete compressive stress approaches the material limit before hoop yield. But, for joints with conventional reinforcement quantities and concrete properties, avoidance of iteration for the hoop yield condition does not have a profound impact on the accuracy of results (14). The procedure without iteration (as an option to the iterative approach in (14)) is summarized below:

- Assume: $E_c = 2f_c' / \epsilon_c$; $N = E_e / E_c$; $\epsilon_h = \epsilon_{yh} = f_{yh} / E_e$

- Solve quadratic for $\tan^2 \theta$:
  $$\left[ \frac{1 + 1/N\rho_h}{1 + 1/N\rho_v} \right] \tan^4 \theta + \left[ \frac{n_v / E_c \epsilon_h}{(1 + N\rho_v) N\rho_h} \right] \tan^2 \theta - 1 = 0$$
  (12)

- Determine shear stress at hoop yield:
  $$\nu_{hy} = \frac{\rho_h f_{yh}}{\tan \theta}$$
  (13)

- Determine vertical strain at hoop yield:
  $$\epsilon_v = \frac{1}{E_s \rho_v} \left( \frac{\nu_{hy}}{\tan \theta} - n_v \right)$$
  (14)

- Determine principal concrete compressive strain at hoop yield:
  $$\epsilon_2 = \frac{\epsilon_v - \epsilon_h \tan^2 \theta}{1 - \tan^2 \theta}$$
  (15)

- Determine shear strain at hoop yield:
  $$\nu_{hy} = \frac{2(\epsilon_v - \epsilon_2)}{\tan \theta}$$
  (16)

- Determine pre-yield effective shear modulus:
  $$G_j = \frac{\nu_{hy}}{\nu_{hy}}$$
  (17)

The effective shear modulus applies to the cracked, pre-yield condition of the joint and is nearly constant up to hoop yield given the same conditions as those stated above for non-iterative solutions. For a rough check on whether the nonlinearity is such that iteration would be necessary, the strain quantity $\epsilon_2$ [Eq. (15)] can be compared to the strain at peak concrete stress $\epsilon_o$. Finally, combining Eqs. (10) and (11), the resulting expression for joint shear contribution to story
displacement is:

$$\Delta_y = \frac{VH^2}{G_j b_o h_o h_b} (1 - \frac{h_o}{L})(1 - \frac{h_b}{H} - \frac{h_c}{L})$$

(18)

**DESIGN EXAMPLE**

To illustrate how the expressions just derived are applied to conduct a displacement-based frame evaluation, a design example from the ACI Committee 352 Recommendations (5) will be considered. Properties for the beam/column assembly in the longitudinal direction for Design Example 4 in the recommendations are summarized in Fig. 6. Units are kept consistent with the 352 design example. In addition to the properties given in the ACI 352 document it was necessary to assign values for story height $H$ (12'-0"), bay width $L$ (20'-0"), column axial stress $n_v$ (0.1$f_v'$), slab thickness (9"), slab reinforcement (0.33%), bond strength $u$ (12 $\sqrt{f_v'} = 0.76$ ksi), and the steel modulus of strain hardening $E_{sh}$ (0.015$E_s = 450$ ksi).

Beam sectional analyses were carried out for both the positive and negative bending senses including the reinforcement and concrete in the floor slab over an effective flange width of $L/4$ (15, 16). Ample confinement was assumed for concrete in the core of the cross sections. As expected, significant asymmetry was observed in computed moment-curvature relationships: $M_{vb} = +7700, -10,600$ k-in; $\phi_{vb} = +0.00009, -0.00011$ in$^{-1}$; $E_{lb} = 85.6 \times 10^6$ (+), 94.2$ \times 10^6$ (-) k-in$^2$. In keeping with the simplifying assumption of symmetrical beam resistances, each of these quantities was averaged for use in both bending senses. Review of detailed output from the sectional analysis routine showed that the ratio $\epsilon_{sm}/\epsilon_{sy}$ in Eq. (8) was suitably approximated by multiplying the curvature ductility ratio $\mu_\phi$ by 1.2 after yield.

Sectional analysis of the symmetric column section was made to establish $M_{vc}$ (13,000 k-in) and $E_{lc}$ (92.7$ \times 10^6$ k-in$^2$ before yield) assuming a constant axial compression of 314 kips.

Joint properties were established in a manner consistent with the mechanical model selected for the analysis (13,14): joint volume $b_c h_c h_b$ (23,520 in$^3$), vertical reinforcement ratio $\rho_v$ (0.0239), horizontal reinforcement ratio $\rho_h$ (0.0106), strain at peak concrete stress $\epsilon_o$ (0.002), and vertical stress $n_v$ (0.4 ksi). With these values, Eqs. (12)-(17) yield $v_{ly} = 0.84$ ksi and $G_j = 226$ ksi. By iterating to convergence, as outlined in Reference 14, values of $v_{ly} = 0.83$ ksi and $G_j = 217$ ksi.
are obtained. The principal concrete compressive strain $\varepsilon_2$ is only 21% of the strain at peak stress, so there is very little nonlinearity before hoop yield. Far less effort was involved in establishing joint properties than what was required for either beam or column moment-curvature analyses.

Expressions for member contributions to story displacement [Eqs. (3), (8), (9), and (18)] were organized in a spreadsheet (Table 1) to carry out the displacement-based analysis. Beam curvature ductility ratio $\mu_\phi$ was selected as the controlling variable for evaluation of member contributions at increasing levels of deformation. Once, each of the individual contributions were computed, the sum was used to establish the interstory drift ratio corresponding to each particular value of $\mu_\phi$. Before the computed results can be accepted as valid, the joint and columns must be checked for yield.

For this illustrative design example, it is assumed that a separate analysis of the overall structure had established that the story drift demand is 2% of the height and that joint shear stress could not be reduced by lowering beam strength or by increasing joint dimensions. The analysis showed that beam yield would occur at a story drift ratio of 0.80% followed soon after by yield of joint hoops at 0.96% drift. A plot of joint shear stress demand [by rearranging Eq. (11)] versus story drift ratio (Fig. 7) indicates that, in order to delay hoop yield until after the target drift level, the joint design had to be altered so that the shear stress at hoop yield was raised by 14%, to 0.95 ksi.

Examination of Eqs. (12) - (17) suggests a number of potential joint design alterations: increase $\rho_h$, increase $f_{vh}$, or decrease the inclination $\theta$ of principal tensile stresses by increasing the vertical reinforcement ratio $\rho_v$ or increasing $f_c'$. Table 2 summarizes the effectiveness of several postulated design alterations to the joint. It is apparent from this analysis that the joint shear stress at hoop yield is far more sensitive to the quantity and yield stress of hoops than to the column reinforcement ratio or concrete compressive strength. A more complete sensitivity analysis was presented in Reference 4.

As increasing concrete strength or column reinforcement were not particularly effective measures, and specifying special higher-strength hoop steel was considered to be impractical, it was decided to decrease hoop spacing from 6 to 3.5 inches, which increased the value of horizontal reinforcement ratio $\rho_h$ to 0.0135. Properties for the modified joint were $v_{hv} = 0.99$ ksi and $G_j = 233$ ksi. Beam yield occurred at a slightly smaller interstory drift ratio (0.78%) than it did with the original joint design. The main consequence of the modified joint design is that hoop yield would not occur until the drift imposed is 2.5% of the story.
height, which is more than the target demand level. For this example, no tendency for column hinging was detected even at very high drift levels.

Element contributions with increasing interstory drift ratio for the modified design are shown in Fig. 8. Before beam yield, beams contributed 41%, columns 17%, anchorage 14%, and the joint 29% to the total. At the demand drift level of 2%, the ratio $M_{mb}/M_{vb}$ reached 1.22, suggesting that flexural hinging had spread over 18% of the beam clear spans. At this level of post-yield bending, significant yield penetration would be expected to increase the flexibility of the anchorage mechanism. The element contributions at 2% interstory drift (Fig. 8) reflect these tendencies, as it can be observed that the beam contribution increased to 60% of the total along with 19% from the anchorage, while the participation of the joint and columns decreased (to 14% and 8%, respectively).

Under these circumstances, joint damage and column hinging have been eliminated as potential detractors from tough cyclic response. A certain amount of toughness will always be lost because of damage to the bond mechanism resulting from penetration of bar yield from flexural hinges into the anchorage zone. The extent to which this can be controlled by the designer was investigated elsewhere (4). Loss of the cyclic toughness of frames (manifested as pinching hysteresis) might also result from shear softening in beam hinges, which was not included as part of this analysis. If it had turned out that the design alterations required to meet the target drift demand were unacceptably extensive, then the structural framing should be stiffened to lower the overall displacement demand.

**CONCLUSION**

This paper has outlined procedures that can be used for displacement-based assessment of RC frame structures including the actions of beams, columns, anchorages, and joints. The expressions provided for member displacement components assume that yield is prevented in the joints and columns of the frame, and that hinging is therefore restricted to beams. Anchorage inelasticity was considered explicitly, as it is unavoidable for bars extending from hinging beams.

To make the equations and overall approach simple enough to be carried out by hand or in a spreadsheet, it was necessary to limit consideration to isolated components removed at member midspans, which were assumed as inflection points, and to treat asymmetric beam resistance in only an average sense. Obviously then, this technique is not a substitute for full frame analysis, nonlinear or otherwise. But its
idealization of structural form is consistent with what is adopted in design recommendations and standards (5, 16).

The principal benefit of the procedure presented in this paper is that it establishes a link between story drift demand and a number of important design and performance questions: Will columns be strong enough? How will the joint perform? What will the curvature demand be in beam hinges? How much transverse reinforcement is needed in beam hinges and columns? How significant will anchorage slip be in the overall response? Will beam hinging dominate the overall response? Traditional design approaches consider most of these questions, but they do so with no clear relationship between requirements stated as force demands and the amount of drift that is likely to develop as the structure responds to ground shaking.

REFERENCES


16. ACI Committee 318, *Building Code Requirements for Reinforced Concrete (ACI 318-89)*, American Concrete Institute, Detroit, 1989.

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**NOTATION**

\[ b \quad = \quad \text{width} \]
\[ d_b \quad = \quad \text{bar diameter} \]
\[ E_c \quad = \quad \text{modulus of elasticity of concrete} \]
\[ E_I \quad = \quad \text{flexural rigidity} \]
\[ E_s \quad = \quad \text{modulus of elasticity of steel} \]
\[ E_{eh} \quad = \quad \text{strain hardening modulus of steel} \]
\[ f_c' \quad = \quad \text{concrete compressive strength} \]
\[ f_s \quad = \quad \text{steel stress} \]
\[ G \quad = \quad \text{shear modulus} \]
\[ H \quad = \quad \text{story height} \]
\[ h \quad = \quad \text{member total depth} \]
\[ L \quad = \quad \text{total span length} \]
\[ L_n \quad = \quad \text{clear span length} \]
\[ N \quad = \quad \text{modular ratio} = \frac{E_s}{E_c} \]
n = axial stress
P = coefficient of post-yield bending
\( \tan \theta \) = inclination of principal tensile direction
u = average bond strength
v = shear stress
\( \gamma \) = shear strain
\( \Delta \) = lateral displacement
\( \varepsilon \) = strain
\( \varepsilon_c \) = concrete strain at \( f'_c \)
\( \theta \) = rotation
\( \mu_\phi \) = curvature ductility ratio
\( \rho_h \) = joint horizontal reinforcement area ratio
\( \rho_v \) = joint vertical reinforcement ratio = column reinforcement ratio

The following subscripts are used extensively throughout the paper (unless specifically noted in the list above, their meanings are as listed below):
b = beam
a = anchorage
j = joint
c = column
y = yield
m = maximum
o = initial
p = post-yield
s = steel
h = horizontal
v = vertical
2 = principal compressive
# TABLE 1 — SAMPLE DISPLACEMENT CONTRIBUTION CALCULATIONS DONE IN SPREADSHEET

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<td>0.310</td>
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<td>1.31%</td>
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<td>1.298</td>
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<td>0.496</td>
<td>1.461</td>
<td>6.256</td>
<td>4.34%</td>
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### Table 2 — Alternative Joint Redesigns

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<th>Detailing</th>
<th>$v_{hy}$, ksi</th>
<th>pre-yield $G_{fy}$, ksi</th>
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<tr>
<td>Hoop $f_y$: 60 to 75 ksi</td>
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<tr>
<td>Hoop sp.: 6 to 3.5&quot;</td>
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<tr>
<td>$f_c'$: 4 to 7 ksi</td>
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<td>Col. $\rho$: 12 to 20#11</td>
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Fig. 1—Representative reinforced concrete frame assembly (column moments not shown)
a) Deflected shape

b) Curvature distribution

c) Idealized sectional response

Fig. 2—Beam displacement contribution

a) Deflected shape

b) Stress distributions

c) Bar-strain distribution

Fig. 3—Anchorage displacement contribution
Fig. 4—Column displacement contribution and curvature distribution

Fig. 5—Joint displacement contribution
Fig. 6—ACI 352 (15) design example #4 (longitudinal beam case)

Fig. 7—Joint shear stress demand for original design
Fig. 8—Element contributions to story drift for modified design
Development of a New Structural Member
— Concrete Filled Steel Plate

by T. Takeda, T. Yamaguchi, and T. Nakayama

Synopsis: An experimental program was carried out to investigate the behavior of concrete filled steel plate walls. Seven wall-panel specimens were tested under repetitive in-plane pure shear loading. Each specimen was made by connecting a pair of surface steel plates with partitioning webs and tie bars, and filling the boxes so-formed with concrete. The parameters investigated were the thickness of the surface steel plate, the number of partitioning webs and the presence or absence of headed stud bolts. Results describing a restoration force characteristic of a large loop area are presented. Rigidity after the onset of cracking approximates the cumulative value of truss rigidity (rigidity of resistance mechanism consisting of longitudinal and transverse tension chord members of steel plates and compression diagonals of concrete) and in-plane shearing rigidity of surface steel plates. The skeleton curve for the shear stress vs. shear strain relationship could be theoretically idealized into a quadri-linear curve with three control points.

Keywords: Composite construction; concretes; ductility; plates (structural members); rigidity; shear properties; steels; strength; trusses
INTRODUCTION

The aging of workers in the construction industry has been extreme in Japan in recent years and a growing shortage of skilled workers is predicted. For construction business to continue smoothly and efficiently in this severe social environment, it is a matter of greatest urgency to promote rationalization of field construction techniques such as by reducing manpower requirements and improving working environments. One way is to introduce "concrete filled steel plate (SC)" as an effective alternative to "reinforced concrete (RC)". Since steel plates are fabricated in the factory, field work is considerably reduced by eliminating the need for formwork erection and reinforcement bar placement, leading to shorter construction periods and improved quality. SC structure comprises, as shown in Fig.1, a flat composite slab made by connecting a pair of surface steel plates with partitioning webs and tie bars, and filling the boxes so-formed with concrete. In the near future, we will embark on a project which utilizes a SC structure comprising large scale block elements through which many kinds of pipes will pass into the INNER CONCRETE STRUCTURE of a nuclear power plant (Fig.2).

The SC structure building are of composite structure with the bearing walls of the main earthquake resisting elements. Mechanical characteristics of SC structures have not yet been clarified, but some studies\(^1\) have been done. This study is being conducted to determine the basic mechanical characteristics of SC panels under in-plane stresses. It aims to clarify the mechanism of force transmission of the surface steel plates and the concrete, and to obtain useful data for design.

RESEARCH SIGNIFICANCE

The response of SC panels subjected to cyclic in-plane shear loading is investigated experimentally, and some of the important behavior aspects are reported. The stable hysteretic as well as the large deformability are shown. The shear stiffness of specimens after cracking is shown to approximate the cumulative value of truss rigidity and in-plane shearing rigidity of surface steel plates.
OUTLINE OF EXPERIMENTAL PROGRAM

Test specimen

The test program is summarized in Table 1. Fig.3 shows the specimen arrangements. The parameters investigated in the experimental program are the thickness of the surface steel plate, the presence or absence of headed stud bolts on the surface steel plate, and the number of partitioning web plates, which will influence the shear characteristics. The partitioning web plates were, if provided, in the Y-direction. The seven specimens are flat composite slabs made by connecting a pair of surface steel plates with partitioning webs and tie bars. The test panels were 1200mm × 1200mm in plan dimension, with a thickness of 200mm. To insure adequate load transfer, additional out-of-plane constraint steel plates were provided at the loading bolts along the perimeter of the specimen. The typical test specimen detail is shown in Fig.4.

Materials

High-workability concrete with coarse aggregate of maximum size 10mm was used. The concrete for panels was cast vertically. The test cylinders were sealed and cured until the panel was tested. The properties of the concrete, determined from cylinders tested at the time of panel tests, are listed in Table 1.

The mechanical properties of the steel plate, determined from coupon tests, are given in Table 2.

Testing setup

The loading apparatus was designed to subject the panels to a simulated uniformly distributed in-plane shear loading, as shown in Fig.5. The loading jack, which loaded reaction beams, provided equal tension in the rods. If necessary, the magnitude of tension in each rod, where electric strain gage measurements were made, was adjusted. Rigid beams were attached to each side of the specimen to keep the boundary straight. A sliding surface was provided between the rigid beam and the attachment to provide a uniform load. The applied loads in each test were repetitively reversed and increased until the specimen failed.

During testing, specimen deformations were continuously monitored using LVDT's placed on both faces of the panel. They were arranged to measure the average strains in the X- and Y-directions, and in the two diagonal directions. Also, electric strain gage measurements were made in the X-, Y-, and diagonal directions on both sides of the steel plate's surface.
ANALYSIS CONSIDERATION

Elastic rigidity

Using parallel rigidities of both the in-filled concrete \( (G_{c0}) \) and the surface steel plates \( (G_{s0}) \), and compatibility of shear strain, the resultant elastic rigidity \( (G_E) \) of SC is obtained as follows.

\[
G_E = G_{c0} + G_{s0}
\]

\[
G_{c0} = G_C(1 - P_w)
\]

\[
G_{s0} = G_S \cdot P_w
\]

Rigidity of cracked section

After cracking, concrete resists diagonal compression force only, while surface steel plates resist corresponding biaxial tension forces and shear force. For the stress condition, two kinds of the mechanism are formed. One is the truss mechanism which consists of the vertical, horizontal steel members and the diagonal concrete strut. The other is the in-plane shear mechanism of the surface steel plate. The mechanisms are shown in Fig.6. Using parallel rigidities of both the truss mechanism \( (G_{t0}) \) and the in-plane shear mechanism of the surface steel plates, the resultant rigidity of the cracked section \( (G_1) \) is obtained. The derivation of the rigidity of truss mechanism is included in the appendix, \( G_{s0} \) is defined in eq. (1).

\[
G_1 = G_{t0} + G_{s0}
\]

\[
G_{t0} = \frac{E_S}{4n} + \frac{(1 - \nu_s)}{P_w} \left[ \frac{2 + (1 + \nu_s) \rho_d}{(1 + \rho_d)} \right]
\]

Cracking strength

Assuming the tensile strength of concrete to be \( 0.313 \sqrt{\sigma_B} \) MPa \( (1.0 / \sigma_B \text{ kgf/cm}^2) \) and the shear resistance provided by concrete to be governed by the ratio of concrete elastic rigidity \( (G_{c0}) \) to resultant elastic rigidity \( (G_E) \), eq. (3) is derived as a cracking load.

\[
Q_C = \left[ 1 + \frac{G_{s0}}{G_{c0}} \right] (1 - P_w) A_0 \times 0.313 \sqrt{\sigma_B} (\text{MPa})
\]
Yield strength

Assuming the shear force distribution factor provided by the truss mechanism \( q_t \) to be governed by the ratio of \( G_{10} \) to \( G_{1} \), and that provided by the surface steel plates \( q_s \) to be governed by the ratio of \( G_{80} \) to \( G_{1} \), distribution factors of the applied shear force become (see Appendix)

\[
q_t = \frac{1 + \nu_s}{2np + (1 + \phi) + (1 - \phi) \nu_s} \\
q_s = \frac{2np + (1 - \nu_s) \phi}{2np + (1 + \phi) + (1 - \phi) \nu_s}
\]

where \( \phi = \frac{1}{2} \left[ \frac{2 + (1 + \nu_s) P_D}{1 + P_D} \right] \)

The truss mechanism produces the normal stresses, in-plane shear mechanism does the shear stress.

\[
\sigma_x = q_t \frac{Q}{2A_s} \\
\sigma_y = \left( \frac{1 + \nu_s \cdot P_D}{1 + P_D} \right) q_t \frac{Q}{2A_s} \\
\tau_{xy} = q_s \frac{Q}{2A_s}
\]

The tensile stress components and the shear stress component come into play. Based on Von-Mises yield criterion, eq. (4) is derived.

\[
Q_y = 2 \alpha \cdot A_s \cdot \sigma_y \\
\alpha = \frac{1}{\sqrt{\left( 3 - 6 \phi + 4 \phi^2 \right) q_t^2 + 3q_s^2}} \quad \cdots (4)
\]

The strain behavior on the steel plates is also evaluated theoretically. Since the stress components \( \sigma_x, \sigma_y, \tau_{xy} \) are known, principal stresses \( \sigma_1, \sigma_2 \) are determined from eq. (5).

\[
\sigma_1 = \frac{\sigma_x + \sigma_y}{2} - \frac{\sigma_x - \sigma_y}{2} \cos 2\theta - \tau_{xy} \sin 2\theta \\
\sigma_2 = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \cos 2\theta - \tau_{xy} \sin 2\theta \\
\text{where} \quad \tan 2\theta = \frac{2 \tau_{xy}}{\sigma_x - \sigma_y}
\]

\cdots (5)
Then, principal stresses become

\[ \sigma_1 = \alpha_1 \frac{Q}{2A_S} \]
\[ \sigma_2 = \alpha_2 \frac{Q}{2A_S} \]

where

\[ \alpha_1 = q_T \cdot \cos^2 \theta + q_T \left[ \frac{1 + \nu_S \cdot P_D}{1 + P_D} \right] \sin^2 \theta - q_S \cdot \sin 2 \theta \]
\[ \alpha_2 = q_T \cdot \sin^2 \theta + q_T \left[ \frac{1 + \nu_S \cdot P_D}{1 + P_D} \right] \cos^2 \theta + q_S \cdot \sin 2 \theta \]
\[ \tan 2 \theta = \frac{2(1 + P_D) \cdot q_T}{(1 - \nu_S) P_D \cdot q_C} \]

The strain in any direction is calculated to convert that in the principal axes which is evaluated from the stresses.

**Ultimate strength**

Based on the shear transfer concept\(^{[2]}\), shear load carried by the truss action is considered. The applied shear is resisted by the components of the strut compression and shear forces acting parallel to the diagonal cracks. Failure will finally occur when the concrete struts fail under the combined action of compression and shear in the struts, while the reinforcement continues to develop its yield strength.

An evaluation method is proposed to calculate the maximum shear force carried by the concrete, which is confined by the steel plate stress corresponding to yield strength. For the concrete, the compression-tension region of the biaxial ultimate strength envelope of Kupfer-Hilsdorf\(^{[3]}\) in eq. (6), as shown in Fig. 7, was adopted.

\[ \frac{r_2}{\sigma_T} = 1 - 0.8 \frac{\sigma_1}{\sigma_B} \cdot \cdots \cdot (6) \]

From eq. (5) and eq. (6), we obtain eq. (7)

\[ Q_u = \beta \cdot \sigma_B \cdot A_0 \]

where \( \bar{\sigma}_x - \bar{\sigma}_y \neq 0 \)

\[ \beta = \beta_2 \tan 2 \theta \]
\[ \cos 2 \theta = \frac{(1 + 0.8 e) \beta_2}{[e + (1 - 0.8 e) \beta_1]} \]

and where \( \bar{\sigma}_x - \bar{\sigma}_y = 0 \)

\[ \beta = \frac{[e + (1 - 0.8 e) \beta_1]}{1 + 0.8 e} \cdot \cdots \cdot (7) \]
in which

\[\begin{align*}
\theta &= \left| \frac{\sigma_T}{\sigma_B} \right| \\
\beta_1 &= \frac{\overline{\sigma}_X + \overline{\sigma}_Y}{2\sigma_B} \\
\beta_2 &= \frac{\overline{\sigma}_X - \overline{\sigma}_Y}{2\sigma_B}
\end{align*}\]

\(\overline{\sigma}_X, \overline{\sigma}_Y\) denote concrete compressive stress in the X- and Y-direction caused steel plates yielding, respectively.

**TEST RESULTS AND DISCUSSION**

**Failure Modes**

In all tests, after the onset of cracking, the surface steel plates buckled before yielding. The phenomenon of buckling was judged from the measured strain behavior. Therefore, buckling waves could not be observed anywhere at this time. The rigidities were substantially unchanged, however, with starting of the surface steel plate buckling. After yielding of the steel plates, local tearing on the welded steel portion along the perimeter of the specimen finally caused failure. A typical failure mode of the test specimen is shown in Fig.8 (a). Also, a crack pattern of the in-filled concrete, sketched after removing the surface steel plate, is shown (b). The cracks developed uniformly at an angle of 45 deg to the axis.

The test results are summarized in Table 3. The initial cracking loads were judged from the measured deformation response.

**Shear load-deformation**

A typical shear load-deformation hysteretic curve is shown in Fig.9. The dotted line denotes the skeleton curve we have theoretically derived. It is idealized into a quadri-linear curve with three control points. The first control point corresponds to the formation of a shear crack, the second to the yield strength, and the third to the ultimate shear strength. A shear strain of \(8 \times 10^{-3}\) at ultimate is assumed herein. Also, except for specimens with local tearing on the welded steel portion, SC structures exhibited large deformability and no reduction in strength appeared. Therefore, the ultimate strength was assumed to be maintained. Based on this evaluation, the load-deformation response of the specimen was simulated with good accuracy.

Skeleton curves and parameters are compared in Fig.10. As would be expected, the specimen with the heavier surface steel plate exhibited greater rigidity, higher ultimate shear strength and a stable load hysteresis loop. The specimen with 3 partitions exhibited more stability. But in this study there was little difference in ultimate strength between the three samples. Between specimens with and without stud bolts, there was no difference in strength, ductility and load hysteresis loop.
Shear load-strain

A typical shear load-strain hysteresis curve for the surface and partitioning steel plate are shown in Fig. 11. For the comparison of the measured strain hysteresis curves in the 45° direction, the ratio of the slope in compression field to that in tension field shows approximately 0.2 which is less than the Poisson's ratio for steel plate $\nu_s$. Therefore, both stresses perpendicular to each other will become positive (in tension). It should be noted that both stresses in the diagonal direction on the surface steel plane of concrete filled steel plate wall could be in tension. It depends on the ratio of surface steel plate to gross area of section. The measured strain hysteresis curve of the partitioning steel plate is similar to that of the surface steel plate in the same direction.

The dotted line in these figures show the calculated load - strain relation. It was derived from conversion of principal strain due to the truss mechanism after cracking and to the in-plane shear mechanism. Good agreement between measured and calculated values is shown.

Strength

Measured and calculated values of cracking, yield and ultimate strengths are shown in Table 3. The values are calculated by eq.(3), eq.(4) and eq.(7), respectively. Except for cracking loads for all specimens and yield strength for the specimen with 2.3mm surface steel plate, good agreement is shown.

CONCLUSION

The following were confirmed from the experiments.

1) Stable hysteresis was exhibited up to a shear strain of $2 \times 10^{-3}$. After that, no reduction in strength occurred since the surface steel plates prevented separation of concrete.
2) The hysteresis loops tended to be large due to the in-plane shear resistance mechanism of the surface steel plate.
3) Rigidity after cracking approximated the cumulative value of truss rigidity and in-plane shearing rigidities of the surface steel plates.
4) Shear buckling waves were observed on the surface steel plates, but the principal stress was not always compressive.

ACKNOWLEDGMENTS

The authors would like to thank Dr. Hiroyuki Aoyama, a professor of Nihon University; and Mitsubishi Heavy Industries, LTD. for their contributions to this project.
NOTATION

\( A_D \) : gross area of section
\( A_S \) : area of one surface steel plate
\( E_C \) : modulus of elasticity of concrete
\( E_S \) : modulus of elasticity of steel plate
\( G_C \) : modulus of elasticity of concrete in shear
\( G_{CO} \) : rigidity of the in-filled concrete
\( G_E \) : resultant elastic rigidity
\( G_S \) : modulus of elasticity of steel plate in shear
\( G_{SO} \) : rigidity of the surface steel plate
\( G^1 \) : resultant rigidity of the cracked section
\( G_{TO} \) : rigidity of the truss mechanism
\( n \) : modulus ratio of elasticity \( = E_S / E_C \)
\( p \) : \( P_w / (1 - P_w) \)
\( p_b \) : ratio of partitioning to surface steel plate \( = p_w / p_b \)
\( p_w \) : ratio of surface steel plate cross section to gross area of section\n\( Q \) : applied shear force
\( Q_C \) : shear force at cracking
\( Q_y \) : shear force at yielding
\( Q_u \) : ultimate shear capacity
\( Q_{SO} \) : shear force applied to in-plane shear mechanism of surface steel plate
\( Q_{TO} \) : shear force applied to truss mechanism
\( \gamma \) : shear strain
\( \nu_S \) : Poisson's ratio for steel plate
\( \sigma_B \) : compressive strength of concrete
\( \sigma_T \) : tensile strength of concrete
\( \sigma_y \) : yield strength of steel plate

REFERENCES


2. Alan H. Mattock,and Nail M. Hawkins; " Shear Transfer in Reinforced Concrete Recent Research ", PCI Journal, March-April, 1972, pp.55-75


APPENDIX

Elastic rigidity

Shear resistance in the in-filled concrete and in the surface steel plates will be in the same ratio as their elastic rigidities.

\[ Q = Q_{c0} + Q_{s0} \]
\[ = G_c \cdot A_c \cdot \gamma + 2G_s \cdot A_s \cdot \gamma \]
\[ = \left[ G_c \left( 1 - P_w \right) + G_s \cdot P_w \right] A_0 \cdot \gamma \]
\[ = \left( G_{c0} + G_{s0} \right) A_0 \cdot \gamma \]

Therefore,
\[ Q = (1 + \frac{Q_{s0}}{Q_{c0}}) Q_{c0} \]

For the in-filled concrete,
\[ Q_{c0} = G_c \cdot A_c \cdot \gamma \]
\[ = G_c \left( 1 - P_w \right) A_0 \cdot \gamma \]

Assuming \( G_c \cdot \gamma = \sqrt{\sigma_B (MPa)} \) for the cracking strength, we obtain eq. (3).

Rigidity of cracked section

The equations for the analysis based on the truss mechanism are derived, using equilibrium and compatibility conditions, as follows.

Assumptions \([1]\) and \([3]\) of the vertical members consist of the surface and partitioning plates (see Fig.6). The surface steel plate is in the biaxial tensile stress condition, while the partitioning web plate in the axial tensile stress condition.

Distribution of the stress for the vertical members

Stress-strain relation for a surface steel plate

\[ \varepsilon_x = \frac{1}{E_s} \left( \sigma_x - \nu_s \cdot \sigma_y \right) \]
\[ \varepsilon_y = \frac{1}{E_s} \left( \sigma_y - \nu_s \cdot \sigma_x \right) \]
Stress of a partitioning plate

\[ \sigma_{YP} = E_s \cdot \varepsilon_Y \]
\[ = \sigma_{YW} - \nu_s \cdot \sigma_X \]
\[ N_{YP} = \frac{A_P}{A_S} (N_{YW} - \nu_s \cdot N_X) \]
\[ = P_D (N_{YW} - \nu_s \cdot N_X) \]

Substituting the \( N_Y - N_{YW} \) instead of \( N_{YP} \), we obtain

\[ N_{YW} = \frac{Q}{2} \left[ \frac{1 + \nu_s \cdot P_D}{1 + P_D} \right] \]
\[ N_{YP} = \frac{Q}{2} \left[ \frac{(1 - \nu_s) \cdot P_D}{1 + P_D} \right] \]

The equivalent Young's modulus

The equivalent Young's modulus of the vertical and horizontal members are defined as follows.

\[ \varepsilon_X = \frac{N_X}{E_{SX} \cdot A_X} \]
\[ \varepsilon_Y = \frac{N_Y}{E_{SY} \cdot A_Y} \]

\( \cdot \cdot \cdot (A2) \)

From eq. (A1) and eq. (A2), we obtain

\[ \frac{N_X}{E_{SX} \cdot A_X} = \frac{N_X}{E_S} \cdot \frac{A_X}{A_{YW}} \cdot \frac{N_{YW}}{E_{SY} \cdot A_Y} \cdot \frac{N_X}{E_S} \cdot \frac{A_X}{A_{YW}} \cdot \frac{N_{YW}}{E_{SY} \cdot A_Y} \]

Then, the equivalent Young's modulus of the horizontal members becomes,

\[ \frac{E_{SX}}{E_S} = \frac{N_X}{A_X} \cdot \frac{A_X}{A_{YW}} \cdot \frac{N_{YW}}{1 + P_D} \]
\[ = (1 - \nu_s) \left[ 1 + (1 + \nu_s) P_D \right] \]
In the same manner,

\[ E_{SY} = \frac{N_Y}{A_Y} - \frac{N_X}{A_X} \]

The rigidity of the truss mechanism

The horizontal deflection in the truss (see Fig. 6) is calculated from the Castigliano theorem in eq. (A3).

\[ \delta = \sum_{i=1}^{5} \frac{N_i \cdot A_i \cdot E_i}{\bar{N}_i \cdot E_i} \cdot \ell_i \quad \ldots (A3) \]

in which,
- \( A_i \): cross-sectional area of the member \( i \)
- \( E_i \): equivalent Young's modulus of the member \( i \)
- \( N_i \): axial stress produced in the member \( i \) of the system by the applied load
- \( \bar{N}_i \): axial stress produced in the member \( i \) of the system by a unit load
- \( \ell_i \): length of the member \( i \)

All members of the system are numbered and their cross-sectional areas and equivalent Young's modulus given in Table A1.

**Table A1** Data for the truss in Fig. 6

<table>
<thead>
<tr>
<th>( A_i )</th>
<th>( E_i )</th>
<th>( \ell_i )</th>
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<tr>
<td>( \frac{(P_w + P_P) A_0}{2} )</td>
<td>( \frac{E_S}{1 - \nu_S} )</td>
<td>( \ell )</td>
</tr>
<tr>
<td>( \frac{P_w \cdot A_0}{2} )</td>
<td>( \frac{(1 + P_D) E_S}{(1 - \nu_S) [1 + (1 + \nu_S) P_D]} )</td>
<td>( \sqrt{2} \ell )</td>
</tr>
<tr>
<td>( \sqrt{2} A_c )</td>
<td>( E_S )</td>
<td></td>
</tr>
<tr>
<td>( \frac{2}{2} )</td>
<td>( \frac{E_S}{n} )</td>
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</tr>
</tbody>
</table>

Using the data given in table A1, we obtain

\[ Q_{T0} = \frac{E_S}{4n} \left[ \frac{(1 - \nu_S) [2 + (1 + \nu_S) P_D]}{(1 + P_D)} \right] \frac{A_0 \cdot \gamma}{P_w} \]
Shear resistances provided by the truss mechanism and by the surface steel plates will be in the same ratio as their rigidities.

\[ Q = Q_{T0} + Q_{S0} = G_{T0} \cdot A_0 \cdot \gamma + G_{S0} \cdot A_0 \cdot \gamma = (G_{T0} + G_{S0}) A_0 \cdot \gamma \]

Consequently, we obtain eq. (2).

**Steel plate stress**  Stresses in the truss mechanism and in the in-plane surface steel plates will be in the same ratio as their rigidities.

\[ Q_{T0} = \frac{G_{T0}}{G_{T0} + G_{S0}} Q = q_T \cdot Q \]
\[ Q_{S0} = \frac{G_{S0}}{G_{T0} + G_{S0}} Q = q_S \cdot Q \]

Eq. (A4) becomes

\[
G_1 = \frac{P_w \cdot E_s}{4np + 2 \phi (1 - \nu_s)} + \frac{P_w \cdot E_s}{2 (1 + \nu_s)} = \frac{P_w \cdot E_s}{2} \left\{ \frac{2np + (1 + \phi) + (1 - \phi) \cdot \nu_s}{[2np + \phi (1 - \nu_s)](1 + \nu_s)} \right\}
\]

from which

\[
\phi = \frac{1}{2} \left( \frac{2 + (1 + \nu_s)P_D}{1 + P_D} \right)
\]
\[ P = \frac{P_w}{1 - P_w} \]
\[ G_S = \frac{E_s}{2 (1 + \nu_s)} \]

Hence

\[
q_T = \frac{1 + \nu_s}{2np + (1 + \phi) + (1 - \phi) \cdot \nu_s}
\]
\[
q_S = \frac{2np + (1 - \nu_s) \phi}{2np + (1 + \phi) + (1 - \phi) \cdot \nu_s}
\]
The truss mechanism produces the normal stresses

\[
\sigma_X = \frac{N_X}{A_X} = q_T \cdot \frac{Q}{2A_S} \\
\sigma_Y = \frac{N_{YW}}{A_{YW}} = \left[ \frac{1 + \nuS \cdot P_D}{1 + P_D} \right] q_T \cdot \frac{Q}{2A_S}
\]

In-plane shear mechanism produces the shear stress

\[
\tau_{XY} = \frac{Q_{SO}}{A_w} = q_S \cdot \frac{Q}{2A_S}
\]

Putting the stress components \((\sigma_X, \sigma_Y, \tau_{XY})\) into the Von-Mises yield criterion in eq. (A4),

\[
\sigma_X^2 - \sigma_X \cdot \sigma_Y + \sigma_Y^2 + 3 \tau_{XY}^2 = \sigma_Y^2 \cdot \cdot \cdot \ (A4)
\]

we obtain eq. (4).

**Notation (shown in Appendix)**

- \(A_p\): area of partitioning webs
- \(A_s\): area of surface steel plates
- \(A_X\): area of steel plates in X-direction
- \(A_Y\): area of steel plates in Y-direction
- \(A_{YW}\): area of surface steel plates in Y-direction (= \(A_s - A_p\))
- \(E_{SX}\): equivalent Young's modulus of the X-direction member
- \(E_{SY}\): equivalent Young's modulus of the Y-direction member
- \(N_X\): axial force in the X-direction (= \(Q_{to} / 2\))
- \(N_Y\): axial force in the Y-direction (= \(Q_{to} / 2\))
- \(N_{YP}\): axial force applied in the partitioning web plates
- \(N_{YW}\): axial force applied in the surface steel plates
- \(Q_{co}\): shear resistance in the in-filled concrete
- \(Q_{SO}\): shear resistance in the surface steel plates
- \(Q_{to}\): shear resistance provided by the truss mechanism
- \(\varepsilon_X\): unit strain in X-direction
- \(\varepsilon_Y\): unit strain in Y-direction
- \(\sigma_{Xp}\): unit normal strain of partitioning web plates in X-direction
- \(\sigma_{YW}\): unit normal strain of surface steel plates in Y-direction
### TABLE 1 — LIST OF SPECIMENS

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Surface steel plate thickness (mm)</th>
<th>Partitioning web number</th>
<th>Concrete properties</th>
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<td></td>
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<td>Young's modulus Ec</td>
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<td>Compressive strength $\sigma_B$ (MPa)</td>
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<td>SC107-2</td>
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</table>

Notes:
1) Steel plate thickness 4.5mm were used for the partitioning webs.
2) Four 5mm dia. tie bars were provided in 480mm square for all specimens.
3) Head stud bolts 5mm dia. 35mm in length provided for SC209-2S and SC150-2S in @160mm.

### TABLE 2 — PROPERTIES OF STEEL PLATES

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<tr>
<th>Plate thickness</th>
<th>Measured thickness (mm)</th>
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<th>Yield strength $\sigma_y$ (MPa)</th>
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<td>Buckling</td>
<td>Yielding strength</td>
<td>Maximum strength</td>
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<td>P_W (%)</td>
<td>P_P (%)</td>
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<td>strain (10^3)</td>
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Fig. 1—Outline of SC structure

Fig. 2—Cross section of PWR Reactor Building
Fig. 3—Specimen arrangement

Fig. 4—Typical test specimen
Fig. 5—Top view of test setup

Fig. 6—Analysis model after cracking

Fig. 7—Biaxial ultimate strength envelope of Kupfer-Hilsdorf
a) Buckling mode of specimen SC209-2S

b) Crack pattern of specimen SC209-2S

Fig. 8—Typical failure mode of SC structure
Fig. 9—Typical shear stress-shear strain relation
Fig. 10—Measured shear strain responses

Fig. 11—Comparison of experimental and theoretical strain responses
Seismic Design of Confined Masonry Walls

by J. Bariola and C. Delgado

Synopsis: The objective of this paper is to present models for the design of confined masonry structures based on the available experimental data. In particular, this study deals with in-plane response of masonry walls subjected to lateral forces, with emphasis on aspects of initial stiffness, strength and deformation capacity. The experimental information used in this work comprises tests performed at the Structures Laboratory of the Catholic University of Peru. Results indicate that stiffness can be calculated considering a wall cross section inertia using the transformed cross-section concept with the appropriate Young's moduli for concrete and masonry. Bending strength can be estimated reasonably well assuming for the cross-section (1) a rectangular compressive stress distribution, (2) zero strength under tension and (3) a linear strain distribution. Unit shear strength could be safely calculated as 0.5\(\sqrt{f_{cm}}\), where \(f_{cm}\) is the characteristic compressive strength of masonry. It is observed that confined masonry can develop drift values larger than 0.5% of wall height which is comparable to that of reinforced masonry. Deformation capacity is observed to increase for increasing (a) wall horizontal reinforcement ratio and (b) column horizontal and vertical reinforcement, and to be reduced with increasing axial load.

Keywords: Bending; codes (standards); confined concrete; deformation; experiments; masonry walls; models; shear strength; stiffness strength
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Carlos Delgado is a structural engineer at JBB Consulting Engineers in Lima, Peru. He received his Civil Engineering Degree from the Catholic University of Peru.

INTRODUCTION AND OBJECTIVES

Confined masonry is a common construction system in Latin America. However, in most countries there are no regulations for design. The system consists of masonry panels, commonly made of clay units and portland cement mortar, and reinforced concrete beams and columns along lateral and top boundaries. Brick strength usually ranges from 50 to 150 kg/cm². Most structures are 1- or 2-story houses but 5-story buildings are also common. Seismic behavior of these structures has been generally satisfactory. However, lack of research and adequate design requirements limits its use. Present research is aimed at extending the knowledge of structural behavior to improve design procedures. For this objective published experimental data was evaluated with emphasis on defining stiffness, strength and deformation capacity.

EXPERIMENTAL DATA

Experimental information was primarily based on tests carried out at the Catholic University of Peru. This information includes studies dealing with parameters such as: (1) slenderness (height/depth ratio); (2) horizontal wall reinforcement; (3) axial load; (4) connection between the confining column and masonry wall; (5) dynamic loading; and (6) horizontal and vertical column reinforcement. Some unreinforced walls were also selected for the data base. Data are summarized in Tables 1 to 3.

The literature reviewed consists generally of cyclic tests of square walls with approximate dimensions of 2.5 x 2.5 x 0.15 m. Monotonic tests were also considered. Masonry compressive strength ranged from 60 to 150 kg/cm², and concrete strength from 150 to 220 kg/cm².
Interpretation of the data was based on the following hypothesis: For monotonic tests, initial lateral stiffness of walls is the slope of the tangent to the load-displacement curve at the origin. For cyclic tests, the tangent was drawn to the virgin load-displacement curve at the origin.

The failure mechanism for walls subjected to lateral loading was classified as either flexural or shear. Flexural failure was associated with cases where (1) experimental evidence indicated that tensile reinforcement had yielded; and (2) lateral strength could be computed by calculations using Eq (1) described below. In such cases, inclined cracks may appear at some stage during the test. In other cases in which bending failure was not the controlling mechanism, and diagonal cracking caused a sudden drop in lateral strength, failure was defined as being of the shear type. All specimens fell in one of these categories, except one which clearly showed sliding-shear failure (specimen A1, Table 1) which was still included as a shear failure.

Deformation capacity was defined as the lateral drift ratio corresponding to a 20% drop in lateral strength.

**COMPUTATION OF STRESSES, STRENGTH AND STIFFNESS**

Shear strength was estimated using a formula described in the next Section. Flexural capacity was computed considering a rectangular compressive stress distribution, a linear strain distribution, and elastoplastic behavior for steel. Therefore, the resisting moment can be expressed as

\[ M = A_s f_y j d \]  \( \text{(1)} \)

where:
- \( M \) = resisting moment
- \( A_s \) = tensile steel area
- \( f_y \) = yield stress
- \( j d \) = distance between compressive and tensile resultants
For stiffness computation, walls were modelled as elastic cantilever beams using the moment of inertia of the transformed cross-section from strength of materials. All three materials: masonry, concrete and steel were considered. Deformations due to bending and shear were taken into account. In most cases the modulus of elasticity of concrete was computed as $E_c = 15000\sqrt{f_c'}$. The exception was a 3-story specimen tested on a shaking table (5) for which the measured value of Young's modulus was used since it differed appreciably from the above formula. For masonry, Young's modulus and shear modulus were assumed equal to $500f_m'$ and $200f_m'$. It was observed that for computation of the moment of inertia of wall cross-sections, the contributions of concrete and steel are important because of their high moduli of elasticity relative to that of masonry.

**DISCUSSION**

Calculated normal stresses at the base of a cantilever wall subjected to lateral loading indicated a 'jump' at the interface between the masonry panel and the reinforced concrete confining element as typified by Fig. 1a. Obviously, this jump is proportional to the ratio of concrete and masonry moduli. The shear stress distribution shows a smooth variation along the base of the wall. Stresses shown in Figures 1a and b correspond to a wall of dimensions $5 \times 5 \times 0.15$ m with a lateral load of 15 Tons.

Computation of wall stiffness is performed in engineering offices using a variety of procedures. It is common to consider the masonry cross section, neglecting concrete. In other cases, the confining frame and walls are considered as independent structures coupled at floor levels. A braced frame is also used to model the confined wall for which a fictitious diagonal strut is defined (8, 9). For light confinement a cantilever beam with a transformed two-material cross section has been proposed. For the available experimental data, only the last model gave acceptable results, as shown in Fig. 2. All other models underestimate excessively the stiffness (Fig. 2). Specimens that best fit the calculated results had square wall panels with horizontal reinforcement and existing axial load.

Shear strength of reinforced masonry walls is normally expressed in terms of $\sqrt{f_m'}$ and the $M/VL$ ratio, where $M$ is the bending moment, $V$, shear force,
and, $L$, length of the wall. The intent is to allow calculated shear to increase with decreasing $M/V_L$ ratios, for $M/V_L < 1$ (e.g. short walls). For all $M/V_L > 1$ (tall walls) strength is kept constant (10). The calculation of shear strength for confined masonry walls is not simple because there are no models that distinguish between the individual contributions of the confining frame and masonry wall. Therefore it is convenient to adopt a conservative value of unit shear strength expressed as

$$V_u = 0.5 \sqrt{f_m} \quad (2)$$

considering the experimental data shown in Fig. 3. As can be observed (Fig. 3) this formula gives conservative estimates for all available experimental data in which shear failure was observed, including unreinforced masonry. The beneficial effect of axial load has been neglected as suggested in the literature (11).

Calculated flexural strength was compared with observed results indicating good agreement (Fig. 4).

Calculated lateral strength of walls was obtained as the minimum value of (calculated) flexural strength and shear strength (Fig. 5). Values of the ratio between observed and calculated strength (Table 3) are in the range 0.96 to 1.96 and, in all cases calculations indicated the failure type (flexure or shear) that was observed in the experiment.

Table 3 indicates the maximum drift obtained from experimental data. As indicated before, maximum drift was determined as the value corresponding to a point where strength was reduced by 20%. However, in several cases tests were stopped before reaching this point. Therefore, results presented in Table 3 are lower bounds. Horizontal reinforcement in the wall and vertical and horizontal reinforcement in confining columns have a beneficial effect on deformation capacity (Table 4). Limited data in Table 5 indicates that axial load above $P/f_mL_t = 0.08$ causes a reduction in deformation capacity. The experimental data from Ref.(1) in which influence of slenderness was studied are presented in Table 6. As can be seen the slenderness ratio $H/L$ did not seen to influence deformation capacity.
SUMMARY AND CONCLUSIONS

The research objective of this paper was to study experimental results of confined masonry walls under lateral loading with the aim of obtaining models for design.

Results indicate that stiffness can be estimated adequately idealizing the wall as a cantilever beam with a cross section defined by the transformed section. Because of the large difference between moduli of elasticity of concrete and steel in comparison with masonry, confining columns contribute considerably to the moment of inertia of the cross section.

It was found that bending strength of walls, calculated assuming for the cross section: (1) a rectangular compressive stress distribution and (2) a linear strain distribution coincides satisfactorily with experimental values.

Unit shear strength was assumed conservatively as 0.5 $\sqrt{f'm}$, where $f'm$ is the characteristic compressive strength of masonry. Based on the available data it was not possible to develop a model to compute shear strength that includes contributions of the masonry wall and the confining columns. Therefore only the contribution of masonry was considered.

Computing lateral strength as the smaller of bending and shear strengths, it was possible to estimate experimental results and anticipate the mode of failure (flexural or shear).

Experiments indicated that deformation capacity of confined masonry walls can be safely assumed to exceed chord rotations of 0.5%. This deformation capacity is comparable to that of reinforced masonry. Deformation capacity was observed to increase with increasing (a) horizontal reinforcement ratio in the wall and (b) horizontal and vertical reinforcement ratios in confining columns. Limited data presented here indicated that axial load reduces deformation capacity for $P/f'mLt$ values larger than 0.08.

ACKNOWLEDGEMENTS

The writers specially thank Ms. Lourdes Montoya who did all word-processing and figure drawing in this paper.
REFERENCES


### TABLE 1 — CHARACTERISTICS OF SPECIMENS

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<th>Thickness (cm)</th>
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### TABLE 2 — REINFORCEMENT OF SPECIMENS

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**Notes:**
1. An additional collar beam at mid-height was used.
2. Nominal yield stress of reinforcement.
# TABLE 3 — TEST RESULTS

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Notes:
(1) Data not available.
(2) Test ended before capacity reduced 20%.
(3) Nominal strength is the minimum of calculated bending and shear strengths.
(4) Compressive strength of masonry too high as compared with other specimens.
Shear strength formula does not give appropriate results.
TABLE 4 — INFLUENCE OF COLUMN REINFORCEMENT ON DEFORMATION CAPACITY

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TABLE 5 — INFLUENCE OF AXIAL LOAD ON DEFORMATION CAPACITY

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TABLE 6 — INFLUENCE OF SLENDERNESS ON DEFORMATION CAPACITY

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Fig. 1a—Normal stresses at wall base

Fig. 1b—Shear stresses at wall base
Fig. 2—Observed and calculated lateral stiffness
Fig. 2 cont.—Observed and calculated lateral stiffness

Fig. 3—Lateral strength versus wall slenderness
BENDING STRENGTH

![Bending Strength Graph]

NOTE: SEE REFERENCES IN TABLE 1

Fig. 4—Flexural strength

LATERAL STRENGTH

![Lateral Strength Graph]

Fig. 5—Lateral strength
Rehabilitation of Reinforced Concrete Structures — The Integration of Experimental Results and Analytical Models

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Synopsis: Research on rehabilitation of non-ductile reinforced concrete structures located in zones of high seismic risk has been underway at The University of Texas at Austin since 1981. A sampling of details and results from selected experimental programs investigating repair and strengthening of reinforced concrete non-ductile frame buildings is presented. Researchers at The University of Texas have integrated knowledge about the behavior of non-ductile elements and systems, retrofitted members, subassemblages, and superassemblages into non-linear time-history analysis models. These models have been used to investigate the response of buildings, retrofitted with techniques studied in the laboratory, to a variety of strong-motion earthquake records. An overview of some of the analytical modeling is presented, and results from two studies investigating the use of different concentric bracing schemes or infill wall systems to retrofit a three-story non-ductile frame building are discussed.

Keywords: Dynamic analysis; earthquake-resistant structures; frames; infill walls; models; reinforced concrete
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INTRODUCTION

Many existing buildings located in regions of high seismicity in the United States that were designed and constructed according to past standards and practices are often found to be inadequate to withstand major earthquakes. Figure 1 demonstrates how the required base shear coefficient for ordinary moment-resisting reinforced concrete frames in zones of high seismic risk has changed from the 1955 Uniform Building Code (1) to the present 1994 UBC (2) and 1991 National Earthquake Hazards Reduction Program (NEHRP) provisions (3). Because the procedures for calculating lateral forces have changed over the years, certain design assumptions had to be made to permit the comparisons in Fig. 1. The empirical equation \( T (\text{sec}) = 0.1 N \) (number of stories) was used to compute an equivalent vibration period for reinforced concrete frames, and a multiplying factor of 1.4 was applied to working stress design forces to account for load factors, strength reduction factors, and load combinations used in more recent ultimate strength design provisions. It should also be noted that the large difference between the 94 UBC and 91 NEHRP forces are due to the high penalty imposed by NEHRP for erection of an
ordinary moment-resisting frame in a zone of high seismic risk; the 94 UBC does not allow construction of an ordinary moment-resisting frame in any seismic risk zone except zone 1 (low seismicity).

Because the level of lateral forces prescribed by older codes was much lower than forces required by modern provisions, reinforced concrete frame design was often governed by gravity loads. As a result, most frames built during the late 1950's and 1960's were detailed for gravity loads, and as a result, have insufficient ductility. In addition, understanding of behavior of reinforced concrete members subjected to reversed cyclic loads has improved since the 50's, and as a result, so have ductile detailing requirements. Examples of some of the non-ductile details used in older frame construction are illustrated in Fig. 2 and include: 1) discontinuous bottom reinforcement in beams, 2) no transverse reinforcement in beam-column connections, 3) short lap splices (typically detailed for compression only) immediately above the floors, and 4) widely spaced ties over the entire height of columns. In addition, lack of consideration of the actual depth of spandrel beams or non-structural elements (such as masonry infills) in close proximity to columns has resulted in strong beam-weak column frame systems which are prohibited by modern design codes.

Since 1981, researchers at The University of Texas at Austin (UT) have been extensively involved in investigating the rehabilitation of non-ductile, moment-resisting, reinforced-concrete (RC) frame systems. Experimental studies have yielded information about the behavior of retrofitted members and systems as well as existing non-ductile details. With this information, researchers at UT have integrated observed behavior for ductile and non-ductile elements into non-linear, time-history analysis models to investigate the behavior of complete structural systems retrofitted to various degrees using procedures studied in the laboratory. A sample of these related experimental studies conducted at UT is presented in the following section. Because of length limitations, only select details and the most significant conclusions are presented for each study.

OVERVIEW OF SELECTED EXPERIMENTAL STUDIES

Strengthening of a Frame Sub-Structure

A large-scale model of a non-ductile RC frame with columns susceptible to shear failure was retrofitted with two strengthening schemes (5,6). The original unstrengthened model, shown in Fig. 3, was first retrofitted with concrete piers over the full height of the frame (Fig. 4a). The load-drift
response for the strengthened frame is illustrated in Fig. 4b along with two low-level load cycles that were applied to the original structure to assess its initial stiffness. The tests demonstrated that the strengthened columns behaved monolithically, and the failure mode was shifted to hinging of the deep spandrel beams. Furthermore, stiffness and strength of the strengthened frame was approximately three and five times that of the original frame, respectively. Strength of the original frame was based on the estimated shear strength of columns. Following testing of the pier-strengthened structure, the piers were removed and the damaged original frame was strengthened using a steel bracing system (Fig. 5a). The load-drift response for the second strengthening scheme is shown in Fig. 5b. The strengthened frame was approximately 1.5 times the stiffness of the uncracked original structure, despite the pre-cracked condition of the spandrel beams. Strength of the retrofitted system was approximately six times the estimated strength of the original structure. Failure of the strengthened system was governed by yielding and buckling of the upper-level braces, and shear capacity of the columns was increased at least 50 percent by addition of steel channels to the column faces.

**Jacketing of RC Frame Connections**

Four identical, large-scale RC beam-column connections designed according to 1960's practice were rehabilitated and tested under a severe bi-directional loading history (7). Connection specimens included flexible columns, strong beams, and non-ductile reinforcing details. Variables investigated in the study included jacketing both columns and beams, column jacketing only, damage or no damage prior to jacketing, and the layout of the longitudinal reinforcement (bundled or distributed) in the column jacket.

Dimensions for the specimens prior to jacketing are shown in Fig. 6. The first specimen, "O" (original connection, see Fig. 7a), was tested to cause column hinging, then the specimen was repaired by jacketing the column with bundled longitudinal reinforcement (see Fig. 7b) and retested as Specimen RB (repaired with bundled bars). The second test specimen was undamaged prior to strengthening with a column jacket (Specimen SB, strengthened with bundled bars). The third specimen was strengthened using longitudinal bars distributed around the perimeter of the column (Specimen SD, strengthened with distributed bars) to determine the influence of bundled bars. For the fourth specimen, the column was jacketed using distributed bars, and the beams were moderately strengthened with additional longitudinal reinforcement (Specimen SB-D, strengthened with distributed bars and jacketed beams). Reinforcement details for Specimens SB, SD, and SB-D are illustrated in Fig. 7b, 7c, and 7d.
The original beam-column joints contained no transverse reinforcement, and because placement of ties during the rehabilitation process can be difficult and labor intensive, a joint confinement cage made up of A36 steel angles and flat bars was used to provide confinement in the rehabilitated beam-column connection (see Fig. 8 for details).

Envelopes for the positive cycles of the story shear vs drift response for the primary loading direction are presented in Fig. 9. It is clear that jacketing of both damaged and undamaged frame elements substantially enhanced the strength of the original connection. Moreover, the jacketing designs tested in this study were successful in converting the original connection behavior from that characteristic of a strong beam-weak column system to strong column-weak beam behavior. The study also demonstrated that the confinement cage performed satisfactorily and can be designed to provide confinement equivalent to that recommended by ACI Committee 352 (8) for new construction; the tests demonstrated that the rehabilitated beam-column joints had shear strengths exceeding those specified in the Committee 352 report.

Steel Jacketing of Reinforced Concrete Members

A series of 17 rectangular columns with inadequate lap splices at their base and 11 columns with insufficient transverse reinforcement over their height were subjected to reversed cyclic loads to investigate: 1) the use of various steel jacket details, steel collars, and welded splice bars to improve strength and ductility of the splice specimens, and 2) the use of steel jackets and collars to improve strength and deformation capacity of the shear specimens (9). A sample of the results from this experimental program is presented here.

Specimen details and load-drift responses for an unretrofitted specimen (Specimen FC-4) and a specimen strengthened using a 1/4 in. thick A36 steel jacket (Specimen FC-9) are shown in Figs. 10 and 11. The unretrofitted specimen contained column bar splices at its base that were 24 bar diameters long, transverse reinforcement that was spaced at 16 in., and concrete with a compressive strength of 2850 psi. Reinforcement details were consistent with ACI 318-63 requirements. The specimen was subjected to reversed cyclic loads and experienced sudden strength reductions associated with splice failures at approximately 1% drift for both loading directions. Strain gages mounted on longitudinal reinforcement at the base of the column indicated that splice failures occurred just prior to reinforcement yield. A comparable column specimen was strengthened using a 1/4 in. steel jacket that was approximately 50% longer than the splice length. A 1 in. thick gap between the column and
steel jacket was filled with non-shrink grout (with compressive strength of approximately 5220 psi). Because the column was reinforced symmetrically, two different strengthening details were used on the long faces of the column to broaden the range of variables investigated in the study. One face had only steel plate. The other had five 1.0 in. diameter epoxy-grouted anchor bolts installed at midwidth of one column face with an embedment into the column of 8 inches. The load-drift response shown in Fig. 11 demonstrates that more than yield strength was attained in both loading directions. However, performance was noticeably better in the "push" direction; strength was maintained to more than 3% drift and declined gradually thereafter. Strength degradation in the other direction began at approximately 1.5% drift and was quite rapid.

Specimen details and load-drift response for an unretrofitted column specimen (Specimen SC-3) with deficient shear strength are presented in Fig. 12. Details and the load-drift response for a shear-deficient column strengthened with a 1/4 in. thick A36 steel jacket (Specimen SC-7) are presented in Fig. 13. The concrete compressive strength for both specimens was roughly 3000 psi. The unretrofitted specimen reached its ultimate strength at a drift less than 2%, while the strengthened specimen maintained its strength to approximately 6% drift. The grouted steel jacket increased the shear strength of the unretrofitted specimen by approximately 40%.

**ANALYTICAL STUDIES**

Inelastic static and dynamic response analyses of a number of low and medium-rise non-ductile frame buildings and rehabilitation schemes have been performed by Jordan, Badoux, and Pincheira (10, 11, and 12) using modified versions of LARZ (13) and DRAIN-2D (14). In the remaining material, an overview of some of the analytical models developed at UT, and selected results from one of the simpler buildings studied with these models are presented. Results of analyses performed by Jordan and Pincheira on a prototype non-ductile three-story building, two concentric steel bracing rehabilitation schemes, and two infill wall schemes were selected for presentation here.

A plan view and elevation of a typical frame in the longitudinal direction of the three-story non-ductile prototype building are presented in Fig. 14. The building was designed to satisfy the force requirements of the 1964 UBC (4) and was proportioned and detailed according to the 1963 American Concrete Institute Building Code (15). Normal-weight concrete with a compressive strength of 3000 psi and Grade 60 steel were used in the design. Because the lateral forces prescribed by the 1964 UBC were
relatively low (compared with modern standards), member design was
governed by gravity loads. As a result, deficiencies in the structure were
typical of non-ductile frame systems (see Fig. 2): short lap splices (24 bar
diameters) at the base of columns, no transverse reinforcement in joints,
light confinement, and short anchorage lengths (6 in.) for beam bottom
reinforcement.

Before presenting results of the analytical studies, underlying assumptions
used in application of the inelastic analysis programs, and details of
selected hysteretic models developed at UT are presented.

Analytical Models

Primary assumptions utilized in the development/modification of the analy­
sis programs LARZ and DRAIN-2D utilized in the analytical studies by
Jordan and Pincheira, respectively, were reasonably similar. Discrep­
ancies are noted where appropriate. These assumptions included the
following:

a) The building was idealized as a series of planar frames linked at
story levels by rigid diaphragms.
b) Masses were assumed to be lumped at each story level.
c) Elastic axial deformations of column and wall members were in­
cluded.
d) Elastic shear deformations of beams, columns, and walls were in­
cluded.
e) Joints (common region at intersection of beams and columns) were
assumed rigid in DRAIN-2D. Inelastic joint deformations were
modeled in LARZ.
f) Gravity loads were applied as initial member end actions.
g) Torsional response of the building was neglected.
h) Earthquake excitation was applied in only one horizontal direction,
and all support points were assumed to move in phase.
i) Effects of soil-structure interaction were included in DRAIN-2D.
j) Secondary moments due to P-∆ effects were neglected.
k) Slip of reinforcement at joints was included in LARZ.

Inelastic behavior of beam and column elements was idealized using the
one-component model originally proposed by Giberson (16). The element
is composed of an elastic portion that possesses flexural, axial, and shear
stiffness, and two nonlinear rotational springs at the member ends
connected in series to the elastic portion. The element is assumed to
deform in double curvature with its inflection point at mid-length. All
inelastic deformations are introduced by means of moment-rotation rela-
tionships for the nonlinear springs. To model the inelastic flexural behavior of reinforced concrete members under reversed cyclic load, modified versions of the Takeda and SINA hysteresis models (17, 13) were developed. Modifications were introduced to the models in order to represent splice failures and pullout of beam bottom flexural reinforcement. Rules incorporated in the Takeda and SINA hysteresis models operate on bilinear and trilinear moment-rotation relationships, respectively, developed for monotonic loading conditions. Figure 15 illustrates the cyclic moment-rotation response representative of a splice failure at the base of a first-story column in the unretrofitted three-story building, computed using the modified SINA model. Although the hysteresis models operate on a primary curve that is derived from basic principles, the rules that govern stiffness and strength deterioration, and pinching of the hystereses are very empirical. The parameters which affect rules that control the shape of the hysteresis loops were established by calibrating the models using results of tests on beam-column connections and jacketed beam-column connections performed by Guimaraes, Kreger, and Jirsa (18) and Alcocer and Jirsa (7).

Structural infill walls were modeled using the equivalent-column analogy (column with moment of inertia of the wall and rigid beams at each floor level). Inelastic behavior was modeled using the same element used for the beams and columns. Because walls are expected to deform in single curvature, each wall element was subdivided into short subelements (typically four or more per story).

Medium-slenderness-ratio braces (50 < kl/r < 120) were used in one of the rehabilitation schemes considered here for the three-story building. Inelastic behavior of the braces was represented using the DRAIN-2D buckling element. The model, which was developed and based largely on tests conducted at the University of Michigan (19), represented elasto-plastic tensile behavior and post-buckling behavior observed for square tube sections and single and double-angle concentric braces. The basic hysteretic behavior is illustrated in Fig. 16.

Shear failure in members and nonlinear joint behavior were modeled in DRAIN-2D and LARZ, respectively (12, 10). Because neither behavior had a significant impact on the response of structures analyzed in this paper, details for these models are not presented here. Shear strength of columns and beams in the unretrofitted three-story structure was sufficient to develop flexural capacity of the members. In addition, even though joints in the unretrofitted structure contained no transverse reinforcement, computed joint shear stresses for perimeter frames did not exceed $13\sqrt{f_c}$ for all but one of the earthquake records considered here, and reached $17.7\sqrt{f_c}$ in a limited number of connections for the other record. Limited experimental data from research conducted by Alcocer
and Jirsa (7) on beam-column joints of non-ductile frames indicated that stresses as high as $15\sqrt{f_c}$ can be sustained up to drift levels of 2%. This and the fact that computed dynamic response histories differed very little for the unretrofitted building with and without the degrading joint shear model, suggest that the joint shear model does not impact the building strength nearly as much as column splice failures and pullout of beam flexural reinforcement.

The effects of soil-structure interaction were approximately accounted for in DRAIN-2D by increasing the fundamental period of the structure and changing the effective damping of the interacting soil-structure system as described by Veletsos (20). Because the computed effects of soil-structure interaction were negligible (12), further details are not presented in this paper.

**Results of Analyses**

The results of "pushover" analyses and time-history analyses conducted on the unreinforced structure and the original structure strengthened using two concentric bracing schemes and two infill wall systems are presented and discussed.

**Selected Ground Motions** -- The three ground motions selected for use included: El Centro 1940 N00E (scaled to a peak acceleration of 0.5g), Corralitos N00E from the 1989 Loma Prieta Earthquake (peak acceleration of 0.63g), and Mexico City - SCT1 N90E from the 1985 Mexico Earthquake (peak acceleration of 0.17g). Ground motions were selected to represent strong shaking on firm and soft soil conditions in regions of high seismic risk. The El Centro record was selected because of the wide frequency band it is capable of exciting. Both El Centro and Corralitos were selected because of their high frequency content, typical of "near-fault" earthquakes on firm soil conditions. Mexico City - SCT1 represents one of the most severe records ever measured for soft soil conditions. Elastic response spectra with 2% viscous damping are presented in Fig. 17.

**Response of Original Structure** -- Using the nonlinear models described earlier and cracked-section properties for members, an estimate of the lateral strength of the building was obtained by applying increments of uniform lateral load in both directions of the structure. A uniform load distribution was used instead of an inverted triangular distribution because preliminary dynamic analyses indicated this was most representative of inertia forces associated with inelastic behavior at times of maximum response. Results of the "pushover" analysis for the longitudinal direction
are presented in Fig. 18 as the base shear coefficient (base shear / building weight) versus drift at the centroid of inertia forces (effective height of building for first mode response).

Three major events are noted on the plot: initiation of bottom beam bar pullout, initiation of splice failure at the base of first-story columns, and failure of splices at the base of all first-story columns. Behavior was linear up to a base shear coefficient of approximately 0.11 when beam bar pullout initiated. Stiffness did not decrease significantly until first-story column splices began to fail at a base shear coefficient of 0.13. First-story displacements increased significantly with small increases in load until all splices at the base had failed at a drift of approximately 0.75%. Because further nonlinear response resulted in marginal increases in strength, failure of splices at the base of all first-story columns appeared to provide a reasonable estimate of the lateral strength of the structure.

Also shown in Fig. 18 are the base shear coefficients required by the 1964 UBC and ATC-22 (21). The computed strength of the building was approximately 1.5 times the lateral strength specified by the 1964 UBC. This was not unusual because the design was governed by gravity loads. The strength required by ATC-22 for an "Ordinary Moment Resisting Frame" (OMRF) was approximately three times the computed strength of the building, and demonstrates the great difference between forces specified in current design and evaluation standards and the strengths available in buildings designed according to obsolete codes. This disparity alone, without regard to the reinforcing details used in the structure, strongly suggests that the structure should be rehabilitated.

Envelopes of interstory drift for the three earthquake records considered are illustrated in Fig. 19. Maximum interstory drifts occurred in the first story for the scaled El Centro and Mexico City records and exceeded 3 and 7%, respectively, while the maximum interstory drift for the Corralitos record occurred in the second story and was slightly less than 2%. The large interstory drifts computed for the soft-soil record and one of the firm-soil records indicate that damage to the frame would be severe. Splice failure was predicted in all first-story and some second-story columns, and pullout of bottom beam reinforcement was predicted for all beams at the first and second level for the El Centro record. Excessive interstory drifts computed for the soft-soil record resulted in loss of flexural strength in most of the beams and columns. The large drifts and loss of strength in many members suggest that the building would probably collapse when subjected to such a record.

Description and Behavior of Retrofit Schemes -- Details of four different retrofit schemes considered here (two utilizing concentric steel braces and two using infill walls) are presented and evaluated. In order
to minimize disruption to occupants of the building, all retrofit schemes were assumed to be installed on the perimeter of the building. With the exception of columns that were jacketed to serve as boundary elements for infill walls, existing columns and beams were assumed to remain unstrengthened. For all schemes, braces or walls were proportioned with the intent of limiting lateral drifts to levels that would not cause significant damage to existing gravity-load-carrying members.

Placement and size of braces in the two bracing schemes are shown in Fig. 20. Braces consisted of double-angle sections that were selected to limit drifts to acceptable levels and to meet the slenderness requirements of the AISC Load and Resistance Factor Design provisions for seismic regions (22). The slenderness ratio for out-of-plane buckling of the braces, which governed the design, ranged from 82 to 98. Yield strength of the steel members was 50 ksi.

The monotonic response of the two braced systems (DA 1 and DA 2) is illustrated in Fig. 21 by the base shear coefficient versus drift response (at centroid of inertia forces). As discussed earlier, these responses were computed for increments of uniform load applied on the structure. Figure 21 indicates that the initial stiffness of the two retrofit schemes was substantially higher than the unretrofitted structure. Initial buckling of braces occurred at approximately 0.2% drift for both configurations, and stiffness reduction following brace buckling was more apparent for DA 1. Yielding of braces occurred at approximately 0.5 and 0.35% drift for DA 1 and DA 2, respectively. Buckling and yielding of braces initiated in the first-story for both retrofit schemes. Ultimate strength of both systems was attained soon after yielding of all braces in the first story. Failure of splices in all first-story columns occurred soon after yielding of first-story braces. This was especially true for DA 2. Differences in distribution of stiffness and strength in the two retrofit schemes resulted in failure of first-story column splices at 60% of the drift required for splice failures in DA 1.

Figure 21 also displays the base shear coefficients specified by ATC-22 for two types of structural systems: an OMRF and a CBF/IMRF (concentrically braced frame with intermediate moment frames capable of resisting at least 25% of the lateral loads). Although the frames in the existing structure did not contain reinforcing details that comply with requirements for an IMRF, these details did not fail prior to developing the strength of either bracing system. Furthermore, the lateral resistance corresponding with yielding of the braces in either configuration exceeds the lateral strength suggested by ATC-22 for either an OMRF or a CBF/IMRF.

In order to further compare the behavior of the unretrofitted and retrofitted buildings, the dynamic response to the different earthquake records and resulting member performance are compared. The fundamental period of
vibration for the structure was reduced from 1.11 sec for the original structure (based on cracked section behavior) to 0.40 and 0.30 sec for the DA1 and DA2 bracing configurations, respectively. Maximum interstory drifts for the braced-building configurations and different earthquake records are presented in Fig. 22. Configuration DA2 was successful in restricting interstory drifts to less than approximately 0.7% for all cases. It was most effective for the soft-soil record where the maximum interstory drift was less than 0.2%. Use of bracing configuration DA2 practically eliminated all hinging of beams and columns in perimeter frames. Failure of first-story column splices was not prevented in interior frames, but the amount of inelastic rotation at the base of these columns was minimal (less than 1.8 times the rotation at peak moment).

Bracing configuration DA1 was successful in reducing drifts below levels experienced in the unretrofitted structure. However, it was not sufficiently effective in reducing interstory drifts resulting from the Corralitos record. The maximum interstory drift for the second story was approximately 1.7%. Even though first-story columns in the perimeter frames did not experience hinging, all but one second-story column and all beams at the first and second level developed hinging. Nearly all first-story columns and beams at the first level of interior frames developed hinges, and all but one of the second-story columns developed hinges at both the top and bottom. The excessive inelastic behavior in the second story is consistent with computed brace deformations; elongation of braces was greatest for second-story braces. It should also be noted that second-story braces in configuration DA2 did not yield. (All braces in the first story yielded for both configurations.) Results for retrofit scheme DA1 indicate that the second story is still susceptible to collapse. In addition, the fact that interstory drifts were largest for the second story for all base motions strongly suggests that the strength of braces in that story should be increased or the entire bracing system reproportioned.

Concentric bracing systems also impart axial tension and compression forces to frame members. For the cases considered here, computed compressive forces in columns reached as high as 65% of the estimated capacity. Estimated tensile capacity was exceeded in most beams at the first and second level and in a number of first-story columns (two-thirds of first-story columns in the perimeter frames associated with DA1). These results indicate that a number of members may need to be strengthened to accommodate the large tensile forces, especially in regions of column splices where high tensile forces will expedite failure of splices.

Details for the two structural wall schemes are presented in Figs. 23 and 24. Wall scheme WRJ consisted of an infill wall added to the center bay of perimeter frames in the short direction of the building (10), as shown
in Fig. 23. The second wall scheme, WJP, is similar to WRJ except that infill walls were added to the perimeter frames in the longitudinal direction of the building (12). Both were intended to limit or preclude inelastic deformations experienced by unretrofitted members in the three-story structure. Figures 25 and 26 show the base shear coefficient versus drift relations for the original building and the two wall schemes. The same relation is shown in Fig. 26 for bracing scheme DA2. It is clear that the walls increased the initial stiffness of the building even more than bracing scheme DA2. However, neither wall system provided as much strength as the bracing scheme. WRJ and WJP yielded at base shear coefficients of 0.3 and 0.46, and at drifts well below those corresponding with failure of column splices. The base shear coefficient associated with failure of each wall retrofit scheme was approximately 0.53 for WRJ and 0.77 for WJP, and corresponded with fracture of longitudinal reinforcement (at a stress of 1.5fy) in boundary elements. Strengths specified by ATC-22 for an OMRF and a dual system consisting of walls and intermediate moment resisting frames (WALL/IMRF) are shown in Fig. 26 for wall scheme WJP. The provisions of ATC-22 do not consider dual systems with a wall and OMRF. Therefore, the base shear coefficient shown in Fig. 26 (WALL / IMRF) can be considered a lower bound for the required strength. Both the yield and ultimate strength for wall scheme WJP exceeded the strength suggested by ATC-22.

Both wall retrofit schemes were successful in reducing drifts and limiting damage in the three-story building. So much so that interstory drifts for the soft-soil record (Mexico City) were less than 0.05%, resulting in elastic behavior of the frame members. This suggests that both wall schemes provided excessive stiffness and strength for soft-soil conditions and should be redesigned. Response of the two wall schemes to the firm-soil records resulted in interstory drifts that did not exceed 0.5% for retrofit WJP and 0.8% for WRJ. Interstory drifts for the Corralitos record were from 20 to 30% larger than drifts for the scaled El Centro record. Wall scheme WJP performed somewhat better than WRJ. Column splice failures in perimeter frames were completely eliminated and beam hinging (pullout of bottom reinforcement) was limited to beams adjacent to the walls for wall scheme WJP. Column and beam hinging in interior frames was non-existent for this case. Splice failures in exterior columns of perimeter frames for wall scheme WRJ occurred during response to the Corralitos record, but not for the scaled El Centro record. Pullout of beam bottom reinforcement was predicted for both records. Damage in the buildings was consistent with curvature ductility demands at the base of the walls. Curvature ductility ratios of 19 and 6 were computed for the Corralitos record for wall schemes WRJ and WJP, respectively.
SUMMARY AND CONCLUSIONS

A sampling of experimental and analytical research conducted since 1981 at The University of Texas at Austin on repair and strengthening of non-ductile reinforced concrete frames was presented. Conclusions from the experimental studies were presented in that section of the paper. Only conclusions from the analytical study of four retrofit schemes (two utilizing concentric braces and two utilizing infill walls) are presented here.

1. A myriad of retrofit schemes can be designed to improve the seismic performance of the original three-story non-ductile reinforced concrete frame structure. "Pushover" (nonlinear monotonic) analyses of the retrofit systems presented here indicated that all of the systems were capable of developing their strength before significant inelastic deformations occurred in non-ductile details in the original structure.

2. Even though pushover analyses seemed to indicate that the selected retrofit schemes were appropriate for the non-ductile frame structure, dynamic analyses of these retrofit systems subjected to both firm and soft-soil conditions revealed conflicting results. Both bracing schemes DA1 and DA2 reduced interstory drifts below levels experienced by the unretrofitted structure, but DA1, which was more flexible and had lower strength, did not control drifts sufficiently to preclude large inelastic deformations in gravity-load-carrying elements. In fact, DA1 appeared to leave the structure susceptible to second-story collapse for strong ground motions on firm-soil.

3. Results were not so disparate for the wall retrofit schemes. Retrofit WJP (the stiffer and stronger of the two schemes considered) was more effective in preventing inelastic deformations in non-ductile elements than retrofit WRJ. Inelastic demands at the base of the walls was approximately three times greater for wall scheme WRJ than WJP. Both wall schemes appeared to be grossly excessive for soft-soil conditions. It should be noted that for either wall retrofit scheme, significant retrofit of the foundation would likely be necessary.

4. For bracing retrofit systems the level of axial forces induced by the steel braces may adversely affect the lateral strength of the existing reinforced concrete member. As a result, jacketing of columns and beams may be necessary to ensure adequate performance of the bracing system.

5. Although the ATC-22 recommendations appeared to be useful for assessing the adequacy of the original, unretrofitted structure, they were not appropriate for determining the level of strength required of the retrofit systems.
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REFERENCES


15. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-63)," American Concrete Institute, Detroit, Michigan, 1963, 144 pp.


![Comparison of base shear coefficient versus building period for ordinary moment-resisting frames](image)

* UBC 94 does not allow OMRSF in zones of high seismicity
Fig. 2—Examples of nonductile details for moment frames designed in the late 1950s and 1960s.

Fig. 3—Original frame model — two bays between third and fifth levels of seven-story prototype exterior moment-resisting frame.
Fig. 4a—Pier-strengthened frame model

Fig. 4b—Load-drift response for original frame and pier-strengthened frame
Fig. 5a—Steel bracing retrofit system

Fig. 5b—Load-drift response for steel bracing system
Fig. 6—Dimensions of beam-column connection specimen
Fig. 7—Reinforcement details for jacketed specimens
Fig. 7 cont.—Reinforcement details for jacketed specimens
$f_c' = 4 \text{ ksi}$

$f_y = 60 \text{ ksi}$

Fig. 7 cont.—Reinforcement details for jacketed specimens
Fig. 8—Details of joint confinement cage
Fig. 9—Response envelopes for jacketed connections
Fig. 10—Reinforcement details and load-drift response for specimen FC-4 with inadequate splice
Fig. 11—Details and load-drift response for retrofitted specimen FC-9
Fig. 12—Reinforcement details and load-drift response for specimen SC-3 with inadequate shear strength
Fig. 13—Details and load-drift response for retrofitted specimen SC-7
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Fig. 16—Hysteretic model for medium-slenderness-ratio brace
Fig. 17—Elastic pseudo acceleration response spectra

Fig. 18—Base shear coefficient versus drift for unretrofitted structure
Fig. 19—Envelopes of interstory drift for unretrofitted structure

Fig. 20—Schematic of bracing schemes DA1 and DA2
• First buckling of braces (schemes DA1 and DA2)
• First yielding of braces in tension
• Splice failure in all columns of the first story

![Base shear coefficient versus drift response for structure retrofitted with braces](image)

**Fig. 21**—Base shear coefficient versus drift response for structure retrofitted with braces

![Envelopes of interstory drift for structure retrofitted with braces](image)

**Fig. 22**—Envelopes of interstory drift for structure retrofitted with braces
SECTION A - A

Fig. 23—Details of infill wall scheme WRJ

SECTION A - A

Fig. 24—Details of infill wall scheme WJP
Fig. 25—Base shear coefficient versus drift for structure retrofitted with wall WRJ

Fig. 26—Base shear coefficient versus drift for structure retrofitted with wall WJP
Novel Dynamic Design Tools for Deepwater Towers that Achieved Breakthrough Results

by D. G. Morrison

Synopsis: The design of deepwater bottom-founded towers (300 to 1,000m) requires a good understanding of the nature of the design environment, the structural response, design force levels and practical member sizing. The novel design tools described in this paper included the "Designer Wave" and the "Quickwave" methods. The "Designer Wave" is a practical short portion of random wave simulation that captures enough of the structural response -- and shear and moment envelopes -- for design purposes. The "Quickwave" method achieves reasonably accurate design forces and member sizes without using time consuming random wave runs and full 3-D structural models. The Designer Wave is essential for the occasional calibration of the Quickwave results. Many design iterations are relatively easy with the Quickwave -- so much so that it was extensively used to derive a new deepwater compliant tower concept. The new tower configuration resulted in breakthrough savings in weight and costs relative to existing solutions.

Keywords: Damping; Novel cements; quickwave; shear properties; towers
Denby Grey Morrison obtained MS and Ph.D. degrees from the Univ. of Illinois, at Urbana-Champaign and did his dissertation with the guidance of Prof. Mete Sozen. He has designed and analyzed numerous deep water bottom-founded towers for water depths from 300m to 1,000m. Recently he and colleagues from Shell Offshore Inc. were awarded patents for the novel Tensioned Riser Compliant Tower concept.

INTRODUCTION

Deep-water bottom-founded towers for oil and gas production have already surpassed the height of land-based structures like the Sears Tower in Chicago. Similar to earthquake resistant design of high-rise land-based structures there is significant dynamics involved in the design of offshore towers. In water depths up to 412 m (1,350 ft) offshore towers have resisted environmental forces mostly by way of stiffness, resulting in dynamic response that amplifies the applied wave forces. Designs for deeper water depths up to 1,000 m have allowed the towers to comply to the environmental forces more flexibly such that the environmental forces are de-amplified in the dynamic response. This paper deals with the latter class of offshore towers intended for use in 300 to 1,000 m and referred to as “compliant towers”.

This paper describes two methods that were important in deriving a novel deepwater compliant tower concept: the Designer Wave, and the Quickwave. These methods were developed to capture the appropriate load levels that the deepwater towers are subjected to, and were used in the many design iterations required to establish the technical and economic feasibility of the new deepwater concept. The novel configuration will not be discussed, rather the emphasis is on the technology developed and used to achieve the goal.

The Compliant Tower Concept in Deepwater Oil & Gas Recovery

Deepwater oil and gas reserves have become a reality with the advances in locating the potential reservoirs, and in the structural concepts -- both floating and bottom-founded -- that can economically provide working platforms for the production and drilling operations.

The compliant tower is a bottom-founded concept that has undergone over a decade of refinement. Exxon was the first to use the notion of
allowing the bottom-founded tower to comply with the waves (1) rather than resist extreme hurricane waves by stiffness alone -- the way bottom-founded towers had in water depths up to 412m (1,350ft) in the Gulf of Mexico. Many other oil companies have compliant tower designs ready for the right combination of water depth, reservoir size, production rates and platform processing requirements that would make the compliant tower the most economical solution.

Substantial research has established that there is a desirable range of structural periods in a compliant tower so that the system does not undergo significant excitation from waves associated with the hurricane spectrum. Usually this requires a long sway-type mode of response with periods between 30 and 50 seconds, and a short bow-shaped mode of response that has periods less than 9 seconds. The main energy in a hurricane spectrum lies around 16 seconds, and the structure is configured to give periods of vibration on either side of the periods of the energetic waves.

The risks and economic exposure in developing deepwater reserves were the reasons for deriving the Designer Wave and Quickwave methods in order to further optimize the compliant tower.

The Design Process and the Need for Methods like Designer Wave and Quickwave.

The design process must address the complexity and size of the structures being discussed, the difficulty in choosing initial member sizes on which to base further analysis, the size of computer models, the time and resources required to do detailed finite element modeling of the towers, the significant dynamic response and time required for detailed dynamic analysis, the randomness and complexity of the environmental wave loads, and the numerous iterations required to optimize the structural configuration. The Designer Wave and Quickwave assist in improving the design process.

The Designer Wave: The Designer Wave is used to find the “right” wave for design purposes. Given a set of environmental criteria that describe the “100-year” hurricane sea state statistically (e.g. the significant wave height, peak periods, type of storm spectrum, associated current, wind) the “right wave” must be found for use in design -- the “Designer Wave”.

The Designer Wave approach takes hours of random wave simulations, and captures a segment of about 100-seconds to generate design loads. This portion of random wave trace, the Designer Wave, must be able to produce level shears and moments as close as possible to the
statistically derived “100-year” extreme force values. Figure 1 (a) depicts this process.

The advantages of the Designer Wave lie in being able to construct accurate level shear and moment envelopes that are used to obtain a reasonable number of load cases for detailed design of members. The method is described in more detail elsewhere (2), and this paper will emphasize how the “Designer Wave” method augments the “Quickwave” method in the design of a 1,000m water depth structure.

The Quickwave: The “Quickwave” method is a series of steps that: (1) Simplify the structure to its most important response components, e.g. piles act as springs, legs as beam elements. (2) Calculate the most important dynamic properties, i.e. the first two modes of vibration. (3) Calculate the response of the tower to random design seas as being represented by an impulsive load followed by a free-vibration. The resulting few cycles of response are used to create design level shear and moment envelopes. (4) Design main legs and braces by using the design envelopes and good design practice, to resist the environmental loads, the in-place hydrostatic loads, and have adequate slenderness and reserve buoyancy properties (See Fig. 1(b)).

The Designer Wave is required to calibrate the Quickwave when deemed necessary. The Designer Wave is an essential step in the detailed design process, whereas the Quickwave is most useful in deriving the initial configuration.

The following sections describe the important features of the Quickwave algorithms, calibration of the Quickwave impulse algorithm, use of Quickwave in initial design and calibration of Quickwave when design criteria change significantly.
IMPORTANT FEATURES OF QUICKWAVE

Simplifying Complex Compliant Tower Configurations

A large "knowledge base" has been developed for compliant towers over the past decade. Despite this knowledge, the Designer Wave and Quickwave methods make design iteration substantially easier.

The designer has a number of physical properties to adjust in order to achieve the desired period ranges. Many references are available on the topic of the relative sensitivity of periods to the tower width, the extension of piles up the tower sides in order to extend the flexibility, location of pile-to-tower connection, soil-tower relative stiffnesses, and member cross-sectional properties (3,4,5,6). It is helpful to visualize the complex system as a beam: very similar to a cantilever if the piles are terminated at the mudline, or a "double cantilever" if the piles are extended up the side of the tower and transferring vertical loads at midheight. Figure 2 indicates the representation of two complex tower models by simple beams (the "Flextower" has piles terminated at the mudline, whereas the compliant piled tower, "CPT", has piles extended up the side of the tower for extra flexibility). In order to define the "beam" to be used in the iterations, the following tabulates those structural characteristics that are most important to the response in most applications (300m to 1,000m):

<table>
<thead>
<tr>
<th>EXPERIENCE &amp; DESIGN CHALLENGES</th>
<th>EXPLICITLY INCLUDED In QW (Quickwave)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Important Features of the Pile-Tower Configuration</td>
<td>Location of centroid of the pile group relative to beam centerline; properties of pile, D and t; length of pile and pile-tower connection above the mudline; axial and lateral pile-soil stiffness below or at mudline. Lateral stiffness is important in most bow-shaped mode calculations, less so in sway mode.</td>
</tr>
<tr>
<td>Important Dynamic Characteristics</td>
<td>Models the long sway mode, and the shorter bow-shaped bending mode by iterating on likely frequency till boundary conditions are met. Approximations can be made for mass distribution in the tower. Mass for payload and piles are included easily &amp; accurately. The tower is usually divided into about 4 main regions to calculate the equivalent &quot;beam&quot; stiffness.</td>
</tr>
</tbody>
</table>

Table 1: Some Quickwave Approximations in Modeling Configuration
The Myklestad (7) method is easily adapted to modern-day spreadsheet formulations. The method quickly determines the mode of vibration that matches a given boundary condition, such as a fixed base or lateral deflection that equals deflections from a known lateral foundation stiffness. The periods calculated with the models depicted in Fig. 2, are given in Table 2.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Quickwave T1 (sec)</th>
<th>3-D Model T1 (sec)</th>
<th>Quickwave T2 (sec)</th>
<th>3-D Model T2 (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flextower, 80 wells</td>
<td>32.9</td>
<td>31</td>
<td>6.5</td>
<td>6.9</td>
</tr>
<tr>
<td>CPT, 80 wells</td>
<td>61.6</td>
<td>61</td>
<td>9.1</td>
<td>8.9</td>
</tr>
</tbody>
</table>

*T1* and *T2* are the sway mode and the bow-shaped bending mode periods.

**TABLE 2: Models Representing a Wide Range of Structural Periods**

**Modeling the Extreme Design Wave as an Impulse**

Quickwave uses a closed-form solution to compliant tower response built on years of research (1,8,9,10,11).

A number of researchers have alluded to the impulsive nature of the response, but Quickwave was the first to base the response calculations on this assumption (12). The modal accelerations are found from the damped transient response to a half-sine impulse. The modal accelerations are used to calculate modal inertia loads that are combined with wave, wind and current loads to give shear and moment distributions along the height of the tower.

The “amplification factor” and damping components of the closed-form solution for the accelerations will be emphasized because these give insight into the physical behavior. The format is as follows:

\[
\alpha_n = \left( \frac{F_o^*}{M^* \omega_n^2} \right) AF \left( -\omega_n^2 \right)
\]

\[
\{ \sin(\omega_n t) + \frac{\omega_n}{\omega_i} e^{-\xi \omega_n t} \left( \xi^2 \sin(\omega_n t) - 2 \xi \cos(\omega_n t) - \sin(\omega_n t) \right) \} \]

(1)

where:

- \( \alpha_n \) = modal acceleration;
- \( F_o^* \) = applied effective modal load;
- \( M^* = \sum_i \phi^T m_i \phi \) where \( \phi = \)mode shape, and \( m_i = \)level masses;
• \( \omega_n \) = natural frequency of the mode;
• \( \text{AF} \) = amplification factor derived from an upper bound to Veletsos (11) response spectra;
• \( \omega_i \) = "input" frequency (of load); the impulse duration was taken as half of a 12-sec period;
• \( \xi \) = critical damping ratio associated with the particular mode

The impulse duration was derived from design approximations that have used 12-sec. waves to represent Gulf of Mexico hurricane-type waves. In the following sections, the importance of the peak periods associated with the design sea state are investigated, and it is shown that the period may have to be lengthened if the sea state has much longer peak periods than the Gulf of Mexico criteria.

The damping associated with the sway bending mode is usually large for extreme wave conditions when the tower is moving with an amplitude of more than 10 ft at the deck level. A damping ratio of 20% of critical is considered to be reasonable. The bow-shaped bending mode is considered to be vibrating at much smaller amplitudes and therefore a damping ratio of 5% is reasonable for this mode.

The "AF" in the closed-form for the modal accelerations, is the dynamic amplification ratio of static displacements. Veletsos (11) subjected compliant tower models to the measured Camille record, as well as to numerous random wave simulations from the Camille storm spectrum. He constructed response spectra relating AF to structural period for different damping ratios. These response spectra are particularly useful because the random nature of the loading is incorporated. In order to use the Veletsos results in a closed-form representation, Quickwave makes a simple upper-bound approximation that holds in the two structural period ranges of interest, the long sway mode and the short bow-bending:

\[
\text{DAF} = 1 / \left( \sqrt{1 - r^2} + 2 \xi r \right) \]

(2)

where:
• \( r = \omega_i / \omega_n \), the ratio of input frequency to natural frequency;
• \( \xi \) = modal damping as % of critical damping.

Figure 3 depicts this approximation, and Table 3 shows the values for the "input" frequencies (representing the waves) and damping ratios in the ranges of interest.
TABLE 3: Frequency and Damping Values Used to Approximate the Upper Bound of Design Response Spectra

<table>
<thead>
<tr>
<th>Mode</th>
<th>1/\omega_v T_v (sec)</th>
<th>\xi (%) critical</th>
</tr>
</thead>
<tbody>
<tr>
<td>\omega_{n1}, sway-bending</td>
<td>15</td>
<td>20%</td>
</tr>
<tr>
<td>\omega_{n2}, bow-shaped bending</td>
<td>12</td>
<td>5%</td>
</tr>
</tbody>
</table>

The combinations are intuitive. It could be expected that the response spectra are going to be more sensitive to wave components with relatively long period content (15 sec.) in the sway mode of vibration. Also, relatively large amounts of equivalent damping are thought to exist from this mode, especially if relative velocity is taken into account in the Morison's equation. The bow-bending modes would respond more to the shorter wave periods (around 12 seconds), and lower damping values have been used in design of short period structures and lower modes (13, 14).

COMPARING QUICKWAVE WITH DESIGNER WAVE: IMPULSE VS STATISTICAL EXTREME RANDOM WAVE

The amplitude of the Quickwave impulse affects the design shear and moment envelopes significantly. The amplitude could be found from expected load levels from previous experience of detailed random wave analyses. The Designer Wave represents the "best" fit to the statistically expected extreme load levels, and a comparison with Quickwave will demonstrate whether the response is truly impulsive in nature.

Figure 4 shows a level shear envelope constructed with the Quickwave and Designer Wave. The Quickwave used the same applied force as did Designer Wave. An excellent comparison at all levels was obtained, which demonstrates that if the same applied force is used the impulse approximation of the tower response is good. Other configurations and moment envelopes were compared with the similar favorable results (Morrison, 1994).

The Quickwave suite of worksheets uses the shear and moment envelopes to design major leg and brace sizes. Figure 5 gives an example of the design of legs in a 1,000-m flextower using regular full 3-D design software compared with Quickwave. The diameter and ring-stiffener spacing (for hydrostatic pressure) of the full design was kept the same in Quickwave, and the wall thickness of the legs was changed in Quickwave to resist the in-place loads. The comparisons of resulting wall
thicknesses are extremely good and differences in wall thickness are rarely more than 1/8 in (3mm) -- the usual increment in thickness that would depend on designer prerogative.

Table 4 serves as summary of the needs met by the Quickwave approach to loading and member design.

<table>
<thead>
<tr>
<th>EXPERIENCE &amp; DESIGN CHALLENGES</th>
<th>EXPLICITLY INCLUDED in QW (Quickwave)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Importance of randomness in the seastates</td>
<td>Capture the random sea by way of adaptation to Veletsos Response Spectra. Approximations are made to Veletsos spectra on the sway-mode frequency range and on the bow-shaped mode frequency range.</td>
</tr>
<tr>
<td>Length and time involved in random sea generation</td>
<td>Model the response as an impulsive load and following free-vibration. The amplitude of the impulse load is equal to the statistical (1-hour maximum) applied load. If this is unknown, the best estimate from prior experience is used.</td>
</tr>
<tr>
<td>Need for initial member sizes, tower configuration (e.g. tower width, pile length, and so on)</td>
<td>Using the design shear and moment envelopes (the maxima occurring at any level during the impulse and subsequent free-vibration), and the known vertical loads (payload, dead load etc.), can be used to design legs and braces.</td>
</tr>
<tr>
<td>Need for numerous iterations to set configuration or investigate alternatives (e.g. &quot;what if the payload or water depth requirement changes?&quot;)</td>
<td>Ease of use to change pile configuration, tower width, steel grade and so on.</td>
</tr>
</tbody>
</table>

Table 4: Some Quickwave Approximations in Modeling Loading and Design Iterations

Usually the designer does not have the benefit of a full 3-D detailed model in the initial stages of preliminary design. The following section illustrates a more typical situation.
COMPARING QUICKWAVE WITH DESIGNER WAVE: THE NEED FOR INITIAL MEMBER SIZES AND CONFIGURATION

In a normal design cycle little is known about the configuration, and initial assumptions must be made. The Quickwave can be used with estimated force levels from expected environmental loads, to set the sizes of legs and braces in the structure. Thereafter, the random wave analyses and Designer Wave can be completed for detailed design phases. Even at this stage comparisons with the Quickwave method can aid in further structural optimization. This also gives insight into what was initially assumed and finally used after iterations.

Figure 6 illustrates at least three steps in the design process. The sequence in which shear envelopes in Fig. 6 were developed were as follows:

1. “QUICKWAVE Initial Estimate”: The envelope resulted from an assumed applied load of 9,000 k. This load was derived from previous CPT designs, with a reduction in lateral load assumed because of the change from larger diameter laterally-supported conductors to 9 5/8 in. tensioned risers in the novel configuration. A complete Quickwave design cycle followed and the pile, leg and brace sizes were established. A preliminary tower weight with allowances for miscellaneous items and fabrication, loadout, transportation and launch steel were calculated. The technical and economic feasibility was established.

2. “STATISTICAL TARGET” represents the beginning of a design effort to confirm the preliminary findings. Initial member sizes in the detailed 3-D model were taken from the Quickwave results. The envelope values were statistically derived from hours of random wave simulations that were used to calculate the 1-hour extreme hurricane estimates. These values are the “target” values for a Designer Wave. As part of this analysis a 1-hour extreme applied force of 7,500 k was calculated.

3. “DESIGNER WAVE” is the envelope from the practical, short segment of random wave that comes closest to matching the extreme value estimates of shears and moments throughout the height of the tower. It is used to construct load cases for the detailed design.

4. “QUICKWAVE After Designer Wave”: is the shear envelope that would have resulted from an initial guess using applied load of 7,500 k (the statistically derived 1-hour extreme value).

Despite the differences throughout the design cycle, initial tower tonnages estimates were well within 5% from the final values, and pile steel was
within 15%. Total tower and pile steel weight differed by less than 5%. Other designs have shown final weights to be within 5%-15% of initial estimates (15).

**COMPARING QUICKWAVE WITH DESIGNER WAVE: RE-CALIBRATE FOR SUBSTANTIAL CHANGES IN ENVIRONMENTAL CRITERIA**

Using the Quickwave under very different combinations of wave, wind and current necessitates re-calibration of the components of the impulse algorithm. The Quickwave method is based on experience in the Gulf of Mexico where design environments are driven by 100-year hurricane conditions in which the wave loads dominate. The effect of environments in which severe currents occur jointly with the waves is also analyzed.

Figure 7 depicts overturning moment as a function of fundamental period of the structure. The dotted line (Quickwave calibrated to GOM criteria) shows the trend for the same applied load as the bold line labeled "Quickwave recalibrated". The GOM calibrated Quickwave gives significantly lower overturning as well as more incremental reduction in moment as a function of incremental lengthening of the fundamental period. The main reasons for these differences are to be found in environmental criteria that include much longer period and higher waves, but also significant superimposed currents that affect tower dynamics:

<table>
<thead>
<tr>
<th>CRITERIA</th>
<th>GOM</th>
<th>NEW SITE</th>
</tr>
</thead>
<tbody>
<tr>
<td>significant wave height (ft)</td>
<td>41</td>
<td>59</td>
</tr>
<tr>
<td>peak period of waves</td>
<td>14.4</td>
<td>21.4</td>
</tr>
<tr>
<td>current at surface (ft/s)</td>
<td>3.4</td>
<td>5.8</td>
</tr>
<tr>
<td>current at (-)1,000 ft (ft/s)</td>
<td>0</td>
<td>4.4</td>
</tr>
</tbody>
</table>

**TABLE 5: Comparison of Environmental Criteria for Two Significantly Different Locations, Gulf of Mexico and North Sea**

The main values that were sensitive to the changed criteria were: the explicit modeling of the current as static and 37% of the total applied load; shifting the period of the impulse from 12 to 16 seconds; and lengthening the "input periods" used in the AF approximations (Table 6). The motions in the changed environment did not warrant changes to damping ratios.
upper bound AF for sway & bow modes in the more severe environment: $1/\omega_v T_v$ (sec) | $\xi$ (% critical)
---|---
$\omega_{n1}$, sway-bending (30-40 sec.) | 18 | 20%
$\omega_{n2}$, bow-shaped bending (6-9 sec.) | 15 | 5%

**TABLE 6:** Frequency and Damping Values Used to Approximate Upper Bound of Design Response Spectra

**THE NOVEL METHODS LEAD TO BREAKTHROUGH RESULTS**

The Quickwave method is very easy to use. Because of the simplifications involved it also gives the designer insight into the effect of structural changes during the design iterations. Initially many configurations were screened in an attempt to arrive at a cost-saving structure, and this would have been much more difficult without Quickwave. Because the Quickwave method gave clear insight into the main load paths it led to a bold, simple framing pattern.

A new “Tensioned Riser Compliant Tower “ (TRCT) concept evolved that: did away with having to laterally support conductors under buckling compression; had 4 main legs; had a smaller launch truss; and had huge X-braces (that were unrestricted by horizontal levels previously demanded by the laterally supported conductors).

Figure 8 depicts the salient features of the TRCT relative to the much denser framing patterns of a more traditional CPT (Compliant Piled Tower). A more complete description of the novel TRCT can be found in Smolinski et al (15).

**SUMMARY AND CONCLUSIONS**

It was essential to develop enabling technology that could be used to iterate quickly until the preliminary feasibility of a new configuration could be established. The approximate design methods had to be accurate enough to allow for a good first estimate of configuration dimensions and details (such as the location of piles), the major brace and leg sizes in the tower, and a good estimate of total structural tonnage for costs. In general the design cycle for deepwater towers is complex and lengthy involving numerous random wave simulations, statistical studies on wave forces and response, and detailed structural models for stress analysis.
and design. To put the size of the problem into context with land-based structures, a tower for a 3,000' water depth is almost 3 times as tall as the Sears tower in Chicago, the tallest land based steel framed structure.

The presented approach found solutions to the following:

1. Find a novel, practical wave, that represents just the right loading to cause the one-in-a-hundred year shear and moment envelopes (the "Designer Wave").
2. Find a novel simplified structural representation for the configurations of thousands of members and joints such that dynamic characteristics (mode shapes and periods) could be well matched in the important first few modes of vibration.
3. Find a novel and simple representation of the dynamic response to the desired load levels, the "Quickwave", that modeled the response with a special impulse-type response such that the resulting shear and moment envelopes matched the "Designer Wave" envelopes from the full 3-D model closely.
4. Find simple design algorithms to size the major braces, legs and piles to within 1/8" (the normal wall thickness increment in tubular members used in offshore space-frame towers).

The methods were then used to derive a completely new structural form and then establish its dimensions, its structural member sizes, and the resulting tonnages. The weights of the new configuration were so much less than other compliant tower solutions for the same water depth that a detailed design was undertaken to confirm the promising results. The weight of the tower after detailed design was within 5-10% of the initial Quickwave estimates, and more than 25% less than "conventional designs".

ACKNOWLEDGMENTS

Significant contributions and advice were given by Shell engineers, in particular Susan Smolinski, Dave Edwards, Allan Reece, John Leonard, Yousun Li, Pat Dunn, Rome Gonzales, Bill Luytles, Peter Marshall, Skip Ward and Dave Huete.
REFERENCES


Long simulations to set statistical response targets, e.g., the 1-Hr. extreme values for Dyn.Base Shear, Applied Force and Moment.

Fig. 1a—Designer wave selected from random waves to obtain close to 100 percent of the average 1-hr maximum response.
STRUCTURAL DATA
(Incl. depth, piles, tower, payload, wells, foundation, mass traps, etc. Structure simplified as beams & springs)

DYNAMIC PROPERTIES:
$T_1$, $\Phi_1$ & $T_2$, $\Phi_2$
(periods & mode shapes calculated by iterating on boundary conditions)

1/2 SINE IMPULSE WAVE
(approximates hurricane wave as an impulse load; "period" of the impulse depends on sea state)

DESIGN ENVELOPES:
MOMENT & SHEAR
(Maximum shear and moment throughout the height of tower)

PILE LOADS, DESIGN, WEIGHT
LEG LOADS, DESIGN, WEIGHT
BRACE LOADS, DESIGN, WEIGHT

Fig. 1b—Quickwave providing load and envelopes for member sizing
Fig. 2—Quickwave structural representation

Approx. \( \text{AF} = \frac{1}{\sqrt{\left(1-r^2\right)^2 + (2r\zeta)^2}} \) where \( r = \frac{\omega_v}{\omega_n} \)

Fig. 3—Approximation to Veletsos response spectra
Fig. 4—Envelope quickwave versus designer wave

Fig. 5—Flextower corner leg calculated quickwave versus full
Fig. 6—Initial quickwave to set configuration and subsequent designer wave on 3D model

Fig. 7—Quickwave must be recalibrated if seastate changes
Fig. 8—The Novel TRCT (tensioned riser compliant tower) showing advantages compared with a conventional piled tower
Research Behind the Success of the Concrete Platforms in the North Sea

by I. Holand and R. Lenschow

Synopsis: The development of the concrete offshore structures is illustrated by briefly describing the background for their functions, the development of structural design, brief examples of concrete research and research results, industry research projects and international standardization. Figures and main specifications of typical structures are shown.

Keywords: Concrete platforms; concretes; offshore structures; reinforced concrete; search; standards; strength; structures
Ivar Holand. Honorary ACI member Ivar Holand is professor emeritus of Structural Mechanics and previous director of Cement and Concrete Research Institute at the Norwegian Institute of Technology (NTH). He graduated from NTH in 1948 and got his doctors degree at the same institute in 1958. After a few years in consulting and contracting, he returned to NTH in 1958, and has spent the rest of his life at NTH and the affiliated research organization SINTEF.

Rolf Lenschow. ACI-fellow Rolf Johan Lenschow is professor of structural engineering at the Norwegian Institute of Technology (NTH), University of Trondheim. He graduated from NTH 1954. After 9 years in consulting he went to University of Illinois and got his Ph.D. degree in 1966 and received later honorary doctors degrees at The Royal Technical University, Sweden, and at Strathclyde University, UK. He became professor at NTH in 1969. He has spent seven years at universities and industry in Denmark, Sweden and USA, and two terms as president of University of Trondheim.

BACKGROUND

"The technological development of concrete platforms is one of the greatest successes of concrete in our time, both from a design point of view as well as the material and construction aspect" /11/, Fig 1.

Fig. 1—Norwegian offshore concrete platforms on 4 oilfields in the North Sea (12)
The beginning of the story of the remarkable concrete structures in the North Sea is only 25 years behind us. When the petroleum industry established activities in Norway in the late sixties, an immediate reaction from the Norwegian construction industry was that concrete should be able to compete with steel that was the traditional structural material in this industry. This assumption proved to come true regarding the price as well as the maintenance costs.

The one after the other of the spectacular structures, 22 in total, have been placed on the sea bed in the North Sea reaching up to 60 m (200') above sea level and down to 330 m (1000') at the deepest location, making this structure one of the tallest concrete structures in the world.

Nobody will claim that the spectacular development of concrete structures is the result of strategic research prior to the decision to place a huge cylindrical concrete structure on the seabed, a structure that could serve as a drilling rig, an oil storage, a plant for oil treatment, and a hotel at the same time.

The first concrete platform was the Ekofisk-platform, a French-Canadian concept that was realised with little, and one would today say with too little, research background.

Hydro-dynamic tests carried out in parallel with the construction showed that wave forces were underestimated. Extra last minute prestressing was the solution. The outer cylindrical containment consisting of a large number of prefabricated elements with holes was supposed to protect the inner cylindrical structure presuming that the outer wall with all the holes should be a wave energy killer. To some extent this was right. However, the total force on the structure was greater than it would have been if the outer wall had been removed. Therefore the next concept, which became the winning concept, had a slim form through the wave zone.

However, the brave decision to launch the Ekofisk platform made way for the unique development and research activity not only for offshore structure as such but also for a spectacular development of the concrete material, design methods, construction methods, load predictions, quality management and safety evaluations.
Fig 2 shows the 17 platforms built by Norwegian Contractors 1973-1995. The Ekofisk platform is found in the upper left corner. Figs 3-5 show the three platforms that are now under construction.

Among innovations in the platforms now under construction, in addition to the use of concrete in buoyant structures of this kind, the application of light weight aggregates should be emphasized. Hadron (Fig 4) is built exclusively with LC 60 (characteristic cube strength 60 MPa). In the upper part of Troll Gas (Fig 3) and in parts of the Troll Oil (Fig 5) a so-called modified normal strength concrete LC 75 has been applied. "Modified" implies in this case that one half of the coarse aggregates has been replaced by high-strength Leca (Leca 800, expanded clay produced in Norway). (The light weight aggregates used in high strength concrete in Norway earlier have mainly been Liapor 8, expanded clay produced in Germany).

The application of light-weight aggregates in highly stressed complex structures is by no means straight forward. The light weight aggregates...
Fig. 3—Troll Gas, the by far largest CONDEEP platform, under construction
- Water depth 303 m
- Height of concrete structure 369.4 m
- Concrete volume 234,000 m³
Operator: A/S Norske Shell
Contractor: Norwegian Contractors a.s

Fig. 4—Heidrun, the first tension leg floater with a concrete hull, under construction
- Water depth 345 m
- Hull draft at field 77 m
- Concrete volume 66,000 m³
Operator: Conoco Norway, Inc.
Contractor: Norwegian Contractors a.s
Fig. 5—Troll Oil, the first catenary anchored floater with a concrete hull, under construction

- Water depth 325 m
- Hull draft at field 40 m
- Concrete volume 43,000 m³

Operator: Norsk Hydro a.s
Contractor: Kværner Concrete Construction a.s

intensify all differences found between high strength and normal strength concretes. Thus, the influence of such parameters as tensile strength and fracture energy on traditional design and dimensioning practice must be taken into consideration. Moreover, the production, transport and compaction present challenges in addition to those known from normal weight high-strength concrete.

The research referred to in this paper is part of the development of the Norwegian concepts for fixed concrete platforms and floating concrete platforms.

Nearly all Norwegian oil production is made by the concrete platforms. The daily production is more than 200000 barrels, which makes Norway to be the second biggest oil exporting country in the world.
THE EARLY PHASES OF THE INVESTIGATION OF INSTALLATION OF AN OIL FIELD

In the early phases the base for good design solutions is worked out and also for the most important decisions. 70-80% of the cost is decided upon in a very early stage.

Out of the investment in an oil field for production, the concrete platform will be in the order of 15% of the total installation, as experienced as a common figure onshore.

In Phase 1 the application for licence is completed and technical solutions are evaluated from an overall viewpoint based on previous experience.

Phase 2 contains a cost-benefit study of the field. Studies are carried out to find the optimal economic and technical solution. In this phase the concrete platform concept enters the scene as one of more possibilities.

Important parameters for the structure are: production volume, number of oil-wells, pipelines and risers, shape and weight of the structure and its installation, storage - and further: water depth, wind, waves and currents, seismic loads and geotechnical data.
In this feasibility study the cost estimates of the structure should be within an accuracy of plus/minus 30%. The study to develop a competitive concept is an iterative process. Alternative structural solutions are tried. Loads are calculated and analyses are carried out. The selected alternatives are dimensioned.

This phase represents a special challenge to the "concrete people" to come up with innovative solutions. If this part of the feasibility study does not open for a concrete offshore structure, it is not likely that a concrete structure will be promoted in later phases of the process.

NORTH-SEA OIL PRODUCTION AND RESEARCH ACTIVITIES

The discovery of oil in the North-Sea launched research activity in many fields driven by the demand of safety requirements that should not be lower than those onshore and the economic competition with onshore oil production especially in the Arabic countries.

The research activity has been performed in parallel with the activities in all phases from seismic prospecting for oil to construction and production.

The geological research and the development of the seismic technique have significantly increased the ability to find new oil deposits. Reservoir-research and a spectacular development of the drilling technique have not only made the estimates of new oil-wells much more reliable but have also led to substantial higher output from the wells and reduced the number of platforms or oil-rigs needed in operation.

The heavy gravity platforms and the lighter piled platforms on the sea bed required more knowledge of the sea bed topography and its soil condition. A new method of foundation has been developed, making it possible to place a million tons heavy platform on soft soil by adding an extra "skirt"-wall, round and under the platform.

The hydrodynamic loads on fixed and floating structures in one of the most hostile seas in the world are a critical factor in design, for safety and economy of the offshore structures. In the beginning both the effect of the waves on the structure and the 100 years period waveheight were underestimated. Hydrodynamic phenomena including interaction with dynamic behaviour of the structure have become a field for extensive scientific studies.

Pipelines on the seabed have become an important engineering discipline. Multiphase-flow (oil, water, gas) in pipelines was almost an
unknown field in the beginning of the oil production in the North Sea, at the same time as it was difficult and unpredictable. Intensive research and investment in the $100 millions in equipment have later saved installation cost for more than ten times the research expenses.

There has all the time been a fierce competition between the concrete gravity platform and the piled steel jacket. The research on steel and steel structures got a real push by the development in the North Sea. The disaster of the oil-rig "Alexander Kjelland" activated an unprecedented research activity on fatigue failure of steel structures.

In this atmosphere of pioneering spirit and in an environment where research was considered to be an investment in competitiveness and in the future, there was an opportunity for concrete structures and concrete technology that was utilised fully, - some of which will be illustrated below.

During this 20 years period, economy has favoured a continuous increase of concrete grades from C45 used for the Condeeps at Beryl A and Brent B, to the present C75-C80 (numbers denote characteristic cube strengths). The increase has been allowed by a steadily increasing level of knowledge accumulated through experience and research. Even the loss of the Sleipner platform in 1991 has had a positive effect by contributing to a better understanding of technical as well as organisational issues.

OFFSHORE CONCRETE STRUCTURES

Structural Design

The offshore platforms are subjected to a great number of loading conditions during the construction, towout, installation, operation and removal phases. Large hydrostatic pressures dominate during deck-mating, while waves, currents, winds and earthquake loads dominate during the operation phase. A typical number of design combinations for the present detail design of the Troll GBS (Gravity Based Structure) (Fig 3) is 25000.

The most innovative period was around 1970 when first the Ekofisk concrete platform was launched and the first of the many Condeep platforms was on the drawing table. The hydrostatic pressure on the structure favoured cylindrical and spherical shapes. The Condeep concept consists of cylinders and domes and the first design was based on simple, classical shell calculations. However, the intersections between
the different shell-elements were highly stressed and the waveloads and other loads introduced additional forces to the hydrostatic ones.

The finite element method was developed a few years earlier and proved to be the ideal tool to handle the complicated design.

For design of offshore concrete structures the finite element method itself and especially the elements have been studied through a number of research projects. A finite element calculation may today handle more than one million degrees of displacement freedoms and involve the use of super computers. To handle the huge amount of data from a finite element program an efficient postprocessing is needed to be able to dimensioning the reinforced concrete sections of the structure.
The primary design is normally based on linear elastic Global Finite Element Analyses. The most important feature of the linear elastic analyses is that the principle of linear superposition is valid. However, reinforced concrete has not an elastic linear behaviour. To perform a complete design also nonlinear structural analyses are adapted particularly in the intersection between the shell type structural members. These intersections belong to the so-called D-regions while the shell structures apart from the intersections represent the B-regions. Design based on linear elastic analyses is often satisfactory for the B-regions. Various types of supplementary models have been used for dimensioning the D-regions.

The concrete research has been focused on bringing forth factors, models and properties needed for more accurate design and dimensioning, and to improve design and material.

**The Sleipner Platform Incident**

The loss of the Sleipner Platform demonstrated that the highly automated analyses by finite element methods and dimensioning by post-processors have their pit-falls, and that the personal design skill should not be forgotten. To illustrate this issue a short description of the Sleipner accident is included.

The gravity base structure of the Sleipner A platform is a concrete structure placed at a depth of 82 m in the North Sea for production of hydrocarbons. The platform (Fig 8) is of the Condeep type and has a base structure consisting of 24 cells, with a total base area of 16 000 m². Four cells were elongated to shafts supporting the platform deck. The triangular spaces between the main cells are denoted tri-cells.

Sleipner A sprang a leak and sank under a controlled ballasting operation in Gandsfjorden outside Stavanger, Norway, on 23 August 1991 /19,20/. The platform had been analyzed using the program system MSC/NASTRAN Version 65 C and the basic 8-noded finite solid element (CHEXA-8).
Fig. 9 shows in the right lower corner the element mesh that was used, as generated by the program. As Fig 9 shows, irregular elements were used in the tri-cell wall. This fact is not according to common practice, which is to modify the generated elements manually, and avoid irregular elements in highly stressed regions.

The main figure shows a planar finite element model that was used for a
local analysis of the tri-cell in the post-failure investigation work. The axial force N was given values taken from the original global analysis.

Fig. 9—Generated element mesh for tri-cell. Planar element model

Fig 10 shows results obtained for shear forces using the NASTRAN program for an analysis of the simplified model. Values in the midpoints were used and extrapolated to the boundary.

Fig 10 shows considerable errors for shear forces in the mid-points of the elements. The parabolic extrapolation enlarges the error, with the result that the boundary shear force is about 60 per cent of the beam theory solution.

The deviations from equilibrium are caused by irregular elements and by the extrapolation scheme. A regular mesh will give the beam theory solution.
In a complementary part of the post-failure investigation, the strength of the tri-cell walls and the corner areas where the tri-cell walls meet, was checked by analyses by formulas in standards, strut and tie models and finite element analyses. Moreover, a model of the corner was tested in full scale /21/. The test programme included ten specimens representing the probable failure area. Seven specimens (Y1-Y6 and Y8) were reinforced as built. A main problem in the "as built" design was that T-headed bars (see Fig 11) used as transverse reinforcement in the haunch, were too short. Thus, one specimen (Y7) was produced with T-headed bars in the haunch elongated to the compressive reinforcement at each side, and two (Y9 and Y10) with a more comprehensive change of the reinforcement, according to a new design of the structure.
Representative failure loads expressed as water heads are:

- failure load as built: 70 m as compared with Sleipner A failure at 67 m
- failure load with elongated T-headed bar: 125 m
- failure load as redesigned: 155 m, as compared with design load 72.5 m

The nature of the early failure caused by too short T-headed bars is illustrated by the photo of the failed specimen in Fig 12. The failure crack to the left in the photo arose suddenly just behind the head plate of the T-headed bar.

The two errors which occurred simultaneously, one in calculation of the shear force in the tri-cell wall and the other in evaluation of the strength of the same wall, caused the failure. The results indicate that the failure would have been avoided (but the safety would not have been acceptable) if only one of the two errors had occurred.
Concrete Research

High strength concrete research: A holistic approach to materials and structures

The concrete research is often divided into two categories: materials research and structural research. The materials research aims at a product with a high structural strength, retaining acceptable ductility; robust production, transport, casting and compaction of this high quality material; as well as a study of the long term development and durability.

The structural research seeks to answer the question: How to use this material in a cost-effective structural design with adequate structural safety?

The mechanical properties of high-strength concrete differ in many ways from those of traditional concrete. The differences materialize mainly in strength, stress-strain relations in compression and tension, fracture energy, influence of confinement, or more generally the failure criterion for three-dimensional states of stress, etc, and hence also in the interaction between concrete and reinforcement. These differences imply that traditional design procedures for reinforced concrete cannot be extrapolated to new strength classes without a thorough study and relevant modifications.
Our experience is that advantages are obtained by combining materials research and structural research in the same environment, where specialists in different fields work together with research engineers with a more general background. In the same manner there should be a continuous interaction between materials research, structural testing and e.g. nonlinear finite element analysis.

Since the material is a high quality material, concrete research will often meet questions that cannot readily be answered by concrete specialists alone. Thus we have had substantial benefits from a close cooperation between a University (The Norwegian Institute of Technology) and a poly-technical contract research institution (SINTEF).

**Effectiveness of reinforcement inclined to the cracks in concrete**

The wall- or shell element faces the same problem as the reinforced concrete panel subjected to a general stress state, see Fig 13.

A comprehensive test series combined with analysis covering slabs subjected to general moments and forces was promoted by Mete Cezanne and Rolf Lenschow /3/. A flexible test-rig was constructed so that the slabs could be subjected to a series of bending and twisting moments, Fig 14. The concrete layers with reinforcement at either surface of the slab were considered as if they were panels when the behaviour of the slabs was studied.

![Fig. 13—A reinforced concrete panel subjected to a general stress state (4)](image-url)
Fig. 14—Test rig and load arrangement at the University of Illinois

Further studies were performed in Trondheim /4/ and several other places directly on reinforced panels /5,6,7,8,9,10,26/.

A main point in the more recent research in Trondheim has been to investigate the strength of concrete in compression when tensioned and cracked in the transverse direction.
Fig. 15—Biaxial failure curve of reinforced concrete specimens showing the reduction of compression strength of concrete under transverse tension (4)

According to an investigation in Trondheim /4/ with concrete of cube strength of 25 MPa (3600 psi) no significant reduction of the concrete strength was observed when the reinforcement was in the same directions as the principle stresses. With the reinforcement deviating from the directions of the principle stresses a reduction of compression strength was observed. For panels with a reinforcement angle = 45 degrees, the reduction varied between 7 and 17%. The reduction is most likely due to stress concentration and eccentricities in the concrete struts between the cracks.

Previous investigations /7/ reported large reduction of the concrete compressive strength with simultaneous transverse tensile strains. An assessment of the detailed test results showed that the failure of the panels was caused by stress concentrations in the corner regions of the panels, leading to local pre-failure. Hence, the failure was not representative for reinforced concrete in general /4/.

Later investigations on panels /27/ have indicated a larger deterioration of the strength due to bi-axial loadings for panels made of 65 and 95 Ma (9500 and 14000 psi) concrete for panels with longitudinal reinforcement and for panels with 45 degree diagonal reinforcement, than reported on
the panels with 25 Ma concrete /4/. Fig 16 illustrates strength predictions based on tests /28,29/.

Tension stiffening in cracked reinforced concrete has a significant influence on the deformation of the structure and the distribution of stresses in a reinforced concrete element. That means that a finite element program has to take tension stiffening into account in calculating a realistic stress distribution in a cracked section. The observed increased stiffness of longitudinal stress-strain relationship with increased transverse compressive stresses is mainly due to increased bond between the bars and the concrete. It is interesting to note that both inclined reinforcement and transverse compression reduce the crack spacing. However, because of increased bond strength the net result is still increased tension stiffening.

The phenomena represented by tension stiffening are not fully clarified, nor are the effects precisely calculable, although some interesting investigations indicate the order of magnitude we are dealing with under various conditions. Results from further research are needed if the powerful finite element program in general shall succeed in calculating a realistic distribution of strain and stresses in concrete structures.

**STRENGTH OF CRACKED CONCRETE**

REF: UNIAX, CONST-STRAIN-RATE STRENGTH

![Strength prediction graph]

Fig. 16—Strength prediction by Collins et al (COL) and by the Norwegian Concrete Code (NS) for high-strength (HSC) and low-strength (LSC) concrete (28)
Long term random loading of concrete structures

The offshore concrete platforms are subjected to wave and wind forces for many years. The longest service life specified so far is 70 years (Troll gas platform). The peak stresses in concrete and reinforcement are caused by these stochastic forces. While steel under fatigue loading in general and in specific cases is thoroughly investigated for many years, a few investigations on concrete under fatigue loading did not provide sufficient knowledge for design of the new offshore structures.

A comparison of available test results /14,15/ had the aim of developing a fatigue relationship which could connect both the minimum and the maximum stress level with the fatigue life. Since some of the collected results were given in the form of Woehler, other in the form of Smith or other diagrams, all the results were expressed by three parameters: mean strength, stress range and number of cycles to failure; and plotted in a three dimensional coordinate system, Fig 17.

Investigation of fatigue of concrete has traditionally followed the test procedures for assessment of the fatigue behaviour of steel, although the material behaviour is different. What is recorded from concrete specimens

Fig. 17—Three-dimensional presentation of representing fatigue strength (14, 15)
under compressive fatigue loading is an increased residual static strength after cyclic loading if the specimen is not too close to fatigue failure.

By studying the remnant residual strengths in the longitudinal and transverse direction and the development of strains in both directions /16/ a steady reduction of tensile strength in the transverse direction under cyclic compression could be observed while the compressive strength in the longitudinal direction is higher after a number of cycles, see Fig 18. The test results are shown in a simplified diagram in Fig 19 of residual strength of concrete after cyclic pre-loading. These tests indicated that the development of the tensile strains was a significant factor for failure and also the most reliable observation for predicting failure of a concrete specimen under cyclic compression. These tests supported the idea that failure of concrete subjected to compression could also be described by a fracture mechanics model /17/.

Fig. 18—Methods of measuring the compressive and tensile remnant strengths (16)
With this background it is more natural to consider a damage rate function for concrete than the conventional Woehler diagram although neither removes uncertainties because of a lack of complete understanding and of experimental results. Figs 20 and 21 show presentations of fatigue capacities from the two different angles. From the damage rate function it is easier to predict the fatigue life when the number of loadings is higher than what is covered by experimental results.
Fig. 20—Comparison of modified damage rate function and experimental fatigue results (16)

Fig. 21—S-N curves obtained with the help of damage rate function; stippled lines present the S-N curves from Reference 18 based on the same experimental report
Concrete exposed to water pressure loading

The behaviour of concrete exposed to water pressure has been in focus during the development and construction of the offshore concrete structures. Under certain conditions water pressure can reduce the strength of a concrete element. Under other conditions water pressure can increase the strength of concrete elements.

The most important result from strain development tests /22/ is the documented differences in the strain recordings between sealed and unsealed, normal density concrete specimens exposed to water pressure loading.

Fig 22 shows how an unsealed concrete specimen under water pressure first behaves like a sealed specimen under confined pressure and thereafter - as the water penetrates the concrete - more and more behaves like a specimen under water with atmospheric pressure.

For a steady state condition with no flow through the concrete, the pore pressure will be equal to the applied water pressure. The internal pore pressure area is smaller than the total cross section of a concrete specimen. A best estimate for the ratios between the inner pore pressure area and the total area (the effective porosity factor) for the ND25 (3700 psi) and the ND80 (12300 psi) concretes tested are 0.91 and 0.65, respectively.

The lightweight aggregate concrete, LWA, specimens behaved differently when it was exposed to water pressure, see Fig 23. Pore pressure development was not registered in the LWA specimens during the

![Graph showing strain development over time](image)

Fig. 22—Strain development during 144 days of water pressure exposure, ND80 (22)
pressurization period. Splitting tests still showed that water had penetrated into the concrete. The observation is explained by the large water storing capacity in the porous Liapor aggregate. No significant pore pressure can develop before the aggregate is water filled. Due to the low permeability of the mortar, this will take considerably more time than the 144 days if there is water pressurization in the performed tests.

Fatigue tests of concrete /23,24,25/ showed that the moisture conditions, and especially drying of small scale specimens, have a significant influence on the fatigue life of concrete specimens. The fatigue life of cylinders tested under simultaneous acting water pressure is, however, not significantly different from that of corresponding cylinders tested under wet condition.

This observation for the fatigue strength corresponds to the results from static strength tests; the static strength does not change if the concrete is exposed to water pressure provided the pore pressure is given time to fully develop.

**Design of material properties, weight, strength and elasticity of concrete**

The weight of the concrete offshore structures, - both for gravity structures that are floated out to the destination and permanently floating
structures, - is an extremely important competitive factor. Therefore lightweight aggregate concrete has become more and more in focus. The compressive strength of normal strength concrete is limited by the strength of the cement paste, and by the paste-interface bond. Hence the strength of the aggregate has not been paid much attention. To produce high strength concrete, the aggregate has to be selected carefully to prevent that the aggregate becomes the strength limiting factor /30/.

The E-modulus is more focused upon for high strength concrete than for normal strength concrete. Higher stress levels cause in general larger deformations because the increase in E-modulus is not proportional to the increase in strength. Therefore, increasing the E-modulus independent of the strength by selecting a more rigid aggregate type is an interesting material design problem.

The weight of the concrete can to a certain degree be chosen by the amount of light weight aggregate, LWA.

More LWA in the concrete mix reduces the E-modulus and the ductility (fracture energy) of the concrete.

![Effect of amount of lightweight aggregate and the type of natural coarse aggregate on the E-modulus](image)

Fig. 24—Effect of amount of lightweight aggregate and the type of natural coarse aggregate on the E-modulus (30)
However, Figs 24 and 25 show how it is possible by material design to obtain the same E-modulus and the same ductility for a lightweight concrete as for a normal weight concrete. Because this concrete in some respects has the same properties as normal weight concrete and in others the same properties as normal lightweight concrete, this has been named moderated normal concrete, MND.

INDUSTRY RESEARCH PROJECTS

Organization of Industry Research

The research in Norway has addressed a number of issues as discussed above. SINTEF Structures and Concrete (until 1 Jan 1992 SINTEF FCB) and Department of Structural Engineering, the Faculty of Civil Engineering at the Norwegian Institute of Technology have been active in industry projects of several types:

- joint industry-research council\(^1\) projects
- joint industry projects

\(^1\) Research Council of Norway, until 1 Jan 1993 The Royal Norwegian Council for Scientific and Industrial Research (NTNF)
projects for one specific industry partner

Projects of the last category often aim at solving a particular problem only, and are of less general interest, even though they supplement the general knowledge. Thus, they are not exemplified here. The most important projects are summarized briefly below. From the description it will appear that the objectives and tasks in the various projects partly overlap. Since the group of people engaged in this work in Norway is small, and since SINTEF is involved in a majority of the projects, the overlap does not imply that research has been duplicated, but that similar activities in different projects have supplemented each other.

We have experienced industry research projects where international and national industries cooperate with a research institute and a university as very stimulating. This is in particular the case when the projects are of some permanence, where the industry participates in the planning, and we feel that they look forward to the results. It is also a stimulating guarantee for the relevance of the research, when industry is willing to pay for it.

COSMAR (Concrete Structures for Marine Production, Storage and Transportation of Hydrocarbons) 1978-1981

COSMAR was a four years German-Norwegian cooperative project with the objective of increasing the knowledge of offshore concrete structures. The project had a total budget of DEM 10.5 million (USD 7 million) and was organized in four sub-projects related to static strength, fatigue, storing and transportation of oil and liquified gas, including temperature effects, and concrete structures exposed to ice and arctic conditions.

Design and Structural Utilization of HSC 1983-1986

The project was a four years project with a budget of NOK 2 000 000 (USD 300 000). In spite of the modest external resources, the work provided much of the background for NS 3473 3 ed 1989, where specifications for high strength concrete are included. The results are summarized in a main report /38/, where references to detailed reports may be found.
Design Model for Cracked Concrete 1984-1987

The objective of the project was to study the behaviour of reinforced plates under tension, from the first cracking to failure, when the reinforcement forms an angle with the tensile direction. Cylinder strengths were in the range 50-60 Ma. Plates loaded in pure tension were of dimensions 1.0×3.0×0.2 m, and loaded in bending 1.2×3.8×0.35 m. The results were used as a basis for adjustments in post-processors used in design of offshore structures. Conclusions and references to test reports are found in a summary report /26/.

Materials Development. High-Strength Concrete 1987-1991

The objective of the project was a further development of the material high-strength concrete adapted for different applications and production processes, in order to increase the use of high-strength concrete. About 20 companies and institutions cooperated in the project, representing material producers, concrete industry, contractors, owners and research institutes. 30 % of the programme was funded by the Research Council of Norway and the remainder by industry. The budget for 1989-1991 was NOK 26 million ( USD 4 million ). Topics covered were
- Aggregates
- Light weight aggregate concrete
- Rheology of fresh concrete
- Testing methods - Mechanical properties
- Material structure
- Information

The project was coordinated by Norcem A.S. 36 reports ( see /3 9/ ) from the project in Norwegian, with summaries and figure titles in English, are available from SINTEF Structures and Concrete.

High-Strength Concrete 1986-93

The High Strength Concrete ( HSC ) project, with a total budget of about NOK 38 million ( USD 5.5 million ), involved the Research Council of Norway, government agencies and ten industry companies with variable duration of participation.
The general objective of the research programme was a combined study of analytical models to simulate the general behaviour of the material, and experimental research to determine the material parameters in the models.

The project proceeded from a pre-project in 1986 through three Phases. The results from Phase 1 provided part of the documentation for the rules in the Norwegian Standard NS 3473, third edition.

Results from Phases 1, 2 and 3 are summarized in Main Reports ( /42/, /43/ and /44/ ). The Marine Report from Phase 3 is so far confidential.

The objective of a Design Guide in Phase 3 /45/ was to prepare suggested design specifications based on conclusions from all three phases compared with high-quality data available world-wide.

The project was governed by a Steering Committee with representatives from the industry partners ( including the Norwegian Petroleum Directorate and the Public Roads Administration ).

**Ductility of High Strength Concrete under Special Loading Conditions 1990-1994**

Relevant loading situations include transient loading ( impact, shock loading, earthquake ), extreme temperatures ( high and low ) and temperature gradients. An extrapolation from normal loads and static strength is inadequate, especially when exemptions are to be made from the ordinary acceptance criteria.

The project is sponsored by the Research Council of Norway and Industry. The general objective is to determine safe barriers for the use of high-strength concrete in a cost-effective design. The need for research was defined and a research programme was established in a Phase 1, finished in 1991. Test methods were examined and test equipment installed at the Norwegian Institute of Technology for later testing. Material properties were determined under high rates of loading, and considerable efforts were made in advanced FE-analyses of concrete structures when exposed to transient dynamic loadings. The project continues in a Phase 2 from 1992 to 1994. Details about the project may be found in /40/.
Light Weight Aggregate Concrete for Floaters 1989-1991

The objective was to develop LWA concretes of grade LC30 and LC40 (numbers denoting cube strengths) with the lowest possible density, and adequate material and production properties for use in floating production platforms.

Topics covered were:
- Mix design and production properties
- Moisture conditions
- Mechanical properties
- Hydrocarbon fire resistance
- Durability
- Structural behaviour
- Design specifications
- Effect of evolved heat on the micro-structure

The major activity has been within mix design. Concretes have been developed with compressive cylinder strengths in the range of 35 and 45 Ma with density (fresh concrete) of approximately 1400 and 1500 kg/m$^3$, respectively, and good workability.

The durability in a marine environment seems to be equal to or even better than that of high-strength normal weight concrete. Results are summarized in /41/.

Lightcon 1993-1995

The objective of the project is to provide documentation for the material and structural properties for light-weight aggregate concrete. The project is a continuation of "Light Weight Aggregate Concrete for Floaters", but with a wider field of application.

The following activities indicate the contents of the project:
- Study of eight bridges built in Norway 1988-1993 (including two floating bridges) and a few test structures
- Guidelines for production and casting based on experience
- Structural performance of concrete with density 1400-1500 kg/m$^3$ including design specifications and instrumentation of structures
- New applications, including evaluation of economical and technical advantages
- Light weight aggregates not currently used in Norway
The project is run in cooperation between six Norwegian industry partners, including the Public Roads Administration, and with Norwegian Contractors as coordinator.

**Nova Analytica 1993-1995**

Present analytical facilities allow complex material models to be applied for analysis of concrete structures. Important applications are studies of structural details with interaction between concrete and reinforcement. Such analyses are frequently undertaken, but material models implemented in available programs leave several questions with respect to the correct simulation of the actual case. The project is aimed at extending the knowledge about modelling of the mechanics of structural reinforced concrete. The knowledge gained will be implemented in available programs. A cooperation with Delft University of Technology/TNO, the Netherlands, has been established, also through membership in the Diana Foundation.

**Brite EuRam 5480. Economic Design and Construction with High Strength Concrete 1992-1995**

Norway participates in a research project under the European Brite EuRam research programme. The participants are based in the following countries: Finland, the Netherlands, Norway, Spain and United Kingdom. Taylor Woodrow Construction Ltd, Taywood Engineering is the coordinator for the total project, which has a budget for 1992-1995 of ECU 3 884 354.

The project contains the following technical tasks
- Mix Design
- Mechanical Properties
- Structural Performance and Design
- Production and Transport Methods
- Site Use Development
- Quality Assurance/Quality Control

SINTEF Structures and Concrete participate as a Norwegian contractor, with industry sponsors. The Research Council of Norway funds 50 % of the budget corresponding to the funding from the Commission of the European Communities for the EC countries. SINTEF is the coordinator for "Mechanical Properties", where we also have our major work load. The type of work to be done is indicated by the list of deliverables for the
various sub-tasks under Mechanical Properties:
- State of the art report on compression testing
- Report on the effect of aggregate type and mix proportion
- Recommended procedures for determining early age shrinkage and cracking, triaxial properties, creep,
- Poisson's ratio, static fatigue under sustained loading
- Report on test equipment and routines for triax-tests
- Report on brittleness performance, ductility and fracture energy of HSC
- Report on reinforcement bond properties
- Compilation report with outline specifications

INTERNATIONAL STANDARDIZATION

A small country like Norway depends much more on international trade than larger countries. Therefore, we have an urgent need for international rules which regulate the international trade. Norway has thus traditionally been an advocate also for international technical standardisation, originally directed towards the Nordic countries and ISO. Since European standardization is in focus at present, and resources are restricted, the largest efforts are now directed towards the European standardization organization CEN.

For a truly international business like the oil business, European standards may be insufficient, also in Europe. Thus, the Norwegian Council for Building Standardization has taken an initiative to work out an ISO-standard for offshore concrete structures. The work is in progress in ISO Technical Committee 67, Sub-Committee 7 Offshore Structures, with a Norwegian secretariat, and the ISO draft will to a large extent rely upon the present Norwegian Standard NS 3473 /36/.

Many of the actual decisions are taken when research results are implemented as suggested rules in a pre-standardization phase, which again necessitates engagement in organizations like CEB, FIP, ACI and RILEM. We have thus e.g. found an engagement in the Joint CEB/FIP Working Group on HSC/HPC (where also ACI is represented) very important for us.
REFERENCES


/7/ M P Collins and F Vecchio: "The response of reinforced concrete to in-plane shear and normal stresses." University of Toronto, Department of Civil Engineering, publ. No 82-03. March 1982.


/12/ Oljedirektoratet: "Årsberetning 1993." Stavanger, Norway, p.64.


/22/ L M Bjerkei: "Water Pressure on Concrete Structures." Dr. Thesis, Norwegian Institute of Technology, University of Trondheim, 1990.


E Thorenfeldt: "Design Model for Cracked Concrete Shells." Summary report, SINTEF Report, STF65 F88070.

I Holand: "High Strength Concrete." Main Report for Phase 2, SINTEF Report, STF70 A93091.

E Thorenfeldt and G Drangsholt: "High Strength Concrete. Plates and Shells." SINTEF Report, STF65 F91013.


S Smeplass: "The Effect of the Aggregate Type on the Strength and E-modulus of High Strength Concrete." SINTEF Report, STF70 A92051.


Haug, A K; Jakobsen, B: "In-situ and Design Strength for Concrete in Offshore Platforms." Second International Symposium on Utilization of High-Strength Concrete, Berkeley, California 1990.


Holand, I : "High-Strength Concrete. Main Report for Phase 1." SINTEF Report, STF70 A92138.

Holand, I : "High-Strength Concrete. Main Report for Phase 2." SINTEF Report, STF70 A93091.


Dyngeland, T; Hansen, E Aa; Holand, I; Johansen, R; Petković, G; Smeplass, S; Stemland, H; Thorenfeldt, E; Tomaszewicz, A : "High-Strength Concrete Phase. Report 1.2 Design Guide." SINTEF Report, STF70 A94044.
Seismic Behavior of Connections in Precast Concrete Walls

by A. E. Schultz and R. A. Magaña

Synopsis: An experimental program is summarized which is aimed at enhancing the knowledge base regarding seismic behavior, analysis and design of precast concrete shear walls. The “emulation design” and “jointed construction” philosophies are described, and an idealization of the behavior of precast shear walls is presented. A compendium of connection details for precast concrete shear walls, seven for vertical joints and four for horizontal joints, is selected for further study and the selection process is described. The connection details are proportioned for a prototype shear wall that is designed as part of a six-story precast concrete office building. A description of all connection details and test procedure is given. Highlights from the cyclic load tests of the vertical joint specimens are documented, including connection resistance, displacement response, initial stiffness and energy dissipation capacity.

Keywords: Connections; construction joints; cyclic loads; energy dissipation; joints (junctions) precast concrete; seismic design; shear walls; structural design
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INTRODUCTION

Precast concrete members are seldom used for lateral load resisting elements in structures exposed to high seismic risk in the United States (U.S.). Current building codes require that precast systems be shown by experiment and analysis to have lateral load resisting characteristics that are equal or superior to those of equivalent cast-in-place concrete systems. This requirement has led to the development of a design philosophy known as "cast-in-place emulation" in which connections between precast members are detailed and reinforced such that they respond in a manner similar to a monolithic, cast-in-place concrete structure. Typically, the joints between precast members are proportioned with sufficient strength to avoid inelastic deformation, thus, plastic hinges form away from the connections. However, the emulation design philosophy has a tendency to undermine the cost-effectiveness of precast concrete systems.

There is a growing belief among structural engineers in the U.S., supported by a limited amount of experimental and analytical evidence, that precast structures which do not emulate monolithic, cast-in-place concrete construction possess inherent characteristics that can be used for seismic resistance. In this design philosophy, known as "jointed construction", certain joints between precast members are allowed to deform inelastically, thereby providing ductility, and, in some cases, energy dissipation capacity to the structural system. The National Science Foundation (NSF) has funded a multi-year program known as PRESSS (PREcast Seismic Structural Systems) to address the limited knowledge base on jointed construction. The Japanese component of PRESSS, which is the Fourth Phase of the U.S.-Japan Natural Resources Panel on Wind and Seismic Effects Large Scale Test Program, addresses the emulation concept exclusively.
This document is a summary of an ongoing project in the Building and Fire Research Laboratory (BFRL) at the National Institute of Standards and Technology (NIST) to develop improved analysis and design methods for precast concrete structures in seismic regions. The overall goal is to advance the concept of jointed construction by investigating the seismic performance of horizontal and vertical joint connections in precast concrete shear walls by means of carefully planned experiments. Specific objectives include (a) identification of the seismic performance, (b) development of behavioral models, and (c) verification of analysis and design methods. This project is funded, in part, by a subcontract from a PRESSS Phase II research project at the University of Nebraska-Lincoln [1,2]. LEAP Associates International, a second subcontractor to the University of Nebraska, has been providing guidance and advice on issues concerning design, fabrication and erection of precast concrete systems.

**PANEL BUILDING CONSTRUCTION**

Due to their high initial stiffness and lateral load capacity, shear walls stand out as an ideal choice for the lateral load resisting system in precast concrete buildings. Furthermore, inelastic deformation of connections offers the potential for high ductility and energy dissipation capacity in jointed construction. However, a distinction must be drawn between residential construction and office buildings. The generous use of wall panels in the former provides ample redundancy and load capacity; thus, seismic force and deformation demands in residential buildings tend to be significantly lower than in office construction. Due to architectural constraints, structural walls in precast office buildings tend to be few and horizontal spans are maintained as short as possible. These walls are usually slender (i.e. total height exceeds twice the total length), and seismic force and deformation demands tend to be high.

The present study is designed to address the behavior of connections between panels in slender precast shear walls. A prototype wall was designed as an integral part of a building system under study in the University of Nebraska PRESSS project and the test specimens in the NIST subcontract were configured to represent portions of this prototype wall. The University of Nebraska system (Fig. 1) is a six-story office building in which gravity loads are supported by interior frames (hollow-core planks, soffit beams and interior columns), and lateral loads are resisted by spandrel frames along the perimeter of the building and structural walls that form part of the service core (elevators and stairwells).
The prototype shear wall (Fig. 2) comprises the precast panels on one side of a stairwell in the short direction of the building (Fig. 1). It is an assemblage of four precast panels of which two are erected side-by-side and the other two are stacked above the first pair. The wall features both vertical joints and horizontal joints, and it is rather slender with a height-to-length aspect ratio equal to 3.9. The panels were configured such that the shear wall included both vertical and horizontal joints, and it employs 3 x 12 m panels can be easily transported to the construction site.

PRECAST SHEAR WALL BEHAVIOR

Since this study addresses jointed construction, it has been tacitly assumed that relative movement between adjacent precast panels is inevitable unless monolithic construction is emulated. Because of wall slenderness, and by virtue of the flexibility of the connections between panels, the idealized response of the prototype wall is assumed to be dominated by rocking (Fig. 3). This rocking motion is a rigid body rotation which is mobilized by opening and closing of horizontal joints between panels. Rocking generates relative vertical displacement between panels as the shear wall drifts laterally in response to seismic loading. However, the stiffness of the vertical joint connections will determine if the relative displacement along vertical joints is proportional to total drift of the panel. Furthermore, the panels will undergo deformation between joints, giving rise to additional drift. For simplicity, the deformation capacity of vertical joint connections in this study is correlated to the mechanism shown in Fig. 3.

By virtue of the size of the relative vertical deflections, vertical joints between precast shear wall panels are considered ideal locations for using energy-dissipating resistance mechanisms to increase wall toughness. In addition, the failure of a vertical joint connection has a less significant impact on lateral load capacity and structural stability than that of a horizontal joint connection. Energy dissipation can also be achieved in certain connection details for horizontal joints, but, ductility (i.e. the ability to deform while maintaining peak load capacity) is the principal requirement for horizontal joint connections. This research program utilizes parts of the connection hardware (i.e. the connectors) to provide ductility and energy dissipation capacity while simultaneously maintaining the resistance required by design for load transfer between the connected panels. A secondary goal of this project is to minimize damage to the concrete panels and anchorages (i.e. bars, plates, welds and bolts).
CONNECTION DETAIL SELECTION PROCESS

The initial focus of this project was a search through the technical literature for possible connection details for vertical and horizontal joints in precast shear walls [3,4]. The objective was to select potential connections that promise improved seismic performance, while maintaining the cost-effectiveness of precast construction. The final selection had to include connections suitable for regions of both moderate and high seismicity. In addition, the selection had to include both bolted and welded connections. The research team, with the active participation of representatives of the U.S. precast construction industry, reduced the number of connections to six details each for vertical joints and horizontal joints.

It was deemed imperative that the results of this research program be quickly incorporated into practice, so the participation of the precast concrete industry was sought. Industry participation was fostered at three levels. First, an Industry Advisory Group was formed to provide guidance at early stages of the project. This group included precast producers, engineering consultants, and a building official. Second, the Prestressed/Precast Concrete Institute (PCI) Seismic Committee formed a Task Group to follow the activities of this project and to report the progress to the PCI Seismic Committee. Third, the research team utilized the 1993 and 1994 PRESSS Industry Seismic Workshops [5] as a forum whereby tentative connection details received critical evaluation from a wide cross-section of the precast concrete industry.

EXPERIMENTAL PROGRAM

The experimental program includes seven vertical joint specimens which represent regions surrounding vertical joint connections in precast shear walls, and four shear wall specimens for the study of horizontal joint connections (Table 1). The vertical joint connection details include three modifications of the commonly-used welded loose plate connection detail (NSP, SFP and IFB), as well as three connections which rely on bolting or a combination of bolting and field-welding (PTS, VJF and UFP). A seventh vertical joint connection detail (XAP) was added to the program in an attempt to improve the observed behavior of one of the details in the initial selection.

The horizontal joint details include only four connections, two in which vertical reinforcement is spliced using grouted sleeves or post-tensioning hardware, as well as two ductile connections which rely on
either shear friction in concrete or plasticity of mild steel for resistance, ductility and energy-dissipation capacity. Two of the potential connections that had been selected by the previously described process were eliminated from the program because they were found to be impractical upon designing the details. The first of these connection details utilizes short lengths of structural steel angles and field welding to splice vertical bars, while the other employs a steel brass-friction device similar to VJF. Both of these connections were deemed inappropriate due to excessive size and overly complicated layout.

The panel specimens were constructed using typical materials, including concrete with a 34-MPa (5000 psi) compression strength, non-shrink grout, Grade 60 reinforcing bars and headed anchor studs (414 MPa), and A36 structural steel (248 MPa). A limited amount of brass plate, Grade 50 steel plate (345 MPa) and 304 stainless steel was also used. In some cases, proprietary devices such as reinforcing bars with upset and threaded ends, grouted splice sleeves, and post-tensioning hardware were used for the connections.

The vertical and horizontal connection specimens were tested in the NIST Tri-directional Test Facility (TTF) using cyclic drift histories that simulate seismic motions. The TTF is a one-of-a-kind, computer-controlled, multidirectional structural testing facility that was designed, fabricated and assembled at NIST one decade ago [6] and has been upgraded on a continuous basis. The TTF can generate the actuator displacements/forces needed to control the translations and rotations of the upper crosshead along three orthogonal axes for a total of six degrees-of-freedom. The drift history used for the tests (Fig. 4) was specifically tailored for the U.S. PRESSS program [7], and it comprises groups of cycles, the pattern of which is repeated until the end of the test. The pattern includes three identical cycles at a given peak drift followed by an "elastic" cycle at a fraction of peak drift, and peak drift is increased monotonically among successive groups of cycles.

A schematic diagram for the vertical joint test setup is shown in Fig. 5. The specimens represent the portion of the shear wall panel which surrounds a connection. The vertical joint has been rotated to a horizontal position to facilitate full utilization of TTF capabilities. The tests proceeded as the previously described drift history was imposed horizontally in the plane of the wall, and the separation of the precast concrete pieces (i.e. joint thickness) was maintained constant. To achieve these conditions vertical translation and in-plane rotation were fully constrained, and all out-of-plane motions were prevented as well.

The horizontal joint test setup is shown schematically in Fig. 6. The specimen represents a portion of the prototype wall, including the panel
above the joint and a stub representing the panel below the joint. Vertical loads are applied to the top crosshead, the sense and magnitude of which is determined as needed to define a vertical stress equal to 0.69 MPa (100 psi) representing dead loading, and a continuity moment equal to 50% of the overturning moment at the base of the panel. This last feature is necessary to properly model the lower portion of a shear wall.

All seven vertical joint connection tests were completed at the writing of this paper, and the four horizontal joint specimens were constructed. The remainder of the paper describes all vertical joint and horizontal joint connection details. Highlights from the experiments of the vertical joint connection tests are also given.

**VERTICAL JOINT SERIES**

The 2/3 scale vertical joint connection specimens were nominally identical except for the blackout region containing the anchor plates and connector. The concrete panels surrounding the blackout regions (Fig. 5) were of identical dimensions with a 1220-mm (4-ft.) dimension parallel to the joint, a 610-mm (2-ft.) dimension normal to the joint and a 15-mm (6-in.) thickness. All panels were reinforced with two sheets of WWF6 x 6 - W3.5 x W3.5 welded wire fabric, which comprises 5.3-mm (0.21-in.) diameter wire at 15-mm (6-in.) spacing, to provide shear strength to the panels. In addition, #4 Grade 60 deformed bars were placed along all edges of each panel. Perimeter bars are often used in practice to protect precast concrete components during transportation and erection. In the present study, 203-mm (8-in.) splice lengths were used to guarantee strength development of these bars to resist in-plane forces generated upon application of the drift history.

All connectors were welded and/or bolted to steel plates that were embedded in and anchored to the concrete panels. In proportioning the connectors for the vertical joint details, it was assumed that first yield is the design limit state, and appropriate factors were used to ensure yielding, plastic deformation, and eventual failure in the selected mode of response. To guarantee strong anchorages for the embedded plates, a redundant system of headed anchor studs (Grade 60) and low-alloy reinforcing bar anchors (ASTM A706) was used. For simplicity in design, it was assumed that the studs resist the vertical shear force generated by the ultimate strength of the connector, and the horizontal components of the rebar anchors resist the corresponding moment.
All connections in the vertical joint series are classified in Table 1 according to the expected mode of response. Details of the connectors and blockout regions are shown in Fig. 7-13, and are described in the following sections. Measured responses for the various vertical joint specimens to the imposed drift history are given in Table 2. Peak shear displacement along the joint is normalized by the measured yield displacement of each connection to define a displacement ductility factor $\mu$. Cumulative dissipated energies are normalized by twice the product of the average yield load $P_o$ and average yield displacement $\Delta_o$, these average quantities being those for connectors with similar yield points (NSP, SFP, IFB, XAP and VJF).

In Table 2, the first normalized energy $e_{0.35}$ is obtained at the end of the cycle group with a peak drift of 0.35%, thus it serves to indicate the energy dissipation capacity of the connection at early stages of inelastic behavior. The second normalized energy $e_{\max}$ is obtained at the end of the test, thus it captures the total capacity of the specimen. The NSP detail was deemed to have failed at the end of the cycle group at 0.5% drift, however, it was loaded another three cycles before it lost all load capacity. Displacement, ductility, drift and normalized energy for this specimen are listed at the end of the cycles at 0.5% drift, and the quantities in parentheses are at the end of the test (0.75% drift).

**Notched Shear Plate (NSP)**

The notched shear plate (NSP) detail (Fig. 7) is a slight modification of the currently-used welded loose plate detail, except that connector cross-section is reduced to guarantee that the welds are stronger than the connector. Experimental observations by Stanton et al. [8] indicate that it is very difficult to provide sufficiently strong welds in rectangular plates for the purpose of preempting weld failure. In addition, the notches reduce connector cross-section such that the NSP can be assured to respond predominantly in shear. This connection is the simplest and most economical vertical joint detail in this program.

The NSP detail demonstrates stable force-displacement hysteresis with ample energy dissipation that resemble those of steel beams (Fig. 14). Cracking of the notched shear plate at the root of the notches led to failure by gradual strength reduction. This occurred only after that plate had been deformed to large plastic strains.

Prior to crack formation, the notched shear plate exhibited strength increases associated with strain hardening up to (a) a displacement ductility factor in excess of 7, (b) a nominal shear strain in excess of
20%, (c) two complete cycles at a peak displacement $\Delta_{max}$ equal to 0.9 cm (0.35 in.), and (d) an equivalent drift ratio equal to 1/2%. In addition, this connector had the highest cumulative energy dissipation after the cycles at 0.35% drift.

**Slotted Flexure Plate (SFP)**

The slotted flexure plate (SFP) detail (Fig. 8) is another minor modification of the currently-used welded loose plate detail in which the connector cross-section is reduced by slotting instead of notching. The two slots create three struts which have a more slender profile than does the shear panel in the NSP detail. This feature allows the connector to respond to the interface shear demand primarily in flexure, thus promising greater deformation capacity than the NSP detail.

The slotted flexure plate detail demonstrates stable force-displacement hysteresis loops with ample energy dissipation (Fig. 15). This detail develops a much larger peak strength than do any of the other details. At peak strength, the flexural capacity of the struts between slots is proportional to the plastic section modulus of the connector cross-section which is 50% larger than the elastic section modulus that defines the yield moment. Cracking of the slotted flexure plate at the root of the slots, after undergoing large plastic strains, led to failure by gradual strength reduction.

Prior to crack formation, the slotted flexure plate exhibited strength increases associated with strain hardening and a fully plastic section up to (a) a displacement ductility factor in excess of 6, (b) two complete cycles at a peak displacement $\Delta_{max}$ equal to 1.4 cm (0.55 in.), and (c) an equivalent drift ratio equal to 3/4%. Energy dissipation after the cycles at 0.35% drift was lower than for specimen NSP, but the greater ductility of this detail enabled it to dissipate more energy by the end of the test.

**Inclined Flat Bar (IFB)**

The inclined flat bar (IFB) detail (Fig. 9) was first proposed by Stanton et al. [8] as a welded plate connection that responds in uniaxial tension/compression. This mode of response is most efficient in terms of material utilization and force/deformation capacity, and it is achieved by welding the long, narrow plate at a small angle with the axis of the vertical joint. Stanton et al. [8] recognized that buckling is likely to dominate the behavior of the connection when loaded in compression.
The inclined flat bar detail demonstrates moderately stable force-displacement hysteresis loops in tension with only a modest amount of energy dissipation (Fig. 16). Out-of-plane bending and buckling under compression loading led to strength deterioration after yielding, and large stress concentrations that eventually resulted in failure. Failure by rupture is due to low-cycle fatigue of the kinked region of the inclined flat bar, as it is bent during the compression cycles and straightened in the tension cycles. Behavior in the tension cycles is affected (pinched) by this kinking action, as it is necessary for the inclined flat bar to undergo finite deflection in the tension direction before the plate is fully straight and sizable tensile stresses can be resisted.

Prior to rupture, the inclined flat bar resisted the simulated seismic displacement history exhibiting (a) a displacement ductility factor in excess of 12, (b) a nominal uniaxial strain on the order of 10%, (c) two complete cycles at a peak displacement $\Delta_{max}$ equal to 2 cm (0.8 in.), and (d) an equivalent drift ratio equal to 1%. Energy dissipation capacity of this specimen was the same as that of the SFP at 0.35% drift, but the buckling-induced pinching severely curtailed energy dissipation capacity by the end of the test.

X-Shaped Axial Plate (XAP)

The seventh specimen in the vertical joint series, the X-shaped axial plate (XAP) detail (Fig. 10), was added to the program in an attempt to improve the observed behavior of the IFB detail. This connector is a symmetric, coupled pair of inclined bars cut from a single steel plate. The intent is to utilize the tension leg to stabilize the out-of-plane deflection of the compression leg. The portion of the legs between the welds and the central node where the legs intersect is assumed to respond primarily in uniaxial tension/compression.

This connector was installed in the concrete panels used to test the IFB connector, thus, this test also served to investigate the feasibility of replacing damaged connections that rely on welded plates. The damaged IFB connector was removed from the panels using an oxygen-acetylene torch, and the embedded plates were ground smooth prior to welding the XAP connector. No deleterious effects from welding were noted in the concrete or anchorages at any time during the test of the XAP detail.

The X-shaped axial plate displayed stable response to the simulated seismic drift history with ample energy dissipation capacity and none of the "pinching" which affected the IFB connection (Fig. 17). Out-of-plane
bending of the connector was controlled throughout most of the test, and resistance to the imposed drifts was symmetric. The tension leg was effective in restraining the compression leg, and the central node served as a bracing point. During the last group of drift cycles, buckling of the connector was observed, and the connector legs kinked next to the central node. Eventual failure of the connector was similar to that for the IFB, with low-cycle fatigue fracture occurring as large strains were reversed in the kinked region.

Prior to failure, the XAP connection resisted the simulated seismic displacement history exhibiting (a) a displacement ductility factor in excess of 8, (b) a nominal uniaxial strain on the order of 10%, (c) one and one-half cycles at a peak displacement $\Delta_{max}$ equal to 1.5 cm (0.8 in.), and (d) an equivalent drift ratio equal to 3/4%. This detail was as good as any of the others over its entire range of drift response.

Pinned Tension Strut (PTS)

The PTS detail (Fig. 11) was developed to improve certain aspects of the IFB. These include the propensity of out-of-plane bending/buckling, and flexural demands imposed by the fixed-end conditions. In addition, the pinned tension strut (PTS) detail uses bolting as the primary field connection technique (i.e. the welding is included only to guarantee ample tolerance), making it an easily replaceable connector following a damaging earthquake.

The results from the pinned tension strut test indicate considerable improvement over the IFB detail. Even though the pinned tension strut exhibited lower stiffness than the IFB, compression strength is higher, and it is maintained over a wider range of displacement (up to 1.5 cm or 0.6 in.). The lower stiffness and marked pinching are due, in part, to the flexibility introduced by the bolted connection. This flexibility is the result of tolerance in the hole, which was minimized in this study by using a closely machined steel sleeve (Fig. 11), as well as deformation of the hole by the stronger and harder bolt. Unfortunately, premature failure of one of the rebar anchors led to large rotation of one of the anchor plates, thus, preventing the connector from being deformed to larger drift demands than those shown in Fig. 18.

It is noted that even though the connector had yielded at the time the rebar anchor failed, the connector itself was not failed. Out-of-plane bending was observed, but the stiffener, which was provided by the web of the structural steel tee from which the loose plate was fabricated, limited these displacements. Prior to failure of the rebar anchor, the
pinned tension strut resisted the simulated seismic displacement history exhibiting (a) a displacement ductility factor equal to 3, (b) a nominal uniaxial strain on the order of 9%, (c) one complete cycle at a peak displacement $\Delta_{max}$ equal to 1.5 cm (0.6 in.), and (d) an equivalent drift ratio equal to 3/4%. The pinching in this bolted connection severely limited its energy dissipation capacity both in early stages of the test, as well as over the entire drift history.

**Vertical Joint Friction (VJF)**

The vertical joint friction (VJF) connection (Fig. 12) relies on field bolting; it is intended for applications where large deformation capacity is needed. The VJF is an adaptation of the slotted, bolted friction connection proposed by Grigorian et al. [9] for steel buildings. In the present study, the connection transfers shear rather than axial load, and there is eccentricity between the planes of frictional shear stress transfer. Two friction interfaces are provided by utilizing a pair of brass shims (above and below the slotted plate), and a top loose plate serves to transfer the clamping force in the bolts. Disc springs are used to maintain a clamping force of approximately constant magnitude.

The vertical joint friction connection displayed nearly elastoplastic response to the simulated seismic drift history (Fig. 19). In comparison to all other details, force capacity was highly predictable and nearly constant over the entire range of displacements, and energy absorption capacity was unmatched. The initial stiffness was lower than expected, as bolt movement introduced measurable in-plane flexibility. During the last cycle of the test, at least one of the bolts hit the end of its slot, and the resulting forces led to a premature failure of one of the rebar anchors. Subsequently, the test was terminated after completing one and one-half cycles at a peak displacement of 4 cm (1.6 in.). As in the previous test, however, the connector itself was far from being failed.

Prior to failure of the rebar anchor, the VJF connection resisted the drift history exhibiting (a) an effective "displacement ductility factor" in excess of 23, (b) one and one-half cycles at a peak displacement $\Delta_{max}$ equal to 4 cm (1.6 in.), and (c) an equivalent drift ratio equal to 2%. Due to flexibility afforded by the bolts in this connection, energy dissipation capacity was not as large at early stages as it was for the NSP and XAP specimens. However, the nearly elastoplastic response and large deformation capacity enabled this specimen to accumulate three times as much dissipated energy as did the SFP and XAP details.
U-Shaped Flexure Plate (UFP)

The U-shaped flexure plate (UFP) detail (Fig. 13) is another bolted connection which is intended for applications where large deformation demands are expected. The UFP was proposed by Kelly et al. [10] as an energy-dissipating flexible connector in which rolling bending action resists vertical shear force. The UFP poses little resistance to in-plane movement that is normal to the vertical joint, so it is well-suited for locations where a control joint is sought. Like the VJF detail, the deformation capacity of the UFP is not limited by strain capacity; rather, connection geometry is the sole constraint on vertical displacement.

The U-shaped flexure plate connection displayed highly stable response with ample energy dissipation capacity over the entire range of imposed displacements (Fig. 20). Due to the nature of the rolling, bending action, localized yielding occurred at all magnitudes of displacement. Thus, the force-displacement loops indicated hysteresis over the entire range of drifts considered. The connector did not exhibit global yielding, and the peak force in a given cycle was observed to increase with peak displacement over the entire range of response. It is noteworthy that this connector, which was supposed to develop only 40% of the specified 98-kN (22-kip) design load, actually attained the target design load in the last two groups of cycles.

This connector was designed assuming Grade 36 structural steel (248 MPa or 36 ksi). However, due to the formation of cracks along the edges of the bent region of the plate during fabrication, a second plate was made using 304 stainless steel which has both a higher specified elongation and higher yield and ultimate stresses. During design, it was assumed that the bolts would provide sufficient clamping force to prevent slip of the plate. However, the stainless steel UFP connector proved to be 2.5 times as strong as was assumed, and friction capacity of the interface was exceeded. Shear force was transferred by bolt bearing, and the reversals of the associated flexural stresses led to low-cycle fatigue of the bolts. The plate was subsequently welded to the anchor plates and the test was continued. The test was eventually terminated when one of the reinforcing bar anchors fractured next to the welded end.

As in the previous test, however, the U-shaped flexure plate did not demonstrate distress when the test was terminated. Prior to failure of the rebar anchor, the UFP connection resisted the simulated seismic displacement history exhibiting (a) one and one-half cycles at a peak displacement $\Delta_{max}$ equal to 4 cm (1.6 in.), and (b) an equivalent drift ratio equal to 2%. As expected, the UFP detail did not display as much
ability to absorb energy by hysteresis at small drifts, but this was due simply to the low force it developed at small drifts. By the end of the test, this detail had accumulated twice as much dissipated energy as did the SFP and XAP details.

HORIZONTAL JOINT SERIES

All connections in the horizontal joint series are classified in Table 1 according to the expected mode of response to the imposed drift history. The gross dimensions of the four horizontal joint specimens are identical (Fig. 21-24), including the 152-mm (6-in.) thickness. Panel reinforcement in all specimens is nominally identical except for the type of vertical reinforcement, and the connection-specific hardware in the vicinity of the joint. As in the vertical joint series, two sheets of WWF6 x 6 - W3.5 x W3.5 welded wire fabric were used to provide shear strength to the panels, and properly-lapped #4 deformed bars were used along the edges of all panels. In addition, three overlapping wire spirals are used along each jamb to confine the concrete and mitigate compression damage from cyclic loading.

In proportioning the connectors for the vertical joint details, it was assumed that first yield is the design limit state, and vertical reinforcement in all panels was proportioned to develop a nominal capacity of 72 kips (two #7 Grade 60 bars or equivalent). In addition, it was deemed important to avoid the use of blockouts, as these reductions in the depth and width of the compression region can curtail the flexural capacity of the walls. In addition, they require placement of dry-pack grout following erection of the panels.

Grouted Splice Sleeve (GSS)

This detail is a relatively simple connection (Fig. 21) that is common in the precast industry. Vertical rebars are spliced using proprietary sleeves (Splice Sleeve) and grout (Master Builders). These splice sleeves allow rebar stresses to be transferred across the joint, enabling the reinforcement to yield under lateral loading and form a plastic hinge at the base of the panel. Aside from the splice sleeves and the necessary grout tubes, this detail also features a 190-mm (7.5-in.) debonded length of rebar in the panel below the joint. By debonding the bar, the length of reinforcement that can deform plastically and dissipate energy is increased. In addition, the concrete surrounding the portion of vertical rebar that is likely to undergo large plastic
deformations is isolated from the associated bond stresses strains, thus mitigating cracking damage to the panel.

Post-Tensioned Tendon (PTT)

If sizable energy dissipation is not necessary, and if gravity stresses are low in the panels, the post-tensioned tendon connection (Fig. 22) is quite attractive. Vertical reinforcement is provided in the form of high-strength prestressing bar, and these are spliced using standard couplers. Like the GSS detail, there are no grout pockets in this connection. Because the tendon is not grouted, it is not bonded to the panel, and this feature is deemed necessary to protect the tendon from inelastic strains that can reduce the effective prestress in the panels. The expected mode of response for this connection is transfer of lateral loads by flexure across the joint. Energy dissipation is not likely to be large, but the debonded tendon should allow ample inelastic displacement of the panel.

Prestressing strand can also be used, and it offers a much larger linear strain capacity than does bar. However, bar (Dywidag Systems) was selected because it is easier to handle and place in a vertical member. The uniformly-spaced tendons are a compromise between the performance-based strategy of protecting the tendons from large strain increases due to rocking and the construction-based need to provide some reinforcement along the jamb for placement of the panels and to limit out-of-plane “walking” as the panel rocks during ground shaking.

Precast Vertical Joint (PVJ)

The precast vertical joint specimen (Fig. 23) features two horizontal joints that are connected using grouted splice sleeves as in the GSS specimen. The innovative feature of this specimen is not the horizontal joint connection detail; rather, it is the ingenious way of introducing energy dissipation in a panel at locations where a vertical control joint may be desirable. The vertical joint is “precast” in the panel by forming vertical v-shaped grooves on both faces of the panel. The effective thickness of the panel at the groove location is tuned to the expected vertical shear force in the panel when the shear wall is subjected to peak load. Upon cracking, shear friction reinforcement, in the form of #4 rebar dowels, prevents separation of the cracked surfaces. Sliding of the panels along the cracked surfaces generates vertical shear resistance, as well as energy dissipation.
Debonded Smooth Bar (DSB)

A connection detail with large deformation capacity (Fig. 24) is created by using smooth bars that are hot-rolled from mild structural steel (Grade 50) for vertical reinforcement. The smooth bars are debonded from the panel over part of their length to allow a greater volume of steel to yield when the wall is subjected to peak lateral load. This feature is included to maximize the amount of energy that the panel is likely to dissipate. The smooth bars are connected to deformed reinforcing bars using proprietary coupling system (Richmond Screw Anchor) which employs upset and threaded bar ends. Once the bars have been connected, the corrugated ducts are filled with high-strength, high-flow grout. Plastic sheathing is used to ensure debonding of the lower portion of the smooth bar.

SUMMARY AND CONCLUSIONS

An experimental research program was undertaken with the goal of advancing the design of precast concrete shear wall connections according to the jointed construction philosophy. Seven vertical joint connection details have been tested under reversed cyclic drift histories, and four horizontal joint connections are scheduled for testing. Preliminary results indicate that the connection details selected for vertical joint tests respond favorably to the imposed drift history, and the observed characteristics, including strength, stiffness, ductility and energy dissipation capacity, make these connections suitable for earthquake-resistant design of precast concrete shear wall buildings.

The research in this program indicates that it is possible to design yielding plates of various configurations for vertical joint connections in precast concrete shear wall panels using current design methods. The desired mode of behavior was observed in all of the connections, even though the failure mode in some of the connections was not as expected. Anchorages and welds can be designed to resist large, random cyclic loadings without failure or significant loss of integrity, even though three of twenty-four rebar anchors fractured prematurely next to the welded ends. It may be necessary to use a more conservative design approach, or a better material than ASTM A706 reinforcing bars for these anchors.

Two variations of the commonly-used welded, loose plate have demonstrated good performance under simulated seismic loading, and the drift and displacement ductility capacities of the NSP and IFB are
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well-suited for use in regions of moderate seismicity. The IFB connection has ample deformation capacity, but out-of-plane bending and buckling eliminate any reliable compression strength. The coupled leg in the XAP connection, as well as the stiffener in the PTS connection, mitigate this adverse behavior. The VJF and UFP connection details display an outstanding ability to resist seismic forces over a wide range of displacement, while the former retains a nearly elastoplastic force-displacement hysteresis loop. Thus, the VJF and UFP are ideally suited for regions of the highest seismic risk.

Even though experimental results for the horizontal joint series are not presented, it is clear that a number of techniques which do not represent dramatic departures from current construction practice can be utilized to advantage. By addressing key aspects of expected behavior, such as compression damage along jambs and strain fracture of vertical bars, a group of four connection details for horizontal joints are presented that appear to satisfy typical construction constraints, as well as structural performance requirements for seismic resistance. Moreover, materials and connection hardware used for these details are commonly available in the U.S.

ACKNOWLEDGEMENTS

This research was carried out as part of the U.S.-PRESSS Program, M. J. Nigel Priestley, Coordinator. The University of Nebraska-Lincoln team, including Maher K. Tadros, Amin Einea, Xiaoming Huo, and Say-Gunn Low, is thanked for their support and collaboration. Financial support of the National Science Foundation (NSF), Mahendra P. Singh and Shih-Chi Liu, Program Directors, Grant BCS 91-23015; Precast/Prestressed Concrete Institute (PCI), Paul Johal, Research Director; Precast/Prestressed Manufacturers Association of California (PCMAC), Douglas Mooradian, Research Director; Precast Association of Nebraska (PCAN); and the Center for Infrastructure Research at the University of Nebraska-Lincoln (CIR-UNL) is gratefully acknowledged. The Industry Advisory Committee guiding this project, especially Jagdish Nijhawan, Vilas Mujumdar, Thomas D'Arcy, and Simon Harton, has provided valuable input. Interaction with other researchers within the PRESSS program, especially John Stanton, Susie D. Nakaki, Catherine W. French, and Michael E. Kreger and Geraldine Cheok, has been very valuable to this study. The assistance of Alvin Ericson is gratefully acknowledged. Also, Harry B. Lancelot (Richmond Screw Anchor), I. Mike Kanoh (Splice Sleeve North America), Bert C. L. Phong (Master Builders), and Ronald J. Bonomo (Dywidag Systems International) are thanked, respectively, for donating the reinforcing
bars fitted with threaded dowels and couplers, the splice sleeves, the specially-formulated grout (for use with the splice sleeves), and the post-tensioning hardware, respectively.

REFERENCES


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<th>Connection</th>
<th>Type of Joint</th>
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<tr>
<td>Debonded Smooth Bar (DSB)</td>
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†Connector added to experimental program
*TCY - tension/compression yielding
NE - nonlinear elastic
SY - shear yielding
FY - flexural yielding
F - friction
SF - shear friction
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†Large strength reductions in last three cycles. Values in parentheses are at end of test.
‡Prior to marked nonlinearity
At onset of marked nonlinearity
‡‡Normalized by $2P_o\Delta_o$, where $P_o = 120$ kN and $\Delta_o = 0.20$ cm
Fig. 1—Typical floor plan for University of Nebraska office building (1,2)
Fig. 2—Prototype shear wall
Fig. 3—Idealized wall response

Fig. 4—PRESSS simulated seismic drift history
Drift History

Moving Head

Spacer Beam

Post-Tensioned Rods

Connection

Joint

Fixed Head

Spacer Beam

Fig. 5—Test setup for vertical joint tests

Drift History

Moving Head

Spacer Beam

Post-Tensioned Rods

Joint

Fixed Head

Vertical Loads for Axial Stress and Top Moment

Fig. 6—Test setup for horizontal joint tests
Fig. 7—Notched shear plate (NSP) connection

All dimensions are in mm (in.)
All dimensions are in mm (in.).

Fig. 8—Slotted flexure plate (SFP) connection
a) Elevation

b) Section

c) IFB Connector  $t = 9.5$ (3/8)

All dimensions are in mm (in.)

Fig. 9—Inclined flat bar (IFB) connection
Fig. 10—X-shaped axial plate (XAP) connection

All dimensions are in mm (in.)
a) Elevation

b) Section

c) PTS Connector $t = 9.5$ (3/8)

All dimensions are in mm (in.)

Fig. 11—Pinned tension strut (PTS) connection
All dimensions are in mm (in.)

Fig. 12—Vertical joint friction (VJF) connection
All dimensions are in mm (in.)

Fig. 13—U-shaped flexure plate (UFP) connection
Fig. 14—Force-displacement response for specimen NSP

Fig. 15—Force-displacement response for specimen SFP
Fig. 16—Force-displacement response for specimen IFB

Fig. 17—Force-displacement response for specimen XAP
Fig. 18—Force-displacement response for specimen PTS

Fig. 19—Force-displacement response for specimen VJF
Fig. 20—Force-displacement response for specimen UFP.
Fig. 21—Grouted splice sleeve (GSS) connection
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Fig. 22—Post-tensioned tendon (PVJ) connection
Fig. 23—Precast vertical joint (PVJ) connection
Fig. 24—Debonded smooth bar (DSB) connection
Synopsis: This paper describes a research program to investigate the behavior of ductile connections between precast beam-column elements. Eight beam-column connections were tested to characterize the overall behavior of the connection details. Each connection specimen was designed to incorporate one of three behavioral concepts in the connection elements: tension/compression yielding, substantial energy dissipation, or nonlinear-elastic response. Based on the behavioral information collected during connection tests, analytical models were developed to investigate the behavior of complete precast frame systems. Results of the experimental study and preliminary results of the analytical work are presented. The objective of the program is to provide rational design recommendations for engineers to detail precast frame connections for use in regions of seismic risk.

Keywords: Connections; ductility; earthquake-resistant structures; energy; frames; friction; joints (junctions); precast concrete; unbonded post-tensioning
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INTRODUCTION

Precast concrete construction represents an attractive and economical solution for many types of multistory buildings. Advantages of precast concrete over cast-in-place concrete include superior quality control, speed of erection, and aesthetic architectural form. However, relatively few precast structures are constructed in seismic areas of the United States because framing methods and current connection details between precast elements suitable for use in U.S. construction practice are not explicitly approved in design codes. The current 1994 Uniform Building Code (UBC) classifies precast frame systems, such as those discussed in this paper, as an "undefined structural system" (1). In order to utilize such a structural system, the UBC requires that the lateral-force resistance and energy absorption capacity for this "undefined structural system" be shown by technical and test data to be equivalent to one of the structural systems defined in the code. In effect, current UBC provisions require that precast frames emulate a cast-in-place reinforced-concrete system.

The coordinated research effort "Precast Structural Seismic Systems" (PRESSSS) was initiated to address this need. The ultimate objectives
of the program are to develop precast concrete systems and corresponding design recommendations for seismic regions. The program has sought innovation, taking advantage of the unique characteristics of U.S. precast construction to provide economical building systems. With this philosophy, the research has focused primarily on development of systems which use ductile connection concepts for concrete frame and panel systems. The connections themselves are designed to accommodate most of the lateral deformation, while the precast members remain relatively undamaged. This approach is in sharp contrast to precast construction practice in Japan where buildings are designed using substantially different details and essentially the same design philosophy as is used for cast-in-place concrete buildings (2). Reinforcement details that are substantially different from those used in monolithic construction are used to accommodate connections between the precast members, but the final product has been shown to behave very similar to cast-in-place construction. Because of this, the Japanese precast design approach has been referred to as "emulation design". Earlier research (3,4) conducted in New Zealand during the 70's examined the cyclic behavior of precast, prestressed beam-column connections. Blakeley and Park recognized the obvious deformation capabilities inherent in precast, prestressed connections. However, even though these connections were not intended to "emulate" cast-in-place connection behavior, specimen response was evaluated using cast-in-place behavior as the reference. Similar precast, prestressed specimens discussed in this paper were designed and evaluated with quite different behavioral objectives in mind.

This paper describes the portion of PRESSS research conducted at the University of Minnesota (UMn) and The University of Texas at Austin (UT) to evaluate the behavior of ductile connections between precast beam-column frame elements. The research was divided into three major components:

1. Development of Ductile Connection Details
2. Experimental Investigation of Beam-Column Subassemblages

DEVELOPMENT OF DUCTILE CONNECTION DETAILS

In this phase of the project, a variety of ductile connection details were developed in cooperation with industry representatives. The details may be grouped into four behavioral categories: tension/compression yielding, energy dissipating, nonlinear-elastic, and shear yielding. Conceptual examples of different connection types are shown in Fig. 1. The first five details (Fig. 1(a)-(e)) represent connections which would dissipate energy through tension and compression yielding of the
connection elements.

Figure 1(f) shows an example of an energy-dissipating device. In this case, energy is dissipated through friction. Holes are slotted in the plates to accommodate the anticipated drifts without bolts going into bearing. Special materials may be used to enhance the friction coefficient.

Figure 1(g) illustrates a nonlinear-elastic connection (depicted by thickened beam end regions, referred to as horizontal "dogbones," with central post-tensioning). The detail was derived from a concept proposed by Priestley and Tao (5). The connection is constructed with unbonded post-tensioning reinforcement located close to beam middepth, and is detailed to maintain reinforcement stresses below the proportional limit during a design-level earthquake. The connection derives its name from characteristics of the associated story shear versus drift response. When the story shear attains a level sufficient to open the joint between each precast beam and the column, the response deviates from the original linear-elastic behavior. Even though behavior becomes nonlinear due to large deformations at the joint interface, the post-tensioning steel remains elastic during design-level events. The still elastic unbonded post-tensioned reinforcement causes the connection to return to its undeformed position when story shear is reduced to zero. The National Institute of Standards and Technology (NIST) has conducted a variety of tests on centrally post-tensioned connections that incorporate mild reinforcement details (6,7).

The shear-yielding concept, shown in Fig. 1(h), was investigated by Popov in steel frames employing eccentric bracing (8). The rigidity of the concrete frame may promote the development of shear yielding in a structural steel element similar to that observed in an eccentrically braced steel frame. The connection in Fig. 1(h) is shown in its original and deformed configuration.

**EXPERIMENTAL INVESTIGATION OF SUBASSEMBLAGES**

To characterize the behavior of the connection details, one-half scale precast beam-column subassemblages were fabricated and tested under reversed cyclic loads. The subassemblages were scaled from a system developed by PRESSS Phase I researchers at Englekirk and Sabol Consulting Engineers, Inc. The floor plan for the prototype structure is shown in Fig. 2. Lateral-force-resisting elements in this building are concentrated along the perimeter of the frame. Because the connections are intended to resist lateral force reversals along one direction, connection subassemblages were subjected to unidirectional reversed cyclic loads.
The one-half scale test specimens represented interior beam-column joints located in the lower stories of the peripheral frame spanning in the short direction of the 15-story prototype structure (also shown in Fig. 2). The required nominal moment capacities for the connections tested in this study were scaled from the prototype structure. The design moment $M_{ud}$ for the prototype structure was 2283 ft-k; the calculated one-half scale design moment was 285 ft-k. A strength reduction factor of 0.9 was used to obtain the nominal moment capacity, $M_n$, of 317 ft-k. The story drift level (ratio of story lateral displacement and story height) under the nominal moment $M_n$ was targeted to be 2 percent.

The nominal "loading" history is shown in Figure 3. The subassemblages were taken to increasing drift levels comprising the following sequence: one cycle each at 0.05, 0.075, and 0.1% drift, followed by three cycles each at 0.25, 0.5, 0.75, 1, 1.5, 2, 2.5 and 3% drift. After each set of three cycles, an intermediate cycle was imposed to a peak load of 75 percent of the previously attained peak load to investigate stiffness degradation. Some specimens were subjected to additional deformations of two cycles at 4% drift, one cycle each at 5, 6, and 7% drift and a final monotonic displacement to 10% drift. For convenience, desired drift levels were applied to specimens by vertically displacing the beam tips in opposing directions (Fig. 4a) rather than by laterally displacing the top of the column.

A total of eight beam-column subassemblages, representative of three of the four connection categories, were tested. An example of the shear yielding concept was not included in this study.

Tension-Compression Yielding Concept

Four of the connections tested represented examples of the tension-compression yielding concept: UT specimens GAP and DB; and UMn specimens GAP and TCV.

UT-GAP was representative of the "gap-joint system" which received very favorable reviews from the precast industry advisory group (Fig. 5). In the gap-joint system, the beam is intended to be restrained from translation at the bottom (or top) of the beam-column interface. Lateral movement is accommodated by rotation about that point through opening and closing of a gap (1 in. wide in this specimen) at the top (or bottom) of the interface. The gap system is desirable for two reasons: (1) It enables ease of fabrication of the bottom connection; the erectors can lower the beam into place and complete both the top and bottom connection while working from the top of the beam. (2) If the gap is provided at the bottom of the beam, the lack of translation at the
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top surface will prevent large cracks from developing across the floor panels at the beam-column interface under cyclic lateral loading. In the tension/compression yielding type connections, the gap at the interface of the beam and column permits both tension and compression yielding of the connecting elements.

Specimen UT-GAP incorporated tapered threaded couplers in the column to mate with mild reinforcement in the cast-in-place topping of the beam. The bottom horizontal connection and resistance to beam-end uplift were provided by four high-strength vertical rods anchored in the corbel. Oversized holes in the beam provided sufficient construction tolerance to slip the beam ends over the rods. Following placement of the beams, nuts were fastened to the ends of the rods, then the voids around the rods were grouted. Each beam was seated on a neoprene pad to accommodate the rotation of the beam without causing damage to the corbel. Fiber-reinforced grout was placed in the bottom of the gap between the beams and column to facilitate direct transfer of compression forces from the bottom of each beam to the column. The tops of the beams were cast after the beam top bars were screwed into the threaded couplers embedded in the columns.

Response of the specimen is illustrated in Fig. 6 using story shear versus drift. Overall response of the connection was reasonably good through cycles to 2.5% drift. However, some pinching of the hysteresis loops was evident with the onset of inelastic behavior at approximately 0.6% drift. Most of the pinching was attributed to shear and flexural deformations that occurred in the vertical rods at the interface between the corbels and beams. This slip between the two members is indirectly demonstrated in Fig. 7 by the moment-rotation response of one of the beams. Note that stiffness was generally less for loading in the positive-moment direction (which corresponded with opening of the gap at the bottom of the beam). Deformation of the vertical rods at the beam/corbel interface became more pronounced at higher deformation levels and eventually dominated the response. Failure of specimen UT-GAP occurred as the result of fracture of the vertical rods during cycles to 3% drift (note strength reductions in Fig. 6). The compliant bottom connection also affected the inelastic response of the beam top bars which were intended to experience most of the inelastic action in the connection. Figure 8 illustrates the stress-strain response of a beam top bar at the face of one of the beams. Note the tensile inelastic strains (corresponding with opening of the gap at the top of the beam) were substantially greater than compressive inelastic strains. This observation will play a critical role in interpreting the behavior of Specimen UMn-GAP discussed later.

UT-DB employed vertical "dogbones" and high-strength threadbars to
connect precast beams and columns (Fig. 9). High-strength fiber reinforced grout was placed between the beam ends and the column before the beams were connected to the column. Ducts that contained the threadbars were grouted after the threadbars were snug tightened. The story shear versus drift response is illustrated in Fig. 10. Connection behavior was reasonably acceptable through 1% drift cycles, although the high-strength threadbars resulted in less energy dissipation than would be anticipated for monolithic beam-column connections. During loading in the positive direction to 1.5% drift, concrete located between the 90 degree hooks on the longitudinal beam reinforcement and the anchor plates at the ends of the dogbones crushed. The same behavior was observed at approximately 2% drift when the connection was loaded in the negative direction.

The use of connections that provided an indirect path for force transfer between precast elements was the common thread that precipitated the premature failure of specimens UT-GAP and UT-DB.

UMn-TCY was a simple system that incorporated block-outs through the beams and embedded corrugated pipes in the column to accommodate placement of reinforcement. Many of the other connections investigated required the use of couplers or other arrangements which caused discontinuities in the load path. With this system, it was intended to avoid the use of couplers by placing continuous reinforcement in the beam block-outs which could be slid through the column during construction, tied in place, and subsequently grouted. The beam reinforcement could be spliced at midspan near the inflection point for reversed cyclic load. The connection reinforcement was designed to yield at the beam-column interface. The transverse beam reinforcement was increased by approximately 30% (No. 3 bars at 3.5 in.) with respect to the calculated minimum amount (No. 3 bars at 5 in.) to increase the confining action on the longitudinal beam bars, and therefore reduce the possibility of buckling of the bars after reaching yielding. The specimen is shown in Fig. 11. The transverse reinforcement and horizontal skin steel is shown only on the East Beam for clarity. The cross-hatched areas in the beam cross section represent the block-out regions which were grouted after placing the connection reinforcement.

Figure 12 shows a plot of story shear versus drift. Specimen UMn-TCY performed quite well for the design level and up to nominal drifts exceeding 4% (actual drift 3.6%); the peak story shear was reached during the first cycle at 4% nominal drift. At nominal drifts of 5 and 6% the corresponding maximum attained story shear did not show any significant sign of decreasing with respect to the peak story shear attained at 4%. The story shear-drift curves show significant signs of
pinching in the latter part of the test (above 4% drift) which can be attributed to several factors: a) slip between the corrugated pipes housing the longitudinal beam reinforcement (grouted within the pipes) and the surrounding column concrete, b) buckling of the longitudinal beam reinforcement in the connection region close to the beam column interfaces, with consequent spalling of the concrete cover in the beam top and bottom surfaces, and c) relative vertical movement (slip) between column and beams as a result of the elongation, due to yielding, and kinking of the beam longitudinal reinforcement.

Strain gage data indicated that yielding of the beam longitudinal bars occurred at locations corresponding to the beam-column interfaces at nominal drift values of about 1.5%.

UMn-GAP was another example of a tension/compression yielding gap-joint system. The connection is shown in Fig. 13. In this case, the bottom connection was made using two lightly post-tensioned bars (Area / bar = 1.25in.$^2$) each stressed to 94 kips/bar. Although not as easy to erect in the field as the gap system with the vertical rod (UT-GAP), the horizontal element was more efficient in carrying the large horizontal restraining force (200 kips). The post-tensioned bottom reinforcement prevented relative opening of the bottom face of the connection and provided a pivot point for the beam rotation. The post-tensioning was used to provide sufficient clamping force to sustain gravity and lateral shear loads while ensuring the post-tensioning bars would not undergo inelastic deformation during the loading history. Nearly all the deformation was concentrated at the top of the connection through tension and compression yielding of three No. 9 reinforcing bars which were connected through couplers at the beam-column interface.

Because of the longitudinal beam reinforcement buckling experienced during the test of UMn-TCY, the beam transverse reinforcement was increased by 75% (No. 3 bars at 2 in.) in the beam within 1 ft. of the beam-column interface. In the post-tensioned regions at the beam ends additional spiral reinforcement was used to increase the concrete compressive strength and deformability.

The connection was assembled as follows: grout was applied and cured within approximately the bottom third of the beam-column interface; the bottom connecting rods were lightly post-tensioned; the beam top reinforcement was threaded into the couplers cast through the column, and the beam block-outs were grouted. The couplers were threaded after post-tensioning to ensure that no significant initial stresses were induced in the beam top reinforcement. A 1 in. wide gap was left open over the top approximately two-thirds (14 in.) of the beam-column
The beam longitudinal top reinforcement, left exposed in the gap region, was intended to yield in tension and compression under cyclic loading. With increasing levels of applied deformation, the inelastic behavior of the reinforcement would eventually propagate into the beam. Another feature anticipated with the use of the "gap concept" was the limitation of concrete deterioration under reversed cyclic loading. Under imposed deformations the gap should be able to accommodate relative rotations between beams and column, therefore limiting the amount of concrete damage in the beam and column members.

Specimen UMn-GAP performed satisfactorily up to 2% nominal drift. Figure 14 shows the plot of story shear versus drift. The hysteretic behavior of the connection indicates good energy dissipation characteristics as the specimen was subjected to load cycles into the inelastic range. While cycling at the 2% imposed drift level, however, the top reinforcement across the beam suffered a brittle fracture at the beam-column interface, at the face of the couplers. Failure occurred first on the east side of the connection, then after two more cycles on the west side. Data obtained from strain gage measurements show that yielding of the reinforcement was not limited to the gap region, but did propagate within the beam over a region varying between one and two feet. The couplers used were the same type as those used in UT-GAP which experienced 3% drifts before a different connection component failed. However, Fig. 8 demonstrated that the inelastic tensile demand on beam top bars in UT-GAP was much greater than the inelastic compressive demand. Because the bottom connection in UMn-GAP behaved as intended, it is quite likely that much greater inelastic demands (perhaps approximately equal tensile and compressive demands) were imposed on the beam top bars in this specimen than on bars in UT-GAP.

A series of tension tests was conducted on the same reinforcement bars as those used in the specimen, and on bars and coupler assemblages. The results indicated that tensile failures in bars connected with the couplers are not as ductile as those observed for individual bar tests. It should be noted that in all cases the ultimate loads carried by the bar plus coupler assemblages were greater than the loads required by the specifications.

The performance of the post-tensioning rods, comprising the bottom connection, was satisfactory. It appears that the beam rotated about the post-tensioning centroid as anticipated. Very little concrete damage was observed in beam and column members in the connection region.
Energy Dissipating Concept

UT-FR was the only connection tested in this program that incorporated special connection hardware to enhance the energy dissipation capacity of the beam-column connection. The connection details are illustrated in a schematic shown in Fig. 15. The top of each beam was connected to the column by a steel plate assembly that was embedded in the beam and bolted onto the side of the column. The plate assemblies contained 4 in. slotted holes that permitted sliding along vertical plate surfaces on the sides of the beams. Consistency in the level of force required to cause sliding along these plate surfaces was obtained by placing 1/8 in. thick brass plates between all sliding surfaces. The friction coefficient for brass sliding on steel was approximately 0.2. The force required to cause slip on these plate surfaces was controlled by the clamping force applied to the plates which was maintained relatively constant by placing conical (belleville) washers beneath the high-strength bolts that clamped the plates together. The concept for this friction connection stemmed from work conducted by Gregorian, Yang and Popov (9).

The bottom connection between the beams and column was developed to replace the corbel which performed relatively poorly in UT-GAP. Note that the bottom connection shown in Fig. 15 provided a direct path for force transfer from the bottom of the beams into the column, as well as a much more aesthetically pleasing detail. Three 1-1/8 in. diameter A490 bolts were used to connect the bottom of each beam with the corbel. The line of bolts was also intended to act as a pivot point for beam rotation.

Specimen UT-FR behaved quite well through the first two cycles to 3% drift (see story shear versus drift response in Fig. 16). Energy dissipation was enhanced, as intended, and was substantially greater than for any of the other specimens tested in this study. Testing was terminated during the third cycle to 3% drift because a weld between plates in one of the top connections failed as a result of larger than anticipated forces being developed in the connections. It was anticipated that the hysteresis loops shown in Fig. 16 would resemble elasto-plastic behavior. However, rotation of the beams forced some bolts in the friction devices to bear against the sides of the slotted holes, and resulted in the post-slip stiffness that is evident in the story shear versus drift plot. Demand on many of the components used in the connection (such as the failed weld between plates) exceeded design capacities as a result. The assemblage of plates used in the top connections and the high-strength bolts used in the bottom connections introduced connection flexibility at low drift levels that was approximately twice that observed for the other specimens. Designers
should be aware that the overall flexibility of precast beam-column connections is very sensitive to the stiffness of the connecting elements, and as result, proportions of connections between precast elements can be controlled by stiffness rather than strength.

**Nonlinear Elastic Concept**

**UMn-PTS/PTB** were two specimens that represented the nonlinear-elastic post-tensioning concept (5). The precast beams and columns were connected with unbonded post-tensioning steel which passed through the joint and was anchored in horizontal "dogbones" located at the beam ends (Figs. 17 and 18). The intersecting spiral reinforcement shown in the top and bottom regions of the beam dogbones was provided to enhance the concrete confinement in those zones.

The difference between specimens UMn-PTS and UMn-PTB consisted in member sizes and post-tensioning types: UMn-PTS used monostrands (multistrands in the prototype), and UMn-PTB had shallower beams and used high strength threaded bars. The beams were deeper in the case of PTS to accommodate the strand anchoring plates. The monostrand/multistrand system offers more economical use of post-tensioning steel while the thread bar is more easily installed and exhibits less significant seating losses than the former.

To achieve nonlinear-elastic behavior, the connection post-tensioning steel should remain elastic even when subjected to large lateral deformation levels. The choice of the initial prestress is critical to guarantee this kind of behavior (5). The initial prestress was limited to the amount required to ensure the beam-column interface joint would remain closed under the design gravity and wind loads. During seismic loading, the connection opens creating a concentrated beam rotation at the column interface. During this process it is intended that the post-tensioning steel not be stressed into the inelastic range. As a consequence, the connection returns to its undeformed position upon unloading. During this process the connection is able to accommodate relatively large deformations while maintaining its load carrying capacity. The nominal required moment capacities of the beams at the interface of the dogbone region were increased by 30 percent to insure that the deformations remained concentrated in the connection regions while the members themselves remained relatively undamaged.

Both specimens, UMn-PTS and UMn-PTB, performed quite well for design level drifts up to 3%. Figure 19 shows a plot of story shear versus drift for UMn-PTS. Figures 20 and 21 illustrate the moment versus rotation response with respect to the superimposed predicted
behavior for UMn-PTS and UMn-PTB, respectively. The moment refers to the moment at the beam-column interface. The measured rotation refers to the rotation of the beam relative to the column measured within 7 in. from the interface. Predicted rotations represent the concentrated rotations calculated from assumed crack openings at the column-dogbone interface.

The predicted behavior was obtained using the model proposed by Priestley and Tao (5) that was used to size and detail both connections. The relationship between load and displacement is assumed to be linear until the compression force in the outermost concrete fiber is zero; this point is defined as \( M_{cr} \). The value of \( M_{cr} \) can be evaluated by force balance within the cross section. The value of the external force, \( F_{cr} \), needed to generate that moment is found by geometric considerations. The behavior of the connection is further considered linear up to a moment of twice \( M_{cr} \). At this moment, designated \( M_2 \), the crack opening is assumed to have propagated to the centroid of the section (the prestressing steel is assumed concentrated along the centroidal axis). The value of the corresponding external applied force is twice \( F_{cr} \). After this point the behavior of the connection will start to deviate considerably from linear. For a point beyond this "linear" limit, the value of the moment was evaluated by internal force balance, as a function of the stress in the prestressing steel and an assumed equivalent rectangular concrete compression block. The change in steel stress at points above \( M_2 \) result directly from the crack opening at the assumed centroid of the prestressing steel. This elongation causes a concentrated rotation to develop in the connection region. This information can be used to determine the increased beam end rotations and drifts.

In reviewing Fig. 20, it can be observed that the prediction gave a reasonably close envelope curve for the experimental data. The sensitivity of the model to the assumed concrete spalling depth is apparent in the plots where the expected behavior of the specimens is presented both for an assumed spalling depth of 0.75 in., equal to the depth of the concrete cover, and for an assumed spalling depth of 1.5 in., equal to the average depth of the concrete not confined by the dogbone spiral reinforcement. Even though extensive spalling was not observed on a large scale during the tests, deterioration of the grout joint and the consequent increase in the relative rotation between beams and column, make the assumption of a spalling depth of 1.5 in. reasonable.

Connection PTS was taken to drifts exceeding 3% while exhibiting stable hysteretic behavior; the peak story shear was reached during the first cycle at 3% drift. In the next set of cycles to 4% drift, the story
shear necessary to reach that drift was lower than the maximum story shear attained during the previous set of cycles to 3%. Due to failure of the strain gages during the post-tensioning operation for specimen PTS (as a consequence of damaging the gages during strand installation) it was not possible to investigate the behavior of the post-tensioning steel during testing to determine at which drift level yielding of the strands initiated. When the specimen was initially displaced to 3% drift the monostrands started to fracture (this was assumed after hearing a loud snapping noise and observing the simultaneous dropping of the load necessary to maintain the specimen at that drift level). The fracture of more strands in the following cycles to 4, 5, 6, and 7% drift levels impaired the clamping action of the post-tensioning strands; this can be seen in the story shear-drift plot where substantial pinching was observed during load reversal. It is important to notice that fracture of the strands occurred at drift levels higher than 3%; at those drift levels elongations of the strands were such that stresses in the steel were well beyond the yield point. During tension tests performed on the post-tensioning steel, the strands did not show much capacity after yielding, therefore the failure of the strands was not considered premature.

Connection PTB was taken to drifts exceeding 4% while exhibiting stable hysteretic behavior; it was only at the first cycle to 5% drift that the peak load necessary to reach that drift was lower than the peak load attained in the previous set of cycles to 4% drift. The moment-rotation plot (Fig. 21) showed little evidence of pinching at low values of moment during the load reversal phases; an indication that no significant loss of the post-tensioning clamping force took place. A story shear-drift plot is not presented for this specimen because movement of the loading frame during testing contaminated the drift data. Data obtained from strain gages mounted on the post-tensioning bars in PTB showed that there was indeed no yielding of the post-tensioning steel up to the design moment. Yielding of the bars started to occur at drift levels of approximately 4%. Spalling of the concrete cover in the dogbone side areas and consequent reduced bearing support under the post-tensioning bar anchor plates initiated at 4% drift and continued until the extent of damage was such that the post-tensioning bars deformed out of plane and the test was considered concluded. This failure was observed at a value in excess of 9% drift while loading monotonically from 6 to 10% drift.

The behavior of both connections was close to nonlinear elastic up to the design drift level. It was only in the final part of each test, when the specimens were taken to failure, that the areas within the hysteresis loops began to increase as a consequence of concrete cracking and yielding of the steel reinforcement (both conventional and
post-tensioning). As mentioned earlier, this inelastic behavior can be considered as a positive phenomenon when energy dissipation is taken into consideration. From the moment-rotation response for PTS (Fig. 20), it can be seen that in the final part of the test, at load levels beyond the design moment of 317 ft-k, rupture of the post-tensioning strands caused severe pinching of the hysteresis loops.

**UT-PTS** was the third connection tested that was intended to investigate nonlinear-elastic behavior. This connection differed from the connections tested at the University of Minnesota in that the precast beam was continuous through the connection region (see Fig. 22), and the beam was pretensioned instead of post-tensioned. The prototype system from which this specimen was derived was envisaged to have precast beam elements that spanned from midspan to midspan over two columns. Beams would be connected at midspan to resist shear and gravity moments only. Precast column elements would span between floors.

Specimen UT-PTS was pretensioned using 20 centrally-located 3/8 in. diameter strands pretensioned to 40% of $f_{pu}$. In order for the strands to remain elastic through specimen drifts of 2%, they were debonded through the joint and for 2 ft. on each side of the column (a total of 69 in.). Spiral reinforcement was placed in the top and bottom of each beam within 18 in. from the column interface to confine the concrete and permit large rotations at the beam-column interface. Approximately 75% of the joint reinforcement recommended by ACI-ASCE 352 (10) was used in the specimen. This was done with the hope of precipitating a joint failure because none of the other specimens exhibited signs of significant distress in the joint region, and because the force transfer mechanism for joints in prestressed specimens (with unbonded prestressing through the connection) is comprised primarily of a direct compression strut (a mechanism which differs substantially from behavior of monolithic joints at low load levels).

Specimen UT-PTS was assembled by first placing the lower column segment into the test setup. The beam/joint segment was then lowered over the 16 column bars that protruded from the top of the lower column segment. The half inch gap that was left between the lower column segment and the beam/joint segment was grouted along with the ducts in the beam/joint segment. The upper column segment was lowered into position and column bars were joined immediately above the beam using threaded couplers. The recess provided around the base of the upper column segment to accommodate the threaded couplers was filled with fiber reinforced grout.

Story shear versus drift response for the specimen is shown in Fig. 23.
Behavior through the 2% drift cycles generally resembled measured behavior of UMn-PTS (Fig. 19), except hysteresis loops for UT-PTS were pinched at low drift levels. Pinching was the result of slip in the column bar couplers once the column cracked. The pinching would have been substantially less if axial load had been applied to the column (no axial load was applied to any of the columns) and if commercial products intended for coupling conventional bars had been used. Commercial bar couplers were not used because of the extreme congestion that existed in the half-scale specimen.

Energy dissipation increased markedly during the cycles to 4% drift, but not as the result of yielding of strands. Strain gages mounted on strands in the debonded regions indicated that strands remained elastic throughout the test. It was presumed that bond between the concrete and strand deteriorated outside the debonded region (increasing the debonded length) as the test progressed. The substantial increase in energy dissipation was attributed to failure of the joint. Although transverse reinforcement began yielding at approximately 1.5% drift, significant dilation of the joint and obvious crushing of concrete along diagonals of the joint were not evident until cycles to 4% drift. Even though concrete spalling adjacent to the column face was as much as 1 in. during cycles to 5% drift (only slightly more than the cover thickness), it is believed that deterioration in connection strength was primarily the result of damage incurred by the joint.

**COMPARISON OF SPECIMEN RESPONSE**

In the interest of brevity, comparisons of specimen behavior, based on the observations contained in the discussions of individual specimen response, will be made in the summary section. The following discussion focuses on stiffness degradation and residual drifts observed during the tests.

**Stiffness Comparisons**

Small amplitude intermediate displacement cycles (to loads representing 75% of the previously attained peak loads) were used to provide information about stiffness degradation in the specimens. Intermediate displacement cycles were not applied until drifts reached 0.25%. For displacement cycles below 0.25% drift, data from the peak response in each cycle was used. Stiffness values for each of these cycles were calculated using the story drift (equivalent column displacement) at zero load and the story shear and drift at peak response (i.e. secant stiffness). Stiffnesses were calculated for both positive and negative loading cycles.
Figure 24 presents secant stiffnesses for all test specimens as a function of the maximum drift level attained during the previous displacement cycles. For drifts less than 0.25%, stiffness was plotted versus actual drift. Stiffnesses for UMn-PTB corresponded with drift cycles administered after instrumentation was installed to monitor movement of the loading frame.

The most significant stiffness reduction for all specimens occurred during the initial displacement cycles, and was primarily due to member cracking. Initial stiffness values varied by as much as 75% (UMn-PTS vs. UMn-GAP). Some differences in initial stiffness can be explained because of differences in member proportions or connection details. For instance, the column in UMn-PTB was larger than that used in most of the other specimens, and the depth of beams in UT-DB was reduced by 25% over much of the beam length. Furthermore, UT-FR incorporated a complicated plate assembly for the top connection between the beams and column and high-strength bolts for the bottom connection that introduced additional flexibility in the specimen.

The trend in stiffness degradation was similar for all specimens, although specimens having the highest initial stiffness tended to experience a higher rate of degradation. This is illustrated in Fig. 25 by normalizing the secant stiffness data presented in Fig. 24 by the maximum secant stiffness for each specimen. Note that specimens with the highest initial secant stiffness in Fig. 24 typically had the lowest normalized stiffness in Fig. 25 for drifts of 0.75% or larger. It should be noted that two of the specimens, UMn-TCY and UT-GAP, demonstrated significant amounts of pinching in the story shear - drift responses at drifts as low as 1%. Stiffness degradation was substantially higher for these specimens considering the tangent stiffnesses (computed at low load levels).

Residual Deformations

To investigate the ability of each connection to return to its original undeformed position following unloading, the residual displacement after each set of cycles was evaluated. Results are given in Fig. 26 as a function of the maximum drift level reached during the previous set of cycles. Residual deformations were evaluated with respect to the original undeformed position at the beginning of each test. The plot can be divided in three regions. The first region, from the start of the test up to an applied drift of 1% shows relatively minor differences in residual drifts for all specimens. Most of these differences can be attributed to slip in the connection details that utilized indirect load transfer paths.
In the second region, between 1 and 3% drift levels, prestressed connections exhibited slight increases in residual drifts that remained between 0.1% (UMn-PTS) and 0.3% (UT-PTS), while the residual drifts for the other connections reached 1% or more. This difference reflects anticipated differences in behavior between the three connection concepts. The nonlinear elastic connections (UMn-PTS and PTB, and UT-PTS) can undergo relatively large deformations without sustaining significant permanent deformations. After unloading, the connections can return to a configuration close to the original undeformed one. Specimens tested to study the tension-compression yielding concept (UT-GAP and DB, and UMn-GAP and TCY) are supposed to deform plastically in tension and compression as a means to dissipate energy through hysteresis. Consequently, these specimens developed permanent deformations evident in Fig. 26. The sharp increases in residual drift values can be seen as a consequence of the many factors that finally led to specimen failure: yielding and elongation of beam longitudinal reinforcement for UMn-GAP; yielding and elongation of longitudinal reinforcement with related sliding of the beams with respect to the column, slip of corrugated pipes within the column due to deteriorated bond, and buckling of longitudinal beam reinforcement for UMn-TCY; and yielding and elongation of beam longitudinal reinforcement combined with inelastic flexural and shear deformations in vertical bars at the interface between beams and corbels for UT-GAP. The specimen tested to examine the influence of an energy dissipating device in a connection (UT-FR) was also supposed to deform "plastically" to dissipate energy. The "permanent" deformations that developed in this specimen as the result of friction plates sliding past each other were greater than for the other connections. These deformations could be relieved by loosening the bolts in the friction connection device and restoring the connection to its original undeformed position.

In the third region, where applied drift levels are higher than 3%, an increase in residual drift was observed for all specimens (test data for specimens UT-DB, UMn-GAP, and UT-GAP did not exist for drifts greater than 3%). In the case of UMn-PTB and PTS the increase in residual drift was primarily due to yielding of the post-tensioning bars and rupture of some post-tensioning strands, respectively. For UT-PTS, the increase was related to deterioration of the joint region. Strands in this specimen did not yield. For UMn-TCY the increase was due to the aforementioned deformation process and the resulting specimen degradation. It is important to notice that the magnitude of the final residual drifts was substantially different for the two types of connection concepts: at an applied drift level of 4%, the residual drifts for UMn-PTB, UMn-PTS and UT-PTS were on the order of 0.5%, while for UMn-TCY it was approximately 1.6%.
The subassemblage tests provided detailed information about the behavior of individual connections, particularly in the form of load-deformation hysteretic response. This information was incorporated into nonlinear dynamic analyses of five- and fifteen-story frame systems subjected to a variety of ground motions. The analyses were conducted using the DRAIN-2DX program developed in PRESSS Phase I. The objective of the analyses was to establish gross measures of response, such as story drifts, global and local ductility demands, etc., necessary to determine the suitability of the frame systems incorporating the various connection details in seismic regions. The results were compared with those of an idealized cast-in-place structure.

The fifteen-story frame system that is being analyzed is one of the perimeter frames of the structure described earlier (Fig. 2) which was designed by Englekirk and Sabol Consulting Engineers, Inc. The five-story frame structure had the same footprint as the fifteen-story building and was proportioned in a manner consistent with the design of its larger counterpart; using the equivalent lateral force procedure in the UBC for Seismic Zone 4, an $R_w$ of 12 and $S$ equal to 1.2.

Two of the hysteretic models used are shown in Figs. 27 and 28: a nonlinear inelastic model and a nonlinear elastic model. These models are idealized to represent the behavior of monolithic reinforced concrete and precast prestressed concrete connections, respectively. The nonlinear inelastic model is currently available in the DRAIN-2DX program; the stiffness degrading model is one of two developed at the University of Minnesota to characterize the behavior of the nonlinear-elastic precast connections observed during the laboratory tests. The model introduces distinct bilinear loading and unloading lines. After unloading occurs, the next loading takes place along the previously defined unloading branch. The slopes of the unloading branches depend on the maximum deformation level reached at the onset of unloading.

The computed roof-level displacement responses for the five and fifteen-story frames with the hysteretic models shown in Figs. 27 and 28 and subjected to the 1940 El Centro NS acceleration record (scaled to a peak acceleration of 0.4g) are shown in Figs. 29 and 30. Maximum displacements of the frame systems with "prestressed" connections exceeded displacements for the "monolithic" systems by 50 and 75% for the five and fifteen story frames, respectively. The calculated displacement waveforms for the precast prestressed frames exhibited slightly longer apparent periods than the waveforms for the
monolithic frames. The most noticeable difference between the two systems is in the magnitude of the residual displacements. At the end of the seismic disturbance, the "prestressed" connections exhibited negligible residual displacements for both the five and fifteen story buildings, while the "monolithic" buildings were left with roof eccentricities of approximately 0.8 and 2.5 in., respectively.

SUMMARY

Research has been conducted to investigate the behavior of ductile moment-resisting connections between precast beam and column elements. The connections evaluated may be classified into three behavioral categories: tension-compression yielding, energy-dissipating, and nonlinear-elastic. Eight connections from these categories were subjected to reversed cyclic load tests to characterize their behavior. Results of the tests were used to develop analytical models incorporating the observed connection behavior to investigate the suitability of details tested in this program for seismic regions and to develop recommendations for the design of ductile beam-column connections.

All of the specimens, except UT-DB and UMn-GAP, performed well through design-level drifts. Specimen UMn-GAP experienced bar fractures at the face of the bar couplers during 2% drift cycles, and UT-DB experienced concrete crushing in the dogbone regions of the beams as a result of using a connection detail that incorporated an indirect path for transfer of forces at the ends of the beams. This same problem (indirect force path in the connection region) contributed to failure of UT-GAP (fracture of dowel bars in bottom connection between beams and corbel) and UT-FR (fracture of a weld in the energy-dissipating connection) during 3% drift cycles. The indirect force path inherent in the connections also resulted in additional flexibility that was most apparent at low drift levels. Particular attention must be paid to designing these connections for appropriate levels of stiffness.

Although not all specimens performed as well as others, most contained at least one connection detail that demonstrated good performance throughout testing. For example, the corbel/dapped beam combination used in UT-FR and the lightly post-tensioned bars used in the bottom of beams in UMn-GAP did not deteriorate during testing, and they provided a well-defined "pivot point" to accommodate deformations developed in the connections at the beam tops. Note that these gap connections could also be inverted, providing the gap at the beam bottom to avoid distortion and damage in the slab/horizontal diaphragm. Care must be exercised, however, in choosing the other
connection to be paired with these connections. Specimen UMn-GAP apparently failed because of the severe inelastic demand that was placed on the coupled bars in the top of the beams.

Prestressed specimens, UMn-PTB and UT-PTS, exhibited the desired nonlinear elastic behavior through 3% drift levels, while UMn-PTS apparently experienced strand fractures during load cycles to the same drift level. All of the unbonded prestressed specimens experienced minimal residual drifts during cycles up through design drift levels. Energy dissipation for these connections was quite low for the same drift levels. In contrast, energy dissipation for the connection containing a friction device (UT-FR) was quite high. The higher cost associated with the friction connection may preclude the use of such a connection throughout the structure, but the superior energy dissipating characteristics suggest that a few of these connections placed at strategic locations in a structure, such as one utilizing prestressed connections, may dramatically improve the overall seismic resistance of the structure.

All but one of the connections tested in this study contained transverse reinforcement in the joint that was quite similar to that recommended by ACI-ASCE 352 (10). None of these connections demonstrated signs of joint distress, indicating that the ACI-ASCE 352 provisions are conservative for the design of most precast beam-column joints. Specimen UT-PTS was constructed with 75% of the joint reinforcement recommended by Committee 352. The specimen maintained strength and showed no signs of distress until the second drift cycle to 4%. More attention will be directed in the future to development of a design procedure for joint reinforcement in prestressed connections.

Analyses have indicated that precast prestressed frame systems may experience moderate increases in displacements as compared with monolithic concrete frame systems. The increased displacements appear to be less for larger period (fifteen-story vs. five-story) structures. The analyses verified reduced residual drifts associated with the nonlinear-elastic connection systems.

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BIBLIOGRAPHY

a) Vertical dogbones with threaded rebar

b) Cross-bars through joint

c) Steel beam detail

d) Welded detail

e) Gap joint

f) Friction plates

g) Horizontal dogbones with central post-tensioning

h) "Eccentric braced frame (EBF)"

— Undeformed and deformed views

Fig. 1—Connection types considered in study
a) Typical floor plan

Fig. 2—Prototype precast frame building
Joint modeled in the lab

b) Elevation view

Fig. 2 cont.—Prototype precast frame building
Fig. 3—Nominal displacement history

Displacement History

Applied Drift (%)
a) Deformed shape in test setup

b) Ideal deformed shape for connection

Fig. 4—Deflected shape of subassemblages
Fig. 5—Schematic of UT-GAP

Fig. 6—Story shear-drift response for UT-GAP
Fig. 7—Moment-rotation response for west beam of UT-GAP

Fig. 8—Stress-strain response at column face for beam top bar in UT-GAP
Fig. 9—Schematic of UT-DB

Story Drift (%) vs Story Shear (Kips)

Fig. 10—Story shear-drift response for UT-DB
Fig. 11—Connection details for UMn-TCY
Fig. 12—Story shear-drift response for UMn-TCY
Fig. 13—Connection details for UMn-GAP
**Fig. 14**—Story shear-drift response for UMn-GAP

**Fig. 15**—Schematic of UT-FR
Fig. 16—Story shear-drift response for UT-FR
Fig. 17—Connection details for UMn-PTS
Fig. 18—Connection details for UMn-PTB
Story Shear - Drift

Fig. 19—Story shear-drift response for UMn-PTS

Moment - Rotation

Fig. 20—Moment-rotation response for west beam of UMn-PTS
Fig. 21—Moment-rotation response for west beam of UMn-PTB

Fig. 22—Schematic of UT-PTS
Fig. 23—Story shear-drift response for UT-PTS

Fig. 24—Secant stiffness versus drift
Fig. 25—Normalized secant stiffness versus drift

Fig. 26—Residual deformations for each set of drift cycles
Fig. 27—Schematic on nonlinear inelastic hysteresis model

Fig. 28—Schematic of nonlinear elastic hysteresis model
Fig. 29—Computed roof-level displacement response for five-story frame subjected to 1940 El Centro
Fig. 30—Computed roof-level displacement response for fifteen-story frame subjected to 1940 El Centro
Inelastic Design of Earthquake Resistant Reinforced Concrete Buildings Considering Displacement and Energy Limits

by A. Shibata, N. Inoue, and N. Hori

Synopsis: In the earthquake resistant design of RC buildings, it is necessary to evaluate inelastic behavior and damage of structures both by maximum displacement and by total energy dissipation. In this study, damage assessment of RC structures is carried out based on energy response. Damaging potential of earthquakes to structures is estimated by total input energy, and damage of structures is estimated by the damage index taking account of both maximum response and cumulative damage.

From the results of parametric inelastic response analyses using simulated earthquakes, it is considered that total input energy depends primarily on earthquake property. The damage parameter proposed by Fajfar, which relates ductility factor to dissipated hysteretic energy, seems to be relatively stable in many cases. The damage parameter is found useful to represent earthquake response pattern of structures.

Using the damage parameter and the damage index, a procedure is presented to find yield force and corresponding ductility factor for given value of damage index. This study shows a possibility of a design concept of RC buildings considering displacement and energy limits.

Keywords: Damage; ductility; earthquake-resistant structures; energy; inelastic response; reinforced concrete
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INTRODUCTION

Earthquake resistant design of RC buildings requires the consideration of the level of damage induced in structures during earthquakes in terms of maximum displacement as well as total energy dissipation. Therefore, it is necessary to evaluate inelastic behavior and damage of RC structures taking account of both maximum displacement and total energy dissipation.

A concept of earthquake resistant design based on energy consideration was first given by Housner[1]. Application of energy-based design to steel structures was done by Akiyama[2]. Fajfar[3] defines a damage parameter which relates total dissipated energy to displacement ductility factor. However, little research has been completed on the energy approach for seismic design of RC structures.

In this study, damage assessment of RC structures is carried out based on energy response. The damaging potential of earthquakes to structures is estimated by total input energy, which is considered to depend primarily on earthquake property. It is considered that earthquake input energy can be used as the basic value in the earthquake resistant design of RC frames.

Generally, damage of structures is estimated by indices based on maximum response, for example, ductility factor or rigidity degrading ratio. However, it is considered that both the maximum displacement and the dissipated hysteretic energy have to be considered in evaluating earthquake damage more appropriately.
The relationship of total dissipated energy to ductility factor is investigated for RC structures in this study. Based on the results of parametric inelastic earthquake response analyses, a design concept considering both displacement and energy limits is proposed.

ENERGY RESPONSE

Models for Analyses

Single degree of freedom inelastic models having various initial periods are analyzed for studying the damage characteristics of RC structures. As for the force-displacement relation, modified Takeda model[4] (degrading trilinear type) in Figure 1 was used. The rigidity degrading ratio at yield point is 0.3 as shown in equation (1).

\[ K_y = 0.3K_0 \]  

where \( K_0 \) is initial stiffness of the system, \( K_y \) is yield point stiffness.

Initial period \( T_0 \) and yield point period \( T_y \) are given in equation (2) and (3), respectively.

\[ T_0 = \frac{2\pi}{\sqrt{K_0/m}} \]  \hspace{1cm} (2)

\[ T_y = \frac{2\pi}{\sqrt{K_y/m}} = \frac{T_0}{\sqrt{0.3}} \approx 1.83T_0 \]  \hspace{1cm} (3)

where \( m \) is the mass of the system. In the elastic range, the response behavior is governed by the initial period. But near or over the yield point, the response is considered to be influenced by the yield point period. In this study, the yield point period is considered as an important parameter as well as the initial period.

Input Ground Motions

Simulated earthquakes are used as input motions instead of observed ground motions commonly used. Phase spectra of simu-
lated ground motions are determined by considering initial random values for phase, and envelope function. Five simulated accelerograms denoted by wave 1 to 5, are generated by inverse Fourier transform, which have response spectra corresponding to level 1 and level 2 design response spectra as mentioned in Japanese guideline[5], respectively. Amplitude spectra were modified iteratively to adapt to the target response spectrum of each level. Envelope function for time history is given in equation(4)[6].

\[ g(t) = 2.32(e^{-0.00t} - e^{-1.49t}) \]  (4)

The acceleration time histories of simulated input motions are shown in Figure 2, and the maximum values are given in Table 1. Target spectra and mean velocity response spectra of five input ground motions are shown in Figure 3. Coefficient of variation is about 5 ~ 10 percent.

Energy Response

Damaging potential of earthquakes to structures is evaluated by input energy \( E_i \) given by equation(5), and damage of structures by energy dissipation is evaluated by \( E_{II} \) given by equation(6).

\[ E_i = \int_0^T (-m\ddot{x}_0)\dot{x} \, dt = \frac{1}{2}mV_i^2 \]  (5)

\[ E_{II} = \int_0^T F(x)\dot{x} \, dt = \frac{1}{2}mV_{II}^2 \]  (6)

where \( F(x) \) is restoring force of the system, \( x \) is relative displacement and \( \dot{x} \) is relative velocity to the ground, \( \ddot{x}_0 \) is ground acceleration and \( T \) is duration time. In many cases, these two energy values, \( E_i \) and \( E_{II} \) are presented by equivalent velocity \( V_i \) and \( V_{II} \), respectively, as shown in equation(5) and (6).

In the following, the result of inelastic earthquake response of single degree of freedom system for the level 2 ground motions is shown. All response spectra are presented in the form of mean spectra of five input motions. Inelastic responses are obtained for specified level of ductility factors given by equation(7).

\[ \mu = \frac{\delta}{\delta_y} \]  (7)

where \( \delta \) is maximum displacement, \( \delta_y \) is yield displacement. It should be noted that ductility factor \( \mu \) is defined using yield displacement, and plastic deformation is observed for \( \mu > 1 \).
Elastic velocity response spectrum $S_V$ and elastic input energy spectrum $V_I$ for various damping ratios are shown in Figure 4 and Figure 5, respectively. $S_V$ and $V_I$ are very similar in the case of no damping. While $S_V$ becomes smaller by the increase of damping, $V_I$ doesn’t vary so much with damping, but only becomes smooth with the increase of damping. Akiyama[2] used elastic input energy spectrum $V_I$ with 10 percent damping ratio as the standard spectrum for the earthquake resistant design of structures.

Inelastic energy response spectra for various ductility factors $\mu$ are shown in Figure 6. Input energy spectra are shown in Figure 6.(a), and dissipated hysteretic energy spectra are shown in Figure 6.(b). Viscous damping ratio $h$ of 0.05 proportional to instantaneous stiffness is assumed. These spectra are computed for constant target ductility factor $\mu$, and are presented with initial period in horizontal axis. Input energy $V_I$ seems to be independent of ductility factor $\mu$. When we change the horizontal axis of Figure 6.(a) from initial period $T_o$ to corresponding yield point period $T_y$, it is seen that elastic input energy spectrum in Figure 5 and inelastic input energy spectrum in Figure 6.(a) are very similar.

The smaller the ductility factor $\mu$ is, the smaller the dissipated hysteretic energy $V_{hh}$ becomes. In this study, trilinear type force-displacement relation is used and cracking point is considered, so it should be noted that $\mu=1.0$ doesn’t mean elastic behavior.

The ratio of $V_{hh}$ to $V_I$ shown in Figure 7.(a) differs according to ductility factor $\mu$, but is largely unaffected by period. The average ratio of $V_{hh}$ to $V_I$ for all range of period is shown by solid circles in Figure 7.(b), where horizontal axis is ductility factor $\mu$. The range of ratio $V_{hh}/V_I$ for various periods is also shown by vertical lines. In the range of $\mu \geq 3$, the ratio $V_{hh}/V_I$ is considered to be constant value of about 0.8. However, in the range of small ductility factor $\mu$, the ratio $V_{hh}/V_I$ seems to vary with period.

The dashed line in Figure 7.(b) represents an approximate relation for the average value given by equation(8). Generally, the ratio $V_{hh}/V_I$ is considered to be a function of damping ratio $h$ and ductility factor $\mu$. But in this paper, damping ratio $h$ is fixed as 0.05, and the influence of various damping ratios is under study.

$$\frac{V_{hh}}{V_I} = 0.87 - \frac{0.6}{(\mu + 0.1)^2} \quad (8)$$
Fajfar[3] studied equivalent single degree of freedom systems of buildings, and proposed damage parameter $\gamma$ in equation (9). $F_y$ is yield force. He states that by using approximate value of damage parameter $\gamma$, it is possible to determine response ductility factor taking into account low-cycle fatigue, and to apply it in earthquake resistant design procedures.

$$\gamma^2 \mu^2 = \frac{E_H}{F_y \delta_y}$$  \hspace{1cm} (9)

In this study energy factor $\mu_e$ defined by equation (10) was used. The energy factor $\mu_e$ is dissipated hysteretic energy $E_H$ normalized by unit energy $F_y \delta_y$, which is considered to be an important parameter in taking account of cumulative damage. By using this energy factor $\mu_e$, equation (9) is rewritten in the form of equation (11). It is considered that maximum response or ductility factor $\mu$, and cumulative damage or energy factor $\mu_e$, are related by the damage parameter $\gamma$, which may be called a response pattern parameter.

$$\mu_e = \frac{E_H}{F_y \delta_y}$$ \hspace{1cm} (10)

$$\gamma^2 \mu^2 = \mu_e$$ \hspace{1cm} (11)

Energy factor spectrum $\mu_e$ is shown in Figure 8. It is considered that because of many numbers of hysteretic cycle, dissipated hysteretic energy $E_H$ becomes larger in the short period range.

Damage parameter spectra $\gamma$ for various values of ductility factor $\mu$ are shown in Figure 9. For $\mu \geq 1.4$, damage parameter $\gamma$, the ratio of square root of energy factor $\mu_e$ to ductility factor $\mu$, seems to be independent of ductility factor $\mu$. In the long period range, the damage parameter $\gamma$ is almost constant value of about 0.7. However in the short period range, it seems that damage parameter $\gamma$ is larger due to the response pattern having many numbers of hysteretic cycles.

From the observation of various response spectra, we notice that spectra have different patterns according to the period range. The boundary period separating the short period range and the long period range seems to be related to the peak period of elastic velocity response spectrum (about 0.6 sec) in Figure 3. In case of
inelastic energy spectra in Figure 6, \( V_{ij} \) and \( V_{III} \), the initial period corresponding to the peak of spectra is about 0.3 sec, which corresponds to the yield point period of 0.6 sec. However in case of energy factor \( \mu_e \) and damage parameter \( \gamma \), the initial period separating the two ranges having different tendency is about 0.6 sec. More study has to be done on the relationship between elastic and inelastic spectra for RC structures.

**INELASTIC DESIGN**

As an index for evaluating damage level quantitatively, damage index \( D \) proposed by Park et.\[7\] is used. Because the damage of structures is considered to be evaluated both by maximum response and by cumulative damage, damage index \( D \) is defined by the following equation.

\[
D = \frac{\delta}{\delta_u} + \beta \frac{E_{III}}{F_y \delta_u} = \frac{\mu}{\mu_u} + \beta \frac{\gamma^2 \mu^2}{\mu_u \mu_u} = \frac{\mu}{\mu_u} + \beta \frac{\mu_c}{\mu_u} \tag{12}
\]

where \( \delta \) and \( \mu_u \) are ultimate displacement and ultimate ductility factor under monotonic loading, respectively. The factor \( \mu_u \) must be defined based on actual loading test, and it is difficult to determine a general value. However in this paper, a tentative value of \( \mu_u \) is assumed. The coefficient \( \beta \) represents the contributing ratio of energy factor \( \mu_e \) to the damage of structures, and based on actual loading tests approximate values of \( \beta \) are proposed\[7\]. A structure with \( D = 0 \) is considered to cause no damage, and \( D = 1 \) is considered to be a limit state. Park et.\[8\] investigated earthquake damage caused in structures using this parameter, and concluded that a structure with \( D \leq 0.4 \) is considered to be repairable.

For the earthquake resistant design of structures, it is necessary to define design limit and to find yield force \( F_y \) corresponding to the design limit. In the following, two procedures for finding yield force \( F_y \) are shown when a value of damage index \( D \) is given.

**Procedure 1 ;** Trial and error method by response analyses

**Procedure 2 ;** Estimation using given input energy and modeled damage parameter

It is assumed that ultimate ductility factor under monotonic loading is \( \mu_u = 4.0 \), and the coefficient \( \beta = 0.15 \)[3] in equation(12).
In procedure 1, design yield force $F_y$ for a given value of damage index $D$ is found numerically through parametric inelastic response analyses for the two sets of simulated input motions corresponding to level 1 and level 2 target response spectra. Response results are given by mean of five response values from the five input motions, respectively.

In procedure 2, calculation step for determining $F_y$ is as follows.

Step 1: Give input energy spectrum $V_I$:

Inelastic input energy spectra in Figure 6.(a) with yield point period in horizontal axis and elastic input energy spectra in Figure 5 are very similar. In this study, elastic input energy spectrum for 10 percent damping plotted with the abscissa scaled by the factor of $\sqrt{0.3}$ is assumed to represent the model for the inelastic input energy spectrum plotted for the initial period. Inelastic input energy spectrum of level 1 and level 2 ground motions used in the following calculation by procedure 2 are shown in Figure 10.

Step 2: Give values of damage index $D$:

For the values of damage index, $D=0.4$ (repairable damage) and $D=1.0$ (limit state) are given as an example.

Step 3: Obtain corresponding ductility factor $\mu$ for a given value of damage index $D$:

The model of damage parameter $\gamma$ is assumed as shown in Figure 11. Corresponding ductility factor $\mu$ is calculated by equation (13) using equations (11) and (12). Estimated values are shown in Figure 12.

$$\mu = \frac{\sqrt{1 + 4D\beta\gamma^2\mu_n} - 1}{2\beta\gamma^2} \quad (13)$$

Step 4: Estimate dissipated hysteretic energy $V_H$ from input energy $V_I$:

By the proposed relation of the ratio $V_H/V_I$ (function of ductility factor $\mu$) given by equation (8), corresponding dissipated hysteretic energy $V_H$ is estimated. Estimated values are shown in Figure 13.

Step 5: Obtain yield force $F_y$ corresponding to ductility factor $\mu$:
Using the relation of ductility factor $\mu$ and energy factor $\mu_c$, yield force can be calculated. The equation (11) can be rewritten as follows.

$$\gamma^2 \mu^2 = \mu_c = \frac{E_{II}}{F_y \delta_y} = \frac{\frac{1}{2} \frac{m V_{II}^2}{F_{y}^2}}{\frac{(\frac{2\pi}{\gamma})^2}{m^2 (\frac{2\pi}{\gamma})^2}}$$

(11')

By equation (11'), yield force $F_y$ can be obtained as follows.

$$\frac{F_y}{mg} = \frac{1}{mg} \sqrt{\frac{\frac{1}{2} m^2 V_{II}^2 (\frac{2\pi}{\gamma})^2}{\gamma^2 \mu^2}} = \frac{\sqrt{2\pi V_{II}}}{\gamma \mu g T_y}$$

(14)

where $g$ is the gravity acceleration. Estimated yield force spectrum $F_y$ is shown in Figure 14 in terms of base shear coefficient $F_y/mg$. Estimated yield force $F_y$ by the procedure 2 seems to be very similar to the obtained $F_y$ by the procedure 1.

**CONCLUSIONS**

In this study, earthquake response behavior of structures is investigated from the viewpoint of energy response, and damage of structures is estimated both by maximum displacement and by total energy dissipation. From results of parametric inelastic response analyses, it is understood that input energy is dependent primarily on earthquake property and that it is possible to use it in the earthquake resistant design of structures. Damage parameter that relates ductility factor to energy factor and that represents earthquake response pattern of structures, is shown to be relatively stable, except for short period and small ductility factor.

By using proposed design procedure using given input energy, damage parameter and damage index representing damage state of structures, the estimated yield force corresponding to design limit is considered to be very similar to the obtained yield force...
by earthquake response analyses. The results obtained from this study show a possibility that a design concept considering displacement and energy limits can be applied to the earthquake resistant design of RC buildings.

REFERENCES


### TABLE 1 — MAXIMUM VALUES OF INPUT GROUND MOTIONS

<table>
<thead>
<tr>
<th></th>
<th>$A_{\text{max}}$ (cm/s²)</th>
<th>$V_{\text{max}}$ (cm/s)</th>
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<tr>
<td><strong>Level 1</strong></td>
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<td></td>
</tr>
<tr>
<td>wave1</td>
<td>268.8</td>
<td>28.3</td>
</tr>
<tr>
<td>wave2</td>
<td>262.3</td>
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</tr>
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<td>249.2</td>
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<td>288.9</td>
<td>22.9</td>
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<td>wave5</td>
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<td><strong>Level 2</strong></td>
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<td></td>
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<td>wave4</td>
<td>451.2</td>
<td>43.2</td>
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<tr>
<td>wave5</td>
<td>418.7</td>
<td>43.4</td>
</tr>
</tbody>
</table>
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(a) Hysteresis loop

\[
\begin{align*}
\mu &= \frac{\delta}{\delta_y} \\
K_y &= 0.3K_0 \\
T_y &= T_0/\sqrt{0.3} \approx 1.83T_0
\end{align*}
\]

(b) Initial period and yield point period

- \(K_0\) ; initial stiffness
- \(K_y\) ; yield point stiffness
- \(F_y\) ; yield force
- \(\delta_y\) ; yield displacement
- \(\delta\) ; maximum displacement
- \(\mu\) ; ductility factor
- \(T_0\) ; initial period
- \(T_y\) ; yield point period

Fig. 1—Model of force-displacement relation
Fig. 2—Input ground motions

- Panel a: Level 1 ground motions
- Panel b: Level 2 ground motions

Graphs show acceleration (cm/s²) vs. time (s) for waves 1 to 5.
Fig. 3—Response and target velocity spectra
Fig. 4—Elastic velocity response for various damping ratios

Fig. 5—Elastic input energy for various damping ratios
Fig. 6—Energy response for various ductility factors

(a) Input energy

(b) Dissipated hysteretic energy
Fig. 7—Ratio of dissipated hysteretic energy to input energy.
Fig. 8—Energy factor

Fig. 9—Damage parameter
Fig. 10—Input energy

Fig. 11—Damage parameter
Fig. 12—Ductility factor

Fig. 13—Dissipated hysteretic energy
Fig. 14—Yield force
Implications of the Choice of Structural System for Earthquake Resistant Design of Buildings

by L. E. Garcia and J. F. Bonacci

Synopsis: An evaluation of the implications in the structural system selected for reinforced concrete buildings with three different plan layout and four different heights -- 5, 10, 15 and 20 stories -- was performed as part of the calibration of the update of the Colombian Seismic Code (10). The buildings had varying amount of structural walls. In total 72 buildings were studied. Expected performance of the buildings under the Code design earthquake was evaluated using elastic and inelastic procedures. Using the amount of concrete and reinforcing steel for all the buildings, and prevalent material and labor prices, a cost of the structure per unit area was determined. Conclusions with respect to behavior and cost implications were obtained for the parameters studied for the different buildings.

Keywords: Earthquake-resistant structures; inelastic response; reinforcing steel; structural systems; walls
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John F. Bonacci is Associate Professor of Civil Engineering at the University of Toronto, Toronto, Canada. He received B. Sc., M. Sc. and Ph. D. degrees at the University of Illinois, Urbana-Champaign. He is a member of ACI committees 352, Monolithic R/C Joints, 368, Earthquake Resistant Members/Systems and 408, Bond, and is an associate member of Committee 442, Response of R/C Buildings to Lateral Loads. He has taught and conducted research at the University of Toronto in reinforced concrete and earthquake engineering.

INTRODUCTION

The choice of the structural system of a building depends on several variables, some of them controlled by the structural engineer and some beyond his control. One aspect that plays a very important role is local usage and tradition. With the upgrade and updating of the earthquake resistant design regulations, brought by awareness of a greater seismic risk or by the occurrence of damaging earthquakes of intensity beyond what was generally accepted, the type of structural systems permitted by the Codes comes under close scrutiny. As a result of this process some systems are penalized with more strict requirements or simply deleted from the list of the accepted systems. Several examples of this type of process having taken place can be found after the 1967 Caracas, Venezuela, Earthquake, the 1971 San Fernando Earthquake, the 1983 Popayán, Colombia, Earthquake, the 1985 Chile Earthquake and the 1985 México Earthquake. The expected changes in requirements for steel moment frame construction after the Northridge Earthquake of January 17, 1994, in southern California is just another example.

Recent studies (6, 8, 13) for the upgrading of the earthquake resistant design requirements of the Colombian Code (10) with respect to the structural systems permitted for reinforced concrete buildings, have shown that not all the structural systems permitted for reinforced concrete
buildings have the same behavior expectancy and that the choice of structural system has important cost implications. The preference of the Colombian engineers for moment resistant unbraced frames, as opposed to systems including walls, as confirmed by a recent survey (9) made for ACI Committee 368, made the results of the study more important given the world wide tendency to a greater usage of walls (2).

THE SCENARIO

Colombian Building Practice

Colombia is located in one of the most active seismic regions of the world. All large cities in the country have seismically active faults nearby, or are close to the Nazca Plate subduction zone. Therefore, earthquake resistant design is required in all construction. Wind plays a secondary role with respect to design concerns, especially in the interior part of the Country. The possibility of having damaging winds is present only in the Caribbean coast.

The main structural construction materials in Colombia are reinforced concrete and masonry. Structural steel is used in large span roofs and in bridges. Its use in the framing of buildings is limited, although it is becoming more popular. For reinforced concrete buildings, the moment resisting frame has been the traditional system and only recently structural walls have appeared as an alternative in low-rise and high-rise buildings. Masonry is used for single family dwellings, and low-rise and medium-rise apartment buildings. Adobe construction is used in some rural areas but its use in Colombia is decreasing rapidly.

Colombian Building Code

The need of a modern mandatory building code was made evident by the 1983 Popayán Earthquake. The Colombian seismic code was enacted in June of 1984 (7,10). The Code tried to take care of the problems brought out by earthquakes that occurred on November 23 of 1979 ($M_s=6.4$), December 12 of 1979 ($M_s=7.8$) and March 31 of 1983 ($m_b=5.5$). These earthquakes emphasized a series of deficiencies in the Colombian building practice, of which one of the most important was excessive flexibility of the structural systems used (7). The reinforced concrete
moment resisting frame was, and still is, one the most widely used structural systems (9). The absence of code, and the lack of awareness of the problems associated with the excessive flexibility of frames responding to lateral loads, was the culprit of much of the bad behavior encountered. Because of these reasons the 1984 Colombian code made strict story drift control one of its main objectives. The results were outstanding, a recent survey among practicing structural engineers, assigned story drift as the controlling design parameter, as opposed to base shear, when determining dimensions for a building structure.

A recent earthquake, October of 1992 ($M_s=7.9$), produced great amounts of nonstructural element damage in Medellin, located more than 120 km from the epicenter. This event increased the concern that the current story drift requirements might be insufficient and that stricter requirements are needed. The update of the Code contains story drift requirements that are more conservative than those contained in the 1984 version of the Code.

**Seismic Design Requirements**

The seismic design requirements of the Colombian Code were inspired by the ATC-3-06 (1,7). The Code covers buildings located in low seismic risk zones up to high seismic risk, and the material requirements are inspired by North American Codes. The country is divided into three seismic risk regions. Depending on this zoning the code regulates the structural systems allowed, the height limits for the structural systems, the type of occupancy and the requirements for the different materials. These requirements are more restrictive for the higher risk zones. The reinforced concrete requirements follow ACI 318, except for moderate seismic risk zones, where the requirements are more strict than those of Section 21.8 of ACI 318-89 (Revised 1992).

The 1984 Code followed the ATC-3 procedure for story drift evaluation: the structure is analyzed using lateral forces divided by a response modification factor, $R$, and the structure lateral displacements from the analysis are amplified by a displacement amplification factor, $C_d$, to obtain the values that are used to verify the story drift. The story drift limit was 1.5% of the story height. The update of the Code requires that the analysis of the structure be performed using lateral forces without being divided by $R$, and that drift be evaluated using the displacements obtained from these forces. The story drift limit in the new Code is 1.0%$h$. 
CHARACTERISTICS OF THE BUILDINGS STUDIED

Building Type and Loads

The buildings studied are apartment buildings located in a moderate seismic risk zone, as defined by the Colombian Code, with an effective peak ground acceleration ($A_g$) of 0.15 and an effective peak velocity in terms of acceleration ($A_v$) of 0.20, which correspond to the two largest cities in the country: Bogotá and Medellín. The underlying soil type was defined as $S_1$, with a value of the site factor, $S$, of 1.0. Live load was taken as 180 kg/m$^2$, as required by the Code for apartment buildings, and given that heavy masonry partitions are used routinely, the value of partitions load was taken as 250 kg/m$^2$. Additional miscellaneous load for finish, mechanical and electrical, etc. was taken as 150 kg/m$^2$.

Structural Layouts

Three structural layouts in plan -- A, B and C -- were selected. All of them give the same area per floor of 720 m$^2$. The floor system consists of reinforced concrete joists which are supported on the beams in the E-W direction. Both beams and joists have the same structural depth. These beams have spans of 5 m for layout type A (Figure 1), 7.5 m for layout type B (Figure 2), and 10 m for layout type C (Figure 3). The beams in the N-S direction, parallel to the joists, have spans of 6, 8 and 12 meters for layouts type A, B and C, respectively. The span for the joists is the same as for the N-S beams.

In layout type A square section columns were used. In layouts type B and C the columns had a section with a height equal to two times the width. In all cases the column section was kept uniform throughout the height of the building. Actual column dimensions varied for the different buildings as a result of the dimensioning for each individual case.

Beam width was 40 cm for both layout type A and B, and 55 cm for layout type C. Beam depth was 45 cm for layout type A, 55 cm for layout type B, and 65 cm for layout type C.

A story height of 3.20 meters was used for all the buildings. Four building heights were employed corresponding to 5, 10, 15 and 20 stories.
Structural Wall Area

As shown in Figure 4, the parameter \( p \) for a given story is the ratio between the total section area of all the walls that act in a given direction, divided by the floor area of the story (12). Six values of \( p \) were used: 0.0%, 0.3%, 0.6%, 1.0%, 1.5% and 2.0%. These values were obtained changing the thickness of the walls. The walls had a fixed length for each of the structural layout types, as shown in Figures 1 to 3. This way the slenderness ratio of the walls was constant for each plan layout type and each building height.

In Figure 5 the buildings used are defined. For each group of wall area \( p \), six in total; three plan layout types -- A, B and C -- are used; and for each combination of \( p \) and plan layout, four building heights -- 5, 10, 15 and 20 stories -- are employed, giving a total of 72 buildings studied.

RESULTS OBTAINED FROM THE DESIGNS

Design Procedure

Buildings for this study were designed following procedures that were much like those applied in a real project. The main steps were as follows:

- Using the loads described before, the beams and joists of the floor system were designed for gravity load only. Two cases were studied for each plan layout: joists in the E-W direction, and joists in the N-S direction. For each of these cases several beams and joists (they have the same depth) were tested. From these results, dimensions for the beams and joists were determined. It is important to note that in all three plan layouts the case that required least materials was the one with the joist spanning in the N-S direction. This is due to the fact that as both beams and joists have the same structural depth, then it is more economical to have the joists in the direction of the longest span.
- Having the dimensions of the beams and joists it was possible to establish the gravity loads for the full structure and thus obtain trial dimensions for the columns. The lateral seismic design forces were obtained using the Code equivalent lateral load procedure.
- Once all dimensions of the elements were determined the analysis of the structure was performed using the computer program COMBAT (5).
• Compliance with the Code drift requirements was checked, and in those cases in which larger than allowable drifts were obtained, the dimensions of the columns were modified until full compliance was obtained. Drift was critical only for the buildings that did not have walls, \( p=0.0\% \).
• Once drift compliance was assured, the design of all the elements was performed. The design included the joist system (same for each plan layout), beams, columns and walls.
• Material listings were obtained for the designed elements.

### Base Shear and Fundamental Period

The Code design base shear, \( V_s \), expressed as a seismic coefficient, \( C_s=V_s/W \), where \( W \) is the total building weight, is shown for all the buildings studied in Figure 7. The value only varies with the height of the building, because the response modification factor, \( R \), is the same for all buildings. \( C_s \) ranges from 0.088 for the 5 story buildings to 0.055 for the 20 story buildings.

Although the equivalent lateral load design procedure was employed, periods and modes of vibration were obtained using a dynamic analysis procedure. The values of the fundamental period of vibration, obtained for each of the principal directions in plan, are shown in Figure 8.

### Story Drift

The elastic story drifts, expressed as a percentage of the story height, obtained during the design procedure as required by the code, are reported in Figure 9 for the X (E-W) direction. Those for the Y (N-S) direction are of the same order of magnitude. For the buildings without walls \( (p=0.0\%) \), the values are close to the maximum allowable of 1.5%\( h \) prescribed by the current code. For the buildings with walls the story drift is always less than 0.5%\( h \).
Quantities of Concrete and Reinforcement

Concrete -- The amount of concrete corresponding to each of the structural elements is reported in Figure 10. The total amount of concrete for each building, and that corresponding to beams and joists, and columns and walls, are shown in Figure 11. The total amount of concrete varies from a minimum of $0.19 \text{ m}^3$ of concrete per square meter of area to a maximum of $0.43 \text{ m}^3/\text{m}^2$. It is important to notice that the concrete corresponding to the columns is the main reason behind the variations in the total amount of concrete.

Reinforcement -- The amount of reinforcing steel corresponding to the different types of reinforcement required in each class of element is reported in Figure 12. The total amount of reinforcing steel for each building, and that corresponding to beams and joists, and columns and walls are shown in Figure 13. The total amount of reinforcing steel varies from a minimum of 18 kg of steel per square meter of area to a maximum of 65 kg/m$^2$. Here again, the steel corresponding to the columns is the main reason behind the variations in the total amount of reinforcing steel.

EXPECTED PERFORMANCE

Parameters for Performance

The expected behavior of the buildings for the seismic design event, was determined using story drift as the qualifying parameter. The story drift was estimated using elastic and inelastic analytical schemes. The elastic procedure was the same prescribed by the Code, as reported before.

The inelastic procedure employed was the one described by Shimazaki and Sozen (11), as employed in several instances (3, 13). The following steps were performed:

- The building period and mode shapes were determined from a linear dynamic analysis using gross uncracked sections of the elements.
- The linear elastic base shear was obtained using a response spectrum corresponding to the design ground motions, as the square root of the sum of the squares of the base shear, of a suitable number of modes for each principal direction. The spectrum employed was not reduced by the response modification factor, $R$. 
• The inelastic story drift of the building in each principal plan direction, X and Y, was calculated using the fundamental mode shape, the fundamental building period amplified by $\sqrt{2}$, and the response spectrum corresponding to the design ground motions.

• The base shear strength of the buildings was calculated for each principal direction. The collapse mechanism leading to the minimum base shear strength was employed. All possible cases of three types of mechanisms were evaluated, as shown in Figure 6.

• A strength ratio was established as the ratio between the linear elastic base shear and the base shear strength.

• A period ratio, was determined as the ratio between the fundamental building period, amplified by $\sqrt{2}$, and the characteristic period of the ground motion, 0.6 sec in this case because the buildings are located on stiff ground (11).

• Period and strength ratios are added for the purpose of checking the displacement ratio. According to Shimazaki (11), the displacement ratio (ratio of expected inelastic displacements to elastic spectral estimate) will exceed one only if the sum of period and strength ratios is less than one. A displacement ratio of less than one means the elastic estimate provides a valid upper bound to the inelastic response.

**Base Shear Strength**

The base shear strength in both principal directions for all the buildings studied are reported in Figure 7. It is important to notice that for the buildings with no walls or with a small amount of walls, p=0.3% and p=0.6%, the values for the base shear strength are of the same order of magnitude of the values of the code base shear, which are prescribed at the strength level, as mentioned before.

**Shimazaki Relationship**

For each building, using the base shear strength value obtained, as reported in Figure 7, and the fundamental period value given in Figure 8, suitable affected by $\sqrt{2}$, the values of strength and period ratios were determined and then added together for purposes of checking the displacement ratio according to Shimazaki's approach. The period and strength ratios are plotted in Figure 14, from which it is apparent that only 2 of the 72 buildings will have inelastic response that exceeds the elastic
estimate (displacement ratios greater than one). They are the five story buildings with $p=0.3\%$, and plan layout types A and B. In both cases the estimated non linear story drift is $0.1\%h$, well under the code limit.

COST IMPLICATIONS

Cost Index

In order to determine a cost for the structure of the studied buildings, a cost per cubic meter of concrete was determined for the designed elements, including the material in itself, formwork, equipment and labor cost for casting the element. For reinforcement the cost charged by the fabricator and the labor cost of placing the reinforcement were included. The total construction cost of the structure was thus evaluated, and later divided by the number of square meters of area of the structure to obtain a cost per square meter. The minimum value, corresponding to the five story building of layout type A with $p=0.0\%$, was used as a basis. A Cost Index was determined for each building dividing the cost per square meter by the minimum value.

Figure 15 reports the Cost Index for all the buildings studied. In this graph the continuous line follows the trend of the same type of structural solution, as the number of stories increases. The broken line corresponds to the same number of stories, for different layouts and wall percentages.

Cost vs. Height

In Figures 16 to 18 the variation of the cost of the structure, per unit area, with the building height is shown for plan layouts A, B and C respectively. The variation is plotted for cases with the same amount of wall area, $p$. The cost of the structure increases with the number of stories of the building, although the rate of increase is not the same for all cases studied, as reflected by the crossing of some of the lines with others.
Cost Implications of the Use of Structural Walls

The use of structural walls as opposed to a framed structure is the sensible solution, in terms of cost, in a number of cases. The first crossing of the line of $p=0.0\%$ in Figures 16 to 18 indicates the building height from which it is more economical to use structural walls. This crossing is 15 stories for layout A, 7 stories for layout B and 12 stories for layout C. For layouts B and C, less wall area does not always mean lower cost, as shown by the crossing of solutions with different wall area percentages.

CONCLUDING REMARKS

The results obtained indicate that the proper choice of structural system not only gives better expected behavior, but in a number of cases leads to more economic solutions. They also confirm that great care should be exerted by the architect, and the structural engineer, in selecting appropriate beam spans and, in general, the plan layout of the structure, not withstanding the height of the building.

Although the performance gauge for the designs in this study was the story drift (as well as the possibility of being wrong when estimating it using routine elastic analysis procedures), it must be pointed out that, for the range of story drifts obtained within the acceptable limit (Figure 9), a considerable variation in performance of structural and nonstructural elements can still be expected between the various framing systems.

REFERENCES


Fig. 1—Layout Type A

Fig. 2—Layout Type B

Fig. 3—Layout Type C
Fig. 4—Definition of wall percentage

\[
p = \frac{\text{Area of the horizontal section of the walls}}{\text{Area of the floor}}
\]

Fig. 5—Definition of the buildings

Fig. 6—Collapse mechanisms for calculation of base shear strength
Fig. 7—Base shear

Fig. 8—Fundamental period
Figure 9—Story drift (percent h)

Figure 10—Concrete use by elements
Fig. 11—Total concrete use

Fig. 12—Steel reinforcement use by element
Fig. 13—Total reinforcing steel use

Fig. 14—Shimazaki relationship
Fig. 15—Cost index

Fig. 16—Cost versus height — Layout Type A
Fig. 17—Cost versus height — Layout Type B

Fig. 18—Cost versus height — Layout Type C
Cyclic Response of Reinforced Concrete Structural Walls

by S. L. Wood and C. Sittipunt

Synopsis: A conceptual model for the behavior of structural walls subjected to lateral load reversals is presented. The primary feature of the model is a reduction in shear strength with increasing levels of deformation. Measured and calculated data from structural walls are evaluated to determine conditions for which the strength and deformation capacity of a wall may be limited by the residual shear strength.

Keywords: Cyclic loads; degradation; flexural strength; hysteresis; nonlinear finite element analysis; reinforced concretes; shear strength; strength; structural walls
**INTRODUCTION**

Structural walls frequently form the primary lateral-load resisting elements in reinforced concrete buildings. Experience during the 1985 Chile earthquake [1] indicated that deformations and structural damage are controlled in buildings with a large number of walls, due to the inherent stiffness of the walls. Far fewer walls are typically used in reinforced concrete buildings in the U.S., however. As a result, the seismic response of many reinforced concrete buildings is directly related to the inelastic behavior of the structural walls.

This paper focuses on the inelastic behavior of isolated structural walls subjected to lateral load reversals. Emphasis is placed on interpreting the consequences of shear failures with respect to the strength and deformation capacity of the walls. Experimental data are presented to highlight important aspects of behavior, and an analytical model is used to study the influence of additional parameters on the cyclic response of structural walls.

**EXPERIMENTAL DATA**

Tests conducted at the Portland Cement Association in the 1970s [2,3,4] represent the most extensive experimental investigation of wall behavior in the U.S. to date. Nineteen isolated, slender walls were tested. All walls were 15-ft. tall and fixed at the base (Fig. 1). Experimental parameters included the cross-sectional shape, applied axial stress, loading history, and amounts of longitudinal and web reinforcement (Table 1).

Lateral loads were applied in the plane of the wall at mid-depth of a stiff girder located at the top of the walls. Most walls were subjected to a series of load reversals in which the specimen was pushed to a specified displacement level three times, and then the amplitude of the displacement was increased (Fig. 1(c)). Five walls (R3, B9, B10, B11, and B12) were subjected to loading cycles in which the amplitude of the...
displacement cycles alternated above and below the measured yield displacement. Wall B4 was loaded monotonically to failure. Walls subjected to load reversals were pushed to approximately the same displacement levels in both directions.

Material properties for the walls are documented in Ref. 2, 3, and 4 and summarized in Ref. 5. Concrete compressive strengths ranged from 3300 to 7800 psi and values of measured yield stress in the reinforcing bars ranged from 62 to 78 ksi. Axial load, in addition to self weight, was applied to nine walls. The average axial stress did not exceed 0.15 $f'_d$ in any wall.

The measured base–shear strength of each wall is listed in Table 1. All walls experienced yielding of the longitudinal reinforcement in the boundary elements prior to failure. The tests were concluded after the lateral resistance of the walls had decreased to less than one–half the maximum resistance. Observed modes of failure are also indicated in Table 1. The displacement capacity of walls characterized as failing in flexure was typically limited by buckling or fracture of the longitudinal reinforcement in the boundary elements. Web crushing or shear–compression failure of the boundary elements typically limited the displacement capacity of walls characterized as failing in shear. All walls withstood a minimum of two complete cycles to displacement levels larger than of 1% of their height without an appreciable loss of lateral strength.

The nominal flexural and shear strengths of each wall, calculated using the procedures defined in ACI 318–89 [6], are summarized in Table 1. The base moment corresponding to the nominal flexural capacity of each wall, $M_n$, was calculated by assuming a linear strain distribution over the depth of the cross section and a peak compressive strain of 0.003 in the concrete. Measured values of maximum concrete compressive stress and steel yield stress were used in the calculations. Strain hardening of the reinforcement was ignored. Because a single lateral load was applied to the top of the wall, the nominal flexural capacity may be expressed in terms of a shear strength at the base of the wall, $V_{nf}$:

$$V_{nf} = \frac{M_n}{H_w}$$  \hspace{1cm} (1)

The nominal strength of the wall in shear, $V_{ns}$, was calculated using Eq. (2) from Chapter 21 of ACI 318–89 [6]:

$$V_{ns} = (2\sqrt{f'_c} + \varrho_n f_y) A_{cv}$$  \hspace{1cm} (2)

where $f'_c$ is expressed in psi and $\varrho_n$ is the horizontal web reinforcement ratio. The limiting value of the nominal shear strength was taken to be
The nominal base-shear strength of the wall, $V_n$, was then taken to be the minimum of the calculated flexural and shear strengths.

As indicated in Fig. 2(a), the calculation procedure defined by the current building code [6] provides a conservative estimate of the strength of all the walls. The ratios of measured strength to nominal capacity ranged from 1.04 to 1.45 for the nineteen walls considered.

Although the design procedures provide a conservative estimate of specimen strength, the calculations do not provide insight into the likely mode of failure of the walls. For example, one would expect a wall to fail in flexure if the nominal capacity in flexure is less than the nominal capacity in shear. Similarly, a shear failure would be expected if the nominal capacity in shear is less than the nominal capacity in flexure. The expected boundary between flexural and shear modes of failure is indicated by the broken line in Fig. 2(b). This idealization of likely failure modes is not consistent with the experimental data from the PCA tests. Walls in which the nominal capacity in flexure was less than sixty percent of the nominal capacity in shear were observed to fail in flexure when subjected to cyclic loading, while walls with a nominal flexural capacity greater than sixty percent of the nominal shear capacity were observed to fail in shear. [5].

Consequences of shear failures in structural walls, in addition to the possibility of sudden crushing of the concrete in the web of the wall, include pinching of the global force-displacement hysteresis curves and concentrations of shear distortions near the base. Measured data from walls B1 and B2 are shown in Fig. 3 and 4, respectively, and illustrate the differences in the measured response of walls that failed in flexure and those that failed in shear.

The lateral-load capacity of wall B1 was limited by fracture of the longitudinal reinforcement in the boundary elements, which first occurred during the first cycle to a displacement level of ±5 in. In spite of this damage, the wall was able to accommodate displacement cycles to ±6 in. before the residual strength of the wall dropped below one-half of the maximum base-shear strength. Wall B2 experienced web crushing, also during the first cycle to a displacement level of ±5 in. However, the residual strength of the wall immediately dropped to less than one-third of the maximum base-shear strength. Plots of average shear distortion in the lower 6 ft of the wall, indicate that distortion levels were nearly two times larger in wall B2 than wall B1 during the initial displacement cycle to ±5 in.

Measured data from tests conducted by Wolschlag [7] highlight the concern that the shear strength of structural walls may decrease when the
walls are pushed to repeated displacement levels beyond yield. Specimens were three stories in height and lateral loads were applied at the elevation of each floor slab (Fig. 5). Two walls were subjected to static load reversals. Measured data for wall W1 are shown in Fig. 6.

The base shear vs. first-story displacement hysteresis curve for wall W1 exhibits a severely pinched shape and degradation of strength and stiffness with increasing deformations. In an attempt to identify the source of the strength degradation, Wolschlag [7] separated the flexural component of deformation from the shear component. Hysteresis curves for the two components of distortion are shown in Fig. 7. The base shear is plotted as a function of the first-story shear distortion in Fig. 7(a) and as a function of the first-story flexural distortion in Fig. 7(b). The differences in the general shapes of the two curves are significant. The shear distortion constitutes more than 75% of the total displacement at the first-story level, and the shear hysteresis curve displays the same pinched shape exhibited in the global hysteresis curve (Fig. 6). In contrast, the flexural hysteresis curve is robust and displays only modest stiffness degradation with cycling.

CONCEPTUAL MODEL FOR CYCLIC SHEAR STRENGTH OF WALLS

Based on the observed performance of structural walls, Wolschlag [7] proposed the conceptual model for cyclic response shown in Fig. 8. Two backbone curves are shown in the figure. The solid line corresponds to a traditional representation of flexural response. Reductions in wall stiffness corresponding to cracking and yielding of the longitudinal reinforcement may be observed. After yielding, the strength of the wall continues to increase, albeit more gradually, due to strain hardening of the reinforcement. The deformation capacity of the wall is limited by failure of the materials at displacement levels well beyond yield. A similar representation of wall response has been proposed by Wallace and Moehle [8, 9] as a means of evaluating the need for confined boundary elements in structural walls.

The broken line in Fig. 8 represents the degradation of shear strength with increasing displacements. At low deformation levels, the shear strength is assumed to be greater than the base shear corresponding to yielding of the longitudinal reinforcement. As the amplitudes of the deformations increase, the available shear strength is assumed to decrease. If the wall reaches the deformation level corresponding to the flexural capacity while the residual shear strength exceeds the nominal flexural capacity, the wall would be expected to fail in flexure. Conversely, if the wall is displaced to
a level at which residual shear strength falls below the nominal flexural capacity, the wall would be expected to fail in shear. This representation of reduced shear strength under cyclic loads is similar to that proposed by Wight and Sozen [10] for columns. Shear strength degradation is considered in current building codes [6] for the design of columns in seismic zones; however, the shear strength of structural walls is assumed to be independent of the loading.

Although algorithms are readily available to calculate the flexural strength and deformation capacity of walls [8, 9], little is known about the nature of the backbone curve for shear. The rate of shear strength degradation with cycling may depend on a number of parameters, including the amount of web reinforcement, the moment to shear ratio in the wall, the average shear stress, and the loading history. An analytical investigation was undertaken to complement the experimental data and explore the influence of key parameters on the cyclic shear strength of structural walls.

**DESCRIPTION OF ANALYTICAL MODELS**

A finite element model was developed [11] to study the response of structural walls subjected to lateral load reversals. The model was verified using measured response from 13 rectangular and barbell walls tested at the Portland Cement Association [2,3,4].

The significant features of the nonlinear material models used in the study are shown in Fig. 9 and 10. Two hysteresis models were developed for concrete: one representing cyclic response when subjected to normal stresses and one modelling cyclic response when subjected to shear stresses [11]. The normal stress model is capable of modelling the influence of compression softening of confined and unconfined concrete, crack closing and reopening, and tension stiffening. The shear stress model includes the influence of aggregate interlock and dowel action. Both models consider degradation of concrete strength with cycling. The material model for the reinforcing steel includes yielding, strain hardening, and Baushinger effects. Anchorage of the reinforcement and deterioration of bond strength under cyclic loading was not modeled in this investigation. Verification of the material models is documented in Ref. 11.

A representative finite element mesh used to analyze the PCA walls is shown in Fig. 11. The walls were modelled as planar systems. Concrete was modelled using four-node, isoparametric, plane-stress elements. Each concrete element is surrounded by four steel elements that were modelled using two-node, truss elements. The thickness of the concrete elements was varied to reflect changes in the cross-sectional geometry.
A row of stiff linear elements was placed at the top of the wall to distribute the applied lateral force. Displacements measured during the experiments were imposed at the upper corner of the analytical models. Nodes at the base of the wall were fixed against translation in the horizontal and vertical directions. Vertical forces were positioned along the top of the wall to represent the applied axial loads.

The reliability of the analytical models was evaluated by comparing three types of measured and calculated responses: global hysteretic response (applied load versus top displacement), local hysteretic response near the base of the wall (applied load versus average shear strain in the lower 6 ft. of the wall), and mode of failure. Comparisons of the calculated and measured global hysteretic response are shown in Fig. 12 and 13, for walls R1 and B7, respectively. Wall R1 failed in flexure after the longitudinal reinforcement in the boundary element fractured, and wall B7 experienced web crushing and failed in shear. The analytical model was successful in reproducing the key features of the measured hysteretic response and the observed modes of failure in both walls.

CALCULATED RESPONSE OF WALLS

Using these finite element models, a parametric study was performed to identify walls that are susceptible to shear failures when subjected to cyclic lateral loads. Three parameters were selected for investigation: the longitudinal reinforcement ratio within the boundary elements, $e_{br}$; the web reinforcement ratio, $e_n$; and the aspect ratio of the wall, $H_w/L_w$. A total of 32 different wall configurations were considered in the parametric study. All walls were assumed to have a barbell cross section and equal web reinforcement ratios in the horizontal and vertical directions. The walls were subjected to a loading history with increasing displacement amplitude (Fig. 14). The boundary elements of all walls were assumed to be confined, such that crushing of the concrete in the boundary element would not limit the strength or deformation capacity of any wall considered.

The ranges of parameters considered are summarized in Table 2. The slender walls analyzed in the parametric study had the same dimensions as the barbell walls tested at PCA. By varying the longitudinal reinforcement ratio in the boundary elements and the web reinforcement ratio, a wide range of ratios of nominal flexural capacity to nominal shear capacity could be considered.

The calculated modes of failure in the slender walls were nearly equally divided between shear and flexural failures, while all the short walls were
calculated to fail in shear. Representative hysteresis plots for slender walls with 0.25 and 0.5% web reinforcement are shown in Fig. 15 and 16, respectively. Flexural failures were calculated in all slender walls with 1% longitudinal reinforcement. The remaining three slender walls with 0.25% web reinforcement (Fig. 15) were calculated to fail in shear. Both the strength and stiffness of these walls degraded with increasing lateral deformations. Strength degradation was not apparent in the calculated response of any of the slender walls with 0.5% web reinforcement (Fig. 16). Web crushing, leading to shear failure, was calculated in the walls with 3 and 4% longitudinal reinforcement and 0.5% web reinforcement. As indicated in Fig. 15 and 16, the displacement capacity of the slender walls was independent of the calculated mode of failure. Walls that were calculated to fail in shear were able to resist the same deformation levels as walls that were calculated to fail in flexure. This observation is consistent with the measured response of walls tested at PCA [5].

When considering the calculated response of the short walls, however, web crushing did limit the displacement capacity of some of the walls. Calculated hysteresis curves for walls with 0.25% web reinforcement are shown in Fig. 17. Walls with 3 and 4% longitudinal reinforcement were calculated to experience web crushing at displacement levels less than those accommodated by walls with less longitudinal reinforcement.

A summary of the calculated modes of failure for all walls considered in the parametric study is presented in Fig. 18. The results for slender walls and the measured data from the PCA tests are shown in Fig. 18(a). Similar data for short walls are shown in Fig. 18(b). The calculated results exhibit the same trends observed in the experimental data: when the nominal flexural capacity exceeds sixty percent of the nominal shear capacity, walls subjected to lateral load reversals tend to fail in shear.

Another representation of the results is shown in Fig. 19. Walls with a maximum calculated average base–shear stresses less than $4\sqrt{f_c'}$ tended to fail in flexure. This observation is also consistent with the measured data from the PCA experiments [5]. Walls with a maximum calculated average base–shear stresses greater than $4\sqrt{f_c'}$ tended to fail in shear. The calculated shear failures have been divided into three categories in Fig. 19, all of which correspond to web crushing. Walls with low web reinforcement ratios are susceptible to degradation of shear strength with cycling (Fig. 15). Walls with larger web reinforcement ratios and maximum calculated average base–shear stresses less than $8\sqrt{f_c'}$ experienced web crushing; however, the displacement capacity of these walls was not limited by the mode of failure (Fig. 16). When the maximum
calculated average base–shear stress exceeded $8\sqrt{f'_c}$, the walls experienced significant web crushing at displacement levels less than those accommodated by comparable walls with lower nominal shear strengths (Fig. 17). Degradation of strength with lateral load reversals was also observed in some walls with low web reinforcement ratios and high average base–shear stresses.

CONCLUSIONS

Evaluation of the measured and calculated response of structural walls subjected to lateral load reversals indicates that standard design procedures are not capable of identifying likely shear failures, walls with low web reinforcement ratios are susceptible to strength degradation with cycling, and the displacement capacity of walls which experience high shear stresses may be limited by web crushing. The results presented in this paper are limited to one assumed loading history, and are not sufficient to completely characterize the conceptual model of the cyclic shear strength of structural walls shown in Fig. 8. However, the results indicate that aspects of the cyclic shear response of structural walls, which are typically ignored during design, may adversely affect the performance of reinforced concrete buildings subjected to strong ground motion.

ACKNOWLEDGMENT

Most of the work described in this paper was sponsored by the National Science Foundation through a cooperative research project to investigate the performance of reinforced concrete buildings during the 1985 Chile earthquake and a subsequent project at the University of Illinois to evaluate wall strength, stiffness, and deformation capacity. The counsel and support of Professor Mete A. Sozen during both of these studies are gratefully acknowledged.
NOTATION

\( A \) = Cross-sectional area of wall.
\( A_{cv} \) = Effective area of the cross section at the base of wall (\( L_w^* t \)).
\( H_w \) = Height of wall.
\( L_w \) = Total length of wall.
\( M_n \) = Moment at base of wall corresponding to the nominal flexural capacity of the cross section (\( \phi = 1.0 \)).
\( P \) = Axial load at base of wall (including self-weight of wall).
\( V_{max} \) = Maximum calculated base shear resisted by wall.
\( V_n \) = Shear at base of wall corresponding to the nominal strength of the cross section.
\( V_{nf} \) = Shear at base of wall corresponding to the nominal flexural strength of the cross section.
\( V_{ns} \) = Shear at base of wall corresponding to the nominal shear strength of the cross section (\( \phi = 1.0 \)).
\( V_{test} \) = Maximum base shear resisted by wall during experimental tests.
\( f'_c \) = Compressive strength of concrete in psi.
\( f_y \) = Yield stress of reinforcement in psi.
\( t \) = Web thickness.
\( \Delta \) = Displacement at top of wall.
\( \phi \) = Strength reduction factor.
\( \gamma \) = Average shear distortion at base of wall.
\( \theta_{be} \) = Reinforcement ratio of longitudinal reinforcement in boundary element.
\( \theta_h \) = Reinforcement ratio of distributed horizontal web reinforcement.
\( \theta_n \) = Reinforcement ratio of distributed web reinforcement.
\( \theta_v \) = Reinforcement ratio of distributed vertical web reinforcement.

REFERENCES


6. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318–89)," American Concrete Institute, Detroit, 1989, 353 pp.


<table>
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<tr>
<th>Specimen</th>
<th>$\Phi_{bb}$</th>
<th>$\Phi_V$</th>
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<th>$V_{nf}$</th>
<th>$V_{ns}$</th>
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Walls are identified by a character which identifies the cross-sectional shape (R: rectangular, B: barbell, F: flanged) and a number.

Loading schemes:

#: cyclic loading scheme with increasing displacement amplitude
*: cyclic loading scheme with alternating displacement amplitude
†: monotonically increasing loading.
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Table 2 — Overview of Parametric Study

Notes: $f'_t = 5500$ psi, $f_y = 60,000$ psi, barbell cross section, cyclic loading scheme with increasing displacement amplitude. Equal web reinforcement ratios were used in the horizontal and vertical directions ($\varphi_h = \varphi_v = \varphi_n$).
Fig. 1—Walls tested at Portland Cement Association (2-4)
a) Observed strength

Rectangular Barbell

b) Observed modes of failure

$V_{nf} = 0.6 \, V_{ns}$

$V_{nf} = V_{ns}$

Fig. 2—Comparison of measured wall strengths with nominal capacities
a) Global hysteretic response

![Graph showing base shear vs. top displacement for wall B1 (2)]

b) Shear distortion near base of wall

![Graph showing base shear vs. average shear distortion for wall B1 (2)]

Fig. 3—Measured response of wall B1 (2)
Fig. 4—Measured response of wall B2 (2)
a) Wall elevations

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<tr>
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<th>Loading</th>
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<tr>
<td>W2</td>
<td>Static</td>
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</tr>
<tr>
<td>W3</td>
<td>Earthquake</td>
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<td>W4</td>
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</tr>
<tr>
<td>W5</td>
<td>Simulation</td>
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<tr>
<td>W6</td>
<td>Simulation</td>
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b) Cross sections

c) Loading histories for static tests

Fig. 5—Walls tested at University of Illinois (7)
Fig. 6—Measured response of wall W1 (7)
Fig. 7—Separation of flexural and shear distortion for wall W1 (7)
Fig. 8—Conceptual model for cyclic response of structural walls (7)

Fig. 9—Nonlinear material model for reinforcing steel (11)
a) Stress-strain relationship for concrete subjected to normal stress

\[ \text{Axial Stress} \]

\[ \text{Tension} \]

\[ \text{Compression} \]

\[ \text{Axial Strain} \]

b) Stress-strain relationship for concrete subjected to shear stress

\[ \text{Shear Stress} \]

\[ \text{Shear Strain} \]

Fig. 10—Nonlinear material models for concrete (11)
Displacements controlled at corner node.

Steel elements

Concrete element

Nodes

Linear elements

Nonlinear, isoparametric, plane-stress elements for concrete.

Nonlinear, truss elements for steel.

All nodes at the base of the wall are fixed in the horizontal and vertical directions.

Barbell section

Rectangular section

Fig. 11—Finite element mesh for walls tested at the Portland Cement Association (11)
Fig. 12—Comparison of measured and calculated response of wall R1 (11)
Fig. 13—Comparison of measured and calculated response of wall B7 (11)
Fig. 14—Walls considered in the parametric study
Fig. 15—Calculated response of slender walls with web reinforcement ratio = 0.25 percent
Fig. 16—Calculated response of slender walls with web reinforcement ratio = 0.50 percent
Fig. 17—Calculated response of short walls with web reinforcement ratio = 0.25 percent
Fig. 18—Variation of calculated modes of failure with nominal capacities
Fig. 19—Variation of calculated modes of failure with web reinforcement ratio, average shear stress, and aspect ratio of wall
On Design Requirements for Reinforced Concrete Structural Walls


Synopsis: Following the strong earthquake in Chile on March 3, 1985, an intensive study was conducted to ascertain why the large inventory of moderate rise buildings in the coastal city of Viña del Mar performed so well during the earthquake. The major findings were that the vast majority of the buildings in this coastal city had a high wall area to total floor area ratio and that the reinforcement detailing in the boundaries of these walls were considerably less than required by U.S. codes. Analytical studies indicated that the high percentage of walls led to significantly lower drifts under severe seismic shaking, thus lowering the ductility demands on the walls. At lower levels of ductility demand, experimental results have demonstrated that wall boundaries did not need special detailing of transverse reinforcement. The findings from the series of research studies following the Chilean earthquake have led to modified U.S. design procedures that relate the need for special detailing in wall boundary elements to expected strain levels along the compression edge of the wall. The expected strain levels are determined based on the aspect ratio of the wall and the percentage of wall area to floor area used in the building.

Keywords: Codes (standards); ductility; earthquake-resistant structures; floors; reinforced concretes; structural design; structural walls
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Introduction

On March 3, 1985, a large earthquake ($M_S = 7.8$) occurred off the coast of Chile. Several relatively modern, moderate rise concrete buildings within and near the coastal city of Viña del Mar survived the strong ground shaking with little or no structural damage. The most significant feature of these structures was the use of structural walls as the primary gravity and lateral load resisting system [1]. Shortly after the earthquake, Mete Sozen initiated the formation of a research team consisting of members from the Universities of Illinois, Michigan and California-Berkeley, as well as the Pontificia Universidad Católica de Chile. Patricio Bonelli of Universidad Técnica Federico Santa María later joined the team and assisted in the evaluation of the buildings in Viña del Mar. This research team carried out a variety of investigations including development of a detailed inventory of the structural characteristics and observed damage in all moderate rise buildings in Viña del Mar, detailed analysis of selected buildings and experimental studies of structural components. These initial studies indicated that the existing design requirements for bearing wall buildings in U.S. codes were too conservative and essentially restricted the use of this type of structure in high seismic zones of the
Follow-up studies aimed at more clearly defining the relationship between the expected dynamic response of bearing wall buildings and their deformation capacity have lead to the relaxation of reinforcement detailing requirements for such buildings. In the following sections, this paper will describe the key steps of the research process that started with the observed good behavior of concrete structural wall buildings during the 1985 Chilean Earthquake and discuss how this research has influenced new design provisions in U.S. building codes. An example that demonstrates the use of new U.S. design provisions is included.

Research Significance

This paper retraces the trial of research from post-earthquake observation of structural behavior to design code modification. Thus, the paper provides a clear example of the importance of research and the transfer of research into practice.

The Chilean Experience and Implications for U.S. Practice

The Chilean Earthquake of 1985 presented a unique opportunity to study a type of building system, i.e. concrete bearing walls, that is seldom used for moderate rise buildings in high seismic risk zones of the U.S.A. The city of Viña del Mar is located on the Pacific coast and was approximately 80 miles northeast from the epicenter of the 1985 earthquake. The Modified Mercalli Intensity was estimated to be VII or VIII in the city [2]. The one instrument recording in Viña del Mar came from the basement of a ten-story building. The instrument recording indicated a peak acceleration of 0.36g in the S20W direction and a 40 second duration of strong shaking (ground accelerations exceeding 0.1g).

A post-earthquake inventory of buildings in the city [3] indicated that there were over two hundred concrete structures in the height range of 6 to 23 stories. A selected set of these buildings were inspected by the U.S. research team [4]. Floor plans for first level of three of the study buildings are shown in Fig. 1. The major findings of the U.S. team were: 1) the Chilean buildings had a significantly higher percentage of structural walls than typically used in U.S. construction, 2) the reinforcement detailing, particularly in the form of confinement reinforcement at the edges of the structural walls, was either very light or nonexistent (Fig. 2), 3) almost no inspection occurred during construction, and 4) the structures, in general, responded very well to the strong shaking with only scattered instances of significant structural damage.
Studies in the U.S. Following the Earthquake

To better characterize and describe the Chilean style of construction of bearing wall buildings, as well as to verify reasons for the good and poor behavior of the buildings during strong earthquake excitation, experimental and analytical studies were undertaken at the three U.S. universities noted above.

Initial Analytical Studies

The successful performance of the structural wall buildings in Viña del Mar seems to contradict the U.S. design philosophy of requiring special detailing of transverse reinforcement at the boundaries of concrete walls to ensure a high degree of ductility. The Chilean buildings relied on a large number of structural walls, an average wall area to floor area ratio in one direction for a typical building being 3% (Fig. 3), to resist vertical and lateral loads [3]. Sozen [5] and Wood [6] attributed the lack of observed damage in the buildings of Viña del Mar to the high percentage of structural walls that provided stiffness and thus limited the displacement response and story drifts during strong earthquake excitation. As is shown in Fig. 4, the fundamental period of Chilean buildings can be approximated as the number of stories divided by 20. For such buildings, studies of expected dynamic response [5] have shown (Fig. 5) that the mean drift ratios for typical Chilean buildings will be much lower than those expected in U.S. buildings that typically have a wall area to floor area ratio of 0.5 to 1.0%. Because the drift demands for most Chilean buildings would be at or below 1%, displacement ductility demands are low and special reinforcement detailing is not required at wall boundaries except for special wall configurations [12], as noted later in this paper.

Experimental Studies

The analytical studies were complimented by a set of experimental studies conducted by Ali and Wight at the University of Michigan [7]. The prototype building for these studies was the Almendral building in Valparíso, a city just south of Viña del Mar. This was a twenty-three story building and the predominant structural elements were the U-shaped walls (Fig. 1b). One interesting feature of the walls in this building was the concentration of longitudinal reinforcement at the wall edges and corners, but a complete lack of transverse confinement reinforcement (Fig. 2).

Four one-fifth scale wall specimens, five stories in height, were subjected to a series of load reversals to increasing levels of average story drift. The wall specimens did not model exactly the walls in the
prototype building, but rather had a barbell shape to somewhat reflect U.S. practice. The transverse reinforcement in the wall boundary elements was only half of what would be required by the existing U.S. codes. The first test specimen (W1) survived the loading program to maximum average story drifts of approximately 3% with no loss of lateral load capacity (Fig. 6). The first signs of compression distress (longitudinal cracking and some spalling) in the boundary elements occurred at average story drifts of approximately 1.5%. The combined axial load and overturning moment due to lateral load at this load stage caused an average compression stress of approximately 0.8 $f_c'$ in the wall boundary element. Readings from strain gages on reinforcement and displacement gages attached to the boundary elements indicated that the average compression strain near the edges of the wall ranged between 0.005 and 0.006 at 1.5% average story drift.

Reinforcement Fracture

One of the analytical studies conducted at the University of Illinois was concerned with explaining the collapse of an eight-story building (Fig. 1a) during the 1985 earthquake [8]. This building, which was located just north of Viña del Mar, featured a large area of walls, but the longitudinal reinforcement ratios at the ends of the wall were low. Fracture of longitudinal reinforcement at the free end of the web of a T-shaped wall section along the M-axis was identified as a likely cause of failure in this building. Through the comparison of this building with results from several previously tested structural walls, some of which experienced tension fracture of longitudinal reinforcement, Wood [9] developed a flexural-stress index, given in Eq. 1, to determine the vulnerability of wall sections to reinforcement fracture.

$$\frac{(\rho_l f_y + P/A)}{f_c'}$$

In this equation $\rho_l$ is the total area of vertical wall reinforcement divided by the gross cross-sectional area of the wall, $f_y$ is the reinforcement yield stress, $P$ is the wall axial load, $A$ is the cross-sectional area of the wall and $f_c'$ is the concrete compressive strength. If this index value falls below 0.15, the wall longitudinal reinforcement is vulnerable to fracture under strong seismic excitation.

Inelastic Drift Response

Initial analyses of individual Chilean buildings [10] were extended into general dynamic response studies of bearing wall buildings by
Moehle and Wallace at the University of California at Berkeley. These analyses were used to develop a method for determining the inelastic drift response of buildings and the corresponding curvature demands at the base of the walls [11, 12]. For their calculations they used an approximation of an inelastic displacement response spectrum, $S_d$, as a multiple of the building fundamental period, $T$:

$$S_d \text{ (in.)} = 6 \, T \text{ (sec.)}$$

$$S_d \text{ (mm)} = 152 \, T \text{ (sec.)}$$

This simple expression is a good representation of design spectrums used in the U.S. (Fig. 7). The results of these analyses confirmed the previous work by Sozen [5] that the lateral displacement of bearing wall buildings is a function of the ratio of wall area to floor area and the height to length aspect ratio of the wall.

By modeling wall members as vertical cantilevers and by assuming a plastic hinge length of one-half the wall length ($l_{w}$), Wallace and Moehle [11] developed relationships between drift demand and the resulting maximum curvature at the base of the wall ($\phi_u$). By relating building displacement to curvature at the base of the wall, the curvature demand can also be expressed in terms of the wall area to floor area ratio and the wall aspect ratio (Fig. 8). Calculated curvature demands could then be compared with calculated curvature capacities for typical wall sections (Fig. 9). Calculated wall curvature capacities were based on the assumption that the wall boundaries can reach a maximum compressive strain of 0.004 without special confinement reinforcement. Their results indicated that rectangular walls with an aspect ratio of less than 5, located in a bearing wall structure with a wall area to floor area ratio more than 1.5%, would be expected to experience maximum average story drifts of less than 1%. The corresponding curvature demands would be well below the curvature capacity for these walls. Thus, the previous UBC [13] requirement for the use of confinement reinforcement at wall boundaries, which were based only on a check of the nominal compressive stresses in the extreme fiber of the wall, would often be too conservative. Wallace and Moehle noted that T- and L-shaped walls have lower curvature capacities when the wall web was in compression and thus, special confinement reinforcement may be required in a portion of the web of these walls.
Drift Demand vs. Capacity

Wallace [14, 15] and Wallace and Thomsen [16] have continued to study the relationship between drift demand and drift capacity of wall sections, with drift capacity defined as a function of maximum compressive strain at the edges of the wall. Based on approximate equations for building period and the determination of expected building drifts, relationships have been developed for extreme fiber strain demand as a function of the wall area to floor area ratio, the wall aspect ratio and the wall axial load (Fig. 10). Assuming that the maximum permissible strain that can be tolerated without the addition of special confinement reinforcement is 0.004, then Fig. 10 indicates that there is a wide range of bearing wall buildings that would not need any special confinement reinforcement. Further, this figure strongly suggests that the U.S. construction tendency of putting in low percentages of structural walls (low wall area to floor area ratio) should be reexamined. Based on these findings, Wallace and Thomsen [15, 16] have developed design procedures based on expected maximum compressive strains at the wall boundaries.

Because the results of the various studies described here strongly indicated that the existing provisions of the UBC were too conservative, the design requirements for structural walls were changed for the 1994 edition of the UBC [17]. The ACI Building Code committee, ACI 318, is also discussing the introduction of new design requirements for boundary elements of structural walls [18].

Example

The building shown in Fig. 11 will be used as an example to compare the UBC and ACI design requirements with the procedure developed by Wallace and Thomsen [15, 16] to determine the need for confinement reinforcement at the boundaries of structural walls. This building was used by Wallace and Thomsen to compare their procedure with the requirements in the 1991 UBC and 1992 ACI Codes [19]. The analyses presented here will be for the building response in the transverse direction, where the lateral load resistance is provided by a total of eight walls, four rectangular walls and four T-shaped walls. Details of wall reinforcement are shown in Fig. 12. All the walls are 12 in. (305 mm) thick and 24 ft. (7.32 m) in length.

This model building is a ten-story structure with a uniform story height of 12 ft. (3.66 m). The assumed gravity load for lateral load calculation was 175 psf (8.40 kN/m²). The height to length aspect ratio ($h_W/l_W$) for all the walls is 5.0, and the ratio of wall area to total floor area, $p$, is 0.016. Using appropriate tributary areas, Wallace and
Thomsen calculated the wall axial loads \( (P) \) to be approximately 0.10 \( A_g f_c' \) and 0.05 \( A_g f_c' \) for the rectangular and T-shaped walls, respectively, where \( A_g \) is the gross wall area and \( f_c' \) is the concrete compressive strength (4 ksi, 27.6 MPa). Using the equation given in the UBC, the fundamental period of the building was estimated to be:

\[
T = 0.02 \left( \frac{h_n}{3} \right)^{3/4} = 0.73 \text{ sec.} \tag{3}
\]

where \( h_n \) is the height of the building in feet. Assuming that the building is located in seismic zone 4 (\( Z = 0.40 \)), is a regular building (\( I = 1 \)), is sited on a firm soil (\( S = 1.2 \)), and that the detailing justifies a force reduction coefficient, \( R_w \), of 12, then the lateral seismic force used for design is:

\[
V = \frac{Z IC}{R_w} \quad W = 1,300 \text{ kips (5790 kN)} \tag{4}
\]

where \( C = \left( \frac{1.25 S}{T} \right)^{2/3} \). This lateral seismic force is to be multiplied by the load factor of 1.4 and then distributed to the shear walls in proportion to their lateral stiffnesses. Based on a gross moments of inertia, each T-shaped wall should be assigned 15% of the lateral shear and each rectangular wall should be assigned 10% of the lateral load. These loads result in average shear stresses of 79 psi (0.54 MPa) on the webs of the T-shaped walls and 53 psi (0.36 MPa) on the rectangular walls.

### Previous Code Requirements

Although the estimated building period is above 0.7 sec., the use of an inverted triangular distribution of lateral loads would be a close approximation of the distribution specified in the UBC. Thus, the ratio \( M_U/V_U \) is approximately equal to \( (2/3) h_w \). Combining the factored base overturning moment with the given axial load results in a compression stress at the extreme compression fiber that exceeds 0.2 \( f_c' \) (800 psi, 5.5 MPa) for the edges of the rectangular wall section and at web edge of the T-shaped wall. Thus, according to the 1991 edition of the UBC and the 1992 ACI Code requirements, wall boundary elements would be required at all these locations. Faced with the need to provide twelve confined boundary zones for these eight walls, the structural engineer might seek an alternative structural system.

The compression stress in the extreme fiber of the flange of the T-section wall depends on the assumed effective width of the flange. If the wall is assumed to act as an isolated T-section, the effective flange
width defined by the ACI Code [19] is limited to four times the web width, and the average compression zone stress would exceed 0.2 $f_{c'}$. If the wall is assumed to act as part of a continuous system (continuity provided by the floor levels), the effective flange width defined by the ACI Code is seventeen times the web width and the average compression zone stress would be less than 0.2 $f_{c'}$. Thus, the interpretation used to find the effective compression flange width would determine whether or not the former UBC and ACI code provisions require special detailing of transverse reinforcement within the flange of the T-shaped walls.

New Code Requirements

The 1994 UBC requirements, as well as the proposed modifications to ACI Code, for determining the need for confined boundary elements in structural walls were applied to the example building. The requirements for both codes were developed using a displacement-based design approach and are essentially the same. The only significant difference will be discussed in this section.

Section 1921.6.5.4 of the 1994 UBC defines the limiting conditions for which structural walls may be designed without specially detailed boundary elements. These limits are used in lieu of estimated maximum deflections and strain calculations at the wall boundaries. The first criteria limits the axial stress in the wall due to factored axial loads to 0.10 $f_{c'}$ for geometrically symmetrical walls and to 0.05 $f_{c'}$ for unsymmetrical wall sections. The unfactored axial loads calculated by Wallace and Thomsen for this example building are at these limits for the rectangular (symmetrical) and T-shaped (unsymmetrical) walls, but it will be assumed that all the walls meet this first requirement.

In addition to satisfying the axial load criteria, the UBC requires that the walls must satisfy either an aspect ratio limit, or a maximum shear stress limit. The aspect ratio limit for structural walls is given in Eq. 5:

$$\frac{M_u}{V_u l_w} \leq 1.0 \quad (5)$$

For the inverted triangular loading assumed for this building, Eq. 5 essentially limits the wall aspect ratio, $h_w/l_w$, to 1.5. Because the wall aspect ratio limit is not satisfied for the walls in this example, a designer would then need to check the limiting shear, which is given as:
\[
V_u \leq 3 l_w h \sqrt{f_c'} \quad \text{(psi units)} \tag{6}
\]

\[
V_u \leq (0.25) l_w h \sqrt{f_c'} \quad \text{(MPa units)}
\]

where \(h\) is the wall thickness. Equation 6 can be converted to an average wall shear stress in either the web of a T-shaped section or over the area of a rectangular section by dividing the shear force by the area, \(l_w h\). For the given concrete strength, the limiting shear stress is 190 psi (1.31 MPa), which is larger than the acting shear stresses calculated at the beginning of this example. Thus, all the walls satisfy the existing 1994 UBC limiting conditions and none of the wall boundaries would need boundary elements with special detailing of transverse reinforcement.

For the ACI Code provisions now under discussion, the axial stress limits are the same as those in the 1994 UBC. If the axial stress limits are satisfied, then the walls must satisfy either a limit on the wall aspect ratio, same as given in Eq. (5), or a shear limit criteria that is combined with a second limit on the wall aspect ratio. As noted above, these walls do not satisfy the aspect ratio limit specified in Eq. 5, so for the proposed ACI provisions the shear limit given in Eq. 6 must be checked in conjunction with the second wall aspect ratio limit given below:

\[
\frac{M_u}{V_u l_w} \leq 3.0 \tag{7}
\]

Assuming an inverted triangular distribution of lateral loads, this limit becomes \((h_w/l_w) \leq 4.5\). All of the walls in this example have an aspect ratio of 5.0, so they would fail to meet this limiting criteria. Thus, the proposed ACI criteria are not satisfied and a check on maximum strain, similar to that discussed in the following paragraph, would be required to determine the need for specially detailed wall boundary elements. It should be noted that in a planned interim update of the 1994 UBC, the wall aspect ratio limit given by Eq. (7) will be combined with the shear stress limit in Eq. (6).

**Alternate Procedure**

If a structural wall fails to meet the criteria stated above, the 1994 UBC offers an alternate procedure for determining if special detailing is required at the wall edges. The procedure involves the calculation of
the maximum strain at the extreme compression edge of the wall as a function of the expected maximum displacement at the top of the wall. If the maximum strain exceeds 0.003, then the walls must have boundary elements.

The UBC procedure for calculating the maximum displacement at the top of a structural wall is given in the following equation:

\[ \Delta_t = \frac{3}{8} R_w (2 \Delta_c) \]  

(8)

where \( \Delta_c \) is the displacement at the top of the wall due to code specified lateral forces, calculated using gross sections properties. The code specified forces are equal to the forces obtained from an elastic dynamic response of the structure subjected to a code specified ground motion spectra, divided by the force reduction factor, \( R_w \). The coefficient 2 accounts for cracking of wall sections and the multiplier \((3/8)R_w\) is to account for inelastic behavior of the structure. Shimazaki and Sozen [20] have shown that this procedure gives unconservatively low values for maximum deflections.

Wallace and Thomsen [15, 16] have developed alternate procedures for calculating maximum displacements at the top of the wall and resulting maximum compression strains at the edges of the wall. Their calculations are based on unreduced seismic displacements and the approximate inelastic displacement response spectra defined by Wallace and Moehle [11, 12]. Wallace and Thomsen presented results in the form of figures, which will be used here to save lengthy duplication of code equations. However, it should be noted that the application of the UBC limiting strain criteria of 0.003, using the figures generated by Wallace and Thomsen for unreduced seismic displacements, is a conservative approach.

Fig. 13 is used to estimate the maximum compression strain in the edges of the rectangular wall sections. Entering the figure with a wall area to floor area ratio of 0.016, the maximum strain value for an axial stress of 0.10 \( f_c' \) is between 0.003 and 0.0035. This calculated strain exceeds the 1994 UBC criteria, but it is within the suggested limit of 0.004 proposed by Wallace, Moehle and Thomsen. Thus, special detailing of wall boundaries would not be required for the rectangular walls using the limiting strain of 0.004. The use of 0.004 is consistent with test results reported in this paper [7], as well as experimental studies summarized by Wallace and Moehle [11].

Fig. 13 can also be used to determine the maximum compressive strain at the flange edge of the T-shaped wall. This is a very
conservative approach because the curves in Fig. 10(a) are for a compression zone with a width equal to that of the web. For the same wall area to floor area ratio and an axial stress of 0.05 $f_{c'}$, the maximum strain is approximately 0.0025, which is below the code limits. Thus, a special boundary element would not be required at the flange end of the T-shaped wall.

Fig. 14 can be used to check the maximum strain at the free edge of the web of the T-shaped wall. Wallace and Thomsen have shown that the difference between the tension steel ratio (flange in tension) and compression steel ratio is near 0.01 for the wall cross-section in Fig. 12, assuming the entire flange width is effective in tension. From the results shown in Fig. 14, it is clear that the maximum strains would be far above the limits set by both the 1994 UBC and Wallace, Moehle and Thomsen. Fig. 15, which was developed by Wallace and Thomsen, indicates that the confined boundary zone should extend along the web a length $0.16l_w$, or approximately 3.8 ft. (1.17 m).

Discussion

This example demonstrates that the 1994 UBC and proposed ACI code provisions are a significant improvement over existing requirements. However, the 1994 UBC criteria for determining if confined wall boundary elements are required is inconsistent, and will changed by an interim update. As now stated, the limiting shear in Eq. 6 is not coupled with the limit on the wall aspect ratio given by Eq. (7), as it is in the proposed ACI provisions. This lack of a limit on the wall aspect ratio allowed all the walls in this example to be exempted for the need for boundary elements, which is clearly not safe for the free edge of the web of T-shaped wall. In addition, estimated displacement response using the UBC approach (Eq. 8), is unconservative for wall area ratios above values typically used in U.S. construction.

Other concerns that developed during the investigation of the Chilean buildings are not contained in the new provisions of either the UBC or ACI. First, there is no general discussion of the benefits of providing a larger percentage of wall area to floor area. As was shown by Sozen [5], there is a substantial reduction in story drifts if the percentage of wall area is increased. This trend is reflected in all the figures used in this example, Thus, considerably lower maximum compression strains are calculated at the edges of structural walls as the percentage of wall area is increased.

Also, there is no consideration of the possibility of reinforcement fracture as discussed by Wood [9]. The flexural stress index is only of concern in the T-shaped wall for bending that puts the flange in
tension. Recall, fracture of reinforcement in the web of a T-shaped wall was shown to be the probable cause of the collapse of the building investigated by Wood et al. [8]. If Eq. 1 is applied using the total reinforcement ratio, the flexural stress ratio exceeds the limit of 0.15 set by Wood. However, if the tension reinforcement ratio in the web (0.00367) is used, which is more appropriate for this case, the flexural stress index is 0.105, indicating that the longitudinal reinforcement at the free edge of the web may be susceptible to fracture when the wall is subjected to lateral load reversals. Reductions of the flange width, or increasing the amount of longitudinal reinforcement in the web, would reduce the possibility of reinforcement fracture.

Summary and Conclusions

This paper has described the chain of research studies that lead from the observed good performance of bearing wall buildings during the 1985 Chile earthquake to the development of new code provisions in the U.S. for structural walls. Key research findings during this process were presented and an example was used to discuss the new code criteria. Based on this study, the following conclusions are reached.

1. The 1994 UBC and proposed ACI criteria for determining the need for boundary elements in shear walls are a substantial improvement and remove the very conservative criteria now in use. However, displacement estimates used by the 1994 UBC provisions significantly underestimate expected displacement response.

2. A change in design philosophy is much more important than adherence to any new detailing requirements. Numerous studies have shown that increasing the wall area to floor area ratio significantly above the value commonly used in U.S. bearing wall buildings will eliminate the need for special confinement reinforcement for most structural walls.

3. There is a significant difference between the limitation criteria used (currently) by the 1994 UBC and those proposed for the ACI Code for determining whether or not a structural wall needs well confined boundary elements. For walls with relatively low axial loads, the 1994 UBC only limits the maximum wall shear stress. For the same walls, the proposed ACI provisions couple this shear stress limit with a limit on the wall aspect ratio.

4. All the code provisions discussed here do not address the possibility of fracture of longitudinal wall reinforcement. Some minimum reinforcement limit, similar to that proposed by Wood [9],
should be investigated further for possible inclusion in the appropriate codes.

5. The 1994 UBC limit on maximum compression strain (0.003) for determining whether or not a wall needs special confinement reinforcement along its boundary is too conservative in comparison with test results. A limit of 0.004 is more reasonable.

References


18. *Minutes of ACI Committee 318* (1993), American Concrete Institute, Detroit, MI.

19. *Building Code Requirements for Reinforced Concrete* (ACI 318-89) (Revised 1992), American Concrete Institute, Detroit, MI.

Fig. 1a—Typical floor plan, El Faro building (all dimensions in meters, 1 m = 3.28 ft)

Fig. 1b—Typical floor plan, Almendral building (all dimensions in meters, 1 m = 3.28 ft)
Fig. 1c—Typical floor plan, Festival building (all dimensions in centimeters, 2.54 cm = 1 in.)
Fig. 2—Detail of reinforcement at wall edges, Almendral building (dimensions in centimeters, 2.54 cm = 1 in.)

Fig. 3—Distribution of buildings in Viña del Mar with respect to wall area
Fig. 4—Measured period for Chilean buildings

Fig. 5—Calculated mean drift ratio versus wall area index
Fig. 6—Measured load versus displacement relationship, specimen W1

Fig. 7—Comparison of ATC and simplified displacement spectra
Fig. 8—Relationship between required ultimate curvature and wall area ratio

Fig. 9—Calculated moment-curvature relationships for typical wall sections
Fig. 10—Computed wall extreme fiber compression strain
Fig. 11—Floor plan of study building (1 ft = 0.305 m)
a) Rectangular walls

\[ \text{1 in.} = 2.54 \text{ cm} \]
\[ \text{1 in}^2 = 6.45 \text{ cm}^2 \]

b) T-shaped walls

Fig. 12—Wall reinforcement details (1 in. = 2.54 cm)
Fig. 13—Extreme fiber compression strain for symmetrically reinforced walls

Fig. 14—Extreme fiber compression strain for unsymmetrically reinforced walls
Fig. 15—Required length of confined zone for unsymmetrically reinforced walls

\[ P = 0.05A_w f'_c \quad h_w/l_w = 5 \]

- \( \rho - \rho' = 0.01 \)
- \( \rho - \rho' = 0.005 \)
- \( \rho - \rho' = 0.0 \)