Recently Identified Aspects of Ductile Seismic Torsional Response of Reinforced Concrete Buildings

by T. Paulay

Synopsis: With few exceptions, code provisions relevant to torsional phenomena in buildings subjected to seismic effects, are based on elastic structural behaviour. The key parameter is stiffness eccentricity. The appropriateness of this approach to the design of systems expected to respond in a ductile manner is questioned. The degree of restraint with respect to system twist, strength eccentricity and the pattern of element yield displacements are considered to be more important parameters. For the purposes of seismic design, bi-linear force-displacement approximations of the elasto-plastic behaviour of reinforced concrete systems and their constituent elements, are considered to be adequate. Strategies aiming at the elimination of undesirable effects of torsional phenomena in ductile systems are addressed.

The findings of this study are based on a re-definition of some common terms of structural engineering, such stiffness, yield displacement and displacement ductility relationships. Contradictions with corresponding terms applicable to elastic systems are demonstrated. The introduction of these features, relevant to bilinear modelling of reinforced concrete elements, precedes the examination of the designer’s options for the control of earthquake-induced displacement demands resulting in system translations and twist.

Keywords: codes; deflections; ductility; earthquake-resistant structures; reinforced concrete; stiffness; strength; structural design; torsion in buildings
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INTRODUCTION

Studies of the assessment of the structural performance of existing buildings with earthquake risk (1) triggered inquiries addressing the likely response of buildings as constructed, rather than their compliance with a particular code. A major perceived need was the estimation of torsion-induced displacements of elements of ductile systems (2,3). In the process several issues emerged with apparent conflict with ingredients of existing design practice. The description of progressively emerging fallacies, firmly entrenched in widely accepted routine seismic design techniques (4), is the subject of this presentation.

The perceived need to address earthquake-induced displacements in ductile systems motivated this study. It led to the introduction of unfamiliar, yet not necessarily new, principles. The freedom available to the designer, when strength is to be assigned to elements of a system, is a particularly interesting feature of the conclusions. Identification of structural behaviour, rationale and transparency of a viable design strategy, combined with simplicity of application, were central issues of this motivation.

DESIGN CRITERIA

The primary purpose of this study was to address means by which performance criteria, conforming with the appropriate limit state, may be rationally executed. The criteria considered were:

- Expected earthquake-induced deformations, including those due to system twist, should be limited to ensure that the displacement ductility capacity of any element of the system, \( \mu_{\Delta_{\text{max}}} \), will not be exceeded.
- Maximum interstorey displacements, to be expected at locations remote from the centre of mass, should not exceed those considered acceptable for buildings, typically 2-2.5% of the storey height.
- More restrictive performance criteria may require displacements associated with a specific limit state to be less than those allowed by the displacement ductility capacity of constituent elements.
TERMINOLOGY USED

- In the study of earthquake-induced displacement of buildings, reference will be made to the structural system.
- A structural system comprises lateral force-resisting elements generally arranged in two orthogonal directions. Due to torsional effects, elements of the system may be subject to different storey displacements. Typical elements are bents of ductile frames or interconnected walls in the same plane.
- A lateral force-resisting element may comprise several components. Components will be subjected to identical displacements. Typical components are beams or columns or walls.

Examples are illustrated in Fig. 1. In this study only simple mass systems are considered.

TRADITIONAL CONCEPTS OF THE THEORY OF ELASTICITY

The requirements for static equilibrium and deformation compatibility in a statically indeterminate structure, are well established. These principles are still widely used when strength is assigned to lateral force-resisting elements of a system. The procedure is consistent with the now abandoned principle, that the resulting stresses should be well within the elastic domain of material behaviour.

With the introduction of equivalent lateral static seismic design forces and the acceptance of ductile response, the same technique continued to be widely used. It implied the notion that strength assigned proportionally to element stiffness will eventually result in the simultaneous onset of yielding in all elements.

As a corollary it is generally assumed that components with traditionally defined stiffness, based on flexural rigidity, EI, will have increased yield displacements as their strength is being increased. The flexural rigidity is the product of the modulus of elasticity of the material, E, such as concrete, and I is the second moment of the cross sectional area of a prismatic member. Its value may be adjusted in recognition of the effects of cracking (5).

Requirements for static equilibrium and deformation compatibility in statically indeterminate structures, are well established. Strength allocation based on these principles implies that the intended strength of all components of a lateral force-resisting element will occur at the same displacement. Subsequent adjustments of strength, relying on strength redistribution in a partially nonlinear system, is assumed to result in corresponding changes in yield displacements. Some fallacies relevant to the traditional modelling of elasto-plastic behaviour, widely used in seismic design practice, have been identified (6,7,8).
It is postulated that for the purposes of seismic design the simulation of the non-linear behaviour of a ductile system, can be adequately modelled with bi-linear force-displacement relationships. Implications in terms of component, element or system response, are briefly summarized. The findings enable displacement relationships, including those due to system twist, to be quantified in a simple form.

MODELLING OF DUCTILE COMPONENT BEHAVIOUR

The typical elasto-plastic behaviour of a reinforced concrete and steel component is shown by the full line curves in Fig.2. The relative unit strength shown corresponds to that developed when yield strain is first developed in the extreme tension fibre. The dashed line for the RC section illustrates the effects of crack developments during first loading. This is of no interest in seismic design based on ductile response.

The associated yield curvature of the critical section is readily determined (9). Even under reversing repeated loading, the stiffness of the component is adequately simulated by the slope of the force-displacement relationship up to the unit relative strength. Once the pattern of bending moments is known, the relevant displacement associated with the onset of yielding may also be determined. For a realistic estimate of this displacement the second moment of the section area, transformed into properties of the concrete, needs to be used. In routine design this and the strength at first yield, involve cumbersome computations, preferably to be avoided. Simpler approaches are suggested.

The evaluation of the nominal strength, \( V_{ni} \), of the component, shown subsequently to be a choice of the designer, is an essential part of the design routine. As Fig.2 shows, a linear extension of the elastic response (\( \Delta_{ye} \)) allows the nominal or reference yield displacement, \( \Delta_{ye} \), to be determined. The dashed lines in Fig.2 show how the nonlinear response, for example of a RC component, can than be simulated. If desired, the designer may allow for post-yield stiffness of the component.

Yield Curvature

The first step in estimating the yield displacement, \( \Delta_{yi} \), of a component is the evaluation of the yield curvature at the critical section of a potential plastic hinge. From the study of strain patterns it was found (10,11) that nominal curvatures linearly extrapolated, as in Fig. 2, from that at the onset of yielding at the extreme tension fibres of typical reinforced concrete sections subjected to flexure with moderate axial loads, are approximately constant. For the purposes of seismic design such estimates should be considered adequate. For example the yield curvature associated with bilinear modelling, as in Fig.2, of a typical rectangular wall section (10), is of the order of
where $\varepsilon_y$ is the yield strain of the steel used and $l_w$ is the length of the wall (Fig. 3(a)).

This simple relationship shows that yield deformations are sensitive to the grade of reinforcement used and are inversely proportional to the overall depth of a component. Yield curvatures for components with different lengths cannot be identical, as assumed when using conventional analyses based on elastic behaviour. An important feature of yield curvature is, that for the purposes of seismic design, it is not affected by the strength assigned to the component.

**Yield Displacements**

As previously stated, once the nominal yield curvature, $\phi_{yi}$, at the critical section or sections of a component is established, the yield displacement, used in the elasto-plastic bi-linear modelling, is readily evaluated. For example, yield displacements of reinforced concrete structural walls, subjected to a particular pattern of lateral forces are found to be

$$\Delta_{yi} = C\phi_{yi} h_{wi}^2 = (2C h_{wi}^2 \varepsilon_y) / l_{wi} \propto 1 / l_{wi}$$

where $C$ is a coefficient which quantifies the pattern of lateral forces, and $h_{wi}$ is the height of the wall. When wall heights are identical and the grade of reinforcement used is the same in one building, the bracketed term in eq.(2) is a constant. Therefore, in these common cases yield displacements of wall components, such as shown in Fig. 3(a), are inversely proportional to the length of the walls, $l_{wi}$.

The simple relationship (eq.(2)) can be conveniently used in seismic design whenever relative values, for example for the estimation of displacement ductilities, are sufficient. Equation (2) is fundamental in establishing displacement relationships between components, elements and the entire system (Fig.1). Implications of these relationships have been previously reviewed (7,8,12). The important conclusion to be drawn is the fact that yield displacements are functions of geometric and material properties and are independent of the nominal strength of components.

**A RE-DEFINITION OF STIFFNESS**

When seismic design is based on bi-linear modelling of ductile behaviour, as shown in Fig.2, the stiffness of the component is simply

$$k_i = V_m / \Delta_{yi}$$
where \( V_{ni} \) is the nominal strength assigned to the component and \( \Delta_{yi} \) is a geometry-dependent predetermined property, as defined in eq.(2). It is seen that, contrary to traditional assumptions which are extensively used in seismic design, stiffness is proportional to strength assigned to the component by the designer. An example to be presented in Fig.3, where the bi-linear force-displacement behaviour of components with different lengths compared, will offer further explanations.

**ASSIGNMENT OF STRENGTHS**

As the bi-linear modelling in Fig. 2 implies, components will develop their nominal strength, \( V_{ni} \), when their nominal yield displacement is imposed. Therefore, components with different yield displacements can never yield simultaneously. Correspondingly, components of an element, such as seen in Fig.1, will develop their nominal strength in a given sequence until all components have yielded, dictated by the attainment of their yield displacements. From this it may be concluded that, within rational limits, strength to components may be assigned arbitrarily. In this and subsequent examples it is assumed that the total nominal strength, \( V_E \), is unity.

The principle offers great possibilities to the designer to exploit this freedom of choice in nominal strengths. Thereby more rational and economic solutions may be achieved. It is a fundamental tool aiding the mitigation or even elimination of the detrimental effects of torsional phenomena in ductile systems.

**STRENGTH, STIFFNESS AND DISPLACEMENT RELATIONSHIPS**

The relevance of these relationships, so important in seismic design, will be illustrated with an example presented in Fig.3. The relative lengths of four interconnected rectangular reinforced concrete cantilever walls with identical width, are such that the traditionally defined second moment of sectional area of the components bear the ratio of 1, 2, 4 and 8, respectively, to each other. Conventional design will assign lateral strength, \( V_{ni} \), in these proportions.

The bilinear simulation of the components and the element, based on this traditional distribution of strengths, is shown in Fig.3(b). The relative strength of component (4) with the greatest length, is thus \( \frac{8}{15} = 0.53 \), while its relative yield displacement is according to eq.(2) \( \Delta_{y4} = \frac{2}{3} = 0.5 \). Therefore, its relative stiffness (eq.(3)) is \( k_4 = \frac{0.53}{0.5} = 1.06 \). The superposition of bilinear component responses results in the total response of the element. This can be again simulated by simple bi-linear modelling. No post-yield stiffness was assumed in this example. The superposition relevant to this traditional strength distribution, shown in Fig.3(b), allows the total
translational stiffness of the 4-component element to be defined as:

\[ k_e = \Sigma k_i = \Sigma \left( \frac{V_{ni}}{\Delta_{yi}} \right) \]  

(4)

This in turn enables the nominal yield displacement of the element to be quantified as

\[ \Delta_y = \Sigma V_{ni} / \Sigma k_i = V_e / k_e \]  

(5)

The element nominal yield displacement, so derived and shown as 0.58 displacement units in Fig.3(b), is the weighted average of the component yield displacements. It is seen that when this displacement is imposed, some components will have yielded while some others would not. The purpose of the element yield displacement, given by eq.(5), is to allow the element displacement ductility capacity to be defined.

It is evident that, when the displacement ductility capacity of the components is specified, for example, as \( \mu_{\Delta_{\text{ymax}}} = 5 \), the element displacement at the ultimate limit state must be limited to \( \Delta_u \leq 5\Delta_{y_{\text{min}}} \). In the example the smallest yield displacement is that of component (4), i.e., 0.5 displacement units. Therefore, the displacement ductility demand on this element, controlled by component (4), should be limited to \( \mu_\Delta \leq 5 \times 0.5/0.58 = 4.3 \).

The results of an entirely different assignment of component strengths, the aim of which will be referred to subsequently, is illustrated in Fig.3(c). With different strengths, component stiffness are also different. The reduced total stiffness in this case, defined by eq.(4), resulted in a small increase of the nominal element yield displacement to 0.66 units and a corresponding reduction of the element displacement ductility capacity to 3.79.

A similar procedure can be used when the corresponding properties of the entire building system, comprising a number of parallel elements is determined.

TORSIONAL PHENOMENA IN DUCTILE SYSTEMS

The Origin of Torsional Actions

The purpose of considering the effects of torque and consequent twist on a system, is to estimate displacements imposed on elements, additional to that which would occur at the centre of the mass. The study is based on the usual assumption that lateral force resisting elements are interconnected by infinitely rigid floor diaphragms.

Under the action of a base shear force, the static torque, resulting from stiffness or
strength eccentricities, may be readily determined. The possibility of resisting such a torque during the elastic or ductile response needs to be studied first. Whenever diaphragm rotation, i.e., system twist, occurs, a dynamically induced torque, due to the rotary inertia of the distributed mass, will also be introduced. This is difficult to predict. However, the conditions under which a static and/or a dynamic torque can develop may be identified. This is subsequently illustrated.

Eccentricities

The cause of torsional phenomena is eccentricity. To define this, familiar locations within the plan of a building need to be identified. With reference to Fig. 4 the centre of the distributed mass is shown as CM. For the assessment of elastic response the centre of element stiffness, commonly referred to as the centre of rigidity, CR, is of importance. After the elements have entered the inelastic domain of response and the base shear capacity, $V_E$, of the system is developed, the associated centre of stiffness becomes meaningless. At this stage the important location is the centre of resistance, i.e., that of the nominal strengths of all the yielding elements, CV. Simple equilibrium criteria enables the location, CV, of the corresponding resultant forces, generated by displacements in either of the principal directions, larger than that causing yielding in the element with the smallest length, to be readily determined. A typical example is shown in Fig. 4.

At certain instants of a seismic event, some elements may be subjected to displacements which are less than the relevant yield displacement. At such a stage the full base shear capacity, $V_E$, will not be developed. Moreover, maximum displacements of the yielding elements are not likely to approach their displacement capacity, unless an extremely large angle of twist is imposed by the earthquake. As a general rule, the maximum displacement ductility demand on any element, which is of major interest to the designer, can be expected to occur with the ductile response of all elements, when the full base shear capacity, $V_E$, is developed.

Torsional phenomena will arise whenever stiffness or strength eccentricities with respect to the base shear force, $V_{Ey}$, acting at the centre of mass, CM, denoted as $e_{ex}$ and $e_{ex}$, respectively, exist. Both can be readily determined. The definition by eq. (3) of the strength-dependent element stiffness implies that the two types of eccentricities are related to each other. This feature is not recognised in current codified design procedures. The key ingredient of the design strategy, relevant to torsional phenomena, is the ability of the astute designer to assign strengths to lateral force-resisting elements in such a way as to obtain a suitable location for the centre of strength, CV.

In the study of the influence of stiffness ($e_{ex}$) and strength ($e_{ex}$) eccentricities, the following four classes of structures are distinguished: (a) $e_{sx} = e_{sy} = 0$, (b) $e_{sx} = 0$ but $e_{sy} \neq 0$, (c) $e_{sx} = 0$ but $e_{sy} \neq 0$, (d) $e_{sx} \neq 0$ and $e_{sy} \neq 0$. Case (a) is the condition for, what is commonly referred to as, a torsionally balanced system. In these torsional
phenomena do not arise, unless rotary motions are introduced at the foundations. Case (d) represents the majority of real structures. If desired, the designer may readily achieve conditions (b) or (c) above.

Mass eccentricity, $e_{max}$, with respect to the geometric centre of the plan, may also exist. This, however, will affect only the rotary inertia of the mass, to be taken into account in the evaluation of dynamic response, a subject absent in current design practice and beyond the scope of this presentation.

Degree of Torsional Restraint

It has been suggested (13) that, as part of routine structural design, when unidirectional seismic attack in the principal directions of the building are considered separately, two types of torsional mechanisms should be distinguished. Special features of the behaviour of each of these are briefly reviewed subsequently.

Torsionally unrestrained systems - When only one element, transverse to the direction of the base shear, $V_{Ey}$, and concurrent with CM, is present, as shown in Fig.5(a), no torque can be introduced to the system after the translatory elements have entered the inelastic domain. This feature is not recognized in existing code provisions (14). Therefore, in terms of rotary motions, the ductile system is unrestrained. Element (3) is effective only in resisting earthquake-induced forces in the x direction.

Figure 5(b) shows a similar example where the sole two-component transverse element is eccentric with respect to CM. In terms of a static base shear, $V_{Ey}$, this ductile system is also torsionally unrestrained. However, during dynamic response system twist will introduce displacements in the x direction to the transverse components (3) and (4) and to CM. The acceleration of the mass in the x direction will thus introduce an inertia force at CM. An equal and opposite force will then be developed in the transverse element, leading to a dynamically induced torque.

To illustrate relationships between stiffness and strength eccentricities, the simple specific model comprising rectangular cantilever walls, as shown in Figs 5(a) and 6, will be considered. The two elements (1) and (2) are required to sustain a base shear in the y direction, $V_{Ey}$. Equilibrium criteria suggest that the nominal strength of element (1) should be twice that of element (2). However, this condition will not be achieved in real buildings because compliance with existing code requirements, and inevitable variance of the strengths of the elements, as constructed, will lead to some strength eccentricity. For example the nominal strength of element (2) may be exactly as intended, i.e., $V_{n2} = V_{Ey}/3$. However, the reinforcement of element (1) is such that its nominal strength turned out to be greater than that required, i.e., $V_{n1} = \lambda_1 (2V_{Ey}/3)$, where $\lambda_1 \geq 1.0$. Similarly situations may be considered when inevitably element (2) may have some excess strength, quantified by the parameter $\lambda_2$. 
Figure 6 shows how strength eccentricities, expressed as the ratio $e_{v}/D$, vary with increasing excess strength of one or the other of the two elements. With increased values of $\lambda$, the total base shear capacity of the system will also increase. This is shown by the dashed straight lines in Fig.6.

In a previous section it was demonstrated by eq.(3), that the stiffness of an element with given sectional dimensions is proportional to its nominal strength. Hence the stiffness eccentricities corresponding with the excess strength of the elements of the model in Fig.5(a) are readily determined. These are shown, again in terms of $e_{v}/D$, in Fig.6.

The purpose of presenting this example is to illustrate that:

- Stiffness and strength eccentricities are interdependent.
- For a wide range of the variation of the excess strength of elements, differences between the respective eccentricities, i.e., $|e_{v} - e_{c}|$, remain essentially constant for systems with a given geometry.
- The designer, having full control over strength eccentricity, thereby also controls stiffness eccentricity. According to current design practice the latter is only a geometry-dependent property of the system, beyond the control of the designer.
- Classes of systems characterized by the relations of the two types of eccentricities, previously presented as (a) to (d), are readily identified. Because in this example structure (Fig. 5(a)) the geometry of the two elements is different, a torsionally balanced condition can never be achieved.
- In terms of dynamic response, a particularly favourable situation arises in this structure when $\lambda_{2} = 1.2$, i.e., when, contrary to indications of statics, the nominal strength of element (2) is made 60%, rather than 50%, of that of element (1). In this case CR and CV are approximately equidistant on opposite sides of CM. In attempting to reduce the adverse effects of torsional phenomena, the designer can readily approach this condition.

**Torsionally restrained systems** - When transverse elements in at least two planes are provided, as in Fig. 4, a torque, for example due to strength eccentricity, can be resisted after all translatory elements have yielded. Thereby rotational deformations of the diaphragm, i.e., twist, are restrained. This is particularly the case when the transverse elements remain elastic while they resist the torque generated. Designers are aware that such systems are preferable. Both models, shown in Fig. 5, are torsionally restrained in terms of a base shear, $V_{Ex}$.

Specific examples will be used to illustrate how the designer can influence effects of torsional phenomena on inelastic element displacement demands.

To facilitate meaningful comparisons, three alternatives of an example structure are presented. For these, important design quantities, such as eccentricities, total translatory system stiffness, system yield displacement and the attainable maximum
system displacement ductility capacity, are distinctly recorded under each of the relevant figures.

An example based on the geometric stiffness-proportional distribution of element strengths - As an illustrative example, consider the previously studied structure shown in Fig. 3(a). However, the walls, instead of being components of an element, will now be used as elements of a system. The corresponding layout of these elements is shown in the plan of Fig. 7(a). The conventional assignment of proportional translatory strengths to elements (1) to (4), recorded in Fig. 3(b), results in this case in a stiffness eccentricity of 0.188A, where A is the distance between elements (1) and (4), as shown in Fig. 7(a). Because strengths have been made proportional to geometric stiffness, (EI), the centres of rigidity, CR, and strength, CV, coincide, as seen in the plan, Fig. 7(a). The translatory stiffness and yield displacement of this system, shown in Fig. 7(c), are the same as those of the corresponding element, presented in Fig. 3(a).

In the absence of the (T) type transverse elements, the resulting torque, $M_t = 0.188AV_e$, could be resisted only within the elastic domain of the 4 wall elements. Therefore, in routine design, a torsional analysis of the elastic system is used with the aim to eliminate strength eccentricity and hence the need to sustain a static torque. This scenario is of no further interest.

After the translatory elements, (1) to (4), have yielded, the torque resulting from the strength eccentricity, $e_v=0.188A$, or any other cause, could be resisted only by the transverse elements. It is assumed that the relevant properties of the two identical (T) type transverse elements are: $\xi_{nt}=1.73$, hence from eq. (2) $\Delta_{nt} = 1/1.73=0.58$ and $V_{nt} = 0.5V_e$. The relative stiffness is thus $k_t = V_{nt}/\Delta_{nt} = 0.5/0.58=0.86$ when $V_{E} = 1.000$. Therefore, the static torque-induced forces in the transverse elements are $V_t = \pm 0.188A/0.5A = 0.376 < 0.5$, i.e., less than the strength of these elements. The corresponding displacements of the (T) elements would be $\Delta_t = V_t/k_t = 0.376/0.86=0.44$ displacement units, resulting in an angle of twist of $\theta_t=2\times0.44/0.5A=1.76/A$. This is shown in the displacement profile within Fig. 7(b).

It is assumed again that the ultimate displacement of the critical element (4) is limited by $\mu_{ul} = 5.0$, as in Fig. 3(b), to 2.5 displacement units. The corresponding displacement profile is shown in Fig. 7(b). The displacement demands associated with the static torque indicate that the system displacement ductility capacity of the system, compared with that in Fig. 3(a) ($\mu_a = 4.31$) could increase to $\mu_a = (2.5 + 0.433A \times 1.76/A)/0.58 = 5.62$, without the displacement ductility limit of 5 being exceeded in any element. This example shows that twisting of the system could in fact better utilize the displacement capacity of elements. Nevertheless, for reasons explained subsequently, this approach is not recommended.

Equilibrium and deformation compatibility requirements, embodied in current codes for elastic systems, would not allow such a distribution of strengths to be
used. Irrespective of the presence or absence of the (T) type transverse elements, the conventionally defined torsional stiffness would be used to determine the torque-induced forces. These would radically change the strength of elements (1) to (4). Hence, their stiffness would no longer correspond to the true values, defined by eq. (3).

The example presented in Fig. 7 intends to illustrate primarily the ambiguity associated with the requirement that element strengths be made proportional to geometry-based stiffness.

An example based on an arbitrary assignment of element strengths resulting in zero strength eccentricity - Common sense, as well as standard design practice, would suggest that a promising distribution of element strengths should result in little or no strength eccentricity. This could be achieved in a variety of ways.

One arbitrary choice of strength distribution, among the four walls, resulting in zero strength eccentricity, is that shown in Fig.3(c). As the strengths of the four elements are now different, their stiffness have also changed. The increased yield displacement (Fig. 3(c)) and corresponding system stiffness are reproduced in Fig. 8(c). As the plan in Fig.8(a) shows, this results in a much reduced stiffness eccentricity, \( e_n = 0.07A \). This important feature is ignored in current codified procedures. In the presence of the (T) type transverse elements this relatively small stiffness eccentricity, relevant primarily to elastic behaviour, is likely to result in only a small twist of the system. The worst scenario, associated with zero twist, depicted in Figs.3(c) and 8(b), would then limit the system displacement capacity to \( \mu = 3.79 \). Some twist that may be introduced due to the stiffness eccentricity, \( e_n \), during the dynamic response of this ductile system, shown by the dashed line profile, is likely to increase its displacement ductility capacity.

An example structure with moderate strength eccentricity - The optimal static strength eccentricity would result in the full utilization of the displacement ductility capacity of at least two of the four elements, seen in Fig.9(a). In the example structure this hypothetical situation would involve simultaneous ductility demands of 5 on elements (3) and (4). This limit is shown by the dashed displacement profile passing through two circled points in Fig.9(b). The extremely large twist, \( \theta_{\text{lo}} = 1.95/A \), would allow in this model structure the largest displacement of CM and hence system displacement ductility capacity of \( \mu = 5.39 \) to be mobilized. The (T) type transverse elements would be close to yielding. This hypothetical situation intends to show only the limits of achievable system twist due to a static torque, and its benefits.

A more realistic and promising approach to the assignment of strengths to the four elements in Fig. 9(a) might be one which results in moderate strength eccentricity, such as say \( e_n = 0.08A \). It may be shown that the corresponding strength-dependent stiffness eccentricity is \( e_n = 0.16A \). This static strength eccentricity would result
in a system twist, controlled by the elastic transverse elements, of only 0.74/A. The corresponding achievable system displacement ductility would be \( \mu_A = (2.5 + 0.433A \times 0.74/A)/0.62 = 4.55 \).

During the dynamic response of the system, mass rotation may impose additional system twist. However, as the displacement profile in Fig.9(b) shows, this deliberately chosen position of the centre of strength (CV) resulted in an extremely tolerant structure. Any possible increase of torque, generated by mass rotations, increasing the angle of twist by a factor as much as 2.6, will deliver increased energy dissipation of the system without exceeding the specified displacement capacity of any of the elements.

The message of this example is, that the elimination or drastic reduction of eccentricities in systems, comprising elements with different yield displacements, will not necessarily result in improved ductile performance.

The predictable magnitude of twist due to a static strength eccentricity only must, however, be kept to moderate values. This will make allowances for significant variations in additionally imposed twists during the dynamic response of the ductile system. This is the reason why the static torque-generated twist shown in Fig.7(b), although only 90% of the optimum value, should not be relied on.

**Directional Effects**

In conformance with code [14] requirements, generally only unidirectional seismic attacks, independently in the two principal orthogonal directions of a building, are considered by designers.

An earthquake-generated instantaneous base shear may, however, act in any direction. Researchers prefer to use the term "bi-directional effects". A study of this involves usually the evaluation of simultaneous orthogonal dynamic responses of ductile systems to a selected suit of earthquake records. This technique cannot be considered presently as a viable tool in routine design. The term preferred here [15] is "skew seismic attack". It is postulated that an in-depth study of unidirectional ductile response, independently in the principal orthogonal directions, can usually provide sufficient information with respect to torsional mechanisms and consequent displacement demands under a skew attack. Details are not considered in this paper.
A CRITICAL REVIEW OF EXISTING CODE PROVISIONS

With priority placed on the designers intention to control earthquake-induced displacements in reinforced concrete structural systems, certain conflicts with widely accepted torsional provisions emerge. A brief review of these is presented here.

(i) Relevant world wide code provisions [14] are based on elastic structural behaviour. Yet the vast majority of buildings in seismic regions is designed for ductile seismic response.

(ii) Strength assignment to elements is made proportional to traditionally defined, i.e., geometry-based, (EI), element stiffness. The displacements of elements with such predetermined stiffness, are assumed to vary with the actual strength assigned to them. This is then fallaciously taken as the yield displacement of the element on which displacement ductilities are based.

(iii) It is now widely recognised that a critical issue of the seismic design of ductile systems, is the estimation of element displacements associated with either performance or ultimate limit state criteria. Some codes also impose limits on the twist of elastic systems. Corresponding inelastic deformations are not addressed.

(iv) Emphasis in codes is placed on providing torsional and enhanced translational strengths, with disregard of the ensuing effects on displacements.

(v) No reference to the estimation of yield displacement of elements and that of the entire system is made in codes. Thereby the differences in displacement ductility demands on various elements, particularly those on critical edge elements, cannot be evaluated.

(vi) The fact that stiffness is strength-dependent, is not recognised. Thereby, when element dimensions are set, stiffness eccentricities, based on traditional strength-independent definition of element stiffness, are fixed. Subsequent changes of element strengths, often radical, may significantly affect stiffness eccentricity. This remains unrecognised.

(vii) Computed stiffness eccentricities are manipulated, using additional nominal eccentricities, in order to arrive at increased strength demands on elements, considered to be adversely affected. Strength eccentricities, which are affected by such manipulations, are not considered.

(viii) No attention is given to the facts that the astute designer can control strength eccentricity, and that stiffness eccentricity is a simple and quantifiable function of the strength eccentricity.

These comments should not be construed as a critique addressed to those researchers who, over several decades of dedicated work, developed techniques meeting the envisaged needs in seismic design. Some of the manipulations with nominal eccentricities, extensively covered in the relevant literature, were intended,
by means of parametric studies, to cater for inelastic displacement demands. Unfortunately these intentions are not transparent in current design provisions. Existing code recommendations grew out of the consideration of elastic behaviour only. As such they were widely endorsed by the design profession, including the author (5). The comments, and indeed the major part of this paper, address issues which emerged from recently identified fallacies in seismic design (6), and from corresponding promising exploration of seismic design strategies (7,8,13).

**CONCLUSIONS**

A definition of component yield displacement enables the properties of appropriate bi-linear approximations of force-displacement relationships for reinforced concrete components, to be defined. These lead to a redefinition of component stiffness.

Within rational limits, the nominal strengths of components, satisfying the strength requirements for the relevant element, may be freely chosen by the designer.

The summation of the stiffness and nominal strengths of components of an element is used to define its reference or nominal yield displacement. A similar summation of element properties enables to nominal yield displacement of the system, at the centre of mass, to be defined. These nominal yield displacements serve to establish displacement ductility relationships between components, elements and the system.

The order of twist resulting from static strength eccentricity and the likely effects of the rotary inertia of the mass during dynamic response, enables displacements of critical elements at the ultimate limit state to be gauged. In general, critical elements are those having the smallest yield displacement.

Because stiffness is postulated to be strength-dependent, stiffness eccentricities are related to strength eccentricities. The latter, however, are under the control of the designer.

The displacement at the appropriate limit state of the centre of mass may be based on a displacement profile of the system, controlled by the acceptable displacement of the critical element and the expected system twist. This in turn will define the displacement ductility capacity of the system, generally used in force-based seismic design strategies to estimate the required base shear strength.

The elimination of eccentricities does not necessarily result in more efficient system response.
ACKNOWLEDGMENTS

Informal dialogues with Professors A.V. Rutenberg of the Technion Israel Institute of Technology, W.K. Tso of McMaster University, Hamilton, Canada, M.J.N. Priestley of the University of California, San Diego, USA and H. Bachmann of the Swiss Federal Institute of Technology, Zürich, have greatly contributed to the clarification of issues, several of which are still considered to be controversial. The generous assistance of Dr A.J.Carr and C. Allington of the University of Canterbury, Dr F. Crisafulli of the National University of Cuyo, Argentina and D. Zamfirescu of the Technical University of Civil Engineering, Bucharest, Romania, who so willingly carried out extensive analytical work, is gratefully acknowledged.

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Fig. 1- Components and elements of a ductile system

Fig. 2- Bi-linear idealization of ductile component behaviour
Fig. 3- Idealized force-displacement relationships for an element with wall components

Fig. 4- Eccentricities arising in system with wall elements
Fig. 5- Torsionally unrestrained systems
Fig. 6- Eccentricities in a two-element torsionally unrestrained system

\[ V_1 = \frac{2V_{Ey}}{3}, \quad V_2 = \frac{V_{Ey}}{3}, \quad \lambda_i = \frac{V_{i,\text{nominal}}}{V_i} \]

Fig. 7- Effects of stiffness-proportional distribution of element strength

\[ \theta_{y} = \theta_{x} = 0.188A \]
\[ \Sigma k_i = 1.72; \Delta_y = 0.58 \]
\[ \mu_A = 5.62 \]
Fig. 8- Effects of zero strength eccentricity

Fig. 9- A system with moderate strength eccentricity
Remembering the Column Analogy

by M. A. Sozen

Synopsis:

To my friend and hero Professor Şükrü Muvaffect Üzümeri, a bouquet of thoughts and an invention of another hero, Professor Hardy Cross.

Keywords: column analogy; Hardy Cross; theory
Mete A. Sözen, Kettelhut Distinguished Professor of Structural Engineering at Purdue University, specializes in teaching and research related to concrete building and massive structures subjected to static and dynamic loads. Currently he chairs the National Research Council Committee for Oversight and Assessment of Blast-effects and Related Research and the National Science Foundation U.S.-Japan Technical Coordination Committee for Cooperative Research in Urban Earthquake Disaster Mitigation.

It has been said that it took an age to understand Aristotle and another age to forget him. Hardy Cross may have been forgotten even before he was understood. If his name is mentioned at all today, he is remembered through his contribution to the solution of bending moments in structural frames and flow in networks. Sometimes he is even dismissed as a sleepwalker who stumbled on the relaxation method for solution of linear simultaneous equations. Actually, his close friends and students have said that Cross tended to advise strenuously against the use of the moment distribution method because he considered it to be too exact for inexact structures. Cross’s influence on the profession of structural engineering is indelible and awesome. At the same time, it is subtle and easy to overlook. To get a flavor of his approach, consider his class notes for a course on indeterminate structures. Before he goes into explanations, he questions:

What is theory? It is perhaps worthwhile to call attention to the double use of the word “theory” in scientific discussions. In some cases it is used to mean a body or group of facts the truth of which is not questioned, in others it means a hypothesis which has strong evidence in its favor though its truth is still open to some question. Thus the theory of elasticity is a group of geometrical relations which are not open to debate, but the idea that time yield of the concrete will delay failure from temperature stresses in a concrete arch is a theory in quite a different sense. Other debatable points in indeterminate structures are not theories at all, but merely convenient assumptions: thus no one holds any theory that the modulus of elasticity is constant throughout an arch ring, the only question being whether such variations as do occur produce an important effect on the results.

Theory, to Cross, is the axiom. He does not think that plane geometry needs to be proven for plane continua. But the student is cautioned against mixing “theory” and “theory in a quite different sense.” All that is based on the observed is refutable. He expands on it.
Much confusion of thought has come from misuse of this term [theory]. We may further cite: as groups of facts not open to experimentation or debate, the theory of the elastic arch, the theory of continuous girders, the theory of deflection; as hypotheses strongly supported but as yet not fully proved. The theories of fatigue failure, the theory of earth pressure, the theory that the strength of concrete in a structure is the same as that shown by a cylinder in a testing machine or that rate of application of load is a negligible factor in producing failure, and finally, as misuses of the word, the “theory” that the moment of inertia of a concrete beam varies as $bd^3$, that the tension rods do not slip in concrete beams, that there is no distortion due to shear. The first group of “theories” is not debatable, the second depend usually on experimental verification, while in the case of the third the important question is how significant is the error. The data often needed in the third group are elementary: when these are available, deductive processes furnish a definite answer as to the importance of the error.

Taxonomy of the conceptual models was not his sole concern. His preoccupation was with engineering thinking and design in general. He never expressed it that way, but his constant quest was to determine whether or in which case an exact analysis of an approximate model was an approximate analysis of the exact model.*

He sought simplicity: “The analysis of a structure for continuity should be less complicated than the determination of anchorage and stirrup spacing....”** It is ironical that his wish came true, not because continuity analysis was simplified but because the determination of anchorage and stirrup spacing was made more difficult by illuminati who preferred the rigidity of rules to the flexibility of principles.

He revered statics: No indeterminate analysis – no structural analysis of any kind – is complete until the computer has satisfied himself

1. that the forces balance, at least within the accuracy of the computation used.

2. that he has not overlooked any forces.***

In our time, this wish was also fulfilled with the exception that tragically “himself” became “itself” and “he” became “it.” The following paraphrase from an announcement by an institution that prides itself on being at the cutting edge of

---

* All analyses are based on some assumptions which are not quite in accordance with the facts. From this, however, it does not follow that the conclusions of the analysis are not very close to the facts. (from Ref. 2, p. 3)

** Ref. 2, p. 2.

*** Ref. 2, p. 3
knowledge captures the intellectual fashion: “The advanced experimental capabilities will enable us to test and validate more complex and comprehensive analytical and computer numerical models to improve design and performance.” In the complex and comprehensive environment envisioned, will the computer (it, he, or she) check simple equilibrium? Fat chance!

Traduttori traditori. It is unfair to Cross to pretend to synthesize his view with a few quotations misplaced in time. The reader is urged to read references 1 through 4. If he/she has already done so, he/she is urged to return to them. They will give him/her different insights, always valuable, at different times. In the text below, a conceptual invention of his is discussed primarily to illustrate Cross’s creativity. How he arrived at his moment-distribution method can be understood, if with difficulty, in terms of deformations and the stiffness method. But his “Column Analogy” can only be classed as an artistic leap of imagination.

The Column Analogy

It is very interesting that Professor Cross started his lectures (Ref. 1) on indeterminate structures by referring to “three easily established principles:”

1. Column Analogy
2. Distribution of Moment
3. Virtual Work

Of the three principles he emphasized, the moment distribution survives, sometimes for the wrong reasons. Virtual work, being a theory and not developed but elegantly defined by Cross, has been a perennial. But the column analogy has been lost. It deserves recycling. The column analogy is essentially a theorem for finding indeterminate moments in a one-span restrained beam. What is important and useful about it is that it applies to straight and curved beams. It can be a very useful tool for determining flexural stiffness properties of nonprismatic beams.

To appreciate Cross’s leap of imagination, let us examine the simplest column-analogy application.

Consider a prismatic beam with fixed ends over a span $L$. It is loaded at mid-span by a concentrated load $P$. What are the restraining moments at the ends?

To solve the problem, Cross takes us to an imagined world. In that world, the beam is represented by a section (section of an imagined or analogous column) with depth $L$ and thickness $I/EI$ where $E$ is the Young’s modulus for the material of the beam and $I$ is its moment of inertia. This imaginary section responds linearly to an imagined load represented by the angle-change diagram, $M/EI$, distributed over the section just as the moment, $M$, is distributed over the span of a simply-supported beam.
The unit stresses at the ends, in the imagined world, are the moments sought:

\[ M_{end} = \frac{P_{analog}}{A_{analog}} = \frac{PL^2}{8EI} \cdot \frac{1}{L} \cdot \frac{EI}{L} = \frac{PL}{8} \]

- \( P_{analog} \): Total Load on the analogous section on \((1/2) (PL/4) (L)\)
- \( A_{analog} \): Area of analogous section or \(L/EI\)

If the load on the beam is uniform,

\[ M_{end} = \frac{2}{3} \frac{wL^3}{8EI} \cdot \frac{EI}{L} = \frac{wL^2}{12} \]

The operation is so simple that the correctness of the results appears to be coincidence, but it is not. Plates I through V present examples.

Plate I contains the column-analogy solution for a concentrated load at any distance \(aL\) from one end of a prismatic beam with fixed ends. The extension of this model to a similar beam with uniform load is described in Plate II. It is to be noted that the stresses in the imagined world are calculated from the familiar expression

\[ \alpha = \frac{P}{A} \pm \frac{Mc}{I} \]

hence the term “column analogy.” The column analogy has its best use in determining fixed-end moments and stiffnesses for nonprismatic beams. An application is demonstrated in Plate III.

The solution, given in Plate IV, for a concentrated load on a prismatic beam with one end fixed and the other free, takes us down the rabbit-hole to the Queen of Hearts. Cross wants us to imagine a section which has an infinite width over an infinitesimal length. The only thing that the observer can say is that she has seen the unimaginable and it works. The application is also a witness to Cross’s rare ability to associate images.

The example in Plate V is simply to show an application involving a frame. Though it is simple, its use cannot be recommended vis a vis other current methods. But it does demonstrate that Cross’s Analogy can be used for “one cell” frames, arches, and curved beams.


Concluding Remarks

Cross's written works are replete with jewels of thought. Some, contained in references 1 and 2, are reasonably well known. But it is not a waste of print to revisit his judgment about the results of analysis rendered in relation to stresses computed in arches in reference 3 (paper #8).

The investigations here recorded indicate:

(1) that for a large part (over one-half) of the stresses in an arch there can be practically no uncertainty arising from assumptions involved in the method of analysis used.

(2) that for the flexural stresses due to live load the true stresses cannot be predicted with absolute precision, because the stresses are a matter of chance.

(3) that the departure of the stresses existing in any arch from the values given by the usual methods of analysis can scarcely be greater than the variations in the quality of the concrete, and will most probably be very much less.

Beyond this it does not seem wise or profitable to draw conclusions, though others are apparently indicated by the data. The important fact is that any wide departure from the predicted values of the moments and thrusts in a concrete arch is not possible unless the variation in the properties of concrete is much greater than is commonly supposed. Within a narrow zone of uncertainty, then, the maximum moments and thrusts due to loads in a concrete arch are given without possible question by the "geometrical" (elastic) analysis. The zone of such uncertainty seems to have a width of about ±10 per cent; the zone of probable uncertainty seems to have a width of about ±5 per cent. The terms "true value" and "real value," however, are meaningless except as applied to a given arch under a given condition of loading and a given atmospheric condition; otherwise the reactions are a matter of chance.

The geometrical theory of analysis for arch reactions appears more dependable than the theory of flexure used to compute the fiber stresses produced by these reactions, and much more dependable than the concrete itself. It is not exact or precise, but it is a safe and convenient guide in design.

Socrates is supposed to have told Antisthenes the Cynic, "Your pride shows through the holes of your rags." Is it intellectual arrogance, confidence, sensibility, or humility to have spent years in building, analyzing, testing concrete arches and then to arrive at such a modest set of conclusions? Whichever it is, Cross ought to be a model to current and future writers.
Acknowledgments

I acknowledge my debt to my teachers at Urbana, Nate Newmark, Ralph Peck, Tom Shedd, and Chet Siess who created the magical-confrontational environment to understand Cross. They are not responsible for what I could not understand. Acknowledgment is also due Mrs. Linda Vail for her expert assistance in the preparation of the manuscript.

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HARDY CROSS
Brief Professional Record

1902 BA Hampden-Sydney College, Virginia
1903 BS Hampden-Sydney College, Virginia
1908 BS in CE Massachusetts Institute of Technology, Cambridge
1911 MS in CE Harvard University, Cambridge
1908-1910 Asst. Eng., Bridge Department, Missouri-Pacific Railway
1912-1918 Asst. Prof. of Civil Eng., Brown University
1918-1921 Practice in New York City and Boston
1921-1937 Professor of Structural Engineering, Univ.of Illinois, Urbana
1938-1951 Head, Department of Civil Eng., Yale University

List of Publications (1936)

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Continuous Frames of Reinforced Concrete, (N. D. Morgan, co-author) John Wiley & Sons, New York
Kidder’s Architects’ and Builders’ Handbook, Chapters X, XIV, XV, XIX.
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No. 203, Dependability of the Theory of Concrete Arches
No. 215, Column Analogy
No. 286, Analysis of Flow in Networks of Conduits or Conductors
The operation of determining the fixed-end moments in a prismatic beam subjected to a concentrated load is simple. It involves three steps.

Step 1. Determine the "load" on the analogous column.
Step 2. Determine the section properties of the analogous column.
Step 3. Determine the unit normal stresses in the analogous column.

Step 1: Load and Moment Acting on Section of Analogous Column

The entity defined by the area of the M/EI diagram for a simply supported beam is $P_{\text{analog}}$, the "load" acting on the section of the analogous column.

$$\begin{align*}
&P_{\text{analog}} = \frac{P \cdot L^2}{2E \cdot I} \cdot \alpha \cdot (1 - \alpha)\\
\end{align*}$$

The centroid of the load represented by the M/EI diagram is at

$$x = \frac{L}{3} \cdot (1 + \alpha)$$

from end A. The counterclockwise moment acting on the analogous column is

$$M_{\text{analog}} = P_{\text{analog}} \cdot \left( \frac{L}{2} - x \right)$$
substituting for \( P_{\text{analog}} \)

\[
M_{\text{analog}} = \frac{1}{12} \frac{P \cdot L^3}{E \cdot I} \cdot \alpha \cdot (1 - \alpha) \cdot (1 - 2 \cdot \alpha)
\]

Step 2: Section Properties of Analogous Column

\[
A_{\text{analog}} = \frac{1}{E \cdot I} \cdot L
\]

\[
I_{\text{analog}} = \frac{1}{12} \frac{E \cdot L^3}{1}
\]

Step 3: Normal Stresses on Section of Analogous Column

The normal unit stress \( \sigma_A \) at end A for the load on the analogous column section corresponds to the moment at in a beam with a concentrated load at \( \alpha L \) from end A.

\[
\sigma_A = \frac{P_{\text{analog}}}{A_{\text{analog}}} + \frac{M_{\text{analog}} \cdot c}{I_{\text{analog}}}
\]

where \( c = L/2 \).

Substituting for \( P_{\text{analog}}, M_{\text{analog}}, A_{\text{analog}}, I_{\text{analog}} \) and \( c \),

\[
\sigma_A = P \cdot L \cdot \alpha \cdot (\alpha - 1)^2
\]

To obtain a solution for the variation of the coefficients for end moments at A and B, we set

\[
P := 1 \quad L := 1
\]

\[
\text{Moment at A}(\alpha) := P \cdot L \cdot \alpha \cdot (1 - \alpha)^2
\]

Similarly,

\[
\text{Moment at B}(\alpha) := P \cdot L \cdot \alpha^2 \cdot (1 - \alpha)
\]
Variation of Fixed-End Moment, Coefficient of PL, with $\alpha$
Start with end moment at A for concentrated load $P$ at distance $\alpha L$ from end A

$$\frac{M_A}{PL} = \alpha \cdot (\alpha - 1)^2$$

$M_A$ = Moment at end A for a concentrated load at $\alpha L$ from end A
$P$ = Concentrated load at $\alpha L$ from end A
$L$ = Beam span
$\alpha$ = Ratio of distance to concentrated load from end A to beam span

Assume unit load $w = P/dL$ and integrate over beam span

$$M_A = w \cdot L^2 \int_0^1 \alpha \cdot (\alpha - 1)^2 \, d\alpha$$

$$M_A = \frac{1}{12} \cdot w \cdot L^2$$
FIXED-END MOMENTS IN A NONPRISMATIC BEAM
Three Segments with Different EI's

Centroid of Analogous Column Section

Definitions and Default Values

- Young's Modulus: $E := 1$
- Standard Beam Moment of Inertia (Segment 2): $I := 1$
- Beam Span: $L := 1$
- First Segment, Ratio of Length of Segment to Beam Span: $\beta := 0.2$
- Third Segment, Ratio of Length of Segment to Beam Span: $\gamma := 0.3$
- First Segment, Ratio of Moment of Inertia to That of Second Segment: $\xi_1 := 4$
- Third Segment, Ratio of Moment of Inertia to That of Second Segment: $\xi_2 := 2$
- Unit Load: $w := 1$
Determination of Centroidal Distance from End A

Segment $\beta$

Area

$$A_\beta := \frac{1}{\kappa_1 E_1} \beta \cdot L$$

First Moment about End A

$$M_\beta := A_\beta \frac{\beta}{2} L$$

Segment Standard

Area

$$A_s := \frac{1}{\kappa_1 E_1} (1 - \beta - \gamma) \cdot L$$

First Moment about End A

$$M_s := A_s \left[ (\beta) + \left( \frac{1 - \beta - \gamma}{2} \right) \right] L$$

Segment $\gamma$

Area

$$A_\gamma := \frac{1}{\kappa_2 E_1} \gamma \cdot L$$

First Moment about End A

$$M_\gamma := A_\gamma \left( 1 - \frac{\gamma}{2} \right) L$$

Centroid

$$x_o := \frac{(M_\beta + M_s + M_\gamma)}{A_\beta + A_s + A_\gamma}$$

$$x_o = 0.51$$

Sheet III: 2
Determination of Moment of Inertia

Segment 1

\[ I_{\beta 1} := \frac{1}{12} \cdot \frac{1}{k_1 \cdot E \cdot l} (\beta \cdot L)^3 \]

\[ I_{\beta 1} = 1.67 \times 10^{-4} \]

\[ I_{\beta 2} := A_{\beta} \left( \frac{\beta \cdot L}{2} - x_0 \right)^2 \]

\[ I_{\beta 2} = 8.43 \times 10^{-3} \]

Segment 2

\[ I_{s 1} := \frac{1}{12} \cdot \frac{1}{k_2 \cdot E \cdot I} \left[(1 - \beta - \gamma) \cdot L\right]^3 \]

\[ I_{s 1} = 1.04 \times 10^{-2} \]

\[ I_{s 2} := A_{s} \left[ \frac{\beta \cdot L + \left(1 - \beta - \gamma\right) \cdot L}{2} - x_0 \right]^2 \]

\[ I_{s 2} = 1.84 \times 10^{-3} \]

Segment 3

\[ I_{\gamma 1} := \frac{1}{12} \cdot \frac{1}{k_2 \cdot E \cdot I} (\gamma \cdot L)^3 \]

\[ I_{\gamma 1} = 1.12 \times 10^{-3} \]

\[ I_{\gamma 2} := A_{\gamma} \left[ \left(1 - \frac{\gamma}{2}\right) \cdot L - x_0 \right]^2 \]

\[ I_{\gamma 2} = 1.73 \times 10^{-2} \]

Moment of Inertia and Area for Analogous Column Section

\[ I_{\text{analog}} := I_{\beta 1} + I_{\beta 2} + I_{s 1} + I_{s 2} + I_{\gamma 1} + I_{\gamma 2} \]

\[ I_{\text{analog}} = 0.039 \quad \frac{1}{EI} \quad x_o = 0.51 \]

\[ A_{\text{analog}} := A_{\beta} + A_{s} + A_{\gamma} \]

\[ A_{\text{analog}} = 0.7 \quad \frac{1}{EI} \]

Sheet III: 3
"Load" on Analogous Column Section

\[ P_a := \frac{1}{2k_1} \left[ \int_0^b (\alpha - \alpha^2) \, d\alpha \right] + \frac{1}{2} \left[ \int_0^1 (\alpha - \alpha^2) \, d\alpha \right] + \frac{1}{2k_2} \left[ \int_1^{l-\gamma} (\alpha - \alpha^2) \, d\alpha \right] \text{ wL/EI} \]

Moment about End A on Analogous Column Section

\[ M_a := \frac{1}{2k_1} \int_0^b \alpha (\alpha - \alpha^2) \, d\alpha + \frac{1}{2} \int_0^1 \alpha (\alpha - \alpha^2) \, d\alpha + \frac{1}{2k_2} \int_1^{l-\gamma} \alpha (\alpha - \alpha^2) \, d\alpha \text{ wL}^2/EI \]

Clockwise Moment about Section Centroid

\[ M_a := M_a - P_a \cdot x_0 \]

Fixed-End Moment at A

\[ M_A := \frac{P_a}{A_{\text{analog}}} - \frac{M_a}{2} \]

Fixed-End Moment at B

\[ M_B := \frac{P_a}{A_{\text{analog}}} + \frac{M_a}{2} \]

\[ M_A = 0.111 \text{ wL}^2 \quad M_B = 0.083 \text{ wL}^2 \]
The area of the M/EI diagram for a simply supported beam is \( P_{\text{analog}} \), the "load" acting on the section of the analogous column.

\[
P_{\text{analog}} = \frac{P \cdot L^2}{2EI} \cdot \alpha \cdot (1 - \alpha)
\]

The centroid of the load represented by the M/EI diagram is at

\[
x = \frac{L}{3} (1 + \alpha)
\]

from end A. The counterclockwise moment acting on the analogous column is
PLATE IV

\[ M_{\text{analog}} = P_{\text{analog}} (L - x) \]

substituting for \( P_{\text{analog}} \)

\[ M_{\text{analog}} = \frac{1}{6} \cdot \frac{P L^3}{E I} \cdot \alpha \cdot (1 - \alpha) \cdot (2 - \alpha) \]

Step 2: Section Properties of Analogous Column

\[ A_{\text{analog}} = \infty \]

Because the centroid of the analogous-column section is at B

\[ l_{\text{analog}} = \frac{1}{3} \cdot \frac{P}{E I} L^3 \]

Step 3: Normal Stresses on Section of Analogous Column

Because \( A_{\text{analog}} = \infty \), the normal-stress term disappears,

\[ \sigma_A = \frac{M_{\text{analog}} \cdot L}{I_{\text{analog}}} \]

\[ \sigma_A = \frac{1}{2} \cdot P \cdot L \cdot \alpha \cdot (1 - \alpha) \cdot (2 - \alpha) \]

for \( P := 1 \quad L := 1 \)

\[ \text{Moment at } A(\alpha) := \frac{1}{2} \cdot P \cdot L \cdot \alpha \cdot (1 - \alpha) \cdot (2 - \alpha) \]

\[ \text{Variation of } M/PL \text{ at } A \text{ with } \alpha \]
Definitions and Default Values

Young's Modulus \( E := 1 \)
Moment of Inertia of Columns \( I_c := 1 \)
Moment of Inertia of Girder \( I_g := 1 \)
Span \( L := 2 \)
Height \( H := 1 \)
Load Applied at Center of Girder Span \( P := 1 \)
Properties of Analogous Section

Centroidal Distance from BC

\[ y_0 := \frac{2 \frac{H}{E \cdot I_c} \frac{H}{2}}{2 \frac{H}{E \cdot I_c} + \frac{L}{E \cdot I_g}} \quad y_0 = 0.25 \]

Moment of Inertia of Analogous Column Section

\[ I_{analog} := 2 \frac{1}{12 \cdot E \cdot I_c} \cdot H^3 + 2 \frac{H}{E \cdot I_c} \left( \frac{H}{2} - y_0 \right)^2 + \frac{L}{E \cdot I_g} \cdot y_0^2 \quad I_{analog} = 0.42 \]

Cross-Sectional Area of Analogous Column

\[ A_{analog} := \frac{2 \cdot H}{E \cdot I_c} + \frac{L}{E \cdot I_g} \quad A_{analog} = 4 \]

"Load" on Analogous Section

\[ P_{analog} := \frac{1}{8 \cdot E \cdot I_g} \cdot P \cdot L \quad P_{analog} = 0.25 \]

\[ M_{analog} := \frac{1}{8 \cdot E \cdot I_g} \cdot P \cdot L \cdot y_0 \quad M_{analog} = 0.063 \]

Moment at B

\[ \sigma_B := \frac{P_{analog}}{A_{analog}} + \frac{M_{analog} \cdot y_0}{I_{analog}} \quad \sigma_B = 0.1 \quad wL^2 \]

Moment at A

\[ \sigma_A := \frac{P_{analog}}{A_{analog}} + \frac{M_{analog} \cdot (y_0 - H)}{I_{analog}} \quad \sigma_A = -0.05 \quad wL^2 \]
Shear Strength of Circular Reinforced Concrete Columns

by M. P. Collins, E. C. Bentz, and Y. J. Kim

Synopsis:

Considering the very large number of circular concrete columns used to support buildings and bridges and the critical importance of ensuring that the shear strength of these members is sufficient to survive a possible earthquake, relatively few studies have been conducted on the shear strength of circular reinforced concrete columns. This paper summarizes the results of three experimental investigations in which a total of 15 large circular specimens were tested in shear. The paper also explains how analytical models based on the modified compression field theory can be used to predict the shear response of circular reinforced concrete columns.

Keywords: circular columns; ductility; high strength reinforcement; seismic resistance; shear strength
Michael P. Collins, FACI, is University Professor and Bahen-Tanenbaum Professor of Civil Engineering at the University of Toronto. He is a member of ACI Technical Activities Committee subcommittee on High-Performance Concrete and Joint ACI-ASCE Committee 445, Shear and Torsion. At Toronto, he has led a long-term research project aimed at developing rational procedures for the shear design of reinforced concrete structures.

ACI member Evan C. Bentz is an Assistant Professor of Civil Engineering at the University of Toronto. His research interests include the mechanics of reinforced concrete and the creation of practical tools that transfer reinforced concrete research into the engineering community.

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INTRODUCTION

For a reinforced concrete structure to survive a severe earthquake it is essential that the columns in the structure continue to support their gravity loads while undergoing the substantial horizontal deformations caused by the ground displacements. If the columns have inadequate shear strength they will lose their load carrying capacity at small deformations and a catastrophic collapse of the structure may result. A dramatic example of such a collapse is the destruction of the Hanshin Expressway during the 1995 Kobe earthquake (1). The 3.1 m diameter circular reinforced concrete columns supporting this elevated roadway contained an inadequate amount of hoop reinforcement. As a result the columns were not capable of deforming more than about 0.75% of their height and hence, could not survive the earthquake. See Fig. 1.

In assessing the response of reinforced concrete columns to flexure and axial load, engineers use a rational, simple, general method, the "plane sections" theory, which is capable of predicting not only the strength, but also the complete load-deformation response of reinforced concrete sections. Because of this, there is little disagreement among different engineers or different design codes as to the flexural strength of a given reinforced concrete section. There is, however, substantial disagreement as to the magnitude of the shear strength of structural elements and the reinforcement requirements needed to ensure ductile shear response. The disparity between the state-of-the-knowledge in flexure and the state-of-the-knowledge in shear can be seen if Chapter 10 Flexure and Axial Loads of the ACI Building Code (2) is compared with Chapter 11 Shear and Torsion. While Chapter 10 gives "Design assumptions" and "general principles and requirements", Chapter 11 gives a collection of restricted, empirical equations. The ACI shear strength equations were based primarily upon testing.
small rectangular reinforced concrete beams and so their applicability to large.
circular columns can be questioned.

During the last three decades a considerable amount of research has been
conducted worldwide with the aim of developing behavioral models for
reinforced concrete in shear comparable in rationality and generality to the plane-
sections theory for flexure (3). One group of such models is based on a collection
of assumptions about the behavior of cracked reinforced concrete that is known as
the Modified Compression Field Theory (MCFT) (4). As part of the development
of this general theory at the University of Toronto a number of circular reinforced
concrete sections have been loaded in shear. This paper will explain the key
aspects of the MCFT and will illustrate the use of models based on this theory to
predict the shear response of circular reinforced concrete columns. In addition the
paper will summarize the experimental results of the 15 circular sections tested at
the University of Toronto and will compare the results of these tests with the
predictions of the MCFT models, the ACI Code equations and with a new
analytical model proposed by researchers from the University of California, San
Diego (UCSD) (5).

MODIFIED COMPRESSION FIELD THEORY

Perhaps one of the reasons why the "shear problem" has been so difficult
to solve, is that the traditional type of shear test, while simple to perform, is
difficult to analyze. In such a test, see Fig. 2, the behavior of the member changes
from section to section along the shear span and also changes over the depth of
the beam. Thus, for example, if a relationship is sought between the magnitude of
the shear force and the strains in the stirrups, it will be found that the strains are
different for every stirrup and also differ over the height of the stirrup. In
developing the MCFT, experiments were conducted on elements subjected to
uniform stresses (Figs. 2 and 3). While these tests were more difficult to perform,
they were easier to analyze. The MCFT was developed by observing the load-
deformation response of a large number of reinforced concrete membrane
elements loaded in pure shear in the University of Toronto's Membrane Element
Tester (6,7) and Shell Element Tester (8). Figure 4 compares the calculated and
observed response for one of these elements called SE6 (8). The problem
addressed by the MCFT is to predict the relationship between the shear stress
applied to such an element and the resulting shear strain.

Cracked reinforced concrete transmits shear in a relatively complex
manner involving opening and closing of existing cracks, formation of new
cracks, and interface shear transfer at rough crack surfaces, significant tensile
stresses in the concrete, and great variation of local stresses in both the concrete
and reinforcement from point to point in the cracked concrete with the highest
reinforcement stresses and lowest concrete tensile stresses occurring at crack
locations. The MCFT attempts to capture the essential features of this behavior.
without considering all of the details. In lieu of following the complex stress variations in the cracked concrete, only the average values of the stresses (that is, stress averaged over a length greater than the crack spacing) and the stresses at the crack locations are considered.

Figure 5 summarizes the equilibrium, compatibility and stress-strain relationships used by the MCFT. In the relationships, $\theta$ is the angle between the $x$-axis, and the direction of the principal compressive average strain. Note that the average strains are measured over base lengths that are greater than the crack spacing. The basic simplifying assumption of both the compression field theory (9) and the MCFT (4) is that "the direction that is subjected to the largest average compressive stress will coincide with the direction that is subjected to the largest average compressive strain". For specified applied loads, the angle $\theta$, the average stresses and the average strains can be determined by solving the given equilibrium equations in terms of average stresses, the given compatibility equations in terms of average strains, and the given average-stress average-strain relationships.

The maximum shear stress that the element can resist may be governed not by the average stresses but rather by the local stresses at the crack locations. The need for checking stresses at crack locations is clear for the simple case of an element containing an amount of reinforcement, $\rho_x$, and subjected to pure axial tension in the $x$ direction. In such a case the relationship between the applied axial tensile stress, $f_x$, and the resulting axial average strain, $\varepsilon_x$, can be found from the equilibrium equation

$$f_x = \rho_x f_{xx} + f_i$$  \hspace{1cm} (1)

and from the average-stress average-strain relationships listed in Fig. 5 for the average tensile stress, $f_{xx}$, and the average tensile stress in the concrete, $f_i$. However, the maximum tensile stress that the member can resist is limited to $\rho_x f_{x,yield}$ by yielding of the reinforcing bars at the crack locations where the tension in the concrete is zero. For the more complex case of biaxial stress, both reinforcing steels may be yielding at the crack locations, neither reinforcing steel may be yielding, or only the weaker reinforcing steel may be yielding at the crack location. As can be seen from the equations in Fig. 5, shear stresses on the crack make it possible to redistribute the load from the weaker $y$-direction reinforcement to the stronger $x$-direction reinforcement.

In checking the conditions at a crack, the actual complex crack pattern is idealized as a series of parallel cracks all occurring at angle $\theta$ and spaced $s_0$ apart. It is assumed that for crack widths greater than about 0.05 mm, no significant tensile stresses can be transmitted normal to the crack. However, shear stresses $v_{ci}$ can be transmitted across the crack. The maximum possible value of $v_{ci}$ is related to the crack width, $w$, and the maximum aggregate size, $a$. 
PREDICTING THE SHEAR RESPONSE OF BEAMS AND COLUMNS

The MCFT is a procedure for predicting the stress-strain response of elements of reinforced concrete subjected to uniform biaxial stresses. In using this procedure to predict the shear response of a beam or a column, a number of additional simplifying assumptions will usually be required. The most accurate method requiring the least assumptions is to incorporate the relationships of the MCFT into a non-linear finite element program such as TRIX (10) and use this program to analyse the complete member, see Fig. 6. Such a method can treat both the “disturbed regions” near the locations of point loads and supports and the “beam regions” which are about the depth of the beam away from such disturbances.

Engineering beam theory assumes that plane sections remain plane and hence can study the response of a beam section without being concerned with the details of how the forces are introduced into the member. The MCFT has been used as the basis for a number of such sectional models. The most detailed of these is called the dual section model (11) and forms the basis for program Response-2000 (12). In this analysis, the biaxial stresses and strains and the manner in which they vary over the height of the member are considered, see Fig. 6. It is assumed that the net stress on planes parallel to the member axis is negligibly small. It is found that the inclination, \( \theta \), of the principal stress changes continuously over the depth of the member, becoming steeper near the flexural tension face and shallower near the flexural compression face. By considering longitudinal equilibrium between two adjacent cross sections, the model can determine the distribution of shear stresses over the cross section. It is found that as failure approaches there can be considerable redistribution of shear stresses resulting in higher shear stresses near the stiffer flexural compression side.

While program Response-2000 is capable of giving detailed predictions of the load-deformation response of sections loaded in shear, for design purposes, often all that is required is an estimate of the shear strength of a section. For this purpose a simple “hand calculation” has been developed. This method has been incorporated in the AASHTO LRFD Bridge Design Specifications (13). In this method (see Figs. 6 and 7) just one biaxial element within the web of the section is considered and the shear stresses and angle \( \theta \) are assumed to remain constant over the depth of the member. The shear strength of a non-prestressed section containing at least the minimum amount of transverse reinforcement can be expressed as

\[
V_n = V_c + V_s = \frac{1}{12} \beta_s f'c b_v d_v + \frac{A_s f_y}{s} d_v \cot \theta \text{ (MPa)}
\]  

(2)

where for a circular member the effective width, \( b_v \), can be taken as the diameter, \( D \), and the effective depth, \( d_v \), can be taken as 0.72\( D \). The minimum amount of transverse reinforcement specified by the AASHTO LRFD requirement is
where \( A_v \) is the cross-sectional area of transverse reinforcement (i.e., area of two legs for section shown in Fig. 7), and \( s \) is the spacing of this reinforcement.

The values of \( \beta \) and \( \theta \) as shown in Fig. 7 depend on the longitudinal strain, \( \varepsilon_x \), at mid-depth of the member and the shear stress on the member, \( \tau \), given by

\[
\tau = \frac{V_u}{b_sd_v}
\]

The value of \( \varepsilon_x \) can be determined by performing a plane sections analysis of the section subjected to moment \( M_u \), axial load \( N_u \) (tension positive, compression negative) and equivalent tension \( V_u \cot \theta \). To calculate the peak shear strength under monotonic loading it is not necessary to consider values of \( \varepsilon_x \) greater than 0.002. However, for evaluating the remaining shear capacity under large curvatures, values of \( \beta \) and \( \theta \) for \( \varepsilon_x \) equal to 0.005 and 0.010 have been included in Fig. 7.

As a simple hand calculation \( \varepsilon_x \) can be taken as half the strain in the flexural tension chord of an equivalent truss.

\[
\varepsilon_x = \frac{M_u/d_v + 0.5N_u + 0.5V_u \cot \theta}{2A_vE_v}
\]

where \( A_v \) is the area of longitudinal reinforcement on the flexural tension side of the member. If the value of \( \varepsilon_x \) is negative then the compressive stiffness of the concrete on the flexural tension side of the member must be taken into account by replacing the term \( A_vE_v \) by \( A_vE_v + A_cE_c \) where \( A_c \) is the area of concrete on the flexural tension side of the member.

Shear causes tensile stresses in the longitudinal reinforcement as well as in the transverse reinforcement. If a member contains an insufficient amount of longitudinal reinforcement, its shear strength will be limited by the yielding of this reinforcement. To avoid this type of failure, the longitudinal reinforcement on the flexural tension side of the member should satisfy the following requirement:

\[
A_vf_y \geq \frac{M_u}{d_v} + 0.5N_u + (V_u - 0.5V_s) \cot \theta
\]

The above equations were formulated for design and consequently are less convenient to use in analysing a given section. A simple spreadsheet to calculate
the shear strength of a given section in accordance with the AASHTO-LRFD specifications is given in Ref. 12. Note that in determining the shear strength by these specifications, the critical section for shear is taken to be at a distance of $d_v$ from the end of the column.

**ACI CODE EQUATIONS**

The shear strength of a non-prestressed section can be calculated according to ACI 318M-99 as

$$V_s = V_c + V_t$$

For members subjected to axial compression, $N_u$,

$$V_c = \frac{2}{12} \left( 1 + \frac{N_u}{14A_g} \right) \sqrt{f'\text{c}b_w d} \text{ (MPa)}$$

where $A_g$ is the gross area of the section.

For members subjected to axial tension,

$$V_c = \frac{2}{12} \left( 1 + \frac{0.3N_u}{A_g} \right) \sqrt{f'\text{c}b_w d} \text{ (MPa)}$$

where $N_u$ is negative for tension.

In either case, the shear contribution of the transverse reinforcement, $V_t$, is given by:

$$V_T = \frac{A_v f_v}{d} \leq \frac{8}{12} \sqrt{f'\text{c}b_w d} \text{ (MPa)}$$

For a circular member the web width, $b_w$, can be taken as $D$ and the depth, $d$, can be taken as $0.8D$.

**UCSD SHEAR MODEL**

While many thousands of rectangular reinforced concrete beams have been tested in shear, relatively few experiments have been conducted on the shear strength of circular reinforced concrete members. Using a database of 47 tests on circular members of which 20 specimens suffered "brittle shear failures," Kowalsky and Priestley recently presented (5) an "improved analytical
model" for the shear strength of circular reinforced concrete columns. In their method

\[ V_n = V_c + V_t + V_p \]  \hspace{1cm} (11)

The concrete mechanism term \( V_c \) is

\[ V_c = \alpha \beta \gamma \sqrt{f'_c} (0.8 A_v) \]  \hspace{1cm} (12)

The term \( \alpha \) accounts for the column aspect ratio and is given by

\[ 1 \leq \alpha = 3 - \frac{M}{V_D} \leq 1.5 \]  \hspace{1cm} (13)

The factor \( \beta \) accounts for the longitudinal reinforcement ratio and is given by

\[ \beta = 0.5 + 20 \rho_t \leq 1.0 \]  \hspace{1cm} (14)

The factor \( \gamma \) accounts for the decrease in concrete shear resisting mechanisms as the displacement of the column increases. For example

\[ \gamma = 0.29 \text{ for uniaxial displacement ductilities less than 2} \]
\[ \gamma = 0.05 \text{ for uniaxial displacement ductilities greater than 8} \]

As the displacement ductility increases from 2 to 8, \( \gamma \) decreases linearly from 0.29 to 0.05.

In terms of uniaxial curvature ductilities, \( \gamma \) is taken as 0.29 for values less than 3, as 0.05 for values greater than 15 and varies linearly between 3 and 15.

The contribution of the transverse reinforcement \( V_t \) is taken as

\[ V_t = \frac{\pi A_t f_y}{4} (D - c - cov) \cot \theta \]  \hspace{1cm} (15)

where \( c \) is the calculated depth of the compression zone under the axial compression, \( P \), and the moment, \( M \), and \( cov \) is the concrete cover to the outside of the longitudinal reinforcement. In the UCSD model, \( \theta \) is assumed to be 30°.

The contribution of the axial compression \( V_p \) is taken as

\[ V_p = \frac{P (D - c)}{2L} \]  \hspace{1cm} (16)

for axial compression and
for axial tension. In Eq. (16), $L$ is the length of the column from the critical section to the point of contraflexure.

OVERVIEW OF TORONTO EXPERIMENTS

Over the past 30 years, a total of 15 major experiments have been conducted at the University of Toronto to investigate the shear strength of circular reinforced concrete members. The first four experiments were conducted by Marcus Aregawi (14) as his 1974 M.A.Sc. thesis project under the supervision of S.M. Uzumeri and M.P. Collins. At that time, the shear failures of the large circular columns of the Macuto Sheraton Hotel during the Venezuelan earthquake of 1967 (15) and the many shear failures of circular bridge columns during the 1971 San Fernando (16) earthquake had shown that more information was needed on the shear response of such members. After searching the literature, Aregawi could find only four previous shear tests (17) on circular members containing transverse reinforcement. In Aregawi's tests, considerable care was taken to simulate the end conditions that might occur for a circular column in a multi-storey building (Fig. 8(a) and Fig. 11). The loads were introduced into the member via 305 mm thick diaphragms. The 457 mm diameter, 1524 mm long test sections were loaded such that the bending moment was zero at mid-length of the test zone. The four test specimens, which were thought of as being about one-half scale models of actual columns, had two different hoop spacings and two different concrete strengths (Fig. 9).

The second series of experiments on the shear strength of circular columns was conducted by Jameel Khalifa (18) as his M.A.Sc. thesis project in 1981. He used a test rig, developed by Sadler (19), which permitted a more convenient application of reversed loading and of axial load (Fig. 8(b)). The transverse loads were applied to the test specimen through two steel yokes clamped by bolts to the end blocks of the specimen. The 445 mm diameter test specimens had a clear length of 1270 mm before joining 445x445 mm square sections which then flared over 200 mm lengths into 560 mm wide by 610 mm deep end blocks. Prior to applying the transverse load, an axial compression of about 1000 kN was induced in the columns by post-tensioning a high-strength steel rod located in a 76 mm diameter ungrouted duct. The prime variables in these five tests were the amount of hoop reinforcement and the concrete cover (Fig. 9).

The third series of experiments were conducted by Young Joon Kim (20) as his M.A.Sc. thesis project in 2000. He used the same test rig as Khalifa but dispensed with the flared ends on the specimen (Fig. 8(c)). His six specimens had a diameter of 445 mm and a clear length of 1670 mm. The prime variables were
the amount of spiral reinforcement and the yield strength of the spiral reinforcement.

In the sections below, the results of these three series of experiments will be discussed and compared with the predictions of the analytical models.

AREGAWI'S EXPERIMENTS

The photographs in Fig. 10 illustrate the typical appearance of Aregawi’s specimens. The initial flexural cracks, which formed near the ends of the specimens, turned to become flexural-shear cracks as they approached mid-depth of the members. At higher loads, new diagonal cracks formed that were inclined at less than 30° to the longitudinal axis of the member. Near failure, the crack pattern was as shown in Fig. 10(a). The final failures involved yielding of the hoop reinforcement over most of the length of the specimen and opening of diagonal cracks near the end of the specimen. The maximum shear capacity of the members was reached when the displacement of the member was about 1.5% of the member’s clear length. The failure loads listed in Fig. 9 include a 6 kN allowance for the shear due to the self-weight of the specimen. Specimen WB2 was retested under reversed loads after reaching its monotonic failure shear of 437 kN. In this second test, it reached a maximum shear of 366 kN or 84% of its monotonic value. While being somewhat weaker, the specimen was very ductile holding most of its load until the drift of the specimen reached nearly 15% of the member length at which time the hoops ruptured. See Fig. 10(b).

Figure 12 compares the observed influence of concrete strength on the shear strength of specimens EB1 and EB2 with the predictions of Response-2000, the AASHTO-LRFD provisions, the ACI equations and the UCSD model. It can be seen that for these specimens, the ACI equations are rather conservative, the two MCFT models are fairly accurate and the UCSD model is somewhat unconservative. The four Aregawi experiments were included in the 47 column database used in the development of the UCSD model. In commenting on the relatively unconservative results for these specimens, Kowalsky and Priestley (5) stated that “because of the nonstandard boundary conditions, the results should probably be omitted.” The other 43 specimens in the database typically had heavily reinforced blocks or stiffened sections at each end of the test length. Further, 30 of these specimens were tested in single curvature with a clear length of less than twice the diameter of the column. St. Venant’s principle would suggest that the behavior of such squat columns will be strongly influenced by the boundary conditions.

In Fig. 13, the results of two TRIX analyses are used to illustrate the influence of boundary conditions on the observed shear strength of a column like specimen EB1. Figure 13(a) shows the predicted crack patterns and deformed shape when the column is tested in double curvature. The column is predicted to
fail at a shear of 360 kN with high straining of the hoops near the mid-height of the column. The bulging strain of the column at this level is calculated to be $7.37 \times 10^3$ (i.e., the diameter increases by 3.4 mm), while near the stiff ends, the bulging is reduced to $1.32 \times 10^3$ and $0.56 \times 10^3$. If rather than testing in double curvature a simpler single curvature test was chosen, it is predicted by the TRIX model that the results shown in Fig. 13(b) would be obtained. Although the $\frac{M}{V}$ ratio at the base of the column is the same in both models, the single curvature case is predicted to be 25% stronger. The stiff end blocks now restrict the bulging strain at mid-height of the short column to only $0.83 \times 10^{-3}$. These analytical results indicate that an empirical equation developed primarily on the basis of tests such as that shown in Fig. 13(b) can yield unconservative results when applied to the situation shown in Fig. 13(a).

KHALIFA'S EXPERIMENTS

The photograph in Fig. 14 illustrates the typical appearance of Khalifa's specimens. The relatively high axial compression applied to these columns ($\frac{N_i}{A_g}$ was about $0.3 f_c^2$) resulted in the diagonal cracks forming at about $20^\circ$ to the longitudinal axis of the member. As the shear was increased, new cracks formed at steeper angles (i.e., closer to $30^\circ$). At several stages during the testing of each specimen, see Fig. 15, the head displacement of the testing machine was held constant while cracks were marked and 198 mechanical strain readings were taken using targets attached to the hoops. These detailed strain readings were taken so that the patterns of strains could be compared with the predictions of the dual section modified compression field theory model which was being developed by Vecchio (21) at that time. Figure 16, which compares the observed and predicted shear strains for specimen SC3, is an example of the excellent agreement that was found. This figure shows both the manner in which the shear strains near one end of the member increase as the shear force increases and the predicted and observed variation of the shear strains over the length of the member for one particular value of shear.

As can be seen in Fig. 8 and Fig. 14, Khalifa's specimens were tested in a horizontal position. While the 1.27 m long test section weighed only about 5 kN, each end block and its attached steel loading yoke weighed about 25 kN. The failure shears listed in Fig. 9 have been calculated for the midpoint of the specimen and account for these dead loads. The dead loads of the loading yokes and the end blocks cause the magnitudes of the moments at the two ends of the specimen to be about 70 kNm different in magnitude with the south end of the specimen having the higher moment. For the failure loads listed in Fig. 9, the distances of the zero moment location from the south end of the five specimens are: 742 mm, 743 mm, 709 mm, 696 mm, and 712 mm. The higher moments near the south end of the specimen is the reason that the observed and predicted shear strains shown in Fig. 16 are somewhat higher at this end.
Figure 9 gives not only the failure loads of Khalifa's specimens but also the displacements at the peak load and the post-peak displacements when the shear had dropped to 80% of its peak value. As shown in Fig 15, the displacement, \( \Delta \), was measured as the deviation of one end block from a "tangent rod" fixed to the other end block. The displacements have been expressed in terms of a percentage of the clear length of the specimen, \( L \), where \( L \) equals 1270 mm. It can be observed that as the amount of hoop reinforcement is increased the deformation at the peak load increases and the post-peak slope of the load-deflection curve becomes flatter. See Fig. 15. Specimen SC4, after reaching its monotonic failure shear of 456 kN, was retested under reversed load. In this second test, it reached a maximum shear of 378 kN or 83% of its monotonic value.

Figure 17 compares the observed influence of amount of hoop reinforcement on the shear strengths of the specimens SC0, SC1, SC2 and SC3 with the predictions of Response-2000, the AASHTO LRFD provisions, the ACI provisions and the UCSD model. For these specimens, it can be seen that the ACI equations are a little conservative, the two MCFT models are fairly accurate and the UCSD model is somewhat unconservative. The observed crack patterns and hoop strains in specimens SC0, SC1, SC2, and SC3 at failure are summarized in Fig. 18.

KIM'S EXPERIMENTS

Priestley and Budek (21) have recently recommended the use of spirals of quarter inch diameter, seven-wire, prestressing strand as a convenient and effective form of confinement reinforcement of circular bridge piers. One question about the use of this very high strength transverse reinforcement is how the shear strength of circular columns containing such reinforcement is to be evaluated. Priestley and Budek suggest using the UCSD model but with the design yield strength of the strand reduced to about 1000 MPa. As Response-2000 accounts for the actual stress-strain characteristics of the transverse reinforcement, it is believed that it should be capable of predicting the shear behavior of this new type of circular column. To investigate these issues, the six columns of the YJC series were fabricated and tested. See Fig. 9. One specimen contained no transverse reinforcement, while the other five were reinforced with spirals made either from a #3 reinforcing bar (R) or from a quarter-inch seven wire strand (W). It is of interest to note that the yield force for the #3 bar was 31.6 kN while the yield (0.2% offset) force for the seven-wire strand was 39.7 kN (26% more than the bar).

The specimens were tested in the same rig as that used by Khalifa. To avoid the difference in moment at the two ends of the specimen, the dead weight of the loading yokes and the end blocks was supported by a small hydraulic jack under each end block (Fig. 19). The typical appearance of the specimens after
failure can be seen in Figs. 19 and 20. In the specimens reinforced with high strength wire, the large "bulging" strains caused the concrete cover to spall even prior to reaching the peak load.

The observed crack patterns and the measured crack widths for specimens YJC100R and YJC100W are summarized in Figs. 21 and 22. Note the very uniform pattern of cracking spreading from both ends of the specimens towards the middle. Comparing these two figures, it can be seen that the specimen reinforced with the less stiff seven-wire strand, YJC100W, had wider cracks (e.g., at \( V = 300 \) kN, the average crack width for YJC100R was 0.20 mm while for YJC100W it was 0.29 mm), had larger deformations and failed at a somewhat lower load (434 kN versus 479 kN). To facilitate the observation and recording of the location and orientation of the cracks, a square grid was marked on the surface of the specimens. The side lengths of each square were originally 116.5 mm, which was one twelfth of the circumference of the circular specimens. It can be seen from the post-peak crack patterns, that the shear failures did not occur by the opening of one inclined plane cutting across the specimen. Rather a series of S shaped cracks, which covered nearly the whole depth of the section, formed and widened.

Figure 23 compares the shear force-hoop strain relationships predicted by Response-2000 for YJC100R and YJC100W at close to the location of the critical section for shear. The calculations were performed for a \( M/V \) ratio of 0.485 m, that is, for a section about 3 squares from the end of specimen. The average transverse strain was computed for one quarter of the depth on each side of mid-depth. The experimental values were estimated by summing the measured crack widths over these lengths and then dividing by the length. Response-2000 predicts the pattern of behavior of the two specimens very well and also makes accurate estimates of the failure shears. YJC100R failed at 101% of the Response prediction while YJC100W failed at 91% of the Response prediction.

Figure 24 illustrates how the \( \beta \) and \( \theta \) values listed in Fig. 7 can be used to estimate the post peak reduction in shear capacity that will occur as the deformations imposed on the member are increased. For given member properties, Eq. (2) and the listed \( \beta \) and \( \theta \) values can be used to calculate the shear resistance of the member for each value of \( \varepsilon_x \). For example, if \( \varepsilon_x = 0.50 \times 10^{-3} \) and \( \omega / f' = 0.075 \) then \( \theta = 30.5^\circ \) and \( \beta = 2.59 \). With these values, the shear resistance of YJC200R, from Eq. (2), is:

\[
V_n = \frac{2.59}{12} \sqrt{40.4 \times 445 \times 0.72 \times 445 + \frac{2 \times 71 \times 445}{200} \times 0.72 \times 445 \cot 30.5^\circ} \\
= 195.6 + 171.9 = 367 \text{kN}
\]

Note that from Eq. (4) \( \omega / f' = 0.064 \) and hence, the correct values of \( \theta \) and \( \beta \) from Fig. 7 were chosen. Repeating these calculations for other values of \( \varepsilon_x \) the line
colling downward and labelled Eq. (2) in Fig. 24 is obtained. Thus, when $\varepsilon_x$ reaches a value of $10 \times 10^{-3}$ the shear resistance is predicted to have reduced to 105 kN. While the longitudinal steel and the concrete remain within the “elastic range” Eq. (5) can be used to determine the manner in which the longitudinal strain at mid-depth, $\varepsilon_x$, will increase as the moments, axial loads and shears increase. For specimen YJC200R this procedure indicates that the peak shear of 339 kN is reached when $\varepsilon_x$ equals $0.64 \times 10^{-3}$. At this time the curvature of the section is predicted to be $6.47 \times 10^{-3}$ rad/m. To estimate the remaining shear capacity when the curvature is increased to say about 10 times this value, the strain $\varepsilon_x$ at this curvature is determined from a plane sections analysis and then the shear corresponding to this $\varepsilon_x$ is calculated. As shown in Fig. 24 this residual shear strength for YJC200R at a curvature of $63.2 \times 10^{-3}$ rad/m is 145 kN or 43% of the peak shear capacity.

The observed load-deflection responses of all six specimens are shown in Fig 25. As would be expected, the addition of transverse reinforcement increased both the failure load and the post-peak ductility of the specimens. For the specimens reinforced with seven wire strands, the final failures involved rupturing of the strand. For the specimens with reinforcing bar spirals, the loading was stopped because the preset maximum displacement of the loading machine was approached.

After specimen YJC200R had passed its peak load of 323 kN and had reached a displacement of about 68 mm ($\Delta/L = 68/1670 = 4.1\%$), the specimen was unloaded and then load was reversed. See Fig. 26. After reaching a drift ratio of about 3% in the other direction at a shear of about 200 kN, the load was once more reversed. At a drift ratio of more than 10%, the load still exceeded 200 kN (i.e., 62% of the peak load). The appearance of the column at this stage is shown in Fig. 27. The load-deflection curve for YJC200R shown in Fig. 25(a) is the envelope curve from the load-deformation response.

Figure 28 illustrates the increase in shear strength that occurs as the amount of spiral reinforcement is increased and how this increase is influenced by the yield strength of the spiral. It can be seen that the use of an effective yield strength of 1000 MPa for the high strength strand in the UCSD model results in accurate estimates of the shear strength of the specimens with seven wire strand spirals. The Response-2000 predictions for these specimens are also reasonably accurate while the AASHTO LRFD predictions are a little unconservative if the full 1728 MPa value is used as the yield strength of the spirals.

**CALCULATED AND OBSERVED SHEAR STRENGTHS FOR TORONTO SPECIMENS**

Table 1 compares the calculated and observed shear strengths of the 15 circular reinforced concrete columns tested at the University of Toronto. In
addition to listing the failure shears, the table gives the shears corresponding to flexural failure at the ends of the specimen. In calculating these flexural failure loads, the beneficial effects of confinement on the concrete properties were ignored and hence the tabulated values should be conservative. The shear strengths calculated from the ACI equations, the University of California at San Diego (UCSD) analytical model and the three MCFT models are listed in Table 1. It can be seen that the ACI estimates for the shear capacities are somewhat conservative but relatively consistent with a coefficient of variation of 16.1%. The UCSD model is very consistent with a coefficient of variation of only 11.3%. However, for some specimens, the method overestimates the shear strength by about 25%. As already discussed it is believed that these unconservative predictions arise because the $\alpha$ term in the method, which accounts for the increased strength of squat columns, does not distinguish between the two cases illustrated in Fig. 13. Instead of using $MVD$ as the column aspect ration in Eq. (13) it would seem to be more appropriate to use $L_c/D$, where $L_c$ is the clear length of the column. If this change is made, the $\alpha$ factor for Aregawi's specimens goes from 1.33 to 1.00, for Khalifa's from 1.50 to 1.00 and for Kim's from 1.12 to 1.00. The overall average of experimental to computed shear strength ratio would then increase from 0.91 to 1.03 while the coefficient of variation would be reduced from 11.3% to 10.3%.

The three MCFT models are all reasonably accurate and consistent. As might be expected, the coefficient of variation decreases as the complexity of the model used increases. For the AASHTO LRFD model the spreadsheet calculations took less than two minutes each and gave a coefficient of variation of 12.8%. Inputting the member properties and running the program for the Response-2000 analyses took about three minutes for each specimen and gave a coefficient of variation of 10.1%. The non-linear finite element analyses were conducted with the aid of a pre-processor, TRIXCOL, and a post-processor, TRIXPOST. With these software packages each analysis took less than five minutes and resulted in the extremely low value of 6.0% for the coefficient of variation.

CALCULATED CAPACITY OF HANSHIN PIERS

The 3.1 metre diameter, lightly reinforced circular reinforced concrete columns supporting the Hanshin Expressway were designed in the late 1960's. At that time, the shear strength of such large columns would have been evaluated using expressions similar to the ACI code equations. These equations are based on test results from specimens about 0.3 m deep. It is now known that for lightly reinforced members, the shear stress at failure can decrease as the size of the members becomes larger (22). This so-called "size effect in shear" is accounted for by the MCFT but is ignored by the ACI and UCSD procedures. It is of interest to compare the predictions of these different methods for the shear behavior of the Hanshin piers.
The details of the piers that will be studied are illustrated in Fig. 29. Note that the reinforcement is reduced significantly at a section labeled B located 2.5 m above the base. It is the strength above this section that will be estimated by the various methods. This region of the piers contains only a small amount of hoop reinforcement with the parameter $A_{hr}/D_s$ being 0.30 MPa, which is 66% of the AASHTO LRFD minimum specified by Equation 3. The loading on the pier can be represented as an axial compression, $N$, a horizontal shear force, $V$, and a moment, $M$, applied at the top of the pier. The rotational inertia of the deck makes it possible for ground movements to generate moments at the top of the pier. For the purpose of the comparative calculations, it will be assumed that the point of zero moment in the pier will occur one sixth of the way from the top of the column.

Figure 30 illustrates the predicted shear capacities of the Hanshin piers as the axial compression varies from zero to 28,000 kN. It is estimated that the compression at the base of the column prior to the earthquake was about 14,000 kN. Ignoring any detrimental effects of shear, a flexural failure at section B would limit $V$ to about 6,900 kN when $N$ equals zero and 8,600 kN when $N$ equals 14,000 kN. These loads are labeled “Plastic Hinge Forms at Section B” in Fig. 29. It would be expected that such flexural failures would be ductile, enabling considerable energy to be dissipated prior to final collapse.

For an axial compression of 14,000 kN, the ACI Code would indicate that the pier has a shear strength of about 10,200 kN, which is considerably in excess of the flexural failure load. The UCSD model indicates an even higher capacity of 12,700 kN for a brittle shear failure. On the other hand, Response-2000 predicts a shear capacity of 7700 kN indicating that a brittle shear failure would be critical.

The load-deformation response of the Hanshin pier predicted by a TRIX97 analysis is illustrated in Fig. 31. For this pushover analysis it is predicted that the pier will suffer a brittle shear failure when the lateral deformation of the top of the column is only about 0.6% of the height of the column. The predicted deformed shape and crack pattern at failure, see Fig. 31, involves a zone of distress at the location where one third of the longitudinal reinforcement is terminated, combined with a band of diagonal cracks inclined at about 18° to the longitudinal axis of the pier.

**CONCLUDING REMARKS**

Considering the very large number of circular columns that are used to support buildings and bridges and the critical importance of ensuring that the shear strength of these members is sufficient to survive a possible earthquake it is surprising that relatively few studies have been conducted on the shear strength of circular reinforced concrete columns. The database recently published by
Kowalsky and Priestly (UCSD) listed just 20 tests involving brittle shear failures of such circular members. This paper has given the details for 4 of the Toronto experiments in the UCSD database, plus 11 more similar experiments conducted at the University of Toronto.

The shear design procedures used in the ACI Code were based primarily on tests of small rectangular beams. Hence, these procedures must be applied with caution in assessing the shear strength of large circular members where very little, or no, experimental data is available. The new analytical model proposed by researchers from the University of California, San Diego (UCSD) was fitted to the available data for circular reinforced concrete members and hence gives much more consistent estimates of the shear capacity of these members than the ACI equations. However because of the manner in which the UCSD model treats the influence of column aspect ratio its estimates can be unconservative for members loaded in double curvature such as those tested at the University of Toronto. Neither the ACI equations nor the UCSD model accounts for the so-called "size effect" in shear and hence it is possible that both procedures will overestimate the shear strength of large, lightly reinforced circular concrete members.

The comparisons of analytical computations and experimental observations given in this paper demonstrate that the modified compression field theory models are capable of accurately predicting the shear response of circular reinforced concrete columns. It should be emphasized that these models were not fitted to the available experimental results but are based on equilibrium, compatibility and the observed stress-strain characteristics of cracked reinforced concrete.

ACKNOWLEDGEMENT

The development of the modified compression field theory at the University of Toronto has been made possible by a series of grants from the National Sciences and Engineering Research Council of Canada. The authors would like to express their gratitude to this organization for their continuing long-term support. The seven-wire strand used in constructing some of the specimens was generously donated by Florida Wire and Cable Inc.

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http://www.ecf.utoronto.ca/~bentz/r2k.htm


Table 1 Comparison of calculated and observed shear strengths for the 15 University of Toronto circular columns

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Concrete Strength</th>
<th>$\frac{f_y}{f_{ck}}$</th>
<th>Shear for Flexural Failure (kN)</th>
<th>ACI 11-3.1 (kN)</th>
<th>UCSD 2000 (kN)</th>
<th>AASHTO 2000 (kN)</th>
<th>Response 2000 (kN)</th>
<th>TRIX97 (kN)</th>
<th>Observed Shear (kN)</th>
<th>Failure Ratios: Experimental/Computed</th>
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<tbody>
<tr>
<td>Aregawi</td>
<td></td>
<td></td>
<td></td>
<td>ANS11-3.1</td>
<td>ANS11-2000</td>
<td>ANS2000</td>
<td>ANS3000</td>
<td>ANS4000</td>
<td>ANS5000</td>
<td>ANS6000</td>
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<tr>
<td>EB1</td>
<td>39.3</td>
<td>0.63</td>
<td>624</td>
<td>280</td>
<td>434</td>
<td>350</td>
<td>343</td>
<td>360</td>
<td>363</td>
<td>1.30/0.84</td>
</tr>
<tr>
<td>WB1</td>
<td>39.3</td>
<td>0.85</td>
<td>624</td>
<td>317</td>
<td>475</td>
<td>393</td>
<td>407</td>
<td>470</td>
<td>467</td>
<td>1.47/0.98</td>
</tr>
<tr>
<td>EB2</td>
<td>27.6</td>
<td>0.63</td>
<td>599</td>
<td>252</td>
<td>383</td>
<td>322</td>
<td>291</td>
<td>360</td>
<td>348</td>
<td>1.38/0.91</td>
</tr>
<tr>
<td>WB2</td>
<td>27.6</td>
<td>0.85</td>
<td>599</td>
<td>289</td>
<td>423</td>
<td>364</td>
<td>352</td>
<td>410</td>
<td>437</td>
<td>1.51/1.03</td>
</tr>
<tr>
<td>Khalifa</td>
<td></td>
<td></td>
<td></td>
<td>ANS11-3.1</td>
<td>ANS11-2000</td>
<td>ANS2000</td>
<td>ANS3000</td>
<td>ANS4000</td>
<td>ANS5000</td>
<td>ANS6000</td>
</tr>
<tr>
<td>SC0</td>
<td>23.4</td>
<td>0</td>
<td>728</td>
<td>189</td>
<td>389</td>
<td>238</td>
<td>285</td>
<td>315</td>
<td>326</td>
<td>1.72/0.84</td>
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<td>SC1</td>
<td>19.3</td>
<td>0.42</td>
<td>710</td>
<td>239</td>
<td>421</td>
<td>257</td>
<td>263</td>
<td>315</td>
<td>324</td>
<td>1.36/0.77</td>
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<tr>
<td>SC2</td>
<td>23.0</td>
<td>1.53</td>
<td>726</td>
<td>430</td>
<td>599</td>
<td>467</td>
<td>459</td>
<td>490</td>
<td>478</td>
<td>1.11/0.80</td>
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<td>SC3</td>
<td>24.5</td>
<td>2.29</td>
<td>721</td>
<td>557</td>
<td>705</td>
<td>585</td>
<td>583</td>
<td>543</td>
<td>578</td>
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<tr>
<td>SC4</td>
<td>26.5</td>
<td>1.29</td>
<td>785</td>
<td>406</td>
<td>595</td>
<td>441</td>
<td>475</td>
<td>525</td>
<td>456</td>
<td>1.12/0.77</td>
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<td>Kim</td>
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<td>ANS11-3.1</td>
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<td>ANS4000</td>
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<tr>
<td>YJC CONT</td>
<td>30.8</td>
<td>0</td>
<td>466</td>
<td>147</td>
<td>224</td>
<td>187</td>
<td>176</td>
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<td>YJC200R</td>
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<td>0.71</td>
<td>479</td>
<td>281</td>
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<td>339</td>
<td>331</td>
<td>330</td>
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<tr>
<td>YJC150R</td>
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<td>309</td>
<td>397</td>
<td>371</td>
<td>373</td>
<td>390</td>
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<tr>
<td>YJC100R</td>
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<td>1.42</td>
<td>470</td>
<td>384</td>
<td>474</td>
<td>447</td>
<td>457</td>
<td>450</td>
<td>479</td>
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<td>0.89</td>
<td>474</td>
<td>293</td>
<td>318*</td>
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<td>339</td>
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<td>1.07/0.99</td>
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<td>YJC100W</td>
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<td>1.79</td>
<td>470</td>
<td>442</td>
<td>413*</td>
<td>503</td>
<td>462</td>
<td>390</td>
<td>434</td>
<td>0.98/1.05</td>
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<td></td>
<td></td>
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<td>ANS11-2000</td>
<td>ANS2000</td>
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<td>ANS4000</td>
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<td>0.91</td>
<td>1.08</td>
<td>1.08</td>
<td>1.01</td>
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<td>C.O.V (%)</td>
<td>16.1</td>
<td>11.3</td>
<td>12.8</td>
<td>10.1</td>
<td>6.0</td>
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<tr>
<td>min</td>
<td>0.98</td>
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<td>0.86</td>
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<td>max</td>
<td>1.72</td>
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<td>1.24</td>
<td>1.11</td>
<td></td>
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</tbody>
</table>

* calculated using $f_y = 1000$ MPa
Fig. 1 Shear failure of piers of Hanshin Expressway

Fig. 2 Testing reinforced concrete in shear
Fig. 3 The University of Toronto’s Shell Element Tester

Fig. 4 Comparison of calculated and observed shear response of membrane element SE6
Average Stresses:
\[
\begin{align*}
\sigma_x &= f_x + v \cot \theta - f_1 \\
\sigma_y &= f_y + v \tan \theta - f_1 \\
\sigma_2 &= v (\tan \theta + \cot \theta) - f_1
\end{align*}
\]

Stresses at Cracks:
\[
\begin{align*}
\sigma_{xc} &= f_c + v \cot \theta + v_{c1} \cot \theta \\
\sigma_{yc} &= f_y + v \tan \theta - v_{c1} \tan \theta
\end{align*}
\]

Crack Widths:
\[
\delta = \frac{v_c}{v_c} \varepsilon_1
\]

where
\[
s = \frac{1}{\left(\frac{\sin \theta}{s_x} + \frac{\cos \theta}{s_y}\right)}
\]

Geometric Conditions:
Average Strains:
\[
\begin{align*}
\varepsilon_x &= \left(\varepsilon_1 \tan^2 \theta + \varepsilon_2 / \left(1 + \tan^2 \theta\right)\right) \\
\varepsilon_y &= \left(\varepsilon_1 + \varepsilon_2 \tan^2 \theta / \left(1 + \tan^2 \theta\right)\right)
\end{align*}
\]
\[
\gamma_{xy} = 2 \left(\varepsilon_x - \varepsilon_y\right) / \tan \theta
\]
\[
\tan^2 \theta = \left(\varepsilon_x - \varepsilon_y\right) / \left(\varepsilon_y - \varepsilon_x\right)
\]

Average Stress-Average Strain Relationships:
Reinforcement:
\[
\begin{align*}
f_{sx} &= E_x \varepsilon_x \leq f_{x,yield} \\
f_{sy} &= E_y \varepsilon_y \leq f_{y,yield}
\end{align*}
\]

Concrete:
\[
\begin{align*}
f_2 &= \frac{f_{c'}}{0.8 + 170 \varepsilon_1 \left[2 \frac{\varepsilon_2}{\varepsilon'_c} - \left(\frac{\varepsilon_2}{\varepsilon'_c}\right)^2\right]} \\
f_1 &= \frac{f_{c'}}{1 + \sqrt{500} \varepsilon_1}
\end{align*}
\]

Allowable Shear Stress on Crack:
\[
\tau_c \leq 0.18 \sqrt{f_{c'}} + 0.31 + \frac{24 w}{d + 16}
\]

Fig. 5 A summary of the relationships used in the modified compression field theory.

Fig. 6 MCFT analysis of beams and columns.
### Typical Section

### Section Forces

### Longitudinal Strains

### Shear Stresses

**Fig. 7 Values of $\theta$ and $\beta$ for sections containing at least the minimum amount of transverse reinforcement**

<table>
<thead>
<tr>
<th>$\frac{v}{L}$</th>
<th>$\epsilon_x \times 1000$</th>
</tr>
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<tbody>
<tr>
<td>$\leq 0.075$</td>
<td>$\theta$</td>
</tr>
<tr>
<td>$\leq 0.100$</td>
<td>$\theta$</td>
</tr>
<tr>
<td>$\leq 0.126$</td>
<td>$\theta$</td>
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<tr>
<td>$\leq 0.160$</td>
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<tr>
<td>$\leq 0.176$</td>
<td>$\theta$</td>
</tr>
<tr>
<td>$\leq 0.200$</td>
<td>$\theta$</td>
</tr>
<tr>
<td>$\leq 0.260$</td>
<td>$\theta$</td>
</tr>
<tr>
<td>$\leq 0.260$</td>
<td>$\theta$</td>
</tr>
</tbody>
</table>
Fig. 8  University of Toronto tests on shear strength
| Investigator | Section | Loading Arrangement | L/D | Specimen | | | | | | | | Hoops | Failure | Failure |
|--------------|---------|---------------------|-----|----------|---|-------------|----------|---|-------------|----------|
|              |         |                     |     |          |   | $A_s$ (mm$^2$) | $f_s$ (MPa) | $s$ (mm) | $A_s$/$D_s$ (MPa) | $f'$ (MPa) | $N$ (kN) | $V$ (kN) | $I_p' / \Delta L'$ (%) | $0.81' / \Delta L'$ (%) |
| Aregawi      | 457 mm  |                     | 3.33| EB1      | 71| 141        | 203      | 0.63      | 39.3       | 0        | 363     | 1.51     | n/a     | 0.39     | 0.63     |
|              | 762     |                     |     | WB1      | 71| 141        | 152      | 0.85      | 39.3       | 0        | 465     | 1.51     | n/a     | 0.87     | 1.18     |
|              | 762     |                     |     | EB2      | 71| 141        | 203      | 0.63      | 27.6       | 0        | 348     | n/a      | n/a     | 2.20     | 2.99     |
|              | 762     |                     |     | WB2      | 71| 141        | 152      | 0.85      | 27.6       | 0        | 437     | n/a      | n/a     |          |          |
| Khalifa      | 445 mm  |                   | 2.85| SC0      | - | -         | -        | 0         | 23.4       | -1022    | 326     | 0.39     | 0.63     |
|              | 1000 kN |                   |     | SC1      | 34| 410        | 150      | 0.42      | 19.3       | -1017    | 324     | 0.87     | 1.18     |
|              | 635     |                     |     | SC2      | 100| 510       | 150      | 1.53      | 23.0       | -1083    | 478     | 1.18     | 2.60     |
|              | 635     |                     |     | SC3      | 100| 510       | 100      | 2.29      | 24.5       | -1085    | 578     | 2.20     | n/a      |
|              | 635     |                     |     | SC4*     | 100| 430       | 150      | 1.29      | 26.5       | -1050    | 456     | 1.42     | 2.99     |
| Kim          | 445 mm  |                   | 3.75| YJC Control | - | -         | -        | 0         | 30.8       | 0        | 212     | 0.48     | 0.54     |
|              | 835     |                     |     | YJC200R  | 71| 445        | 200      | 0.71      | 40.4       | 0        | 323     | 1.28     | 4.67     |
|              | 835     |                     |     | YJC150R  | 71| 445        | 150      | 0.95      | 36.0       | 0        | 411     | 1.74     | 3.29     |
|              | 835     |                     |     | YJC100R  | 71| 445        | 100      | 1.42      | 36.0       | 0        | 479     | 3.29     | 7.01     |
|              | 835     |                     |     | YJC200W  | 23| 1728      | 200      | 0.89      | 33.2       | 0        | 315     | 1.50     | 2.51     |
|              | 835     |                     |     | YJC100W  | 23| 1728      | 100      | 1.79      | 36.0       | 0        | 434     | 2.40     | 5.39     |

*clear cover over hoops for SC4 = 8 mm

Fig. 9 Details of Toronto tests
Fig. 10 EB1 at 92% of maximum load and WB2 post-failure under load reversal
Fig. 11 Testing arrangement for Aregawi's specimens

Fig. 12 Influence of concrete strength on shear strength
Fig. 13  Influence of boundary conditions on observed shear strength
Fig. 14  Appearance of specimen SC1 after peak load, $\Delta/L = 1.2\%$
Fig. 15  Comparison of load-deflection curves of specimens SC0 and SC2

Fig. 16  Comparison of observed and predicted shear strains for specimen SC3
Fig. 17 Influence of amount of transverse reinforcement on predicted and observed shear strengths of SC specimens
Fig. 18 Crack patterns, hoop strains and failure loads for specimens SC0, SC1, SC2 and SC3
Fig. 19  Specimen YJC100R after peak load. $\Delta/L = 8.7\%$

Fig. 20  Specimen YJC100W after peak load. $\Delta/L = 11.5\%$
Fig. 21 Crack patterns and crack widths (mm) for specimen YJC100R.
Fig. 22 Crack patterns and crack widths (mm) for specimen YJC100W
Fig. 23 Predicted and observed average hoop strains for specimens YJC100R and YJC100W

Fig. 24 Predicted reduction in shear capacity of JYC200R as deformations increase
Fig. 25 Observed load-deflection response of YJC specimens
Fig. 26 Reversed cyclic loading of YJC200R

Fig. 27 Appearance of YJC200R at end of test
Fig. 28 Influence of yield strength of transverse reinforcement on predicted and observed shear strengths of YJC specimens

Fig. 29 Details of Hanshin Expressway piers B501 – B510
Fig. 30 Predicted shear capacities of Hanshin Expressway piers

Fig. 31 Load-deformation response and crack pattern of the Hanshin Piers predicted by program TRIX97
New Seismic Design Provisions in Japan

by S. Otani, H. Hiraishi, M. Midorikawa, and M. Teshigawara

Synopsis: The seismic design requirements in the Building Standard Law of Japan were revised in June 2000 toward a performance-based design framework. The performance objectives are (a) life safety and (b) damage control of a building at two corresponding levels of earthquake motions. The design earthquake motion is defined in terms of acceleration response spectrum at engineering bedrock. The amplification of ground motion by surface geology and the soil-structure interaction must be taken into consideration. The response is examined by so-called “capacity spectrum method” by comparing the linearly elastic demand spectrum of design earthquake motions and the capacity curve of an equivalent single-degree-of-freedom (ESDF) system. The structure as designed is reduced to an ESDF system using a nonlinear static analysis under monotonically increasing horizontal forces. Equivalent damping is used to modify the demand spectrum taking into account the energy dissipation capacity of a structure at the prescribed limit states.

Keywords: capacity curve; capacity spectrum; damage initiation; demand spectrum; life safety; limit states; performance-based design; pushover analysis; SDF system; seismic design
INTRODUCTION

The first Japanese building law, Urban Building Law, was promulgated in 1919, to regulate building construction in six major cities at the time. Building Law Enforcement Regulations specified structural requirements. The regulations introduced a design seismic coefficient of 0.10 in 1924 after the 1923 Kanto (Tokyo) Earthquake.

Building Standard Law, applicable to all buildings throughout the country, was proclaimed in May 1950. The objectives were to safeguard the life, health, and properties of people by providing minimum standards concerning the site, structure, equipment, and use of buildings. The law outlines the basic requirements, and the technical details are specified in the Building Standard Law Enforcement Order (Cabinet Order) and in a series of Notifications by Minister of Construction.

The law requires that design documents and drawings should be submitted to a municipal government for the confirmation of the design documents to satisfy the legal provisions. This requirement made the code prescriptive because building officials must be able to judge the conformity. The structural design was based on the allowable stress design framework, using different allowable stresses for
long-term (gravity) loading and short-term loading (seismic and wind forces). The structural requirements were revised from time to time after devastating earthquake disasters.

The seismic design provisions of the Building Standard Law Enforcement Order were significantly revised in 1981; major revisions in seismic design were listed below:

1. Structural calculation is required for (a) story drift under design earthquake forces, (b) lateral stiffness distribution along the height, (c) eccentricity of mass and stiffness in plan, and (d) story shear resisting capacity at the formation of a collapse mechanism,

2. Earthquake resistance is specified (a) in terms of story shear rather than horizontal floor forces, (b) as a function of fundamental period of a building and soil type, and (c) separately for the allowable stress design and the examination of story shear resisting capacity, and

3. Required story shear resisting capacity is varied for construction materials and with the deformation capacity of hinging members under earthquake forces.

The Building Standard Law was substantially revised in 1998 to introduce a performance-based design procedure to the existing prescriptive framework. New technical specifications in the form of the Law Enforcement Order and a series of Notifications of Ministry of Construction were issued in June 2000, including the definition of performance objectives for design limit states and the specifications for verification methods.

This paper introduces briefly the concept and framework of new provisions with emphasis on earthquake resistant building design.

PERFORMANCE-BASED REQUIREMENTS

The performance-based requirements in building codes are expected to expand the scope of structural design, especially for the application of new materials, construction and structural systems. It is further expected to remove international trade barriers in the construction markets and to encourage the engineer to develop new construction technology and engineering.

The new procedure, introduced in the Building Standard Law, deals with the evaluation and verification of performance (response) at a given set of limit states under gravity loads, snow loads, wind and earthquake forces. The structural specifications include the method of structural calculation, the quality control of construction and materials, durability of buildings, and the performance of nonstructural elements. For continuity, the design loads and forces were maintained at the same levels as the existing provisions. However, a new format of seismic design forces was introduced; i.e., the response acceleration spectrum of
the earthquake motion is specified at engineering bedrock, having shear wave velocity in the range of several hundred meters per second. The amplification of ground motion by surface geology above the engineering bedrock must be duly taken into account in defining the design ground motion at the free surface.

Two limit states are considered as the minimum standards for building structures to safeguard the life and property of the inhabitants; i.e., (a) life safety and (b) damage initiation. Two sets of design loadings are considered, each having a different probability of occurrence. The structural damage should be prevented in events that may occur more than once in the lifetime of the building for the protection of properties; i.e., the damage must be prevented in structural frames, members, interior and exterior finishing materials. A return period for such events may be 30 to 50 years. For the protection of human life, no story of the building should collapse under extraordinary loading conditions. The maximum possible earthquake motion level is determined on the basis of historical earthquake data, recorded strong ground motions, seismic and geologic tectonic structures and identified activities of active faults. A return period of several hundred years is assumed in defining the design earthquake motions.

DEFINITION OF DESIGN EARTHQUAKE MOTIONS

The design seismic forces were previously specified in terms of story shear forces as a function of building period and soil conditions. In other words, the design seismic forces were specified as the response quantities of a structure without defining the ground motion and the response amplification by a specific structure.

In the revised provisions, the acceleration response spectrum $S_d(T)$ of free surface ground motion at a 5% damping factor is represented as follows;

$$S_d(T) = Z \cdot G_s(T) \cdot S_0(T)$$

where $Z$ is the seismic zone factor, $G_s(T)$ is the amplification factor by surface geology, $S_0(T)$ is the response spectral acceleration ordinate of ground motion at exposed engineering bedrock, and $T$ is the period of a building in sec at the damaged state.

Earthquake Motion at Engineering Bedrock

The ground motion is defined by an acceleration response spectrum at exposed engineering bedrock. The engineering bedrock is defined as a thick soil stratum whose shear wave velocity is on the order of 400 m/s or higher. The exposed
engineering bedrock is used in the definition to eliminate the effect of the surface geology on the ground motion.

The acceleration response spectrum at the engineering bedrock consists of a uniform acceleration portion in a short period range and a uniform velocity portion in a long period range. For the sake of continuity in seismic design provisions, the intensity of ground motion at the engineering bedrock was established to yield design seismic forces that are comparable to those for intermediate soil condition before the revision of the Building Standard Law. Therefore, the constant acceleration and velocity response spectral ordinates for the life-safety events are specified to be 8.0 m/sec² and 815 mm/sec, respectively, at a 5% damping ratio on exposed engineering bedrock.

The design spectrum $S_a(T)$ at exposed engineering bedrock is shown in Fig. 1 or given by Eq. (2) for the life-safety limit state:

$$S_a(T) = \begin{cases} 
3.2 + 30T & \text{for } T < 0.16 \\
8.0 & \text{for } 0.16 \leq T < 0.64 \\
\frac{5.12}{T} & \text{for } 0.64 \leq T 
\end{cases}$$

where $S_a(T)$ is the spectrum ordinate (m/sec²), $T$ is the period (sec) of the building at the life-safety limit state. The design spectrum for the damage-initiation limit state is to be reduced to one-fifth of the spectrum for the life-safety limit state.

**Amplification Factor for Surface Geology**

The amplification of ground motion by surface geology is evaluated using the geological data at the site. The nonlinear amplification of ground motion by surface soil deposits is estimated using the equivalent linearization technique. A lumped-mass shear-spring model (Fig. 2) was used to represent a layer of soil deposits; the stiffness of soil layers was represented by secant shear modulus at maximum response shear strain under the first mode oscillation. The shear modulus reduction factors and damping factors are specified for cohesive and sandy soils at various shear strain levels in Notification 1457 of the Ministry of Construction.

The equivalent shear wave velocity and impedance of an equivalent uniform soil layer were estimated for the equivalent linear shear model. The amplification of ground motion by a uniform soil layer above the engineering bedrock is obtained by considering one-dimensional wave propagation in the frequency domain. The
dynamic amplification function by the surface geology is modified by connecting
the two peak points of the first and second modes by a straight line. The basis and
reliability of this procedure was examined for different soil deposits and the
results were reported in Ref. (1).

The following expressions are given for the amplification function $G_s(T)$ by
surface geology in Notification 1457 of the Ministry of Construction;

$$ G_s = G_{s2} \frac{T}{0.8T_2} \quad \text{for } T \leq 0.8T_2 $$

$$ G_s = G_{s2} + \frac{G_{s1} - G_{s2}}{0.8(T_1 - T_2)} \cdot (T - 0.8T_2) \quad \text{for } 0.8T_2 < T \leq 0.8T_1 $$

$$ G_s = G_{s1} \quad \text{for } 0.8T_1 < T \leq 1.2T_1 $$

$$ G_s = G_{s1} + \frac{1.0 - 1.2}{1.2T_1} - 0.1 \cdot \left( \frac{1}{T} - \frac{1}{1.2T_1} \right) \quad \text{for } 1.2T_1 < T $$

where $T_1$ and $T_2 = (T_1/3)$ are the dominant periods of surface soil deposits, $G_{s1}$
and $G_{s2}$ are the amplification factors of the soil deposits in the first and second
modes. The first-mode period is estimated on the basis of the depths, shear moduli
at strain amplitude and mass density of the soil layers. Empirical formulae are
provided to determine the first- and second-mode amplification factors $G_{s1}$ and
$G_{s2}$ considering the hysteretic energy dissipation and impedance ratios.

If the detailed analysis is not used, the following simple expression can be used;
1) For soil type I (soil layer consisting of rock, stiff sand gravel, and
pre-Tertiary deposits);

$$ G_s = 1.5 \quad \text{for } T < 0.576 $$

$$ G_s = \frac{0.864}{T} \quad \text{for } 0.576 \leq T < 0.64 $$

$$ G_s = 1.35 \quad \text{for } 0.64 \leq T $$

where $T$ is the period of a structure (sec).

2) For soil type II (soil layer other than types I and III) and type III (alluvium
layer mainly consisting of humus and mud whose depth is more than 30 m, or
filled land of more than 3 m deep and worked within 30 years);
\[ G_s = 1.5 \quad \text{for} \quad T < 0.64 \]
\[ G_s = 1.5 \left( \frac{T}{0.64} \right) \quad \text{for} \quad 0.64 \leq T < 0.64 \left( \frac{g_v}{1.5} \right) \]  \( (5) \)
\[ G_s = g_v \quad \text{for} \quad 0.64 \left( \frac{g_v}{1.5} \right) \leq T \]

where \( g_v = 2.03 \) for type II soil and 2.7 for type III soil.

**Seismic Zoning Factor**

The advancement in the simulation methodology and the collection of strong motion records in near-source regions in this decade made it feasible to estimate realistic intensity and characteristics of earthquake motions at engineering bedrock. The seismic zone factor evaluates (a) relative difference in expected ground motion parameters, such as peak ground acceleration or peak ground velocity for strong and intermediate intensity earthquake motions, and (b) frequency content for acceleration and velocity waveforms. Two levels of ground motion are defined; i.e.,

1) Large earthquake: largest annual maximum in 500 years, and
2) Intermediate earthquake: 10\textsuperscript{th} largest annual maximum in 500 years.

The historical earthquake data over the last 500 years in Japan and fault parameters identified for major earthquakes were used in the study.

Two empirical attenuation formulae for near-source and far-field events estimated peak ground acceleration and velocity amplitudes for various subdivided regions of the country as a function of earthquake magnitudes, distance to the fault plane and average shear wave velocity of the upper 30-m surface soil deposit. Regional seismic maps were drawn for peak ground acceleration and velocity amplitudes, separately, expected in 50 years and several hundred years.

The expected intensity levels estimated for the 500-year return period are comparable with or slightly larger than the level of seismic force currently in use for type-II soil. Therefore, the seismic zone factors, varying from 0.7 to 1.0, in the previous Building Standard Law Enforcement Order are maintained in the revised design requirements.

**VERIFICATION OF STRUCTURAL PERFORMANCE**

The performance of a building is examined at the two limit states under the two levels of design earthquake motions; i.e., (a) damage-initiation limit state and (b) life-safety limit state.
The damage-initiation limit state is attained when the allowable stress of materials has been reached in any member or when the story drift reaches 0.5 percent of the story height at any story. The initial elastic period is used for a structure.

The life-safety limit state is attained when the structure cannot sustain the design gravity loads in any story under additional horizontal deformation; i.e., a structural member has reached its ultimate deformation capacity. The ultimate deformation of a member must be calculated as the sum of flexure and shear deformations of the member and deformation resulting from the deformation in the connection to adjacent members. The ultimate flexural deformation $\theta_{\mu}$ may be estimated as

$$\theta_{\mu} = \frac{\phi_y}{3} a + (\phi_u - \phi_y) \ell_p (1 - \frac{\ell_p}{2a})$$

where $\phi_y$ is the curvature when allowable stress is first reached in the member, $\phi_u$ is the curvature at the maximum resistance, $\ell_p$ is the length of plastic region, $a$ is the shear-span or one-half of clear member length.

Equivalent SDF Modeling

A multi-story building structure is reduced to an equivalent single-degree-of-freedom (ESDF) system (Fig. 3) using the results of a nonlinear static analysis under constant-amplitude gravity loads and monotonically increasing horizontal forces (often called a “pushover analysis”). The distribution of story shear coefficients (design story shear divided by the weight supported by the story) is defined by the following expression, consistent with the previous Building Standard Law;

$$A_i = 1 + \left( \frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \cdot \frac{2T}{1 + 3T}$$

$$\alpha_i = \frac{\sum_{j=1}^{i} W_j}{\sum_{j=1}^{n} W_j}$$

where $W_i$ is the sum of dead and live loads at $i$-th floor, and $T$ is the fundamental period of the structure. Horizontal force acting at a floor level is calculated as the difference of story shears immediately above and below.
The deflected shape of the pushover analysis is assumed to represent the first-mode shape of oscillation. The deflected shape does not change appreciably with the distribution of horizontal forces along the structural height; therefore, the constant force distribution is used during the pushover analysis.

The modal participation factor $\Gamma_1$ is necessary to relate the SDF response and modal response of a structure under horizontal ground motion; i.e.,

$$\Gamma_1 = \frac{\{\phi\}_1^T [m] \{1\}}{\{\phi\}_1^T [m] \{\phi\}_1}$$ (9)

where $\{\phi\}_1$ is the first-mode shape vector (normalized to the roof-level displacement), $[m]$ is the lumped floor mass matrix (diagonal matrix), and $\{1\}$ is a vector whose elements are unity.

For a spectral response acceleration $S_A(T)$ and displacement $S_D(T)$ at the first-mode period and damping, the first-mode inertia force vector $\{f\}_1$ and displacement vector $\{d\}_1$ are defined as follows;

$$\{f\}_1 = [m] \{\phi\}_1 \Gamma_1 S_A(T)$$
$$\{d\}_1 = \{\phi\}_1 \Gamma_1 S_D(T)$$ (10)

For the mode shape vector normalized to the roof-level displacement, the roof displacement $D_{R1}$ is equal to $\Gamma_1 S_D(T)$. The first-mode base shear $V_{B1}$ is the sum of inertia forces at each floor level. For the lumped floor masses, the base shear is calculated as follows;

$$V_{B1} = \{1\}^T \{f\}_1$$
$$= \{1\}^T [m] \{\phi\}_1 \Gamma_1 S_A(T)$$
$$= M_1^* S_A(T)$$ (11)

where $M_1^*$ is the effective modal mass as given below;

$$M_1^* = \Gamma_1 \{\phi\}_1^T [m] \{1\}$$ (12)

The effective mass must be not less than 0.75 times the total mass of the structure.
In general, the roof-level displacement $D_R$ and the base shear $V_b$ are governed by the first-mode response. Therefore, the base shear $V_b$ divided by the effective modal mass $M_1^*$ and the roof displacement $D_R$ divided by the participation factor $\Gamma_1$ represent the response spectral acceleration $S_a(T)$ and displacement $S_d(T)$:

$$S_a(T) = \frac{V_b}{M_1^*}$$
$$S_d(T) = \frac{D_R}{\Gamma_1}$$

(13)

The relation between $S_d(T)$ and $S_a(T)$ may be plotted for a structure under monotonically increasing horizontal forces. The relation is called the “capacity curve” of the structure (Fig. 4).

The effective first-mode period $T_e$ of a structure at a loading stage is approximated by the following relation (Fig. 4):

$$T_e = 2\pi \sqrt{\frac{S_d(T)}{S_a(T)}}$$

(14)

It should be noted that the effective period changes with the amplitude of horizontal forces and displacements. The effective period may be modified by the following factor $r$ taking into account the effect of soil-structure interaction:

$$r = \sqrt{1 + \left(\frac{T_{ro}}{T_e}\right)^2 + \left(\frac{T_{sw}}{T_e}\right)^2}$$

(15)

where $T_{sw}$ is the period of sway oscillation, and $T_{ro}$ is the period of rocking oscillation. The sway and rocking periods must be evaluated for the stiffness of soil corresponding to the excitation level of the super-structure.

Equivalent Damping Ratio

Equivalent damping ratio for the first mode is prescribed to be 0.05 for the damage-initiation limit state because the state of a structure remains elastic at this stage.
Equivalent viscous damping ratio $h_{eq}$ at the life-safety limit state is defined by equating the energy dissipated by hysteresis of a nonlinear system and the energy dissipated by a viscous damper under resonant steady-state vibration:

$$h_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W}$$  \hspace{1cm} (16)

where $\Delta W$ is the hysteresis energy dissipated by a nonlinear system during one cycle of oscillation, and $W$ is the elastic strain energy stored by a linearly elastic system at the maximum deformation (Fig. 5).

Naturally, such equivalence does not hold in the case of response under random earthquake excitation. The equivalent damping ratio must be effectively reduced to correlate the maximum response of an equivalent linear system and a nonlinear system under a random earthquake excitation.

A series of nonlinear SDF systems having different hysteretic characteristics (bilinear, degrading bilinear, slip bilinear and Takeda models) and equivalent linear SDF systems were analyzed under natural and artificial earthquake motions in Ref. (2). Analytical parameters include elastic periods and yielding resistance. The maximum response of equivalent linear SDF systems was reported to be comparable to that of nonlinear systems when the equivalent damping ratio is reduced to approximately 70 percent of that calculated by Eq. (16). The reduction was observed to increase with ductility demand of nonlinear systems.

Therefore, the equivalent damping ratio $m_{h_{eq}}$ of a structural member $i$ is estimated by the following expression;

$$m_{h_{eq}} = \frac{1}{4} \left(1 - \frac{1}{\sqrt{\mu}}\right)$$  \hspace{1cm} (17)

where $\mu$ is the ductility factor of the member attained at the life-safety limit state of the structure. If the hysteresis shape of a member exhibits a slip-type characteristic, Eq. (18) must be used;

$$m_{h_{eq}} = \frac{1}{5} \left(1 - \frac{1}{\sqrt{\mu}}\right)$$  \hspace{1cm} (18)

The equivalent damping ratio of an SDF system is estimated as the weighted average with respect to strain energy;
where $w_i$ is the strain energy stored in member $i$ at the life-safety limit state.

The equivalent damping ratio may be modified considering the soil-structure interaction effect:

$$h_{eq} = \frac{1}{r^3} \left( h_{sw} \left( \frac{T_{sw}}{T_e} \right)^3 + h_{ro} \left( \frac{T_{ro}}{T_e} \right)^3 + h_b \right)$$

where $r$ is the period modification factor defined in Eq. (15), $h_b$ is the damping ratio of the super-structure, $h_{sw}$ is the damping ratio of sway oscillation of surface soil layers corresponding to shear strain level considered, but the value is limited to 0.30, $h_{ro}$ is the damping ratio of rocking oscillation or surface soil deposits corresponding to shear strain level considered, but the value is limited to 0.15, $T_{sw}$ and $T_{ro}$ are the sway and rocking oscillation periods at the life-safety limit state, and $T_e$ is the period of a structure at the life-safety limit state as defined by Eq. (14).

Demand Spectrum

Response spectral displacement $S_D(T)$ is estimated from the linearly elastic design acceleration response spectrum $S_A(T)$ at the free surface by dividing the spectral acceleration by the square of a circular frequency ($= T / 2\pi$);

$$S_D(T) = \left( \frac{T}{2\pi} \right)^2 S_A(T)$$

The demand spectrum (Ref. 3) is constructed by plotting an SDF response acceleration in the vertical axis and corresponding displacement in the horizontal axis along a straight line with slope equal to the square of a circular frequency. The period (circular frequency) of SDF systems was gradually varied in the plot (Fig. 6).

Demand spectra are prepared for a damping ratio of 0.05 up to the damage-initiation limit state, and for an equivalent damping ratio at life-safety limit state. For the life-safety limit state, the response spectral acceleration and displacement are reduced by the following factor $F_h$: 
where \( h_{eq} \) is the equivalent damping ratio defined by Eq. (19).

**Performance Criteria**

The performance of a structure under a given design earthquake motion is examined by comparing the capacity diagram of the structure and the demand spectrum of design earthquake motions. The intersection (called performance point) of the demand spectrum for an appropriate equivalent damping ratio and the capacity curve represents the maximum response under the design earthquake motion if the damping ratio of the demand spectrum is equal to an equivalent damping ratio of the SDF system evaluated at the deformation at the performance point (Fig. 7). A continuous capacity curve is not necessary in design, but two points on the capacity curve must be evaluated for the two limit states.

The Building Standard Law Enforcement Order requires that spectral acceleration of a structure, defined by Eq. (13), at a limit state should be higher than the corresponding acceleration of the demand spectrum using the equivalent damping ratio, expressed by Eq. (18) or (19) at the same limit state.

The Building Standard Law Enforcement Order further requires that the exterior finishing and curtain walls should not fail under the design loads and displacements at the life-safety limit state. This requirement is intended to limit the story drift of the structure to a reasonable range.

**SUMMARY**

This paper presented an evaluation procedure of structural seismic performance under major earthquake motions introduced in June 2000 in the revised Building Standard Law and associated regulations. Life safety and damage control of a building are two performance objectives in the seismic provisions.

The earthquake motions are defined by acceleration response spectrum that is specified at engineering bedrock in order to consider the soil condition and soil-structure interaction effect. The return periods of the earthquake motion of approximately 500 years and approximately 50 years are considered for life-safety and damage-initiation limit states, respectively.

The capacity spectrum method is used to examine the damage initiation and
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life-safety of a building. A structure as designed is reduced to an equivalent SDF system by the use of a pushover analysis. A linearly elastic capacity spectrum at a 0.05 damping ratio is modified for an equivalent damping corresponding to the displacement of the nonlinear SDF system at the two limit states.

REFERENCES


Fig. 1: Design earthquake acceleration response spectrum for life-safety limit state on exposed engineering bedrock

Fig. 2: Equivalent single-layer soil model (\(\rho\): mass density, \(G\): shear modulus, \(V\): shear wave velocity, \(h\): damping factor, \(d\): layer depth, \(m\): mass, \(K\): stiffness, and \(c\): damping coefficient)
Fig. 3: Reduction of a structure to a single-degree-of-freedom system by pushover analysis and equivalent viscous damping using hysteretic energy dissipation.

Fig. 4: Capacity curve of a structure in terms of spectral acceleration $S_a(T)$ and displacement $S_d(T)$. 

$Q = 1 \cdot M \cdot x \cdot S_a$

$\Delta = 1 \cdot S_d$
Fig. 5: Equivalent viscous damping ratio for hysteresis energy dissipation

Fig. 6: Formulation of demand spectrum of design earthquake motion
Fig. 7: Performance criteria using demand spectrum of design earthquake motions and capacity curve of an equivalent SDF system.
Lessons from Recent Earthquakes in Turkey and Seismic Rehabilitation of Buildings

by U. Ersoy and G. Ozcebe

Synopsis:

In this paper recent earthquakes in Turkey are briefly discussed. Large-scale seismic rehabilitation projects carried out by the Middle East Technical University faculty and staff on moderately damaged R/C buildings are summarized. Research at METU related to seismic rehabilitation is presented, emphasizing infilled frames, a system extensively used in Turkey.

Keywords: infilled frames; rehabilitation; repair/strengthening; seismic assessment
INTRODUCTION

Turkey is one of the most earthquake-prone countries in the world. In Turkey, nearly 70 percent of the total population lives in zones of very high seismic risk. In addition, approximately 80 percent of the industrial facilities and 75 percent of the power plants are also located in these regions. Between 1925 and 2000, about 435,000 dwelling units have been destroyed due to earthquakes and casualties have reached 76,000.

In the last eight years five major earthquakes shook the country: the 1992 Erzincan, 1995 Dinar, 1998 Ceyhan, 1999 Marmara and 1999 Duzce earthquakes. In the two recent earthquakes in 1999, more than 100,000 dwelling units either collapsed or were heavily damaged (1). The death toll reached 20,000.

After each earthquake, thousands of moderately damaged buildings had to be rehabilitated. Rehabilitation is not feasible and effective unless the general characteristics of the buildings and causes of seismic damage are identified. In the paragraphs to follow, common causes of seismic damage and common characteristics of Turkish residential buildings will be discussed.

The main objective of this paper is to present the procedure followed in large-scale rehabilitation projects and to discuss related experimental research at Middle East Technical University.

COMMON CAUSES OF SEISMIC DAMAGE IN TURKEY

A re-evaluation of structural damage observed in the aftermath of the earthquakes of the last decade has revealed that the causes of earthquake damage in Turkey can be classified into four groups. These will be discussed briefly.
Structural System -- Apart from masonry construction and rural dwellings, most of the residential buildings in Turkey are R/C frame structures having inadequate lateral stiffness. Discontinuities in plan and elevation, soft and weak ground floors, strong beams-weak columns and short columns are very common in such frame buildings. A few examples are shown in Figures 1 through 5.

Detailing -- Detailing errors and inadequate detailing of the reinforcement result in considerable seismic damage in Turkey. Usually the ends of columns and beams are not properly confined, ties with 90° hooks are generally used, there are no ties in beam-column joints, and column longitudinal bars are lap spliced at floor levels. Figures 6, 7 and 8 show some typical examples of the structural damage resulting from inadequate detailing.

Construction -- Until very recently most residential buildings in Turkey were built with practically no inspection. As a result, dimensions and reinforcement of members in the built structure could be different from those on the design drawings. Poor concrete quality is also very common in residential buildings.

Soil -- Soil has not been one of the main causes of seismic damage until the August 17 1999 Marmara earthquake. In the Marmara earthquake liquefaction and bearing failure of soil caused extensive building damage in the city of Adapazari.

REHABILITATION PROJECTS AND METU INVOLVEMENT

In Turkey, according to the present laws, the structural rehabilitation of a building that has experienced moderate damage during an earthquake is among the responsibilities of the State. After the three major earthquakes, Erzincan (1992), Dinar (1995) and Ceyhan (1998), the Turkish Ministry of Public Works and Settlement engaged the three leading national universities as consultants in the rehabilitation work. Special protocols between the universities and the Ministry were signed after the earthquakes.

Structural rehabilitation of buildings damaged in an earthquake requires special expertise both at the design and construction stages.

The METU approach in the seismic rehabilitation of structures has been based on the accumulated knowledge and experience gained from experimental research and field applications in the past thirty years.

In this section, the rehabilitation project undertaken by the Middle East Technical University (METU) after the 1998 Ceyhan earthquake will be presented (2). In the section to follow, the experimental research at METU on repaired/strengthened members and systems will be briefly discussed.
Procedure Followed in the Ceyhan Project

In Ceyhan, METU undertook the task of assessment and rehabilitation of approximately 2000 dwelling units, totaling 210,000 m$^2$ of floor area. METU's responsibilities included the seismic safety evaluation of the buildings and preparation of structural drawings for rehabilitation. To carry out the work involved, METU engaged the assistance of a leading consulting firm based in Ankara.

Assessment -- The buildings were first surveyed by the consultant firm personnel. The as-built structural and architectural plans of each floor were prepared in AutoCad™ file format. On these floor plans, the observed damage was also marked by using special designation techniques, capable of indicating both the type and the severity of the damage. In producing the as-built structural drawings and damage assessment, the basic principles, rules and procedures set by the Coordinating Committee at METU were followed.

After receiving the as-built drawings in Ankara through the net, five teams consisting of METU staff were organized and sent to the site. These teams made the final evaluation on the damaged structure and marked the spots where cores could be taken from concrete. They also re-evaluated the marked damage. Core strength was evaluated together with strength obtained using other non-destructive methods.

The consulting engineering firm conducted the analysis of the structures involved, using the existing properties including the damage, following the principles set by the Coordinating Committee. In the analysis, the actual in-situ measured dimensions of members, reinforcement and concrete strengths were used, and the existence of the hollow-clay-tiles was not taken into account. The objective of the analysis was to see the overall safety of the structure and the weaknesses involved. On the summary sheets, the shear in each member was compared with the estimated shear capacity, and moments were compared with the moment capacities of the members under the same axial load. Interstory drift ratio of each floor was also given on the summary sheets together with the allowable limits given in the Turkish Seismic Code (3).

Evaluation of the results of the structural and strength analyses made by the METU staff gave indications of seismic vulnerability and displayed the local and overall weaknesses of the structure. The comparison of weaknesses determined by the analyses with the observed damage was very helpful in understanding the damage mechanism. A clear understanding of what had happened was essential for a sound rehabilitation.
Rehabilitation Philosophy -- The rehabilitation philosophy of METU was developed by considering the common potential weaknesses of residential buildings in Turkey, which include:

- Inadequate lateral stiffness.
- Irregularities in the frame system both in plan and elevation.
- Inadequate reinforcement detailing.
- Features, which would lead to undesirable seismic behavior, such as weak ground floors, short columns and strong beam – weak column applications.

To summarize, it can be said that in most of the buildings the lateral-load resisting system consists of frames, which have irregularities and weaknesses. In addition, these frames cannot be classified as ductile frames and the drift ratios are high. A design philosophy which ignores these facts, would not be realistic. In the METU approach, which is called "system behavior improvement", a new lateral-load resisting system is introduced. The new lateral-load resisting system consists of providing structural walls in both directions, formed by filling selected bays of frames with reinforced concrete infills, properly connected to the frame members. The existing frames are used mainly to resist the gravity loads.

The other alternative would be to rehabilitate the existing frames and use them as the main lateral-load resisting system. Such an approach would not be feasible, because it would require strengthening of all beams and columns. Even if all these members are strengthened, the frame structure could still suffer damage due to local weaknesses and interstory drift limits may not be satisfied. System behavior improvement by introducing structural walls (infilled frames) has been the basic METU philosophy in the seismic rehabilitation projects carried out after the Erzincan (1992), Dinar (1995) and Ceyhan (1998) earthquakes (2).

Design for Rehabilitation -- Preliminary design for structural rehabilitation was the responsibility of the METU staff. The buildings involved were rehabilitated by introducing structural walls by infilling selected bays of the frame. Frame bays to be infilled were carefully chosen to form a reasonably symmetrical and effective wall pattern, to produce acceptable structural behavior under seismic action. Based on some simple strength and drift calculations for low and medium-rise buildings (up to 10 stories), the structural wall area in each direction was set to be at least 0.25% of the total floor area of the building, but not less than 1.0% of the floor area at the base. Openings in the structural walls were allowed only if unavoidable. In choosing the wall locations, architectural conditions and functional requirements were also taken into account.

The design drawings on which the wall pattern and dimensions are marked were sent to the consulting firm. The consulting firm analyzed the modified structural system using the guidelines given by the Coordinating
Committee. The analyses were carried in accordance with the present Turkish Seismic Code (3). In these analyses the existence of hollow-clay-ties was not taken into account.

The consulting firm presented the results of the analyses to METU. Similar to the analyses made prior to modifications, they also submitted summary sheets where the load effects and strength for each beam, column and structural wall were given. Drift ratios at each floor were also shown on these summary sheets. Using these sheets, the METU staff evaluated the compliance of the modified structure with the requirements of the current code. If the code requirements were not satisfied, the structural wall pattern and/or the wall dimensions were changed and the analysis was repeated until satisfactory results were obtained. In addition to the system improvement by structural walls, reinforced concrete or steel jackets were introduced to the moderately or severely-damaged members and to those which were found to be critical in analysis.

The existing foundation system was checked using the results of the analysis made for the modified structure. Usually new foundations are needed for the structural walls introduced. However if the building has stiff and strong continuous footings in both directions, as was the case in Ceyhan buildings, these footings were found to be adequate and no additional footings were introduced.

**Design Drawings and Final Reports** – Subsequently, the consulting firm was asked to prepare the detailed design drawings in accordance with the rules and principles set by the Coordinating Committee, considering the current codes and the results of experimental research conducted at METU since 1968.

The dowels embedded into the drilled holes in the existing structural members provided the connections of the infills to the frame members and to the foundation. A special epoxy was used for bonding the dowels. Wall reinforcement was designed and detailed in accordance with the requirements of the current seismic code. The reinforcement detailing of columns to which the infills had been connected was checked carefully. METU tests have revealed that inadequate lap splices in column longitudinal bars (made at the floor level) lead to the premature failure of the infilled frame (2). In such cases special boundary reinforcement was introduced in the infill, adjacent to the columns.

A final report was prepared summarizing the assessment and design stages for each building. The design calculations and the final design drawings were checked and signed by the METU staff members responsible for that building. The design drawings, calculations and the final reports were then submitted to the Ministry for approval.
Construction Stage -- Application of the approved design drawings in the field is always more difficult and complicated as compared to new construction. Rehabilitation work should be performed by qualified and experienced contractors. Due to the complicated, difficult and risky nature of this type of construction work, continuous inspection by experienced engineers is essential.

METU RESEARCH ON REHABILITATION

In the late sixties, METU was involved in the seismic rehabilitation of buildings damaged in the Bartin earthquake. Rehabilitation was made by introducing structural walls (infilled frames) in both directions. In those years limited data existed on the behavior of such infilled frames. During the design and construction stages, most of the decisions had to be based on engineering judgment and intuition. In 1968 an experimental research program was initiated at METU to investigate the behavior of infilled frames under lateral loads (5, 6). This was the first experimental research at METU on structural rehabilitation. In the past thirty years numerous experimental research projects related to seismic rehabilitation have been conducted. These include:

a. Slabs rehabilitated by introducing a new reinforced concrete layer.
b. Tests on epoxy-bonded dowels.
c. Jacketed columns tested under uniaxial loading.
d. Jacketed columns subjected to axial load and reversed cyclic bending.
e. Frames infilled with hollow clay tiles, tested under reversed cyclic loading.
f. Reinforced concrete frames with reinforced concrete infills tested under reversed cyclic loading (three projects).

A summary of the research at METU related to seismic rehabilitation is available in reference 5. Here, only the tests on infilled frames with reinforced concrete infills will be briefly discussed.

One-Story Infilled Frames

Three of the one story frame buildings in the Bartin cement plant, damaged during the 1968 earthquake, were rehabilitated by the METU staff by filling selected bays of the frames with reinforced concrete infills. In 1968 a test program was initiated at METU (6) to examine the effectiveness of connections between the existing frames and reinforced concrete infills. These connections consisted of dowels welded to the longitudinal bars of the frame members. Infilled frames were tested under monotonically increasing lateral loads. The performance was found to be satisfactory.
Two-Story, One-Bay Infilled Frames with Strong, Undamaged Frames

Thirteen infilled frames and one bare frame were tested (7). Each of the three series had a reference specimen in which the frame and the infill were cast together (monolithic). The frames, to which infills were introduced, were detailed in accordance with the current seismic code. The average concrete strength was about 25 MPa. Since infills were introduced to undamaged frames, they were considered to be strengthening rather than repairing. All specimens were tested under reversed cyclic loading. The cyclic lateral loading was applied at the second story level only (Figure 9). The loads, displacements at each story level, rotations at the base, and shear deformations on the infill were measured and recorded. All infilled specimens failed by sliding shear at the base. At this stage the vertical reinforcement of the wall had already yielded and both the frame and infill were extensively cracked (Figure 9).

The main variables investigated were the effects of column strength, column axial load, type of connection between the infill and frame members and the reinforcement pattern in the infill. Two types of connections were tested; (a) dowels welded to the longitudinal bars of the frame members and (b) dowels bonded by epoxy into drilled holes in the frame members. Three types of infill reinforcement were tested; (a) regular mesh, (b) diagonal reinforcement, and (c) reinforcement concentrated at the infill boundaries. At the end of the test program, the following conclusions were reached:
- Increasing the capacity of the columns improved the capacity of the infilled frame significantly.
- Among the two types of connections tested, dowels embedded into the frame members were found to be more effective.
- Three types of infill reinforcement patterns were tested. Infilled frames with any of the three patterns showed satisfactory performance. However the infilled frame with regular mesh in the infill behaved somewhat better than the others. In Figure 10, envelope curves of specimens having different infill reinforcement patterns are shown together with those of the reference monolithic specimen and the bare frame.
- Infills properly connected to the frame members increased both the strength and the stiffness significantly (Figure 10). Ratios of strength and initial stiffness of infilled frames to those of bare frames were 7 and 30, respectively. Thus, shear strength of the infilled frames was about $0.8 \sqrt{f_c}$ (in MPa).

Two-Story, One-Bay Infilled Frames with Weak, Damaged Frames

Test Specimens -- In the previous research project, infills were introduced to undamaged frames, detailed in accordance with the code requirements. The main objective of this third experimental research project initiated in 1994 was to observe the behavior of infilled frames in which the
infills were introduced to damaged frames with inadequate lateral stiffness, having strong beams and weak columns (4, 8). The test frames were not properly detailed and had low concrete strength, as encountered in practice. The main deficiencies in detailing were: (a) lack of confinement at member ends, and (b) all lap splices in column longitudinal bars made at the floor level. In some specimens the lap length was made shorter than required by the code, i.e. 12 and 15 bar diameters.

Tests were made in the Structural Mechanics laboratories of METU and Bogazici University under the supervision of the authors. For the test program, a two-story, one-bay frame having the same geometric dimensions as those of the previous tests conducted at METU were chosen (Ref. 7, Figure 9). The test specimen was a 1/3-scale model of a one-bay, two-story R/C frame. Material properties and reinforcement of each specimen are summarized in Table 1. The thicknesses of the infill in "A" and "B" specimens were 50 mm and 60 mm, respectively.

Dowels used to connect the infill to the frame members consisted of deformed bars. They were placed into holes drilled on the inner faces of the frame members and were anchored by using a special epoxy. In all of the "A" specimens the dowels inserted in both columns and beams were 8 mm bars, spaced at 120 mm. Dowels connecting the infill to the foundation beam were 8 mm bars spaced at 120 mm for specimens A2, A4 and A6. In A8 and A10 the dowels were 12 mm bars spaced at 120 mm. In "B" specimens, 10 mm bars were used as dowels. Spacing was 200 mm in beams and 150 mm in the columns. The only exception was specimen B12 in which 10 mm bars were replaced by 12 mm bars. For all "B" specimens dowels at the foundation level were 10 mm bars spaced at 100 mm. The embedment length of dowel bars in the frame members varied from 10 to 15 bar diameters. The yield strength of dowel bars was 480 MPa in the "A" specimens and 400 MPa in the "B" specimens.

After damaging the bare specimens, the dowels were placed. The infill was cast two days later. The infill reinforcement consisted of a regular mesh, placed on two faces of the infill. The infill reinforcement in all specimens was 6 mm bars spaced at 150 mm in both directions, on each face. In three of the frames, local strengthening techniques together with R/C infills were used.

**Test Procedure** -- At the beginning of the test, axial loads were applied to the columns through prestressing cables stressed by a hydraulic jack. The cyclic reversed lateral load was applied only at the second floor level. The axial load was kept constant throughout the test. The test specimens were instrumented to measure applied loads, displacements at each floor level, base rotations and shear deformations in the infills.

**Test Results and Discussions** -- The results are summarized in Table 2. In Series 1, the bare frame A1 was intended to have light damage. Light
damage was defined as the beginning of the formation of the sway mechanism. The bare frame A3 was identical to Specimen A1. This frame was loaded until heavy damage was observed. Heavy damage was defined as crushing of concrete at column or beam-ends, formation of cracks in joints and buckling of column longitudinal bars. The damaged specimens were rehabilitated by introducing identical infills. The response and strength of these two specimens were not very different.

The difference between repaired (damaged frame) and strengthened (undamaged frame) specimens can be seen from the comparison of specimens A6 and A10. The only difference between these two specimens was that, the frame of specimen A10 was undamaged and the frame of specimen A6 was damaged. The load-displacement envelope curves for these specimens are shown in Figure 11. As can be seen from this figure, stiffness degradation in the specimen with the damaged frame was more rapid when compared to the other specimen. Also, the strength and stiffness of the specimen having the damaged frame were about 20 percent less than for the undamaged frame.

An examination of test results for Series 2 reveals that inadequate lap splice length in the column longitudinal bars reduced the capacity of the infilled frame by 30 percent (compare A6 with A8). As can be seen from Table 2 and Table 4, although the frame to which the infill was introduced was damaged and the lap splice was inadequate, there was significant improvement in both strength and stiffness (compare A7 and A8). Since the axial load on the columns was low, concrete strength in the frame did not influence the capacity of the infilled frame significantly.

In specimens A6, the failure mode was combined flexure and dowel slip in the foundation. To cope with this slip problem, the dowels used in specimen A8 were increased in number. The increase in dowels did not improve the behavior of the infilled frame. This observation indicated that inadequately lap spliced regions of the columns must be strengthened locally. In Series 3, this point was taken into consideration. Envelope load-displacement curves for Series 3 specimens are shown in Figure 12. This figure illustrates the effectiveness of the local strengthening techniques applied together with the reinforced concrete infills.

Strength of infilled frames having different lap splice lengths are compared in Table 3. The results suggest that the local strengthening techniques prior to the construction of the infill can significantly improve the behavior. Similar conclusions were reached as a result of tests carried out at The University of Texas (9). Specimen B4 (no splice, continuous longitudinal reinforcement) and B8 (12 bar diameter splice) were the reference specimens. Although the confinement at the column ends was not sufficient, the behavior of the specimen B4 was quite satisfactory. On the other hand, the behavior of specimen B8 was the worst. Compared to the bare frame B7, the strength of the infilled frame increased by more than 10 times.
The specimen B2 had a premature failure. The failure was initiated by pullout of the connecting dowels under the first floor beam. It was found that the dowels were not properly bonded in the drilled holes. The problems observed in this specimen illustrate the importance of workmanship in placing the dowels.

Reinforcement detailing of the bare frames B5, B7, B9, and B11 was exactly the same. The longitudinal reinforcements was lap spliced at the floor level. The length of the splice was very small, i.e. 12-bar diameters. In specimen B6, to rehabilitate the weakness arising from inadequate lap-splice length, a steel jacket was introduced in the regions where these splices were made. The jacket consisted of steel plates bonded by epoxy to the concrete and welded to each other at the corners. The jacket made the infilled frame behave as well as those without any lapped splices.

In specimens B10 and B12, the lap splice length in column longitudinal bars was about 12 bar diameters. In specimen B10, concentrated longitudinal reinforcement was provided in the infill panel adjacent to the columns of the bare frame. This reinforcement was partially confined with hairpin shaped bars. In specimen B12, new columns were formed at the boundaries of the infill panel. The detailing of the columns was done in accordance with the Turkish Seismic Code. This strengthening technique required more workmanship as compared to specimens B6 and B10. As can be seen from Table 3, the nominal shear stress in the infilled frame B12 was about 13 percent more when compared to that of B10. The stiffness comparison in Table 4 revealed that both specimens had similar stiffness characteristics. It was concluded that local strengthening by introducing concentrated longitudinal reinforcement in the infill adjacent to the columns in B12 seemed to be an efficient and practical rehabilitation technique.

The concrete strength of the frame columns did not affect the strength of the infilled frame significantly, since the failure mode was basically flexure. However, the quality of concrete in the frame members has a direct influence on the resistance of the connecting dowels, since the success of the dowel anchorage is closely related to the concrete quality.

The stiffness comparisons of the test specimens are provided in Table 4. The initial stiffness of test specimens was calculated as the slope of the load-displacement curve during the first elastic cycle. The stiffness values in this table clearly show the significant increase provided by the infill. If Specimen B2, which experienced a premature failure due to dowel anchorage problems, is disregarded, the ratio of the initial stiffness of the infilled frame to that of the bare frame varied between 13 and 24. This ratio is somewhat lower when compared to those measured in the previous tests at METU, which had undamaged frames (7).
When the initial stiffnesses given in Table 4 are compared, it is apparent that the stiffness of infilled specimens with frames having lower concrete strength was somewhat lower than for infilled frames having higher concrete strength. However, the difference was not significant.

**Observations** -- The following observations can be made on the basis of the test results. It should be noted that the quality of materials, scale effect and workmanship might affect the results. In addition, the behavior of actual structures may be different from that of the sub-assemblages.

- Reinforced concrete infills introduced to damaged frames increased the lateral strength significantly. The ratio of the strength of the infilled frame to that of the bare frame ranged from 8 to 20.
- Infill walls also increased the lateral stiffness significantly. The ratio of initial stiffness of the infilled frame to that of the bare frame varied from 13 to 24. This significant increase occurred in spite of the heavy damage in the frame columns.
- When infilled frames with damaged and undamaged frames were compared (A6 and A10), it is observed that both strength and stiffness were reduced due to the frame damage. The reductions in strength and stiffness were about 20 percent and 30 percent, respectively.
- The strength of the frame concrete did not seem to affect the strength of the infilled frames significantly. However, quality of concrete was influential on the anchorage of connecting dowels.
- Lap-splices in column longitudinal bars, made immediately above the floor levels, decreased the strength of the infilled frame when the lap length was inadequate (B4 and B8). Although there was a decrease in strength due to inadequate lap-splice length, the strength of the infilled frame was still 10 times greater than that of the bare frame.
- The success of connecting dowels depends on the effectiveness of anchorage in the frame members. The anchorage was very sensitive to workmanship.
- Introducing concentrated longitudinal reinforcement in the infills adjacent to columns (B10) and forming new columns at damaged frame-infill boundaries (B12) improved both the strength and the behavior. Although the benefits of both techniques are comparable, the latter requires more workmanship.

**FINAL REMARKS**

Turkey is located in a very high seismic risk region and major earthquakes shake the country frequently. Most of the residential buildings in Turkey are frame buildings. The lateral stiffness of these buildings is usually inadequate and the frames cannot be classified as ductile frames.
Considering the general characteristics of the damaged buildings, METU prefers to introduce a new lateral-load-resisting system composed of structural walls that infill selected bays of frames, over the height of the structure. METU staff have rehabilitated a large number of buildings after the 1992 Erzincan, 1995 Dinar and 1998 Ceyhan earthquakes.

The experimental research on infilled frames has been carried out at METU since the late sixties. Tests were aimed to bring solutions to problems encountered in practice and to supply the data needed for design. The METU rehabilitation approach integrates research and practice.

REFERENCES

Table 1. Properties of Test Specimens

<table>
<thead>
<tr>
<th>Series</th>
<th>Specimen Id.</th>
<th>( f_c^{(2)} ) (MPa)</th>
<th>( f_y ) (MPa)</th>
<th>Column Reinforcement</th>
<th>Lap Splice Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A1</td>
<td>22</td>
<td>520</td>
<td>478</td>
<td>No splice</td>
</tr>
<tr>
<td>1</td>
<td>A2</td>
<td>23</td>
<td>520</td>
<td>478</td>
<td>No splice</td>
</tr>
<tr>
<td>1</td>
<td>A3</td>
<td>23</td>
<td>473</td>
<td>478</td>
<td>No splice</td>
</tr>
<tr>
<td>1</td>
<td>A4</td>
<td>21</td>
<td>473</td>
<td>478</td>
<td>No splice</td>
</tr>
<tr>
<td>2</td>
<td>A5</td>
<td>12</td>
<td>317</td>
<td>478</td>
<td>40( \phi )</td>
</tr>
<tr>
<td>2</td>
<td>A6</td>
<td>26</td>
<td>317</td>
<td>478</td>
<td>40( \phi )</td>
</tr>
<tr>
<td>2</td>
<td>A7</td>
<td>11</td>
<td>317</td>
<td>478</td>
<td>15( \phi )</td>
</tr>
<tr>
<td>2</td>
<td>A8</td>
<td>23</td>
<td>317</td>
<td>478</td>
<td>15( \phi )</td>
</tr>
<tr>
<td>2</td>
<td>A9</td>
<td>12</td>
<td>317</td>
<td>478</td>
<td>40( \phi )</td>
</tr>
<tr>
<td>2</td>
<td>A10</td>
<td>20</td>
<td>317</td>
<td>478</td>
<td>40( \phi )</td>
</tr>
<tr>
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<td>B1</td>
<td>25</td>
<td>224</td>
<td>224</td>
<td>No splice</td>
</tr>
<tr>
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<td>224</td>
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<td>3</td>
<td>B3</td>
<td>15</td>
<td>224</td>
<td>224</td>
<td>No splice</td>
</tr>
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<td>B4</td>
<td>32</td>
<td>224</td>
<td>224</td>
<td>No splice</td>
</tr>
<tr>
<td>3</td>
<td>B5</td>
<td>28</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B6</td>
<td>23</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B7</td>
<td>20</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B8</td>
<td>34</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B9</td>
<td>18</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B10</td>
<td>32</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B11</td>
<td>18</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
<tr>
<td>3</td>
<td>B12</td>
<td>40</td>
<td>224</td>
<td>224</td>
<td>12( \phi )</td>
</tr>
</tbody>
</table>

(1) Odd numbered specimens are bare frames, even numbered specimens are infill frames.
(2) For odd numbered specimens concrete strength of frame, for others concrete strength of infill.
(3) For Series 1 and 2 the infill thickness is 50 mm, for Series 3 it is 60 mm.
(4) In Series 1 and 2, beam longitudinal reinforcement is 4-\( \Phi 8 \) top and 4-\( \Phi 8 \) bottom.
In Series 3, beam longitudinal reinforcement is, 3-\( \Phi 8 \) top and 3-\( \Phi 8 \) bottom.
For all specimens infill reinforcement was \( \Phi 6 \) spaced at 150 mm in both directions, on both faces.
(5) Value given in parentheses refers to tie spacing at confined regions at each end of the member (in mm).
Beam ties of Series 1: \( \Phi 4/80 \) mm in span and \( \Phi 4/40 \) mm at the ends, beam ties of other series \( \Phi 4/100 \) mm in span, no confinement at the ends.
(6) Continuous longitudinal reinforcement is provided along the entire length of the column.
(7) Partial steel jacketing is applied.
(8) Concentrated longitudinal reinforcement is provided (2\( \Phi 12 \)) at the ends of the infill, next to the columns.
(9) Additional columns were made at the infill boundaries, column reinforcement is 4\( \Phi 8 \).
### Table 2. Summary of Test Results

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Axial Load, (kN)</th>
<th>$V_{max}$ (kN)</th>
<th>Nominal Shear Stress ($\tau$)</th>
<th>2nd Story Drift Index ($\delta_{2}/\delta_{1}$)</th>
<th>1st Story Drift Index ($\delta_{1}/H_{1}$)</th>
<th>Level of frame damage in the bare frame and failure mode of the infilled frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>146</td>
<td>33.3</td>
<td>0.23</td>
<td>0.0263</td>
<td>0.0306</td>
<td>Light damage</td>
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<tr>
<td>A2</td>
<td>146</td>
<td>286.4</td>
<td>1.00</td>
<td>0.0100</td>
<td>0.0089</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>A3</td>
<td>146</td>
<td>34.1</td>
<td>0.23</td>
<td>0.0203</td>
<td>0.0364</td>
<td>Heavy damage</td>
</tr>
<tr>
<td>A4</td>
<td>146</td>
<td>310.9</td>
<td>1.10</td>
<td>0.0067</td>
<td>0.0054</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>A5</td>
<td>58</td>
<td>13.5</td>
<td>0.13</td>
<td>0.0134</td>
<td>0.0161</td>
<td>Heavy damage</td>
</tr>
<tr>
<td>A6</td>
<td>58</td>
<td>137.3</td>
<td>0.45</td>
<td>0.0087</td>
<td>0.0068</td>
<td>Flexure and dowel slip (foundation)</td>
</tr>
<tr>
<td>A7</td>
<td>58</td>
<td>11.8</td>
<td>0.12</td>
<td>0.0102</td>
<td>0.0148</td>
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</tr>
<tr>
<td>A8</td>
<td>58</td>
<td>98.1</td>
<td>0.33</td>
<td>0.0047</td>
<td>0.0058</td>
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<tr>
<td>A9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>No damage</td>
</tr>
<tr>
<td>A10</td>
<td>100</td>
<td>180.4</td>
<td>0.68</td>
<td>0.0040</td>
<td>0.0054</td>
<td>Flexure and dowel slip (foundation)</td>
</tr>
<tr>
<td>B1</td>
<td>100</td>
<td>12.6</td>
<td>0.17</td>
<td>0.0173</td>
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<td>Heavy damage</td>
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<tr>
<td>B2</td>
<td>100</td>
<td>137.5</td>
<td>0.33</td>
<td>0.0172</td>
<td>0.0228</td>
<td>Premature dowel failure</td>
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<tr>
<td>B3</td>
<td>100</td>
<td>12.2</td>
<td>0.21</td>
<td>0.0058</td>
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<td>Heavy damage</td>
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<tr>
<td>B4</td>
<td>100</td>
<td>153.5</td>
<td>0.38</td>
<td>0.0075</td>
<td>0.0121</td>
<td>Flexure</td>
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<tr>
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<td>100</td>
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<td>0.0099</td>
<td>Heavy damage</td>
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<tr>
<td>B6</td>
<td>100</td>
<td>200.0</td>
<td>0.58</td>
<td>0.0099</td>
<td>0.0148</td>
<td>Flexure and shear sliding</td>
</tr>
<tr>
<td>B7</td>
<td>100</td>
<td>13.7</td>
<td>0.21</td>
<td>0.0096</td>
<td>0.0133</td>
<td>Heavy damage</td>
</tr>
<tr>
<td>B8</td>
<td>100</td>
<td>142.7</td>
<td>0.34</td>
<td>0.0097</td>
<td>0.0083</td>
<td>Flexure and sliding shear</td>
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<tr>
<td>B9</td>
<td>100</td>
<td>9.2</td>
<td>0.15</td>
<td>0.0084</td>
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<td>Heavy damage</td>
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<tr>
<td>B10</td>
<td>100</td>
<td>176.8</td>
<td>0.43</td>
<td>0.0014</td>
<td>0.0122</td>
<td>Flexure and sliding shear</td>
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<tr>
<td>B11</td>
<td>100</td>
<td>9.9</td>
<td>0.15</td>
<td>0.0027</td>
<td>0.0067</td>
<td>Heavy damage</td>
</tr>
<tr>
<td>B12</td>
<td>100</td>
<td>224.1</td>
<td>0.49</td>
<td>0.0046</td>
<td>0.0135</td>
<td>Flexure and sliding shear</td>
</tr>
</tbody>
</table>

(1) $\tau_{nom} / \sqrt{\gamma_{t}} = V_{max} / \left[ (0.8 \times 1500 \times t) \sqrt{\gamma_{t}} \right]$

For odd numbered specimens $\gamma_{t}$ refers to the frame concrete
For even numbered specimens $\gamma_{t}$ refers to the infill concrete
Table 3. Effect of Dowels on the Behavior of Infilled Frames

<table>
<thead>
<tr>
<th>Series</th>
<th>Spec. Id.</th>
<th>Splice Length</th>
<th>Nominal Shear Stress((kN/mm))$^{(1)}$</th>
<th>Failure mode of infilled frame</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>A2</td>
<td>No splice</td>
<td>1.00</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>1</td>
<td>A4</td>
<td>No splice</td>
<td>1.10</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>2</td>
<td>A6</td>
<td>40°</td>
<td>0.45</td>
<td>Flexure and dowel slip (foundation)</td>
</tr>
<tr>
<td>2</td>
<td>A8</td>
<td>15°</td>
<td>0.33</td>
<td>Flexure and dowel slip (foundation)</td>
</tr>
<tr>
<td>2</td>
<td>A10</td>
<td>40°</td>
<td>0.68</td>
<td>Flexure and dowel slip (foundation)</td>
</tr>
<tr>
<td>3</td>
<td>B2</td>
<td>No splice</td>
<td>0.33</td>
<td>Premature dowel failure</td>
</tr>
<tr>
<td>3</td>
<td>B4</td>
<td>No splice</td>
<td>0.38</td>
<td>Flexure</td>
</tr>
<tr>
<td>3</td>
<td>B6</td>
<td>12°</td>
<td>0.58</td>
<td>Flexure and shear sliding</td>
</tr>
<tr>
<td>3</td>
<td>B8</td>
<td>12°</td>
<td>0.34</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>3</td>
<td>B10</td>
<td>12°</td>
<td>0.43</td>
<td>Flexure and sliding shear</td>
</tr>
<tr>
<td>3</td>
<td>B12</td>
<td>12°</td>
<td>0.49</td>
<td>Flexure and sliding shear</td>
</tr>
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</table>

(1) $\tau_{nom} = \frac{V_{nom}}{t} = \frac{V_{max}}{t} \left[ \frac{(0.8 \times 1500 \times t)}{\sqrt{f_c}} \right]$. Here $f_c$ refers to the infill concrete

Table 4. Stiffness of Test Specimens

<table>
<thead>
<tr>
<th>Series No.</th>
<th>Specimen Name</th>
<th>Corrected initial stiffness (kN/mm)$^{(1)}$</th>
<th>Ratio of the initial stiffness of infilled frame to that of bare frame</th>
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<tbody>
<tr>
<td>1</td>
<td>A1</td>
<td>4.0</td>
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</tr>
<tr>
<td>1</td>
<td>A2</td>
<td>96.1</td>
<td>24.0</td>
</tr>
<tr>
<td>1</td>
<td>A3</td>
<td>4.5</td>
<td>1.0</td>
</tr>
<tr>
<td>1</td>
<td>A4</td>
<td>91.0</td>
<td>20.2</td>
</tr>
<tr>
<td>2</td>
<td>A5</td>
<td>4.5</td>
<td>1.0</td>
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<tr>
<td>2</td>
<td>A6</td>
<td>86.9</td>
<td>19.3</td>
</tr>
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<tr>
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<td>A8</td>
<td>88.2</td>
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<td>A9</td>
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<tr>
<td>3</td>
<td>B12</td>
<td>61.7</td>
<td>19.3</td>
</tr>
</tbody>
</table>

(1) Stiffness values are normalized by multiplying $\sqrt{20}/\sqrt{f_c}$
Figure 1. Building collapse (1999 Marmara Earthquake)

Figure 2. Reinforced concrete joist floor with shallow beams (1967 Adapazari Earthquake)
Figure 3. Soft and weak ground floor (1999 Marmara Earthquake)

Figure 4. Damaged captive column (1999 Marmara Earthquake)
Figure 5. Strong beam - weak column (1992 Erzincan Earthquake)

Figure 6. Lap splices at the floor level (1999 Marmara Earthquake)
Figure 7. Lack of anchorage at the tie ends (1999 Marmara Earthquake)

Figure 8. Damage at beam - column joint (1999 Marmara Earthquake)
Figure 9. Loading of the infill frames

Figure 10. Load - displacement envelope curves of specimens with different infill reinforcement patterns
Figure 11. Envelope load-displacement curves

Figure 12. Comparison of various strengthening techniques (Series 3)
Strengthening Buildings for Earthquake Resistance with New Concrete

by L. A. Wyllie, Jr.

Synopsis:

Many buildings built in the 1950s and 1960s in regions of high seismicity are extremely vulnerable to extreme damage or collapse in future earthquakes. The most vulnerable and hazardous of these buildings are the unreinforced masonry buildings, with non-ductile concrete frame buildings considered the next most hazardous class. Concrete is a logical choice to strengthen these buildings, either with new shear walls or infilled walls or sometimes with jacketing. The most common and probably the best system is to add new reinforced concrete shear walls. It makes a building much more rigid, reduces seismic drifts or deformations and thus reduces damage and prevents the potential of collapse. A variation of adding shear walls is adding infilled walls, which are wall panels of reinforced concrete (or sometimes masonry) added between floor beams and columns. Concrete jackets consisting of a layer of concrete, usually about 4 inches (100 mm) thick, containing closely spaced ties, can also provide confinement and add shear capacity to concrete frame members. This paper will summarize the pros and cons of the application of these three seismic strengthening techniques.

Keywords: concrete jackets; infilled walls; shear walls
Loring Wyllie’s 34-year career spans a wide variety of accomplishments, honors, and service to the structural engineering profession. In 1990 his contributions were recognized by his election to the National Academy of Engineering. An honorary member of the American Concrete Institute, he has played an active role in the Provisions Update Committee for the National Earthquake Hazards Reduction Program of the Building Seismic Safety Council and Committee 318 and the Technical Activities Committee of ACI.

INTRODUCTION

Many older buildings in regions of high seismicity are extremely vulnerable to extreme damage or collapse in future earthquakes. Many of these “older” buildings are not all that old, having been constructed in the 1950s or 1960s and sometimes even more recently. These buildings present the greatest risk and potential source of loss of life when major earthquakes occur.

We have observed this situation in many recent earthquakes. In Turkey in 1999, many reinforced concrete buildings constructed in the past 50 years collapsed leading to a great loss of life. In Taiwan in 1999, again it was buildings built in the last 50 years that lead to the greatest loss of life, with sometimes more damage to the newer buildings. Although the life loss was significantly less, similar trends were evident in California in Northridge in 1994 and Loma Prieta in 1989.

The most vulnerable and hazardous of these buildings are the unreinforced masonry buildings which are stiff but quite brittle once failure begins. Concrete in the form of new shear walls can greatly strengthen these buildings and prevent their collapse. The non-ductile concrete frame buildings are not far behind and many engineers consider them the next most hazardous class of buildings, especially those with short stiff columns with only nominal ties. Concrete is a logical choice to strengthen these buildings, either with new shear walls or infilled walls or sometimes with jacketing.

Concrete is the natural material to strengthen concrete and masonry buildings. Concrete provides the stiffness needed to be more compatible with the relatively stiff concrete and masonry buildings. Structural steel diagonal bracing systems are often utilized to strengthen concrete and masonry buildings, but the more flexible steel systems rely on the original concrete or masonry buildings to crack before they can become effective, often leading to a more heavily damaged building after being tested by the eventual earthquake.

ADDED SHEAR WALLS

Shear walls added to a concrete or masonry building is the most usual and often the most effective way to seismically strengthen. They provide a high level of increased lateral stiffeners to a building thereby reducing its lateral drift in an
earthquake. When the drift is reduced there is a greater protection of brittle, non-ductile elements which will fail if lateral drifts are substantial.

In a concrete building, the new shear walls usually have to be added between existing columns so the dead load of the columns can hold the wall down and resist overturning force. It is usually desirable to add enough shear walls so the forces on the wall balance and net uplift is prevented, thereby preventing expensive foundation modifications. Another good way to solve this potential problem is to locate walls above basement walls, although the existing basement wall may need supplemental reinforcement. Walls can also be extended an additional bay in the lowest floor or floors to engage additional columns for stability, resulting in L or T shaped walls. Walls are ideally solid but they can certainly contain doors or windows provided a proper design is performed.

Plan location of the walls is an important detail that affects cost and constructability. Often there is an existing beam between columns so if the shear wall is centered on the column line it is difficult to install the needed continuous vertical reinforcing without using a very thick wall or shoring the slabs and removing the beam. A more desirable solution usually involves placing the wall adjacent to the beam and passing through slots in the floor. If slots about 2 feet (600 mm) long are made with equal portions of the slab left in place, shoring may become unnecessary and dowelling can be done to the sides of the beam and columns. If the building must remain occupied while their work is done, adding shear walls on the perimeter minimizes disruption and conflicts with utilities, although it changes the building’s appearance. Some building owners like this as a solution to give a face lift to an ugly building at the same time.

Since shear walls tend to concentrate the lateral resistance of the building in a few locations, it is necessary to insure that the roof and floor diaphragms have sufficient chords and collectors to resist tension and other forces to connect all parts of the building to the new shear walls. The existing beams may be sufficient for this need, although if the number of new shear walls is limited, new collectors or added reinforcement may be necessary.

Another common variation is to add reinforced concrete shear walls to old unreinforced masonry bearing wall structures. These are among the most hazardous class of building in an earthquake and the added concrete walls match the stiffness of the masonry and minimize damage following an earthquake. In contrast, steel diagonal bracing is often added but considerable cracking and damage must occur within the brick masonry before the steel can be effective. Most of this concrete is installed pneumatically as shotcrete which is quite economical as forming costs are minimal. Some of these old masonry buildings are now historic buildings and successful strengthening while preserving historic fabric has involved removing several wythes of the brick masonry, adding the reinforced shotcrete wall and then replacing the original face brick as veneer and the historic appearance remains unchanged.
A few more words need to be said about the use of shotcrete. In all of these retrofit projects, concrete is installed as shotcrete whenever possible. With the high labor rates of North America, the most expensive component of reinforced concrete construction is new conventional formwork. Shotcrete eliminates the formwork on one side. In many low rise buildings, we have used existing wood or metal stud partitions as back forms and added our new shotcrete/concrete walls adjacent to the existing partition to almost eliminate forms. This approach has led to many economical designs. In larger structures, the added shear walls sometimes require boundary members with 6 to 10 larger bars confined within ties at 4 or 6 inch centers. This congested reinforcement pattern presents a true challenge for the shotcrete nozzleman to achieve dense concrete and test panels are essential. On some projects these boundary members are conventionally placed with adjacent shotcrete and stay-in-place forms for a successful outcome.

ADDED INFILLED WALLS

A variation of adding shear walls is adding infilled walls. Infilled walls are wall panels of reinforced concrete (or sometimes masonry) added between floor beams and columns. The infill panel has the characteristic of no direct continuity from floor to floor and relies on shear transfer at its perimeter to the existing beams and columns and the existing columns must provide the vertical chord or boundary member.

Shear transfer at the perimeter of an infilled panel is always critical for this type of strengthening. It involves roughening the existing concrete and installing many dowels epoxied into the existing beams and columns. If the panel is not well connected for shear, the frame will try to act independently and the columns will bear against the rigid infill panel and cause shear failures in the column just below the beam above. In countries with high labor wages as in North America, the cost of adding all of the needed dowels makes this approach non-economical, although in parts of the world where labor is inexpensive, infill walls are frequently chosen.

For a properly dowelled and constructed infilled panel wall, the most critical and controlling element will most likely be the lap splice of the existing column reinforcement. The lap splices of the existing column are undoubtedly short compression splices which will not develop the column longitudinal bars in tension. Infilled walls are typically only one bay long and tension will easily develop in the wall edge under seismic loads. Tests have demonstrated that this tension failure of the short column lap splice is the initiation of system failure in a properly built infilled wall. Strategies to strengthen this deficiency can include jacketing the column with confined concrete or steel, or adding new continuous vertical steel at the ends of the panel adjacent to the column in holes drilled through the beams or enlarge the end of the panel so these bars can pass the side of the beam. Another solution that has been used involves chipping the cover off the column at the lap splice and welding the spliced bars together. With this last
solution closely spaced ties should be added before the cover is replaced to resist
the lateral pressures from the eccentricities of the bars welded together when
tension develops.

ADDED CONCRETE JACKETS

Concrete jackets consist of a layer of concrete, usually about 4 inches or 100 mm
thick containing closely spaced ties to provide confinement and added shear
capacity to concrete frame members. Jacketing a column is quite straightforward
as one simply encases the entire column with the new concrete. Jacketing a beam
is considerably more difficult as the floor slab interferes. Even if one drills
extensively or removes the slab to replace it, then one faces the reality of having
to add concrete above the floor slab, not acceptable unless you have an existing
depression for floor finishes. Jacketing the joint is even more difficult as all the
beams and slab are in the way. Some attempts have been tried experimentally
including steel angles vertically between beam ends with substantial ties top and
bottom within the column jacket. Another attempt is drilling holes through the
beam, but hooking the tie ends must be field hooked with a “hickey” which is not
easy erection. Jacketing really depends on which elements need to be
strengthened and it is practical for columns, difficult for beams and very difficult
for joints.

The basic question for column jackets is if dowels are also required.
Confinement can be effectively provided for round columns or small columns
with jackets without dowels. However, normal sized rectangular columns with
sides 16 to 18 inches (400 to 450 mm) or larger will need dowels drilled into the
core and hooked around the confinement reinforcement to make the jacket
effective in confining the entire column. If ties in the jacket are at 4 to 6 inch
(100 to 150 mm) centers, drilling dowels at that spacing is clearly very
expensive. Acceptable performance can be achieved with dowels about 12 inch
(300 mm) longitudinally and a good sized longitudinal bar such as a #6 (18 mm)
bar held within the dowel hooks. From what the author has seen of the available
research, it is recommended that the dowels be at about 12 inch (300 mm)
centers in plan with any spacing between dowels or the corner not exceeding
about 14 inches (350 mm). Jacketing concrete can be installed as shotcrete or as
cast in place concrete.

An alternative to concrete jackets is steel jackets. Research has shown these to be
effective but similar dowels into the core are needed, similar to the concrete
jacket, to provide effective confinement. These are also many proprietary carbon
fiber and similar fabrics to “wrap” columns. While these work fine on circular
columns and undoubtedly increase shear strength on rectangular columns, their
ability to provide confinement on rectangular columns has yet to be
demonstrated to the author and users need to be thorough in their evaluation of
these products to correct deficiencies identified in the structure.
CONCLUSION

I have tried to summarize the various ways we strengthen buildings with concrete for improved seismic performance. The most common and probably the best system is to add new reinforced concrete shear walls. It makes a building much more rigid, reduces seismic drifts or deformations and thus reduces damage and prevents the potential of collapse.

I chose this topic perhaps a year and a half ago when I was asked to give this talk at this symposium honoring Mike Uzuemeri. I am deeply honored to be asked and be able to participate in celebrating Mike's career here in Toronto. When I chose the topic, little did I know that an August 1999 earthquake would occur in Mike's homeland of Turkey with many concrete buildings collapsing, causing a great loss of life. I am writing this paper in May 2000 having just returned from a World Bank workshop in Ankara advising on how to strengthen the damaged buildings that did not collapse but have been vacated for safety considerations. I felt it was very presumptuous of me, an engineer from California, to go to Turkey and tell Turkish engineers how to strengthen their buildings. So, I put aside my engineering credentials and spoke as a salesman, since salesmen can be pushy and aggressive. I was a salesman selling shear walls or structural walls and I hope I was effective. Most agreed that if the Turkish buildings had more shear walls that performance would be markedly improved.

So today, I honor the career of Mike Uzuemeri and his contributions to our knowledge of reinforced concrete structures and how they perform in earthquakes. The vulnerability of concrete buildings in the earthquake prone regions of the world is extensive, as evidenced by the recent Turkish event. Adding new shear walls or infilled walls to these buildings is extremely effective, and I hope the motivation and funding becomes available to implement these suggestions and to reduce this potential for devastation and loss of life throughout the world.
A Link Between Research and Practice: ACI 352 Recommendations for Design of Joints in RC Structures

by J. O. Jirsa

Synopsis: The mission of most ACI technical committees is to “develop and report information” and a large number of missions include “develop and maintain standards” for use in design, construction, maintenance or other practice-related application. In general, the reports technical committees develop become prime sources of information in their assigned area. The objective of this paper is to document the formation of the committee and the development of the first report of ACI 352 “Joints and Connections in Monolithic Concrete Structures” published in 1976. The reports of ACI 352 have had considerable impact on the design and construction of concrete structures. The activities of ACI 352 provide a case study in the role of a committee in providing a vital link between research and practice. It seemed appropriate to discuss the work of ACI 352 at the Uzumeri Symposium because Mike Uzumeri served as a member of the committee. His research on the behavior of beam-column joints was an important part of the data on which the first report of ACI 352 was based.

Keywords: codes; construction; design; joints and connections; practice; reinforced concrete; research
Jirsa

James O. Jirsa is the Janet S. Cockrell Centennial Professor of Civil Engineering at the University of Texas at Austin and the Chairman of the Department of Civil Engineering. He is the President of ACI and has served on a number of technical and administrative committees of the Institute, including ACI 352, 421, 408, 318, Board of Directors, and the Technical Activities Committee.

INTRODUCTION

In 1960, Siess (1) published a paper entitled “Research, Building Codes, and Engineering Practice.” In that paper he stated, “The practice of engineering as a profession is, by its very nature, based on knowledge. There are basically only two sources of this knowledge: research and experience.” In order for an engineer to advance his knowledge beyond that which was current at the time of his/her formal education, it will be necessary to “keep on learning.” While this can be done on an individual basis, it may be more efficient to do this collectively. Associations of people in any field are formed for this purpose and ACI is no different. In fact, the value of most technical societies is that they provide a forum for such collective activity and they support, maintain and encourage participation. Siess outlined the process by which committees “study collectively the results of research and current practices” and use their “collective judgement” to prepare “a summary of existing knowledge.” Although Siess discussed the process as it related to code development, the same statements apply to other technical documents.

For many committees, it seems that the only validation of their work that is acceptable is the inclusion of their “knowledge” in a code or standard. The success of a committee in developing knowledge should not be judged in this way. The objective of this paper is to provide a case study of a committee document that was widely used in practice for many years before any of its recommendations were incorporated into a code or standard.

The Committee Process

Figure 1 is from the paper by Siess and represents the complex interactions between research, codes, and practice. The interactions are as valid today as they were 40 years ago—little has changed even though the flow of information is much faster with today’s technology. The process of discussing research results and tempering them with the experience gained in practice is a very human process that has not yet been computerized. Although today’s information technology makes participation in committee work possible through electronic means, it cannot replace the camaraderie of face-to-face discussions and social interactions that occur at meetings. These are features of committee and association activity that must be retained.
One of the benefits of committee participation is the exposure to diverse opinions—often expressed very forcefully. ACI has always followed a rigorous consensus process for developing codes, standards, and technical documents. Consensus is not unanimity, but it is a process by which general agreement can be reached on an issue, that is, a position is reached that all involved can live with. Compromises are made and different points of view are heard and respected.

There are indications that associations are having difficulty in recruiting new members and volunteers, and in retaining current members. Volunteers must find value in committee membership. Generally, value is found in the opportunities for continued learning and professional growth that committee work provides. Furthermore, volunteering must be enjoyable. Certainly, interaction with others having like interests is a key element; but developing long term friendships and gaining an understanding of the spirit of cooperation, mutual respect, and compromise are important benefits.

Time from Research to Practice

"One of the greatest ills of the engineering profession is the long lag between research and practice—the time elapsing between the discoveries and findings of research and their general acceptance and application by engineering practitioners." This familiar criticism is a quote from the first lines of a paper by Sheets (2) that was delivered in 1938 at the 34th Annual Dinner of the American Concrete Institute. He makes a "frank analysis of the existing situation" by examining the sort of people who make up the practitioner and research groups. The "manner of men" who make up the practitioner group (in 1938 all practitioners were men) include:

- Executives who are too busy to study research but have the ability to do so.
- Executives who are non-scientific, impatient, and "given to direct action."
- Close-minded individuals who know everything and "pooh-pooh" any new proposal.
- Engineers who "scoff" at any problem that cannot be solved with arithmetic and simple equations.
- Know-it-alls.
- Those who belong to "do-it-the-way-we-learned-in-school crowd."
- Those who belong to the "copy-cat, tradition-loving, mentally lazy flock."
- A very few who are "just plain dumb."
- Engineers who are "smart, open-minded searchers" for improving their knowledge.

The research group includes:

- "Inhabitants of the technical stratosphere" who are unable or unwilling to express themselves using the "language" of the practitioner.
- "Cloud riders" who are not as abstract as the "stratospherists" but are not understandable to the practitioner.
- "Technical adventurers" who conduct research without assuming any responsibility for interpreting or completing their studies.
• A few technical "snobs" who "like to talk a ten cent idea in ten dollar language."
• Self-advertisers who use research as a ladder to their own success rather than as contributors to a profession.
• The "conscientious thinkers, unselfish servers of fellow engineers, and the rare species that can think deeply, conclude soundly, and talk simply."

Sheets discusses the characteristics of a good research report and the need for an "interpreter" to fill the gap between research and practice. Although there are individuals who have such capabilities, for example textbook authors, he suggests that a joint committee of researchers and practitioners could be an "interpreter." Obviously, the people described last in each of Sheets' groups are the ones who would constitute the ideal committee for filling the gap. Through the collective judgement of a committee with diverse membership, the time required to convert research results to information that can be implemented by practitioners will be reduced.

Siess (1) also comments on the lag between research and practice and cites several cases where the findings of research projects did not change codes or practice for 30 years or more. It must be remembered that the prime role of codes is to provide public safety. Therefore, code-writers tend to be conservative in applying research findings and they wait until there is verification by other researchers or the body of knowledge is sufficient to permit extrapolation to a variety of cases. Such long lags are frustrating and discouraging to researchers, however, codes are not the only avenue for implementation of findings. Well-written reports and papers on topics of interest that address critical issues in design and construction may have just as large an impact as code provisions. The work of ACI 352 is an example of a report that had significant impact on design well before some of the provisions were incorporated into the ACI Building Code (3).

ORGANIZATION OF ACI 352

The Joint American Concrete Institute-American Society of Civil Engineers Committee on Joints and Connections in Monolithic Structures (ACI 352) was official organized in 1966. The first meeting of the committee was held in October 1967 at the ACI Convention in Des Moines, Iowa. Prof. Mete Sozen was appointed to chair the committee. The mission was: "Study the detailed design of joints and connections in monolithic concrete members and develop practical aids for the designer and the constructor." The roster of members in 1967 and 1976 are shown in Appendix A. The mission and the membership was selected to address what was considered to be a gap in the design and construction of monolithic concrete structures—the joints between elements. Prestressed and precast structures were considered to be outside the scope of the committee.

Des Moines, October 1967--The discussion at the first meeting (4) was of a general and probing nature to attempt to define the problem the committee faced. The need for defining categories or classifying joints was discussed. There was general agreement that two broad categories were needed—beam to
column and slab to column joints. There was disagreement as to the need to consider seismic and non-seismic loading as separate cases. Reese who had been a proponent for the organization of the committee suggested that the goal of the committee should be to help the designer create a good joint, and that while complex problems need to be studied, there were no rules or guides existing for even simple beam-column joints. He stated that there was a need for standard joint specifications to eliminate joints being designed by detailers rather than by engineers but it was not clear whether these should be in the form of standard details or guides for good design. Several members agreed to submit some typical joint details. The members were asked to submit references about joint failures and Sozen asked the members to consider the course of action to be taken.

Immediately after that meeting, Reese sent a letter to the members in response to questions that had arisen in the discussion in Des Moines. His letter is a masterpiece in which his years of experience in joint design were evident but so was his frustration with the state of the art. First he asked, “What is being done now?” and then posed a series of questions that defined the state of joint design as follows:

1. Detailed design of joints was being left to reinforcing bar detailers.
2. Design drawings usually cover (joints) with a few notes such as “Follow ACI 315” or “Comply with all of ACI 318.”
3. Seldom are even a few typical details provided.
4. Technical high school graduates and self-trained draftsmen have replaced graduate civil engineers as detailers.
5. Detailers cannot “design” a joint because they do not know the forces involved or the type of building and its intended use. They must rely almost exclusively on manuals and handbooks.
6. Computer detailing will make the mechanization of joint design even more desirable.
7. Typical joints are sufficiently alike that careful study and standardization is possible and desirable.
8. It may be possible to categorize joints by the forces they will need to withstand.
9. A review of the state of affairs in joint detailing clearly establishes the need for the committee to act.

Reese asked “Is it not short-sighted, and possibly dangerous, to spend so much time in making involved frame analyses and pursuing sophisticated methods which are directed almost entirely to the main members, and leave the joints and connections to detailers?” After considering various options, he advised the committee to develop “a manual covering joints only in all phases from routine to complex, making all material available under one cover and not enlarging other manuals, which are already large enough.” He further asked if “people will use the material when it is completed?” and answered that “If it comes out in final form as usable as the Detailing Manual, the Design Handbooks, and similar aids, it will not only be used but will be a best seller.” Reese suggested two approaches that might be considered: (a) Show by discussion and illustration how joints should be designed, and (b) for a group of typical joints, provide requirements that are “more in geometric than stress terms” that detailers can use. He thought it was unlikely that there was sufficient research to prepare such a document but
that the committee could also suggest research that would broaden the knowledge base.

The remainder of his letter identifies various groupings of typical joints and lists problems and questions about each group. Reese’s letter set the course of the committee for the next eight years when the first report was published. Many of his questions remain unanswered today. His insight and his ability to pose the right questions were key factors in directing the committee to developing a document that would influence design and construction of reinforced concrete structures.

Los Angeles, March 1968—Problems in joint construction were discussed by the practitioner members of the committee. Broyles and Black described current detailing practices. Congestion problems were often not solved until the steel was placed in the field. Details were not checked by designers for congestion of bars and for ease of placement. The committee discussed a variety of issues related to beam-column joints. Pinkham suggested that beam-column connections probably were the most general case and the simplest to consider is a joint where the beams and columns are approximately the same size or where the beam is slightly smaller than the column so that the beam bars fit inside the column bars. Degenkolb suggested that the connection should develop the strength of the members and should be ductile enough to “hang together” under the applied forces or deformations. Various ideas for “keeping failures out of connections and in the members” were discussed.

The committee assembled a number of reports from the literature on failures in which some aspect of joint design or construction was involved (6-10).

DEVELOPMENT OF REPORT

Following the Los Angeles meeting a draft outline of “Requirements for Beam-Column Joints” was prepared by a subcommittee for discussion by the entire committee. By attempting to set specific, sometimes quantitative, requirements for beam-column joint design, various concerns and suggestions for changes were elicited from the committee members. Four joint cases and the primary design criterion for each case were proposed:

1: Strength
2: Energy absorption and strength (blast loading, for example)
3: Limited ductility and strength (ordinary joint, ultimate strength design)
4. Resistance to several repetitions of overload with plastic deformations developed (structure in a high seismic zone)

A key element of that first draft was the requirement that the joint must be designed to develop specified levels of steel strains (given as a multiple of yield strain) for each type without impairing its ability to transfer or resist the forces acting on the joint. The stresses in the bars (or forces acting on the joint) had to be consistent with the strains. The intent was to require consideration of strain
hardening when large deformations were expected in the members at the joint. Where ductility is required, a “Curvature multiplier” was defined.

A Case 1 joint should have sufficient strength so that the strength of the members is developed before the joint reaches its capacity. A cantilever beam framing into a column is an example of such a case. Ductility is not a concern. For Case 2, energy absorption during large deformations under unidirectional overloading was of prime concern. Overloads due to blast or wind loading were considered to produce such deformations. For Case 3, a joint in a frame designed using ultimate strength design principles was envisioned. In order to develop the strength of all the elements, some moment redistribution is needed. The elements framing into a joint must undergo inelastic deformation for redistribution to occur. For Case 3 joints, some minimum level of ductility is needed and the joint strength must be maintained as the inelastic deformations are developed. In Case 3 joints, the strains are not expected to go into the strain hardening range. However, Case 4 joints were expected to develop a rotation capacity of five times that corresponding to first yield for at least five load/deformation oscillations (load reversals). The details had to be such that the strength of the joint did not fall below 75 per cent of the first cycle strength. The focus of Case 4 was frame structures located in zones of high seismicity.

Since there was little test data available regarding joint response, the strength of the joint was determined using shear and bond values from ACI 318.

Memphis, Nov. 1968—During this meeting, many issues regarding the document for beam-column joints were clarified as the draft outline was discussed. It was agreed that detailing to avoid congestion was a critical issue and that design of joints should be considered to include detailing. There was disagreement as to how much of the structure would be considered part of the joint. Many felt that parts of the members framing into the joint should be included while others argued that if large segments of the members are included, the whole structure could be considered joint design! A major point was the decision that the joint must be designed for the capacities of the members framing into the joint and not only to develop the design forces. The idea of a multiplier on reinforcement stresses to replace a strain multiplier was proposed.

Jennings discussed the problem of specifying required ductility and load reversals for joints. He used an example to illustrate that local or joint ductility could be considerably greater than overall frame ductility. Using measured accelerograph records of a reinforced concrete building from the 1964 Niigata earthquake, he showed that number of plastic excursions were fewer than the number of cycles of alternating load. He indicated that the proposed requirement of 5 times yield deformation for 5 reversals seemed reasonable for Case 4 joints.

Subsequent Meetings and Discussions—Work on the report on beam-column joint design was completed in 1975 and it was published in the July 1976 ACI Journal (11). Many meetings were devoted to reaching agreement on various parts of the report. At one point, there was concern that the report was becoming too “academic.” However, these concerns were addressed through the development of illustrated design examples. The number of joint cases was eventually reduced to two—Cases 1-3 were considered to be covered by a Type 1
Joint and Case 4 became a Type 2 Joint. A stress multiplier was introduced using the results of tests in which strains were measured in hinging regions of beams. For Type 1, strains were not expected to reach strain hardening so the stress multiplier was 1.0. After considering a range of stress multipliers for Type 2 from 1.1 to more than 2, the committee adopted a value of 1.25 for joints where large deformations and deformation reversals into the inelastic range were expected.

Finally, it was decided that the joint included only “the portion of the column within the depth of the beams framing into the column” as shown in Fig. 2. The recommendations were limited to joints where the column width was equal to or greater than the beam width. Perhaps the most contentious aspect of the report was the calculation of the shear strength and the requirements for transverse reinforcement within the joint. The common practice was to terminate the column transverse reinforcement above and below the joint (or floor members). There was concern that the addition of joint transverse reinforcement would raise costs and affect the competitiveness of concrete construction. The committee decided that it would present the “Recommendations” in a format similar to that used in the Building Code. In this way, an explanation of the requirements would be immediately available to the user.

THE 1976 REPORT

The abstract indicated, “The recommendations are based on laboratory as well as field experience and provide a state-of-the-art summary of current information. Areas for needed research are identified. Design examples are presented to illustrate the use of the design recommendations.” The format of the report was as follows:

Recommendations:
   Introduction
   Classification of beam-column joints
   Design considerations
      Forces
      Critical Sections
   Requirements for joints
      Serviceability
      Strength
   Areas of needed research
   Design Examples

Recommendations

Joint classification--The report identified the differences between joints in structures where strength was the prime concern (Type 1) and those where large inelastic, cyclic deformations were expected (Type 2). Where large inelastic deformations were imposed on the members, the joint had to be designed to withstand the increases in bar stresses that would occur under strain hardening.
Tests (12) showed that strains at the face of a support, where the member was hinging, would pass through the yield plateau very quickly as shown in Fig. 3. It was also shown analytically that such increases would be expected where there was a moment gradient in the member at the face of the column. As a result, the bars would be stressed beyond yield and a stress multiplier of 1.25 was to be used to determine shear forces in the joint and for anchorage of bars in or through the joint. The transfer of forces from the members to the joint was shown to produce diagonal cracking similar to that observed in the field.

Shear and transverse reinforcement—Although there were suggestions that the shear capacity of the concrete in the joint should be taken as zero for a Type 2 joint, the committee decided that there was insufficient data to warrant such a recommendation. Transverse reinforcement was needed to provide whatever shear could not carried by the concrete.

In addition, there were minimum area and spacing requirements for the transverse reinforcement. The need for a “basket” to contain the joint core was established. Because joints often had members framing in from more than two directions, designers implicitly considered the joint to be confined without considering the size of those members or the efficiency of beams and columns in confining the joint. A review of joint failures under earthquake loading indicated that for transverse reinforcement to be effective, the hook extension needed to be anchored into the confined core of the joint. The geometry of the transverse reinforcement was described so that designers understood the need to provide 135° hooks to properly confine the joint core in Type 2 joints. Further, crossties were to be bent around peripheral ties (a requirement that was changed in later reports because it was difficult to achieve in the field and there was no evidence that indicated such cross-ties substantially improved confinement).

Column/beam strength—The flexural capacity of the columns was required to be greater than the capacity of the beams framing into the joint. In Type 2 joints, the beam capacity had to be determined using the stress multiplier.

Hooked bar anchorage—Finally, the provisions for anchorage of hooked bars in the joint required a straight bar anchorage length needed to make up the difference between the hook capacity and the expected bar stress at the critical section defined at the face of the core (column bars or ties). The straight bar length could not be added to the tail extension as was the practice in many cases. This provision was included so that designers or detailers would not use large hooked bars in small columns or walls.

Needed Research

Perhaps the most important aspect of the report was the identification of research needs. Eleven areas of research were listed corresponding with those issues where there was found to be inadequate information available for developing the design recommendations and answering some of the questions that had been posed. These areas included effectiveness of confinement, influence of lateral members (including size and location), shear strength of joint (effective area,
effect of axial loads), anchorage of both straight and hooked bars, and influence of loading in orthogonal directions. It should be noted that over the next twenty years, many of the research areas were addressed through experimental studies in the United States, Japan, New Zealand, Canada, and China. Much of the research conducted in this period was reported in a 1991 ACI Special Publication “Design of Beam-Column Joints for Seismic Resistance” (13). Subsequent revisions of the report have been based on the research that was stimulated by the initial report.

Design Examples

Interior and exterior joints of both of joint types were included in the examples. Calculations, free-body sketches, and reinforcement details were included. Three-dimensional illustrations are shown in Fig. 4 for an exterior Type 1 joint and in Fig. 5 for an exterior Type 2. The illustrations were intended to show details of the transverse reinforcement, as well as, the locations of bars and the checks needed to avoid steel congestion and fabrication problems. The closer spacing of transverse steel and the additional crossties in the Type 2 joint can be seen. The hook details (135° vs. 90° hooks) and the engagement of crossties around the peripheral tie illustrate the considerably more rigorous detailing for Type 2 joints.

When the report was published, there was considerable criticism of the requirements. Many felt that the cost of the added detailing requirements would make concrete structures less competitive and the congestion of reinforcement in the joint would make it nearly impossible to construct such frame systems in seismic zones. However, it was found that the cost of the added details represented a very small fraction of the overall cost of a building and contractors quickly developed methods for fabricating and placing reinforcement that satisfied the design requirements.

IMPACT OF REPORT

The 1983 ACI Building Code Requirements (3) included an appendix on “Special Provisions for Seismic Design. Sections of that appendix drew heavily on the ACI 352 report for beam-column joint design procedures. In 1982, a code published in New Zealand (14) included significant new design procedures that reflected studies of reinforced concrete structures conducted there. In 1989, design provisions for beam-column joints were included in a new design document in Japan (15). There were some significant differences in the ACI, NZ, and Japanese codes in the approaches to beam-column joint design and detailing. The differences were of concern to designers and their concerns were noted in a 1983 Applied Technology Report (16) in which a number of topics that coincided with those of the ACI 352 report were identified as needing further research. Between 1984 and 1990, a series of test programs in Japan, New Zealand, China, and the United States were conducted and discussed in meetings of researchers and designers. Many of the papers presented at the final meeting held in Hawaii in May 1989 were included in ACI SP-123 “Design of Beam-Column Joints for
Seismic Resistance “(13). A statement in the Preface of SP-123 best summarizes the experience of the participants in the research program and of those that attended the meetings. “The project demonstrated that researchers working cooperatively can accomplish a great deal through a coordinated, focussed effort.” The task that started in 1966 when the committee was appointed led to exactly the type of process that Siess and Sheets described. “Study collectively the results of research and current practices” and use that “collective judgement” to prepare “a summary of existing knowledge.” The committee served as an “interpreter” to fill the gap between research and practice.

CONCLUSIONS

As was stated in the Introduction, many committees and many committee members see the inclusion of their “knowledge” in a major code or standard such as ACI 318 as the only acceptable or satisfactory validation of their work. The success of a committee in developing knowledge should not be judged in this way. Codes and standards have different objectives. In the case of ACI 318, the objective is to provide the minimum standards for structural concrete for public safety. The code cannot be the only source of new technology and innovation. In fact, as Siess has suggested, it may be desirable to wait until there is sufficient data and understanding of an issue before including it in a Code such as ACI 318.

ACI 352 provides a case study of a committee document that was widely used in practice for many years before any of its recommendations were incorporated into codes or standards. The development of a report permitted the committee to include much more explanatory material than is generally included in a code, to discuss research needs, and to develop design examples that showed how the recommendations could be implemented.

Our profession and the concrete community are best served when the views of a diverse group of individuals, all having experience in a topic, are heard and collectively the group applies its judgement to provide guidance to the rest of the community.
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Appendix A: Roster of Members
Joint ACI-ASCE Committee on Joints and Connections in Monolithic Structures
(ACI 352)

Membership in 1967

W. C. Black, Bethlehem Steel Corp., Bethlehem, PA
C. W. Broyles, Jr., United States Steel, Pittsburgh, PA
R. S. Fling, Fling and Eeman, Columbus, OH
E. Ference, Naval Facilities Engineering Command, Washington, DC
N. W. Hanson, Portland Cement Association, Skokie, IL
P. C. Jennings, California Institute of Technology, Pasadena, CA
J. O. Jirsa, Secretary*, Rice University, Houston, TX
G. F. Leyh, Concrete Reinforcing Steel Institute, Chicago, IL
R. F. Mast, ABAM Engineers, Tacoma, WA
C. W. Pinkham*, S. B. Barnes and Assoc., Los Angeles, CA
R. C. Reese, Vice Chairman*, Raymond C. Reese Associates, Toledo, OH
H. L. Scoggins, Hinsdale, IL
R. E. Shewmaker, Atomic Energy Commission, Bethesda, MD
R. G. Smith, Erico Products, Inc., Harrisburg, PA
M. A. Sozen, Chairman*, University of Illinois, Urbana, IL

*Members of ASCE Control Group

Membership in 1976 when “Recommendations” were published

From original group: Black, Broyles, Hanson (Secretary), Jirsa (Chairman),
Leyh, Pinkham, Reese, Sozen

Members added:

J. E. Barry, Erico Products, Inc. Cleveland, OH
V. V. Bertero, University of California, Berkeley, CA
J. P. Colaco, Colaco Engineers, Inc., Houston, TX
C. R. Hays, Rust Engineering Co., Birmingham, AL
C. Hernandez, Caracas, Venezuela
R. Park, University of Canterbury, Christchurch, New Zealand
A. L. Parme, Consulting Engineer, La Jolla, CA
S. M. Uzumeri, University of Toronto, Canada
L. A. Wyllie, Jr., H. J. Degenkolb & Associates, San Francisco, CA
Fig. 1. Relationship of research, codes, and practice—From Siess (1)

Fig. 2. Portion of structure defined as a joint—From Ref. 11.
Fig. 3. Relationship between measured strain and applied deflection of a cantilever beam—Ref. 12.
Fig. 4. Design Example of Type 1, Exterior Joint—From Ref. 11.

Fig. 5. Design Example of Type 2, Exterior Joint—From Ref. 11.
Design of Confining Reinforcement in Columns for Seismic Performance

by S. A. Sheikh

Synopsis: Professor S. M. Uzumeri and the author initiated a research program on the seismic resistance of reinforced concrete columns in mid seventies. The first phase of this work concentrated on the behavior of rectilinearly confined concrete columns under axial compression. In addition to carrying out an extensive experimental program, an analytical model for the mechanism of concrete confinement by rectilinear transverse reinforcement was developed. This was followed by an investigation of square columns under combined axial load and flexure. After identifying the important variables that affected the mechanism of confinement and section behavior, the research advanced to a study of column behavior under simulated earthquake loads. Distribution of longitudinal and lateral reinforcement, level of axial load, spacing of ties and the type of lateral support provided to the longitudinal bars were found to significantly affect the ductile performance of a column.

Based on the results from extensive experimental and analytical research, a design procedure was developed in which the amount of tie reinforcement and the detailing of both longitudinal and lateral reinforcement are determined for a required ductile performance of a column subjected to a given axial load.

This paper presents selected results from the work carried out over the last several years. An example demonstrating the application of the design procedure is also included.

Keywords: axial load; confined concrete columns; seismic performance
ACI Fellow Shamim A. Sheikh is a Professor of Civil Engineering at the University of Toronto. He worked for his graduate research on the seismic resistance of concrete columns in the mid-seventies under the supervision of Professor S.M. Uzumeri. Currently he is the Chair of the Committee ACI-ASCE 441, Reinforced Concrete Columns. In 1999, he received the ACI Structural Research Award for a paper on the design of ductile concrete columns.

INTRODUCTION

Lateral forces prescribed by various seismic codes (1, 2) are significantly less than the elastic inertia response loads for most framed structures. The safety of the structure during a major earthquake then depends on its ability to dissipate seismic energy in an inelastic mode which requires the structure to deform beyond elastic stage while maintaining near maximum load carrying capacity. Although it is preferable to dissipate seismic energy by post-elastic deformations in beams, column hinging cannot be avoided as evidenced by post-earthquake inspections of numerous structures damaged to varying degrees. Ductile performance of columns is therefore an important aspect of good seismic design and requires appropriately designed and detailed longitudinal and lateral confining reinforcement in the potential plastic hinge regions.

A research program on confined concrete columns was initiated in 1975 by Professor S.M. Uzumeri and the author at the University of Toronto. This program continued over the next several years and involved many other researchers. Part of the work was carried out at the University of Houston in the eighties. In this paper, a summary of this work that describes important aspects of the behavior and design of rectilinearly confined concrete columns is presented. The first comprehensive report on the results from the tests on concentrically compressed confined normal strength concrete was published in 1980 (3) following which an analytical model was developed which can be used to determine the complete behaviour of concrete confined by rectilinear transverse reinforcement (4). Further tests on normal and high strength confined concrete columns were conducted to corroborate the analytical results from the model and to confirm the stress patterns within the confined concrete core that were theorized in the model. In the next phase of work, tests on square concrete columns were carried out under a combination of axial load, shear and flexure, some under monotonic loads and most under cyclic excursions to simulate earthquake loads (5-8). Concrete strength in these tests was varied between 30 MPa and 60 MPa. From the results of analytical and experimental research, a set of design criteria was developed that established the level of performance of columns with respect to ductility and energy dissipation capacity. A performance-based design procedure for confining reinforcement was suggested for concrete strength up to 60 MPa (9). The most recent work is aimed at extending the understanding of mechanism of concrete confinement and design procedure to
concrete with strength up to 100 MPa. In addition, the use of fibre reinforced polymers (FRP) as a replacement for lateral steel to confine concrete is being investigated.

COLUMN BEHAVIOR UNDER AXIAL LOAD

It is generally believed that circular confinement is superior to rectilinear confinement. By virtue of its shape the circular lateral reinforcement provides continuous uniform confinement to the concrete along its perimeter. In the case of rectilinear confinement, however, the lateral pressure is primarily applied at the corners of the ties. This makes the efficiency of confinement to be a function of the distribution of steel i.e. configuration of lateral and longitudinal reinforcement. In both cases, the effectiveness of confinement is affected by the spacing of the lateral reinforcement.

Figure 1 shows results from specimens tested under axial compression by various researchers (3, 10, 11) along with steel configurations for all the specimens. The unconfined concrete strength varied between 33.8 MPa and 45.5 MPa. From a comparison of the specimens, it can be concluded that rectilinearly tied columns can be detailed to provide ductility comparable to that of columns with circular spirals provided that the longitudinal bars are well distributed around the core and supported by closely spaced ties to create near-uniform lateral pressure. Steel configuration and spacing of lateral steel are thus shown to be important parameters in the design of confining steel.

The amount of lateral steel determines the intensity of the confining pressure on concrete. An increase in the lateral steel content results in an increase in strength and ductility of concrete in well-confined columns (Figure 2). Increases in the volumetric ratio of tie steel from 0.76% to 1.5% and 2.3% showed steady improvements in the performance of the columns. The effect of variation in tie spacing in these specimens is considered to be small compared with that of the amount of lateral reinforcement. All the columns were 305 mm square and 1.96 m long. The core size measured from the centre line of the perimeter ties was 267 mm square.

In the development of the analytical model for confinement in tied columns, it was hypothesized that the lateral pressure is applied to the concrete core only at tie corners (4). Under concentrated forces applied at the corners only part of the concrete in the core is subjected to lateral confining pressure. This effectively confined concrete separates from the rest of the core concrete as shown in Figure 3. A similar division between the effectively confined concrete and the rest of the core takes place in the longitudinal direction of the member as well as shown in the figure. Based on this conceptual model, a stress-strain curve for confined concrete was proposed as shown in Figure 4. A standard second degree parabolic equation represents the ascending part OA of the curve followed by three straight lines that describe the complete behavior of concrete.
Constitutive relations needed to develop the entire curve for a square core section are given below.

\[ f'_{cc} = K_s f_{cp} \]  
\[ K_s = 1.0 + \frac{B^2}{0.14 P_{occ}} \left[ \left( 1 - \frac{nC^2}{5.5B^2} \right) \left( 1 - \frac{s}{2B} \right)^2 \right] \sqrt{f_{s} f'_c} \]  
\[ \epsilon_{s1} = K_s \epsilon_0 \]  
\[ \frac{\epsilon_{s2}}{\epsilon_0} = 1 + \frac{2.48}{C} \left[ 1 - 5 \left( \frac{s}{B} \right)^2 \right] \frac{\rho_s f'_s}{f'_c} \]  
\[ \epsilon_{s85} = 0.225 \rho_s \frac{B}{s} + \epsilon_{s2} \]  

where \( P_{occ} = K_p f'_c (A_{co} - A_s) \); \( A_{co} \) = area of core measured from center to center of the perimeter tie; \( A_s \) = area of longitudinal steel; \( B \) = core size measured from center to center or perimeter tie; \( C \) = distance between laterally supported longitudinal bars of \( 4B/n \); \( f_c \) = cylinder strength of concrete in MPa; \( f_{cp} \) = strength of unconfined concrete in the column = \( K_p f'_c \); \( f'_s \) = stress in the lateral steel in MPa; \( K_p \) = ratio of unconfined concrete strength in the column to \( f'_c \); \( n \) = number of arcs containing concrete that is not effectively confined, also equal to the number of laterally supported longitudinal bars; \( s \) = tie spacing; \( \rho_s \) = ratio of the volume of tie steel to the volume of core; and \( \epsilon_0 \) = strain corresponding to the maximum stress in unconfined concrete. The linear dimensions are in mm.

During a more recent test program, two 205 mm square confined concrete specimens were tested to a point just beyond the peak load and then unloaded. The specimens were then cut across and along the test zone. The cut surfaces were painted, polished and washed to highlight the cracks inside the core. Figure 5 shows the pictures of these surfaces along with a cross section with assumed separation curves between the effectively confined concrete and the rest of the core. A comparison of the sketches in Figure 3 and Figure 5 shows that the distribution of confining stress and the resulting area of effectively confined concrete assumed in the analytical model simulate the actual conditions in a reasonably accurate manner.

The analytical model was used along with a few other available models to evaluate the behavior of concrete confined by rectilinear transverse reinforcement in concentrically loaded columns tested by various researchers (12). Figure 6 shows an excellent comparison of the experimental behavior with the analytical response from the proposed model for a column specimen tested by Scott et al (13).
COLUMN BEHAVIOR UNDER AXIAL LOAD AND FLEXURE

The variables, steel configuration, tie spacing and amount of transverse reinforcement, that significantly influenced the performance of confined concrete under concentric compression were further investigated with tests on columns under combined axial load and monotonic flexure (5, 6). A total of 16 specimens, 305 mm square and 2.74 m long, were tested. An additional parameter studied for its effect on flexural response of confined concrete was the level of axial load. The moment - curvature responses of three column specimens are shown in Figure 7. Specimens D-5 and D-15 were exactly similar and cast together. Specimen D-5 was tested under an axial load level $P/P_0$ of 0.39 whereas $P/P_0$ for D-15 was 0.60. Both specimens were subjected to axial load first followed by the application of lateral load that caused shear-free zone in approximately the middle third of the column length. The tests were terminated when the specimens were unable to maintain the applied axial load. As is obvious in Figure 7, an increase in the axial load caused a significant decrease in the energy dissipation capacity and ductility of the column section. An examination of load-deflection responses yields similar conclusions. Specimen D-14, also included in Figure 7, can be compared with D-15 to directly evaluate the beneficial effects of an increase in the amount of lateral reinforcement in columns.

Figure 8 shows the curvature ductility ratio of the specimens as influenced by the level of axial load, lateral reinforcement ratio and tie spacing. The letter designations A, D, E and F refer to the steel configurations used in this series of tests. It can be seen that an increase in axial load and decrease in the amount of lateral steel resulted in significant reduction in the ductility of all column sections. The least effect of these variables is on Configuration E, because the efficiency of confinement is very low in columns with only four laterally supported longitudinal bars. The effect of tie spacing was not very pronounced particularly in the range of the spacing needed for avoiding premature buckling.

COLUMN BEHAVIOR UNDER SIMULATED EARTHQUAKE LOAD

In this series of tests (7, 8) each specimen consisted of a 305 x 305 x 1473 - mm column cast integrally with a 508 x 762 x 813 - mm stub which represented the beam column joint area or footing. The column part of the specimen simulated a column in a framed building between the mid point in a storey (point of contraflexure) and the floor level or the footing. The critical section (point of maximum moment) of the column was adjacent to the stub.

Each specimen was subjected to a pre-determined axial load first that remained constant throughout the test, followed by the application of a standard lateral displacement sequence. The displacement history consisted of one cycle to 0.75 $\Delta$ followed by 2 cycles each to $\Delta$, 2$\Delta$, 3$\Delta$ ... so on until the column was unable to sustain the applied axial load. The displacement $\Delta$ was calculated theoretically and may be defined as the yield or elastic lateral displacement for the
unconfined concrete specimen, at which the specimen behavior departs significantly from a straight line. The potential plastic hinge region was thus subjected to a constant axial load and cyclic shear and flexure.

Failure in all the columns initiated at about 250 mm to 300 mm away from the stub. The moment curvature responses of the most damaged section for a few of these specimens are shown in Figure 9 to stress the importance of the variables that significantly influence the behaviour of confined concrete columns under simulated earthquake loads. The variables that can be studied in this figure are unconfined concrete strength, axial load level, steel configuration and the amount of lateral reinforcement.

Nominal concrete strength in these specimens varied between 31 MPa and 58 MPa. The level of axial load is generally measured by indices $P/f_c' A_g$ and $P/P_0$. For columns with similar $f_c'$, both these indices provide similar comparison. However, for different $f_c'$ values in columns the comparison using $P/f_c' A_g$ may not remain valid.

The effect of a change in axial load on column response can be evaluated from Specimens AS-3 and AS-17 which are almost identical in every regard. Since normal strength concrete was used in both the specimens, either of the two indices, $P/f_c' A_g$ or $P/P_0$, can be used to describe the level of axial load. Increase in axial load from $0.60 f_c' A_g (0.50 P_0)$ to $0.77 f_c' A_g (0.63 P_0)$ caused a significant reduction (45%) in curvature ductility and energy dissipation capacity of the column. Specimens AS-3 and AS3H contained the same amount of lateral reinforcement and were tested under similar axial loads as represented by $P/f_c' A_g$. Specimen AS-3H, with $f_c' = 54$ MPa, displayed relatively more brittle behavior compared to Specimen AS-3 in which $f_c = 33$ MPa. If the amount of lateral reinforcement is increased in proportion to the strength of concrete, then Specimen AS-18H instead of AS-3H, can be compared to AS-3. Both these specimens contained about 45 percent more tie steel than that required by the ACI Code (14) and were tested under $P/f_c' A_g$ approximately equal to 0.6. Ductility and energy dissipation capacity of higher strength concrete specimen were still considerably lower.

Another pair of specimens that can be compared in Figure 9 consists of Specimens AS-17 and AS-18H. Both these specimens contained the lateral reinforcement that was approximately 150 percent of that required by the seismic provisions of the ACI Code (14). The axial load in both the specimens was about 62 percent of $P_0$ although $P/f_c' A_g$ was 0.77 for AS-17 ($f_c' = 31$ MPa) and 0.64 for AS-18H ($f_c' = 55$ MPa). The moment-curvature responses, ductility parameters and energy dissipation capacities of these specimens are very similar. A similar conclusion can also be drawn from a comparison of Specimens AS-18 and AS-20H in Figure 9. Use of index $P/P_0$ appears to be more rational than $P/f_c' A_g$ to evaluate the effects of axial load level on the performance of confined concrete columns particularly if the strength of unconfined concrete is a variable.
Specimen AS-19 is included in Figure 9 to demonstrate that for lower axial loads smaller amount of lateral reinforcement can produce very ductile column.

Effect of steel configuration can be studied by comparing Specimens ES-13, FS-9 and AS-17. The amount of lateral and longitudinal reinforcement, level of axial load and concrete strength in these specimens were similar and they were tested in exactly the same manner. Specimen AS-17 displayed the most ductile behaviour of the three columns while Specimen ES-13 was able to dissipate the least amount of energy. In Specimens AS-17 and FS-9 all the bars were supported by tie bends but only in Specimen AS-17 all the hooks were anchored in the confined core. In Specimen FS-9 the middle bars were laterally supported at alternate points by cross ties with 90° hooks that were not anchored in the core. At small deformations, the support provided by the cross ties to the middle bars is reasonable. However, at large deformations, the cross ties with 90° hooks not anchored in the confined core, were unable to effectively support the bars particularly when the column was subjected to large axial load. In Specimen ES-13, only four corner bars were effectively supported by the perimeter ties.

PERFORMANCE-BASED DESIGN OF CONFINING STEEL

A procedure to design rectilinear confining steel has been suggested (9) for columns in which the amount of steel required is determined for a required ductile performance under a given axial load. The distribution of longitudinal and lateral reinforcement forms an integral part of the design process. Steel configurations are divided into three categories as shown in Figure 10, Category I being the least efficient and Category III the most efficient. Figure 11 outlines the limiting conditions for steel configurations that can be used to design ductile columns for a given performance.

The ductile performance of a column is defined in terms of curvature ductility factor, $\mu_\phi$, cumulative ductility ratio, $N_\phi$, and energy index, $E$, as shown in Figure 12. Based on the tests conducted it was observed that a reasonable correlation existed between $\mu_\phi$, $N_\phi$, and $E$. For $\mu_\phi = 16$, the values of $N_{\phi,80}$ and $E_{80}$ are 64 and 575, respectively. The subscript 80 refers to the response cycle in which the moment is dropped to 80 percent of the maximum value. A column section with this level of deformability is defined as highly ductile. With $\mu_\phi$ value of between 8 and 16, the section is defined as moderately ductile. A limited ductility column has $\mu_\phi < 8$ and is considered to be unsuitable for seismic resistance.

The amount of rectilinear confining steel can be calculated from the following equations.

$$A_{sh} = \alpha \left[ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right] \left( \frac{\mu_\phi}{29} \right)^{1.15} A_{sh,c}$$

(6)
A_{sh,c} in Equation 6 is the total cross-sectional area of transverse reinforcement as required in the ACI Code (14). A_{sh} is the A_{sh,c} modified for the required ductile performance under a given axial load. A simplified version of Eq. 6 is given below

\[ A_{sh} = \alpha \left[ 6 \frac{P}{P_0} - 1.4 \left( \frac{\mu_+}{18} \right) \right] A_{sh,c} \geq \alpha \frac{\mu_+}{18} A_{sh,c} \]  

(7)

Factor \( \alpha \) is unity for Category III configurations (Figure 10) and for Category II configurations as long as the prescribed limiting conditions (Figure 11) are met.

For Category I configurations \( \alpha \) is approximately equal to 2.5. As suggested in Figure 11, these steel configurations may be reliable only for moderate ductility columns under axial load below the balance point. For this low levels of axial load, \( 1 + 13 \left( \frac{P}{P_0} \right)^5 \approx 1.0 \). Equations 6 and 7 thus reduce to Equations 8 and 9, respectively for Category I steel configurations in which only four corner bars are laterally supported by closed ties and the hooks are anchored in the confined core.

\[ A_{sh} = \frac{(\mu_+)^{1.15}}{11.5} A_{sh,c} \]  

(8)

\[ A_{sh} = \frac{\mu_+}{7.2} A_{sh,c} \]  

(9)

Hoop spacing plays a significant role in the mechanism of concrete confinement such that the larger spacing results in reduced efficiency. However, in columns designed to behave in a ductile manner, when subjected to seismic excursions in the inelastic range, the hoop spacing is severely limited to avoid premature buckling of longitudinal bars. It is generally agreed that the hoop spacing in such columns should be the smallest of B/3, 6d_b and 200 mm where B is the core width and d_b is the diameter of the longitudinal bars (9, 15). The proposed procedure can thus be used to design the confining steel for a given column performance as long as the hoop spacing is within the range suggested above.

The proposed procedure is used to design the confining reinforcement for a 600 mm square column. The amount of tie reinforcement is calculated for curvature ductility factors of 10 and 20 using Equations 6 and 7. The amount of lateral steel (\( A_{sh}/s_{hc} \); \( s \) = tie spacing, \( h_c \) = core dimension measured from the centreline of the perimeter tie) is plotted against the axial load in Figure 13. Also shown in the figure is the amount of tie reinforcement required according to the provisions of ACI 318-95. It is obvious that for moderate ductility demand the code requirements are very conservative for most of the axial load range. However, for higher ductility demands, the code-required amount of transverse
reinforcement is seriously inadequate, particularly under higher levels of axial load.

SUMMARY AND CONCLUSIONS

The work carried out over the last several years on reinforced concrete columns confined by rectilinear transverse reinforcement is summarized. The work was initiated by Professor S.M. Uzumeri and the author in 1975 at the University of Toronto. Initial work dealt with columns with 305 mm square section under concentric compression. An analytical model was developed in which constitutive relations for the behavior of confined concrete were proposed. The model was corroborated with the tests conducted by various researchers. This was followed by the experimental and analytical work on columns subjected to axial load and monotonic flexure. All the parameters that affected the mechanism of confinement and hence the section behavior were identified and investigated by this stage of the work. The final phase of the experimental work presented here involved investigation of a typical column in a building which was modelled between the point of contraflexure at the mid height of a storey and the beam-column joint or a footing. These column specimens were tested under axial load and cyclic shear and flexure simulating earthquake loads.

Experimental evidence indicates that ductile behavior of confined concrete columns under cyclic lateral excursions depends significantly on the level of axial load as well as the distribution of steel and the lateral support provided by the longitudinal bars. These factors and the column ductility demand are not considered in the current code provisions for the design of confining reinforcement. Columns designed according to the current code may display brittle behavior under certain circumstances while in other cases they may be overly conservative in terms of ductility and energy dissipation capacity. To address these issues, a design procedure for tied columns is summarized in this paper in which the amount of confining steel is determined based on curvature ductility demand, level of axial load and the distribution of longitudinal and lateral steel. An example explaining the application of the suggested method is also presented.

ACKNOWLEDGEMENT

The work reported here was carried out using several grants from the Natural Sciences and Engineering Research Council of Canada, U.S. National Science Foundation and State of Texas through Texas Advanced Research and Technology program. Advice from Professor S.M. Uzumeri and Professor T. Paulay in the initial phases of this work played a vital role in the development of this research program. Former graduate students, Dr. C.C. Yeh, Dr. S.K. Khoury, S. Patel and D. Shah made significant contributions to this research which is gratefully acknowledged.
REFERENCES


14. ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-95) and Commentary (ACI 318R-85), American Concrete Institute, Detroit, 1995.

Fig. 1 Concrete behavior as affected by steel configuration

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Long. Steel</th>
<th>Lateral Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: 6” dia. x 12” L</td>
<td>None</td>
<td>0.26” spiral @ 1.8”</td>
</tr>
<tr>
<td>2: 4.5” square prism</td>
<td>None</td>
<td>3/16” ties @ 1.8”</td>
</tr>
<tr>
<td>3: 12” square x 77” L</td>
<td>16 #5</td>
<td>1/4” ties @ 3.75”</td>
</tr>
<tr>
<td>4: 12” square x 77” L</td>
<td>16 #5</td>
<td>1/8” ties @ 1”</td>
</tr>
</tbody>
</table>

Fig. 2 Concrete behavior as affected by the amount of transverse reinforcement and tie spacing
4-bar, 1 hoops configuration

8-bar, 2 hoops configuration

Effectively confined concrete at tie level

Effectively confined concrete at the level between the ties

Tie level

Effectively Confined concrete

$A_{eff} = \lambda A_{co}$

$A_{eff} = \lambda^* A_{co}$

Fig. 3 Effectively confined concrete in the core

Fig. 4 Proposed general stress-strain curve of concrete
Assumed in the Model

Effectively confined concrete

From Experiments

Fig. 5 Verification of assumptions in the analytical model

Fig. 6 Comparison of experimental and analytical behavior of confined concrete under concentric compression
Fig. 7 Effects of axial load and amount of lateral steel

<table>
<thead>
<tr>
<th>$\rho_t$ (%)</th>
<th>$s$ (in)</th>
<th>$\frac{P}{f_c A_g}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-5</td>
<td>1.68</td>
<td>4.50 0.46</td>
</tr>
<tr>
<td>D-14</td>
<td>0.81</td>
<td>4.25 0.75</td>
</tr>
<tr>
<td>D-15</td>
<td>1.68</td>
<td>4.50 0.75</td>
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</tbody>
</table>

Fig. 8 Effect of different variables on curvature ductility
Fig. 9 Behavior of columns under simulated earthquake loads
Fig. 9 Behavior of columns under simulated earthquake loads, continued
Fig. 9 Behavior of columns under simulated earthquake loads, continued

Fig. 10 Categories of steel configurations

Fig. 11 Limiting conditions for steel configurations
\[ \phi_i = \frac{1}{2} (\phi_i^+ + \phi_i^-) \]
\[ M_{i,\text{max}} = \frac{1}{2} (M_{i,+} + M_{i,-}) \]
\[ S_i = \frac{1}{2} (S_{i,+} + S_{i,-}) \]
\[ \mu_b = \frac{\phi_2}{\phi_1} \]
\[ N_b = \sum_{i=1}^{i=m} \phi_i \]
\[ E = \frac{1}{M_{\text{max}} \phi_1} \sum_{i=1}^{i=m} e_i \frac{L_f}{t} \left( \frac{S_i}{S_1} \right) \left( \frac{\phi_i^-}{\phi_1} \right)^2 \]

Fig. 12 Ductility parameters

Column section:
600mm x 600mm
Concrete cover 40 mm
\[ f'_c = 30 \text{ MPa} \]
\[ f_y = 400 \text{ MPa} \]

Fig. 13 Application of the design procedure
Aspects of Seismic Evaluation and Retrofit of Canadian Bridges

by D. Mitchell

Synopsis: The approach for seismic evaluation and guidelines for seismic retrofit in the Canadian Highway Bridge Design code are presented. A number of common seismic deficiencies found in bridge structures are discussed. The reversed cyclic loading behavior of columns with lap splices at their bases, outrigger beam-column connections and columns with hinges at their bases are discussed. Methods suitable for retrofit of a number these deficiencies are described. The responses of these elements after these retrofit strategies had been carried out are also presented.

Keywords: bridges; codes; ductility; earthquake-resistant structures; evaluation; reinforced concrete; retrofit
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INTRODUCTION

Fig. 1 shows a map of Canada with the contours of peak horizontal ground accelerations, having a probability of exceedance of 10% in 50 years. This map, together with the contours of peak horizontal velocities, having a probability of exceedance of 10% in 50 years, have been prepared by the Geological Survey of Canada and appear in the 1995 National Building Code of Canada (1). The Canadian Highway Bridge Design Code (CHBDC) (2) uses the map shown in Fig. 1 as a basis for determining the seismic design forces or the ground motion characteristics for analysis. Cities on the west coast of Canada, such as Vancouver and Victoria are located in regions of significant anticipated seismic activity, while cities such as Montreal, Ottawa and Quebec City are in regions of more moderate seismic activity.

While many important bridge structures on the west coast have undergone seismic evaluation and retrofit, the major evaluation and retrofit work in eastern Canada has been focussed on problems associated with the improvement of deteriorating infrastructure, rather than on seismic upgrading. This paper presents the seismic evaluation approach adopted by the Canadian Highway Bridge Design Code (2, 3), reviews some of the deficiencies in existing bridges in eastern Canada and discusses results from research programs to investigate these deficiencies and possible retrofit techniques.

CHBDC APPROACH TO SEISMIC EVALUATION OF EXISTING BRIDGES

The Canadian Highway Bridge Design Code (CHBDC) has adopted a similar philosophy to the AASHTO requirements (4) in distinguishing the "importance" of the bridge structure, but uses different terms for classifying importance. The CHBDC uses importance factors of 3.0 for "Lifeline Bridges", 1.5 for "Emergency-Route Bridges" and 1.0 for "Other Bridges". The CHBDC provides provisions for the evaluation of existing structures for "emergency-route bridges" and "other Bridges", with "lifeline bridges" requiring special studies. The minimum analysis requirements for seismic evaluation are summarized in Table 1. The Seismic
Performance Zone is determined from the peak ground acceleration for 10% probability of exceedance in 50 years and from the Importance Category. The designation "LE" refers to limited evaluation and involves providing minimum seat widths or longitudinal restrainers and a minimum capacity for bearings. In addition, the potential for soil liquefaction, slope instability, approach fill settlements and increases in lateral earth pressures must be considered. The designation "SM" refers to single-mode elastic analysis and "MM" refers to the multi-mode spectral method. Push-over analysis and time-history analysis methods are also permitted. Regular bridges are defined as having less than seven spans, no abrupt or unusual changes in weight, stiffness or geometry and no large changes in these parameters from span-to-span or support-to-support (excluding abutments).

The CHBDC recognizes the important role that the regulatory authority plays in setting appropriate analysis and design requirements for evaluating existing bridges. Therefore, adjustments to the evaluation procedure are permitted if approved by the regulatory authority, such as the required analysis method, accounting for the remaining service life of the bridge and load cases to be considered. The load factors and load combinations for evaluation purposes are given as:

\[ 1.0 \, D + 1.0 \, EQ \]  

where \( D \) is the dead load and \( EQ \) is the earthquake load expressed as a design force. This represents a reduction from that required for new bridges where the earthquake effects are combined with minimum (0.8D) and maximum (1.25D) gravity loads. Different orthogonal loading cases must be combined in the same manner as for the design of new bridges.

The evaluation procedure involves the calculation of the required response modification factor, \( R_{req} \), from the following:

\[ R_{req} = \frac{S_e}{C} \]  

where

- \( S_e \) = seismic force effect assuming all members remain elastic, except as limited by capacities of other members
- \( C \) = member reserve capacity after the effects of dead load have been considered.

Member capacities are calculated from the unfactored nominal resistances of the members. In the determination of the nominal resistances of existing members the code emphasizes the need to take account of the effects of all differences in those members from the design and detailing requirements for new bridges. For concrete elements these effects include:

(a) the influence of premature bond failures due to inadequate anchorage or splice length,
Mitchell

(b) the influence of reduced concrete contribution to the shear resistance as the
ductility demand increases in reinforced concrete members,
(c) the influence of inadequately detailed beam-column and column-footing
joints, and
(d) the influence of defects or deterioration on member performance.

After determining $S_e$, $C$ and $R_{req}$, the engineer must determine the appropriate
response factor required for the existing substructure, $R_{prov}$. The code requires that
the determination of the overall performance and the $R_{prov}$ must account for the
following:

(a) the consequences of the specific detailing,
(b) consideration of all possible failure modes, and
(c) the expected length of inelastic deformations.

The CHBDC permits the use of results from reversed cyclic loading tests of
structural components, which are constructed to simulate the as-built details, in
determining a suitable $R_{prov}$.

Elements which have $R_{prov} \geq R_{req}$ are deemed acceptable, while those not meeting
this requirement must undergo rehabilitation unless it can be demonstrated by non-
linear analysis that the consequences would not be detrimental to the performance
of the bridge.

CHBDC GUIDANCE FOR SEISMIC REHABILITATION OF BRIDGES

The CHBDC provides guidance on the seismic rehabilitation of bridges indicating
the following retrofit techniques:

(a) base isolation,
(b) increasing ductility without strengthening,
(c) addition of energy-dissipating devices,
(d) installation of restrainers,
(e) alteration of load paths,
(f) increasing support lengths,
(g) making provisions for inelastic hinging to occur,
(h) strengthening,
(i) improvement of liquefaction-prone soils, and
(j) stabilization of approach fills and adjacent slopes.

The CHBDC requires that the following design aspects be investigated when
assessing seismic rehabilitation measures:

(a) increased stiffness due to strengthening must be accounted for,
(b) the influence of rehabilitation on fatigue life must be assessed,
(c) the influence of rehabilitation on the alteration of load paths must be considered,
(d) the influence of member strengthening on the force demands on other members and joints must be assessed,
(e) rehabilitation measures should avoid damage to inaccessible foundations,
(f) if uplift occurs then guiding of the associated movement and prevention of support loss must be considered,
(g) if base isolation is used then consideration must be given to other loading cases (e.g., wind),
(h) the durability of the rehabilitation measures must be addressed,
(i) the restraint of thermal movement due to added restrainers must be considered,
(j) soil improvement may induce movements or tilting which must be addressed,
(k) the consequences of stage-wise rehabilitation must be considered,
(l) adequate inspection and maintenance of the rehabilitation must be addressed, and
(m) a complete reanalysis of the rehabilitated structure must be carried out to assess performance.

EXAMPLES OF STRUCTURAL DEFICIENCIES

A number of seismic deficiencies in Canadian bridges are discussed below, along with the results of reversed cyclic loading tests on bridge sub-assemblages.

Bridge columns with lap-splices

In eastern Canada in the 1960's, bridge columns were typically constructed with lap splices at their bases. In the Montreal area these columns had lap splices with about 40 bar diameters of lap length. In addition, these columns contained peripheral hoops (some with 90 degree bend anchorages and others with 135 degree bend anchorages), but lacked crossties and had excessive hoop spacings of 18 to 24 in. (457 to 610 mm). Fig. 2 gives the details of Specimen C6, tested at McGill University (5), simulating these deficiencies. The amount of vertical reinforcement in the column corresponds to a reinforcement ratio of 1.23%. The constant applied axial load of 312 kN represents the superstructure dead load and corresponds to 4% of the gross concrete section capacity. Fig. 3 shows the applied shear versus tip deflection response of this specimen, as well as the appearance of the column after testing. The reversed cyclic loading of this specimen resulted in the following behavioral features:

(a) the lap splices severely limited the length over which hinging occurred at the base of the column. This was also observed by Priestley and Park (6),
(b) the maximum displacement ductility, defined as the maximum displacement reached without significant loss in capacity divided by the displacement at "general yielding", was about 3.3, and

c) deterioration of the hysteretic response occurred due to inadequate confinement offered by the poorly detailed transverse reinforcement and the loss of bond over the lap splice.

Fig. 4 compares the predicted flexural nominal resistance and the predicted probable resistance (including strain hardening) with the moment corresponding to "first yield" and the predicted maximum moment. The measured yield stress of the vertical column bars is 400 MPa and the concrete strength at the time of testing was 36.4 MPa. Fig. 4 illustrates the influence of the lap splices on the flexural capacity and also shows how the lap splices restrict the development of hinging over the height of the column.

Fig. 5 and 6 illustrate the design philosophy used and the details of the retrofit of companion Specimen C6R (5), with the same lap splice details as Specimen C6. A reinforced concrete footing block was added to the top of the existing footing to shift the critical flexural section in the column up near the top of the lap splice. This added footing block permits the column to develop a reasonable length of plastic hinging above the added footing block as shown in Fig. 5, and also serves to reinforce the footing, such that the footing becomes a capacity-protected element. In addition, the column was confined by adding a concrete-filled steel jacket over the column. The steel jacket had a thickness of 3.2 mm and the steel had a yield stress of 361 MPa. The concrete infill had a compressive strength of 39.1 MPa and the sleeve was over the full height of the column, except for a 25 mm gap at the base of the column. Fig. 7 shows the applied shear versus tip deflection response of Specimen C6R as well as a photograph of the specimen at the end of testing. The retrofit column was able to develop significant flexural hinging, a 74% increase in the strength of the original column, and a displacement ductility of about 8.3.

Because of the concern for deterioration of the retrofitted structures, particularly in eastern Canada where a considerable amount of deicing salts are used on highways, an additional specimen, Specimen C7R was constructed and tested (7). Specimen C7R had the same added footing block details as Specimen C6R, but used a reinforced concrete sleeve to add confinement to the column. The reinforced concrete sleeve utilized No. 10 circular inter-locking hoops at 100 mm spacing and six 8 mm diameter vertical bars as shown in Fig. 8. The No. 10 bars had a yield stress of 424 MPa and the 8 mm diameter bars had a yield stress of 602 MPa. A high-performance, fiber-reinforced concrete was used for the concrete in the sleeve to improve the durability of the concrete. The fiber-reinforced concrete contained 1.0% of 0.5 mm diameter by 30 mm long steel fibers, as well as a water-reducing agent and a superplasticizer. The concrete was pumped to the top of the wax coated cardboard form. The fiber-reinforced concrete had a compressive strength of 39.9 MPa. Fig. 9 shows the applied shear versus tip deflection response of Specimen C7R, as well as a photograph of the specimen after testing. A small
amount of additional vertical reinforcement was provided in the reinforced concrete sleeve so that the yielding at the base of the column could spread over the height of the column. This small amount of vertical reinforcement resulted in very large flexural cracks starting at general yielding of the column. The specimen exhibited excellent hysteretic response with no significant strength decay or pinching up to a displacement ductility level of 6.0. At a displacement ductility of 8.0 the interlocking circular hoops began to rupture near the base of the column. Specimen C7R achieved a strength which was 47% higher than Specimen C6 and exhibited considerably greater ductility.

Deficiencies in joints

Fig. 10(a) shows a portion of the double-deck structure of Highway I-880 that collapsed in the 1989 Loma Prieta earthquake (8, 9) and Fig. 10(b) shows a bent of this freeway that was severely damaged. This structure failed in the joint region, due to inadequate joint shear reinforcement and the curtailment of the vertical reinforcement. Fig. 10(c) shows some exposed reinforcement during the replacement of the deteriorated concrete cover on a double-deck freeway structure in eastern Canada. It is evident that the confinement reinforcement in the column and the shear reinforcement in the joint region are deficient. It is important that these types of deficiencies be investigated for Canadian bridges in significant seismic regions. Fig. 10(d) shows an outrigger frame beam-column joint on a double-deck freeway structure. The deficiencies in these sub-assemblages include:

(a) inadequate amount of shear reinforcement in the beam,
(b) insufficient embedment of the bottom beam reinforcement in the joint region,
(c) inadequate amount of joint shear reinforcement, and
(d) large hoop spacings leading to poor column confinement and low shear capacity.

Fig. 11 illustrates the details of Specimen OJ1, tested under reversed cyclic loading at McGill University (10). This test specimen represents an outrigger beam-column connection with seismic detailing deficiencies as described above and was tested upside down. A 125 kN vertical load was applied to the column to simulate the dead load of the superstructure. This dead load is about 3% of the gross concrete section capacity. The concrete had a compressive strength of 39.9 MPa for the beam and footing and 38.3 MPa for the column. The No. 15 bars had a yield stress of 446 MPa and the 6.4 mm diameter bars used for the column hoops and beam stirrups had a yield stress of 437 MPa. Specimen OJ1 exhibited a poor response with a low ductility and pinched hysteretic loops (see Fig. 11). Shear failures in both the joint region and in the outrigger beam limited the strength and severely limited the ductility and energy absorption.
Fig. 12 shows the retrofit details for Specimen OJ2 (10), which was a companion specimen to Specimen OJ1. In order to force hinging in the column, the beam was strengthened, both in flexure and in shear, by adding a reinforced concrete sleeve. This sleeve also added a considerable amount of joint shear reinforcement and was connected to the existing beam by reinforcing bars with heads on one end, which passed through the existing beam through drilled holes. After placing these bars through the holes, plates were welded to the other ends of these bars. The beam sleeve was constructed with a high-performance, steel fiber-reinforced concrete, which together with the added reinforcement, would also control cracking at service load levels. The retrofit also included placing a steel jacket filled with concrete around the existing column to increase the confinement and add shear reinforcement. The reversed cyclic loading response and a photograph of the beam at maximum deflection are shown in Fig. 12. The properties of the steel jacket are the same as those described for Specimen C6R. The retrofitted specimen exhibited a stable hysteretic response with no appreciable loss in strength and large energy absorbing loops up to a displacement ductility of 8.0 in both loading directions. Final failure occurred by low-cycle fatigue rupturing of the column bars at a displacement ductility of 10.0. The fiber-reinforced concrete sleeving over the outrigger beam served to provide excellent crack control, with a maximum crack width of 0.25 mm at a displacement ductility of 6.0.

Concrete hinges

A number of bridge structures in eastern Canada were constructed in the 1960's with columns having concrete hinges at their bases. These hinges were formed by reducing the cross-sectional dimensions of the column over a short vertical distance, with dowel reinforcement passing through the hinge to connect the column to the footing. While this reduces the temperature and shrinkage restraint effects on the structure, large superstructure movements are expected in significant earthquakes. Research (11, 12, 13) indicates that the concrete in columns with two-way hinges is well confined and gives reasonably ductile response until the hinge gap closes. It was found from carrying out reversed cyclic loading tests on columns with two-way hinges (12) that a temporary, but significant improvement to the response can be made by simply dry-packing the hinge gap to give a larger hinge area. It was also found that one-way hinges have significant shear problems in both the parallel and perpendicular direction to the hinge. For these cases, it was necessary to pack the hinge and then sleeve the column in order to improve the ductility and energy absorption.

Deterioration of concrete and corrosion of reinforcement

There are many examples of deteriorated concrete bridge structures. Concrete deterioration problems include spalling of the concrete cover due to poor freeze-
thaw response of the concrete, as well as from alkali-silica reactivity. Poor quality concrete, together with cover spalling, has resulted in severe corrosion of the reinforcement. Rehabilitation projects in eastern Canada have focussed on improving deteriorated bridge structures and seismic retrofit has received little attention. It is hoped that in the future it may be possible to combine rehabilitation due to deterioration with seismic upgrading to reduce the threat of seismic damage and collapse.

CONCLUSIONS

The 2000 Canadian Highway Bridge Design Code provides up-to-date requirements for the seismic design and evaluation bridges across Canada. These new provisions should have a major impact on both seismic design practice and on the assessment of existing structures, since different jurisdictions have been using different seismic design standards, some of which are outdated. A number of seismic deficiencies of existing bridges in eastern Canada have been presented to illustrate their influence on the seismic performance. Some methods of retrofitting these deficient structures and the seismic performance after retrofit have been presented to illustrate effective means of improving the responses. It is hoped that the problem of seismically deficient bridge structures will receive more attention in the future, particularly in eastern Canada. One effective approach is to combine seismic retrofit with the rehabilitation of deteriorated bridges to reduce the overall risk of severe damage and collapse.

ACKNOWLEDGEMENTS

The research funding by the Natural Sciences and Engineering Council of Canada is gratefully acknowledged. The significant contributions to this overall research program by Andrew Griezic, William Cook, Philip Zeyl and David Dunwoodie are gratefully acknowledged.

REFERENCES


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Table 1 Minimum analysis requirements for evaluation of existing bridges

| Seismic Performance Zone | Single-span | Multi-span | | | |
|--------------------------|-------------|------------|---|---|
|                          | Emergency-route | Other | Emergency-route | Other | | Regular | Irregular | Regular | Irregular |
| 1                        | None | None | None | None | | None | None | None | None |
| 2                        | None | None | LE | LE | | None | None | None | None |
| 3                        | LE | None | SM | MM | | LE | LE | LE | LE |
| 4                        | LE | LE | MM | MM | | SM | MM | MM | MM |

LE = limited evaluation required
SM = single-mode elastic analysis required
MM = multi-mode spectral analysis required

Fig. 1 Contours of peak horizontal ground accelerations, having a probability of exceedance of 10% in 50 years (National Building Code of Canada (1))
Fig. 2 Details of “as-built” Specimen C6
Fig. 3 Reversed cyclic loading response of "as-built" Specimen C6
Fig. 4 Expected hinging length for as-built Specimen C6

Fig. 5 Expected hinging length for Specimen C6R
Fig. 6 Details of retrofit Specimen C6R
Fig. 7 Reversed cyclic loading response of retrofit Specimen C6R
axial load, $P$

lateral load, $V$

Fibre Reinforced
High Performance
Concrete

8 mm ties
@ 300

16 - No. 15
column bars

4 - 8 mm dia.
hook anchorage
bars

2 - 8 mm dia.
hoop support bars

SECTION B-B

(b) Column retrofit details

No. 15 dowel

32 - No. 15 dowels

SECTION A-A

(c) Dowel details

Notes

• 300 mm vertical drill
  and bond, typ.

• 200 mm horizontal
  drill and bond, typ.

• dowel anchorage
detail:

PL 6.35x50x50
fillet weld

(d) Added footing block details

Fig. 8 Details of retrofit Specimen C7R
Fig. 9  Reversed cyclic loading response of retrofit Specimen C7R
Fig. 10 Deficiencies in double-deck structures

(a) Collapse of I-880 structure
(b) Joint failures in I-880 structure
(c) Poorly detailed column and joint
(d) Outrigger beam-column joint
axial load, $P$
lateral load, $V$

12 - No. 15

6.4 mm ties
@ 240 mm

SECTION B-B

6 - No. 15

6.4 mm,
2 each face

6.4 mm
U-stirrups
@ 160 mm

SECTION A-A

Notes
- all dimensions in mm
- column cover: 25 mm
- beam cover: 25 mm all sides except 40 mm on bottom

"As-built" outrigger specimen reinforcing details

load-deflection response

shear distress in beam and joint

Fig. 11 Details and reversed cyclic loading response of "as-built" Specimen OJ1
Reinforcing details of retrofit outrigger Specimen OJ2

Fig. 12 Details and reversed cyclic loading response of retrofit Specimen OJ2
Prediction of Strength and Shear Distortion in R/C Beam-Column Joints

by G. J. Parra-Montesinos and J. K. Wight

Synopsis: The behavior of beam-column connections in R/C frame structures has been extensively studied since the 1960's. These studies have served as the basis for design guidelines for R/C joints, in which detailing requirements and stress limits are given to control damage and deterioration of strength and stiffness in the connections. However, no particular attention is paid to the deformation in the joint region and its relation to the connection strength. In this paper a model is presented to evaluate the shear strength of R/C joints for various levels of joint shear distortion. The joint model is based on the state of plane strains in the connection through the development of a ratio between the joint principal strains, which was determined from experimental results. Based on the joint model, stress limits are proposed for interior and exterior joints for a shear distortion of 1%. These limits are similar to those recommended by ACI-ASCE Committee 352 for exterior connections. However, they are lower than those recommended for interior joints. The detrimental effect of eccentricity on joint strength is also estimated. Further analyses are required to fully quantify the effect of joint type and details on the principal strain ratio, and thus on joint strength.

Keywords: beam-column joints; deformation; earthquake-resistant structures; principal strains; reinforced concrete; shear distortion; shear strength
INTRODUCTION

Design and detailing of beam-column connections in earthquake-resistant R/C frame structures gained attention in the 1960’s. Hanson and Connor (1) and Hanson (2) reported on the testing of several interior, exterior and corner R/C joints under load reversals in the late sixties and early seventies. Those tests were primarily aimed at studying the influence of joint type, confinement, and column size and axial load on the response of the connections. From these studies it was concluded that R/C beam-column joints, if properly detailed, should exhibit a stable response without significant damage and loss of strength. From that time on several experimental programs on R/C connections have been conducted in the United States, New Zealand, Japan and Canada to understand the behavior of joints and develop design guidelines to assure a proper connection response, especially under seismic events. Detailed information of some of those experimental programs can be found in references (3) thru (8).

Since 1976, ACI-ASCE Committee 352 has issued recommendations for the design of joints in R/C frame structures (9,10). Throughout the years these guidelines have evolved due to the results from new research programs, focusing primarily on establishing minimum detailing standards for joint confinement, anchorage length of beam and column bars passing through the joint, and moment strength ratio between columns and beams framing into the connection. In addition, maximum joint stress limits, based on experimental observations, have been developed to control the level of damage and deterioration of strength and stiffness in the connection during cyclic displacements. However, little attention has been paid to the deformation in the joint region and its relation to the connection strength. Pantazopoulou and Bonacci (11) developed a set of expressions from
strain compatibility and stress equilibrium in beam-column joints. However, no explicit stress limits were derived for design of connections in R/C frames.

In this paper, a simple analytical model has been developed to predict the shear force vs. shear distortion response of R/C beam-column connections. The proposed model is based on the definition of the state of plane strains in the joint through the development of a factor relating the principal tensile and compression strains. The development of a model based on the state of plane strains in the joint allows the evaluation of its strength for different levels of shear distortion. In addition, it permits the estimation of the corresponding strains in the joint transverse reinforcement and concrete, which allows the prediction of the shear force and shear distortion required to produce yielding of the joint stirrups and/or crushing of the concrete in the connection.

STATE OF PLANE STRAINS IN BEAM-COLUMN CONNECTIONS

This paper deals with the development of a model capable of predicting the shear strength of R/C beam-column joints for various levels of joint shear distortion based on the state of plane strains in a joint panel. The state of plane strains in a beam-column joint can be defined by the following expressions (Fig. 1),

\[
\varepsilon_c = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2} \cos(2\theta) + \frac{\gamma}{2} \sin(2\theta) \]  

(1)

\[
\varepsilon_t = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2} \cos[2(\theta + 90^\circ)] + \frac{\gamma}{2} \sin[2(\theta + 90^\circ)] \]  

(2)

\[
\gamma = \tan(2\theta)(\varepsilon_x - \varepsilon_y) \]  

(3)

where \(\varepsilon_c\) and \(\varepsilon_t\) are the principal compression and tensile strains, respectively, \(\varepsilon_x\) and \(\varepsilon_y\) are the horizontal and vertical strains, respectively, \(\gamma\) is the joint shear distortion and \(\theta\) is the principal compression angle. By evaluating Eqs. (1) thru (3) it can be noted that for any particular joint shear deformation, and assuming that the principal compression angle is known, four unknown variables are present, and thus a fourth relationship is required to define the state of plane strains in the joint model. This has been overcome by defining a factor, \(k_{tc}\), that relates the principal tensile and compression strains as follows,
To the knowledge of the authors, the use of the $k_{tc}$ factor to model the behavior of beam-column connections was first introduced by Parra-Montesinos and Wight (12) in hybrid joints between steel beams and R/C columns. This $k_{tc}$ factor is affected by the joint type (i.e. interior, exterior, eccentric) and confinement, and may vary depending on the level of joint shear distortion. For modeling purposes a linear relationship between the $k_{tc}$ factor and the joint shear distortion $\gamma$ can be assumed as follows,

$$k_{tc} = 2 + k_s \gamma$$

(5)

where $k_s$ represents the slope of the $k_{tc}$ vs. $\gamma$ relationship and is strongly influenced by the amount of joint confinement. For highly confined connections a significant restraint in the growth of diagonal cracks is imposed on the joint. Therefore, lower values of $k_s$ and $k_{tc}$ are expected compared to lightly confined connections. The minimum value of $k_{tc} = 2$ for $\gamma = 0$ in Eq. (5) indicates that a cracked response is assumed for all levels of joint shear distortion. By combining Eqs. (1) thru (5), the state of plane strains in the joint can be defined in terms of $k_s$. Therefore, if $k_s$ is known the strains in the concrete and steel transverse reinforcement can be estimated for any level of joint shear distortion.

**SHEAR STRENGTH MECHANISMS IN R/C JOINTS**

It has been generally accepted that the shear strength of R/C beam-column connections is provided by the contributions from two mechanisms: 1) a strut mechanism and 2) a truss mechanism (Fig. 2). The strut mechanism is activated by direct bearing on the concrete from the adjoining beam and column compression zones. The contribution from the truss mechanism is dependant on the amount of force transferred to the joint by bond between the beam and column bars passing through the connection and the concrete. If bond is completely lost in these bars the joint strength is given only by the action of a diagonal compression strut. However, for R/C joints with acceptable anchorage lengths for longitudinal beam and column bars, such as those recommended by ACI-ASCE Committee 352 (10), only a partial loss of bond should occur, and thus the contribution from the truss mechanism is difficult to estimate.

In this paper it is assumed that the strength of the joint is provided by an equivalent diagonal compression strut activated by transfer of shear forces
to the joint by direct bearing from the beam and column compression zones and by bond between the beam and column bars and the surrounding concrete. After a few reversed displacement cycles it is expected that some deterioration in the bond of these bars will occur. In addition, for a building designed with a strong column-weak beam philosophy, it is assumed that a higher rate of bond deterioration takes place after yielding of the beam bars in tension. Yielding of the beam compression reinforcement is less likely to occur because of the need for very high bond stresses and because part of the compression force component is resisted by the concrete. Therefore, it is generally assumed that a better transfer of forces to the joint via bond will occur near the compression zones of the adjoining members. Transfer of forces to the joint region via bearing is achieved over a depth equal to the compression zones of the beams and columns framing into the joint. For lightly reinforced sections the depth of the compression zone represents only a small percentage of the total member depth.

Combining the two force transfer mechanisms described above, it is reasonable to assume a linear variation of shear stresses, $\tau$, over the joint and column depths, as shown in Fig. 3. This linear variation of shear stresses leads to a parabolic distribution of shear force transferred into the connection for which the centroid is located at $0.25d_{\text{joint}}$ and $0.25h_{\text{col}}$ from the beam compression reinforcement and extreme column compression fiber, respectively. Therefore, the equivalent diagonal compression strut is assumed to be activated over a distance equal to 50% of the joint and column depths, as shown in Fig. 3. The angle of inclination of the diagonal compression strut with respect to the beam axis, $\theta_{\text{strut}}$, is assumed to be fixed and can be estimated as follows,

$$\theta_{\text{strut}} = \tan^{-1} \left( \frac{d_{\text{joint}}}{h_{\text{col}}} \right)$$  \hspace{1cm} (6)

where the joint depth, $d_{\text{joint}}$, is approximated as,

$$d_{\text{joint}} = d - d'$$  \hspace{1cm} (7)

where $d$ is the distance from the extreme compression fiber to the centroid of the beam tensile reinforcement and $d'$ is the distance from the extreme compression fiber to the centroid of the beam compression reinforcement. By determining the angle of inclination of the strut, its depth can be obtained as follows,

$$d_{\text{strut}} = \frac{d_{\text{joint}} h_{\text{col}}}{\sqrt{d_{\text{joint}}^2 + h_{\text{col}}^2}}$$  \hspace{1cm} (8)
Once the strut angle $\theta_{strut}$ is known, and assuming that it coincides with the principal compression angle $\theta$ in the joint panel, the compression strains in the strut can be evaluated for various levels of joint shear distortion and $k_s$. Fig. 4 shows a plot of the principal compression strains, $\varepsilon_c$, vs. joint shear distortion, $\gamma$, for $\theta = 30^\circ$ and $45^\circ$, and $k_s$ ranging from 0 to 1200. The maximum limit for $\gamma$ was set as 2% (0.02 rad), for which significant damage in the connection region is expected. As shown in Fig. 4, increasing $k_s$ leads to a decrease in the concrete compressive strain, $\varepsilon_c$. This is because for a particular level of joint shear deformation a higher value of $k_s$ leads to a larger contribution from the principal tensile strains, $\varepsilon_t$, to the joint distortion. The lower limit of $k_s = 0$ represents an extreme case for which a sufficient amount of confinement is provided in the joint such that the principal tensile strains are double the principal compression strains for any level of joint shear distortion. The curves shown for values of $k_s$ ranging from 200 up to 1200 represent more realistic cases for which an increase in the joint shear distortion is accompanied by an increase in the $k_{tc}$ ratio. It should be noted that for values of $k_s$ ranging from 400 to 1200, the maximum concrete compressive strains are approximately equal to or lower than 0.002, which is generally taken as the strain at peak stress for unconfined normal strength concrete. As is also shown in Fig. 4, increasing the principal compression angle from $30^\circ$ to $45^\circ$ leads to a slight decrease in the compression strain demand for the diagonal compression strut.

In terms of principal tensile strains, a nearly linear relationship with respect to the joint shear distortion was obtained for various levels of $k_s$, as shown in Fig. 5. For a joint shear distortion $\gamma = 1\%$, the principal tensile strains range from approximately 0.7% to 1.1% for $\theta = 30^\circ$, and for $\gamma = 2\%$ the principal tensile strains vary between 1.5% and 2.2%. It should be mentioned that these principal tensile strains only represent average strains across the connection region. After the joint is fully cracked a large portion of the tensile strain in the joint is concentrated at crack openings, and thus the transverse tensile strain in the concrete between cracks only represents a small percentage of the average principal tensile strain.

As was done for the principal strains, the strain demand in the joint transverse reinforcement can be estimated for any level of joint shear distortion by determining the horizontal strain, $\varepsilon_x$, from Eqs. (1) thru (5). Fig. 6 shows the $\varepsilon_x$ vs. $\gamma$ response for $\theta = 30^\circ$ and $45^\circ$, and $k_s$ ranging from 0 to 1200. As Fig. 6 indicates, increasing $k_s$ leads to a higher demand for tensile strains in the joint hoops. If the yield strain of the joint transverse reinforcement and $k_s$ are known, the level of shear distortion for which yielding of the joint hoops would occur can be approximated using Fig. 6. For a joint with Grade 60 hoops and $k_s = 1200$, this value of joint shear distortion can be as low as 0.5% (0.005 rad) for $\theta = 45^\circ$. However, a
reduction in the value of \( k_s \) and/or a decrease in the principal compression angle leads to an increase in the joint shear distortion required to produce yielding of the joint transverse reinforcement.

**K_{tc} - \gamma RELATIONSHIP FOR R/C BEAM-COLUMN JOINTS**

As discussed in the previous section, the evaluation of the state of plane strains in the joint involves the use of the slope factor \( k_s \) in the \( K_{tc} \) vs. \( \gamma \) relationship for the connection. In order to determine appropriate values of \( k_s \) for different types of joints, the \( K_{tc} \) vs. \( \gamma \) response was obtained for nine R/C beam-column connections tested under large load reversals (6-8). These nine specimens represented three types of joints: 1) concentric interior joints, 2) eccentric interior joints, and 3) concentric exterior joints (Fig. 7). In the following a brief description of the test specimens used for determining the \( K_{tc} \) vs. \( \gamma \) response of R/C joints is given.

**Concentric Interior Joints**

The shear strength vs. shear distortion response obtained from testing of two R/C beam-column subassemblies by Durrani and Wight (6) was used to determine the values of \( k_s \) for interior concentric joints. These subassemblies were Specimens X1 and X2 in that research program. The subassemblies consisted of a column with a beam framing into two opposite sides of the column so that the beam and column axes intersect in the joint region. Relevant information regarding overall dimensions and reinforcement of beams and columns for Specimens X1 and X2 is given in Table 1. Specimens X1 and X2 had the same details except for the amount of confinement in the connection. For both specimens, diamond and square shaped hoops were used for each set of stirrups in the column region over the joint depth. In Specimen X1 two sets of stirrups were used, which represented a joint hoop volumetric ratio of 1.8%. The joint hoop volumetric ratio is obtained as,

\[
\text{Joint Hoop Volumetric Ratio} = \frac{\text{Volume of Joint Hoops}}{b_{\text{core}} d_{\text{core}} d_{\text{joint}}} \tag{9}
\]

where \( b_{\text{core}} \) and \( d_{\text{core}} \) are the column core width and depth, respectively. For Specimen X2 the joint hoop volumetric ratio was increased to 2.6% by using three sets of square and diamond hoops in the joint region. Specimens X1 and X2 were subjected to seven displacement cycles of increasing ductility from 1 to 4. For these two specimens a displacement ductility of 4 corresponded to story drifts slightly larger than 5.0%.  


Eccentric Interior Joints

The results of four tests of eccentric interior R/C beam-column subassemblies by Raffaelle and Wight (8) were used to determine the effect of eccentricity on $k_t$. The four subassemblies represented Specimens 1 thru 4 in their investigation. The specimens consisted of cruciform shaped subassemblies with spandrel beams framing into the column. The desired eccentricity was achieved by having the exterior faces of the beams and column flush (Fig. 7b) and changing the width of the spandrel beams. Overall dimensions and reinforcement for Raffaelle's Specimens 1 thru 4 are listed in Table 1. All four specimens had the same R/C column. For Specimens 1 thru 3 the beam depth was kept the same and the beam width was varied such that the subassembly eccentricities ranged between $b_{col} / 7$ and $b_{col} / 4$, where $b_{col}$ is the column width. Specimen 4 had a deeper beam compared to the other three specimens and the eccentricity was approximately equal to $b_{col} / 4$. The joint confinement consisted of square and diamond shaped hoops. For Specimens 1 thru 3 three sets of hoops were used, which represented a joint hoop volumetric ratio of 1.6%. In Specimen 4, because of the increase in the beam depth, five sets of hoops were used in the connection, corresponding to a joint hoop volumetric ratio of 1.7%. These four specimens were subjected to fourteen cycles of increasing lateral displacements with story drifts ranging from 0.25% to 5.0%. Because the beams framed eccentrically into the column, shear distortions in the joint region were measured on both the flush and offset faces of the connection. The joint shear distortion used to determine the values of $k_{tc}$ was based on an effective joint distortion along the centerline of the effective joint width, as will be described later.

Concentric Exterior Joints

Experimental results from the testing of three exterior R/C beam-column subassemblies (Specimens 2 thru 4) by Ehsani and Wight (7) were used to determine the $k_{tc}$ vs. $\gamma$ response for exterior R/C connections. These specimens consisted of a beam framing into a column from only one side of the column. Overall information for these three specimens is listed in Table 1. For these exterior connections the confinement was also provided through diamond and square shaped hoops. Two layers of hoops were used for Specimen 2, while three layers were used for Specimens 3 and 4. This reinforcement represented joint hoop volumetric ratios of
2.5%, 3.0% and 3.4% for Specimens 2 thru 4, respectively. The beam-column subassemblies tested by Ehsani and Wight were subjected to six displacement cycles of increasing ductility up to a maximum ductility of 4.

It should be mentioned that for all nine specimens used to determine the $k_{lc}$ vs. $\gamma$ response of R/C beam-column connections, a small amount of axial force was applied to the columns, but in all cases it represented less than 10% of the nominal axial load capacity of the R/C column.

**PROCEDURE TO DETERMINE $k_{lc}$**

The determination of the $k_{lc}$ vs. $\gamma$ response for the test specimens described above first involved the evaluation of the state of plane strains in the joint for a pre-established level of joint shear distortion. Values of $k_{lc}$ were varied for each level of joint shear deformation until the shear force obtained in the analytical model for that particular distortion corresponded to the measured shear force in the test specimens. For the concentric interior and exterior connections tested by Durrani and Wight (6), and Ehsani and Wight (7), the joint shear distortion was measured on one lateral face of the beam-column connection through the use of linear displacement transducers. However, for the eccentric connections tested by Raffaelle and Wight (8), the joint shear distortion was measured on both the flush and offset lateral sides of the joint because of the significant variation in the shear distortion across the joint width for eccentric connections. Therefore, for Raffaelle's specimens an equivalent joint shear distortion at the centerline of the effective joint width was used to determine the $k_{lc}$ vs. $\gamma$ response of the connections. This was obtained by linearly interpolating between the shear distortions measured on both lateral sides of the joint (Fig. 8).

The shear strength of the connection was assumed to be given by the strength of an equivalent diagonal compression strut, as was explained previously. This strut was assumed to have an angle of inclination $\theta_{strut}$ as defined in Eq. (6). Based on the strains obtained in the direction of the principal compression angle $\theta$ and assuming that $\theta = \theta_{strut}$, the stresses in the concrete strut were estimated. The concrete stresses were obtained using the model for confined concrete developed by Sheikh and Uzumeri (13). Based on this model the stress vs. strain relationship for the concrete consists of four regions, as shown in Fig. 9. The ascending branch of the curve in Fig. 9 was modeled using a parabolic relationship given by the following expression,
where $\varepsilon_c$ is the concrete strain, $f'_c$ is the unconfined concrete strength, $\varepsilon_{s1}$ is the strain at peak compressive stress and $k_c$ is a parameter that accounts for the increase in the compressive strength of confined concrete. The strain at peak stress, $\varepsilon_{s1}$, is obtained as a function of the concrete strength and $k_c$. Several parameters affect the confinement factor $k_c$, such as the joint hoop volumetric ratio, the confining pressure, the distance between the longitudinal column bars, hoop spacing and section dimensions. It should be noted that a factor $\beta$ has been added to the stress vs. strain relationship in Eq. (10) to account for the softening of the concrete strut due to transverse tensile strains. The softening parameter $\beta$ was obtained from the following relationship in terms of the $k_{tc}$ factor (14),

$$\beta = \frac{1}{0.85 + 0.27 k_{tc}}$$

A minimum value of $\beta = 0.60$ was used to account for the fact that after significant cracking occurs in a beam-column joint, the transverse tensile strains in the joint are governed by opening of cracks without an increase in the tensile strains of the concrete struts between the cracks. After the concrete has reached its peak stress, the stress remains constant until reaching the strain $\varepsilon_{s2}$ (Fig. 9), which is determined as a function of the joint confinement and dimensions. For strains larger than $\varepsilon_{s2}$, the concrete behavior is described by a linear descending tail until a minimum stress equal to 30% of the concrete peak strength is reached. The linear descending tail is defined by the following relationship,

$$f_c(\varepsilon_c) = f'_c [1 - Z (\varepsilon_c - \varepsilon_{s2})] k_c \beta$$

where $Z$ is a factor defining the slope of the descending branch. The determination of the slope factor $Z$ was based on the Kent and Park relationship (15) with the modifications suggested by Sheikh and Uzumeri (13).

The horizontal shear force in the joint, $V_{jh}$, was then obtained as,

$$V_{jh} = C_{strut} \cos (\delta_{strut})$$

where $C_{strut}$ is the compression force in the diagonal strut and is obtained as follows,

$$C_{strut} = f_c(\varepsilon_c) A_{strut}$$
where \( A_{\text{strut}} = b_{\text{strut}} \times d_{\text{strut}} \). The strut width, \( b_{\text{strut}} \), was obtained based on the effective joint width, \( b_j \), recommended by ACI-ASCE Committee 352 (16). However, the unconfined joint regions were not included in the determination of \( b_{\text{strut}} \). For the case of eccentric connections, a relationship proposed by the second author to ACI-ASCE Committee 352 was also used for determining the effective joint width as follows,

\[
b_j \leq b_b + \frac{0.3 h_{\text{col}}}{2}
\]  

(15)

where \( b_b \) is the beam width. The depth of the diagonal compression strut, \( d_{\text{strut}} \), was obtained from Eq. (8) based on the effective shear stress transfer zones shown in Fig. 3. After the procedure described above was performed for a particular level of joint shear distortion, the analytically determined shear force in the connection was compared to that obtained experimentally. The assumed value of \( k_{\text{tc}} \) was then adjusted until the theoretical and experimental joint shear forces were equal.

Fig. 10 shows the \( k_{\text{tc}} \) vs. \( \gamma \) response obtained for the nine interior and exterior R/C beam-column connections described above. As shown in this figure, the \( k_{\text{tc}} \) factor increased at an almost constant rate with respect to the joint shear distortion, especially for shear distortions of up to 1%. The lower values of \( k_{\text{tc}} \) were obtained for the two interior and three exterior joints tested by Durrani and Wight, and Ehsani and Wight, respectively. Values of \( k_{\text{tc}} = 12-16 \) for \( \gamma = 2\% \) were obtained for these five specimens. Although higher values of \( k_{\text{tc}} \) were expected for exterior joints compared to interior connections, similar values were obtained, as shown in Fig. 10. This might be due to the fact that Ehsani's specimens reached joint shear distortions of 2% after a very few loading cycles, and thus the shear response obtained at this level of shear distortion did not reflect the potential softening effect caused by several cycles of load reversals. In addition, higher hoop volumetric ratios were used in Ehsani's specimens, which might also have led to lower values of \( k_{\text{tc}} \). When the beams framed eccentrically to the columns, a significant increase in the \( k_{\text{tc}} \) ratio was observed compared to concentric joints. This increase in \( k_{\text{tc}} \) was found to be related to the ratio between the joint eccentricity, \( e \), and the column width, \( b_{\text{col}} \).

Based on the obtained \( k_{\text{tc}} \) vs. \( \gamma \) responses, values of the slope factor \( k_s \) are proposed for different joint types and for joint shear distortions up to 1%. For interior R/C connections the following relationship is proposed in terms of the eccentricity \( e \) and the column width \( b_{\text{col}} \).
For exterior joints a single value of $k_s = 500$ is proposed, assuming that no eccentricity exists between the beam and column axes. Fig. 11 shows the proposed $k_{tc}$ vs. $\gamma$ relationships for both interior and exterior connections and for values of $\gamma$ ranging from 0 to 1%. By using the proposed expressions of $k_s$, the shear strength of R/C beam-column joints can be estimated for joint shear distortions of up to 1%. Clearly, more data is needed to validate these proposed expressions of $k_s$.

Applying the obtained values of $k_s$ to Fig. 6, it can be seen that for values of $k_s$ ranging from 500 to 1000 the shear distortion required to produce an average horizontal tensile strain of 0.002 in the joint varies from approximately 0.5% for a principal compression angle $\theta = 45^\circ$, to 1.3% for $\theta = 30^\circ$. With respect to the compression strain demand on the diagonal strut, Fig. 4 shows that for the values of $k_s$ obtained from experimental results, the compression strains in the concrete are approximately equal to or lower than 0.002 for shear distortions of up to 2%.

NOMINAL STRENGTH OF R/C CONNECTIONS

In R/C frames total story drift comes primarily from inelastic flexural rotations at beam ends and inelastic shear distortions in the joints, plus elastic flexural deformations in the beams and columns. To limit the contribution from joint shear deformations, a maximum permissible shear distortion of 1% (0.01 rad), which roughly represents the resulting story drift, was selected. Therefore, based on the model described above, nominal shear strength equations were developed for both interior and exterior R/C beam-column joints at a shear distortion of 1%. The proposed strength equations are expressed in terms of horizontal joint shear stresses, $\sigma_{hj}$, in order to be consistent with the current shear stress limits recommended by ACI-ASCE Committee 352 (16). The procedure used to develop the nominal stress factors ($\sigma_{hj} / f_c$) was similar to that used for determining the factors $k_{tc}$ and $k_s$. However, $k_{tc}$ and $k_s$ are known in this case, so the only unknown variable is the concrete stress in the diagonal compression strut.

Because infinite combinations of column dimensions and layout of steel reinforcement can be used in the design of R/C frame structures, a single value of the confinement factor $k_c = 1.2$ was used. Also, the region of constant stress for strains ranging between $\varepsilon_{s1}$ and $\varepsilon_{s2}$ (Fig. 9) was not considered in these analyses because of the dependence of $\varepsilon_{s2}$ on joint confinement and dimensions. Thus, after the peak strength is reached the
stress vs. strain concrete response is defined by a linear descending tail, as expressed in Eq. (12), assuming a single value of the slope factor $Z = 50$. Figs. 12 and 13 show the nominal stress factors vs. $k_s$ response obtained for concrete strengths, $f'_c$, ranging from 28 MPa (4000 psi) to 69 MPa (10000 psi) and for principal compression angles, $\theta$, ranging from $30^\circ$ to $45^\circ$. As these two figures show, an increase in $k_s$ beyond 200 has a detrimental effect on the nominal stress factors. Similarly, an increase in the concrete strength leads to a reduction in the stress factors due to the increase in the strain at peak stress for higher strength concrete. The more rapid change in the stress factor between the curves for $f'_c = 28$ and 48 MPa, compared to that for $f'_c = 48$ and 69 MPa in Fig. 12 for $k_s \geq 200$, suggests that a nonlinear relationship exists between the stress factor and the concrete strength.

The angle of inclination of the strut, $\theta_{\text{strut}} = \theta$, also affects the stress factor, as shown in Fig. 13. However, the difference in the values of the stress factor for $k_s = 200$ was less than fifteen percent and this difference diminishes for higher values of $k_s$. Therefore, it is conservative and sufficiently accurate to use the stress factors obtained for $\theta_{\text{strut}} = 30^\circ$ in R/C connections with $30^\circ \leq \theta_{\text{strut}} \leq 45^\circ$.

Combining the effect of the concrete strength, strut angle and $k_s$ on the horizontal stress factor, the following expression is proposed for estimating the horizontal shear stress capacity of R/C beam-column joints,

$$V_{jh} = \alpha_1 \alpha_2 f'_c$$

where $\alpha_1$ and $\alpha_2$ are nondimensional parameters to account for the influence of $k_s$ and $f'_c$ on the joint strength and are expressed as follows,

$$\alpha_1 = 0.34 - 0.00018 k_s$$

$$\alpha_2 = 0.00018 f'_c^2 - 0.03 f'_c + 1.7$$

where $f'_c$ is in MPa.

The nominal horizontal shear strength of the connection is then obtained as,

$$V_{jh} = v_{jh} b_j h_{col}$$

where $b_j \leq b_{\text{core}}$. 
Fig. 14 shows a comparison between the stress limits obtained from the analytical model at a joint shear distortion of 1% and those recommended by ACI-ASCE Committee 352 (16). As can be observed, the proposed values for concentric interior and exterior joints are very similar to those recommended by Committee 352 for exterior connections. However, they are approximately 30% lower than the recommended stress limits for interior beam-column joints. For the case of eccentric joints, a decrease in the proposed stress limits is clearly shown in Fig. 14, as a consequence of the increase in $k_s$. It should be emphasized that more data is needed to validate the proposed values of $k_{tc}$ and $k_s$ for various types of R/C connections. As new data become available the proposed stress limits can be easily adjusted.

**SUMMARY AND CONCLUSIONS**

An analytical model has been proposed for estimating the shear strength of R/C beam-column connections. The joint model is based on the definition of the state of plane strains in the joint through the use of a $k_{tc}$ factor relating the principal tensile and compression strains. Experimental results from testing of R/C connections subjected to load reversals were used to obtain the $k_{tc}$ vs. shear distortion response for various types of R/C beam-column joints. The development of a deformation-based joint model allows the prediction of the connection strength for a particular level of shear distortion, and the estimation of the strain demand in the steel transverse reinforcement and concrete. Stress limits are proposed for exterior and interior R/C connections for a maximum permissible joint shear distortion of 1%. These stress limits are similar to those recommended by ACI-ASCE Committee 352 for exterior R/C connections. However, they are lower than those recommended for interior connections. The detrimental effect of eccentricity on joint strength is also estimated. Further analyses are required to fully quantify the effect of joint type and details on the principal strain ratio, and thus on joint shear strength.

**REFERENCES**


Table 1 – Main Features of Specimens Used to Determine $k_{ic}$

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam Size (cm)</th>
<th>Column Size (cm)</th>
<th>Beam Reinf.</th>
<th>Column Reinf.</th>
<th>Joint Hoop Vol. Ratio</th>
<th>Joint Hoop Layers</th>
<th>e (cm)</th>
<th>$f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durrani X1</td>
<td>28 x 42</td>
<td>36 x 36</td>
<td>4-#22 4-#19</td>
<td>8-#25</td>
<td>1.8%</td>
<td>2</td>
<td>0</td>
<td>34.3</td>
</tr>
<tr>
<td>Durrani X2</td>
<td>28 x 42</td>
<td>36 x 36</td>
<td>4-#22 4-#19</td>
<td>8-#25</td>
<td>2.6%</td>
<td>3</td>
<td>0</td>
<td>33.6</td>
</tr>
<tr>
<td>Ehsani 2</td>
<td>26 x 44</td>
<td>30 x 30</td>
<td>3-#22 3-#19</td>
<td>10-#19</td>
<td>2.5%</td>
<td>2</td>
<td>0</td>
<td>34.9</td>
</tr>
<tr>
<td>Ehsani 3</td>
<td>26 x 48</td>
<td>30 x 30</td>
<td>3-#22 3-#19</td>
<td>8-#19</td>
<td>3.0%</td>
<td>3</td>
<td>0</td>
<td>40.9</td>
</tr>
<tr>
<td>Ehsani 4</td>
<td>26 x 44</td>
<td>30 x 30</td>
<td>3-#22 3-#19</td>
<td>10-#19</td>
<td>3.4%</td>
<td>3</td>
<td>0</td>
<td>44.6</td>
</tr>
<tr>
<td>Raffaelle 1</td>
<td>25 x 38</td>
<td>36 x 36</td>
<td>3-#19 3-#16</td>
<td>8-#19</td>
<td>1.6%</td>
<td>3</td>
<td>5.5</td>
<td>28.6</td>
</tr>
<tr>
<td>Raffaelle 2</td>
<td>18 x 38</td>
<td>36 x 36</td>
<td>2-#19 2-#16</td>
<td>8-#19</td>
<td>1.6%</td>
<td>3</td>
<td>9.0</td>
<td>26.8</td>
</tr>
<tr>
<td>Raffaelle 3</td>
<td>19 x 38</td>
<td>36 x 36</td>
<td>2-#16 2-#16</td>
<td>8-#19</td>
<td>1.6%</td>
<td>3</td>
<td>8.5</td>
<td>37.7</td>
</tr>
<tr>
<td>Raffaelle 4</td>
<td>19 x 56</td>
<td>36 x 36</td>
<td>2-#16 2-#16</td>
<td>8-#19</td>
<td>1.7%</td>
<td>5</td>
<td>8.5</td>
<td>19.3</td>
</tr>
</tbody>
</table>
Fig. 1 - Plane Strains in R/C Joint

Fig. 2 - Strength Mechanisms in R/C Joint

a) Strut Mechanism

b) Truss Mechanism
Fig. 3 – Equivalent Strut Mechanism

Fig. 4 - Principal Compression Strains in Joint Panel
Fig. 5 - Principal Tensile Strains in Joint Panel

Fig. 6 - Horizontal Strains in Joint Panel
Fig. 7 – Joint Types Analyzed

Fig. 8 – Shear Distortion in Eccentric Joints
Fig. 9 – Stress-Strain Relationship for Concrete

Fig. 10 – $k_{tc}$ vs. $\gamma$ Response
Fig. 11 – Proposed $k_{lc}$ vs. $\gamma$ Response

Fig. 12 – Effect of $f'_c$ on Stress Factor vs. $k_s$ Response
Fig. 13 – Effect of $\theta$ on Stress Factor vs. $k_s$ Response

Fig. 14 – Proposed vs. ACI Joint Stress Limits
Gravity Load Collapse of Building Frames During Earthquakes

by J. P. Moehle, K. J. Elwood, and H. Sezen

Synopsis:

Earthquake reconnaissance has identified failure of reinforced concrete columns as a primary cause of collapse of older existing reinforced concrete building frames during earthquakes. Apparent column failure, however, does not always result in building collapse. A study of columns tested in the laboratory examines loss of lateral and vertical load capacities. Correlations with geometric, materials, and loading characteristics are identified.

Keywords: axial load; buildings; collapse; columns; design; earthquakes; experimentation; reinforced concrete; shear
INTRODUCTION

Before the introduction of special requirements in the 1970s, reinforced concrete building frames constructed in zones of high seismicity in the US had details and proportions similar to frames designed primarily for gravity loads. Columns generally were not designed to have strengths exceeding beam strengths, so column failure mechanisms often prevail. Relatively wide spacing of transverse reinforcement was common, such that column failures may involve some form of shear or flexure-shear failure. As shear failure proceeds, degradation of the concrete core may lead to loss of axial load carrying capacity of the column. As the axial capacity diminishes, the gravity loads carried by the column must be transferred to neighboring elements. A rapid loss of axial capacity will result in the dynamic redistribution of internal actions within the building frame and may progressively lead to collapse. This sequence is the focus of the ongoing study reported here.

Particular incentive for this research has been provided by the experience of engineers involved in the seismic retrofit of buildings in California. Many have found, using prevailing rehabilitation methodologies, that it is not economically feasible to limit the building design displacements such that the columns are protected from shear failure. Thus, there is a need to improve understanding of column shear strength, as well as to understand how the gravity loads will be supported after a column fails in shear.

Reconnaissance of recent earthquakes provides evidence of the importance of column shear failure on collapse, as well as the possibility that shear failure of individual columns need not lead to collapse of the building. Laboratory experiments provide corroborating evidence, and suggest improvements to methods for estimating shear strength and deformation at loss of gravity load capacity. This paper summarizes the findings and the status of ongoing studies.
OBSERVATIONS FROM EARTHQUAKES

Earthquakes [Northridge, 1994; Kobe, 1995; Kocaeli, 1999; Chi-Chi, 1999; and others] have demonstrated that columns in older reinforced concrete building frames may be vulnerable to shear failures (Figure 1). While several experimental programs have illustrated that the lateral resistance of these columns is limited after shear failure, the residual axial capacity and stiffness have not been adequately investigated. Methods for reliably assessing the conditions under which axial load capacity is exhausted have not been identified.

In many cases, column damage in recent earthquakes has all but eliminated their axial capacity, yet the building has not collapsed (Figure 2). These examples illustrate the need to consider the whole system when evaluating a building for the collapse limit state. Mechanisms that may contribute to the capacity of a system to resist collapse include:

- catenary action of slabs and beams allowing gravity loads to span to adjacent elements,
- vierendeel truss action from the moment frame above a damaged column, and
- gravity load support provided by shear walls or non-structural elements such as partitions and infills.

Some gravity load collapses during earthquakes can be attributed to shear failure, and the subsequent loss of axial load carrying capacity, of multiple columns in a single story (Figure 3). The story-wide failures may be the result of massive internal redistribution of internal forces, possibly amplified by dynamic effects; however, the specific mechanisms leading to gravity load collapse of reinforced concrete frames are not well understood.
TEST ON LIGHTLY-CONFINED BUILDING COLUMNS

The literature of earthquake engineering documents a large number of simulated earthquake tests of columns representative of older existing building frames. In almost all cases, though, the reports provide unconvincing or incomplete results. Whereas tests of nearly full-scale columns under realistic loading and boundary conditions were sought, almost all tests were on smaller-scale columns tested as cantilevers. Furthermore, tests characteristically were discontinued after lateral load failures were observed, regardless of whether axial load capacity had been exhausted.

To better understand shear and axial load failure of columns, additional tests were done [Lynn, 1996; Sezen, 2000]. Figure 4 illustrates a typical test column configuration. The columns were constructed at full scale. Because the focus of this project was to study only the behavior of the column, the end beams were made relatively stiff and strong. The loading routine subjected the column to nominally constant axial compression and maintained nominally zero rotation between column ends while the column was subjected to series of lateral displacements at increasing amplitude, with three cycles at each amplitude. The two exceptions were Column 2CVD12, which had variable axial load ranging from 56 kips tension to 600 kips compression, and Column 2CLD12M, which was subjected to essentially monotonic loading to failure. For all columns, loading continued until axial load capacity was lost. The first twelve entries of Table 1 summarize specific column characteristics, material properties, applied loads, and measured responses.

Figure 5 plots measured relations between lateral force and lateral displacements for test columns 2CLH18, 2CLD12, and 2CHD12, illustrating three different failure modes. The data for 2CLH18 show moderate flexural ductility, followed by loss of lateral resistance due to apparent shear failure, followed at somewhat larger displacements by axial load failure. Column 2CLD12 had low flexural ductility interrupted by loss of lateral resistance due to
shear failure, but sustained vertical load capacity to relatively large
displacements. Column 2CHD12 had low flexural ductility interrupted by
sudden shear and gravity load failure.

Additional data were gathered to supplement the test results of Lynn and
Sezen. In selecting data, the following criteria were applied: cross sections were
rectangular, with one side not less than 2/3 the dimension of the other side and
minimum dimension around 8 in.; shear span ratio $2 \leq a/d \leq 4$; concrete
compressive strength range $2500 \text{ psi} \leq f'_c \leq 6000 \text{ psi}$; reinforcement nominal
yield stress $40 \text{ ksi} \leq f_y \leq 80 \text{ ksi}$; longitudinal reinforcement ratio $0.01 \leq \rho_l \leq 0.08$;
hoop spacing $s \geq d/2$; lateral load reversed and cyclic in application; failure
apparently attributable to shear distress. Although tests with contraflexure were
desired, among those tests satisfying the other criteria only cantilever tests were
identified. Tests reported by Bett, Klingner, and Jirsa (1985), Ikeda (1968),
Umemura and Endo (1970), Kokusho (1964), Kokusho and Fukuhara (1965)
were added (Table I). None of the cited works systematically reported response
beyond lateral load failure to the point of axial load failure.

In Table I, all quantities were obtained from the references with the exception
of the yield displacement, ultimate displacement, and resulting calculation for
displacement ductility. Each of the references used a different definition of these
terms. To provide uniformity among the data, the following procedure was used.
A secant was defined by the origin (zero load and zero displacement) and the
point where a horizontal line at 70 percent of the maximum applied shear
intersected the envelope curve. The yield displacement was then defined by
where that secant intersected a horizontal line passing through the envelope
curve at the maximum applied shear. The ultimate displacement was defined as
the displacement corresponding to termination of the test or where there was a
loss of more than 20 percent of the maximum applied shear.
DEVELOPMENT OF A SHEAR STRENGTH MODEL

There is considerable uncertainty in calculating the shear strength of lightly-confined reinforced concrete columns. Data from the tests discussed above are used to evaluate shear strength models from ACI 318-99 and FEMA 273. A new model is proposed that provides better correlation with the test results.

ACI 318-99

The ACI 318-99 Building Code provides requirements for design of new buildings. In the absence of specific guidelines for existing buildings, engineers have used some of the ACI 318-99 specifications to evaluate existing buildings. For columns with an axial load greater than $A_g f'_c / 20$, ACI 318-99 calculates the shear strength as follows:

$$V_n = V_c + V_s$$  \text{Equation 1}

$$V_c = 2 \left( 1 + \frac{P}{2000 A_g} \right) \sqrt{f'_c b_w d}$$  \text{Equation 2}

$$V_s = \frac{A_{sw} f_y d}{s}$$  \text{Equation 3}

where $V_n$ = nominal shear strength, $V_c$ = contribution from concrete, $V_s$ = contribution from ties, $P$ = axial load, $A_g$ = gross concrete area, $f'_c$ = concrete compressive strength (psi), $b_w$ = width of section, $d$ = effective depth, $A_{sw}$ = area of the tie steel, $f_y$ = yield strength of the tie steel, and $s$ = tie spacing.

Figure 6 plots ratios of experimental shear strength to $V_n$ (calculated by Equation 1) as a function of displacement ductility achieved in the test. Values exceeding unity are cases where the column developed strength exceeding the strength calculated by ACI 318-99. Previous studies on columns with higher quantities of transverse reinforcement have indicated that the ACI 318 equations can be excessively conservative, particularly when ductility demands are low (Priestley et al., 1994; Aschheim and Moehle, 1992). This same conclusion
apparently does not to apply for these lightly reinforced columns. The mean ratio of test to calculated strength is 1.11; the mean minus one standard deviation is 0.87.

**FEMA 273**

In 1997 the US Federal Emergency Management Agency published the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, commonly referred to as FEMA 273 (1997). Those guidelines contain a column shear strength evaluation method that was based mainly on tests of columns with relatively high amounts of transverse reinforcement. According to the guidelines, the concrete contribution is dependent on the ductility demand, as defined in equation 4.

\[
V_c = 3.5 \left( k + \frac{P}{2000A_g} \right) \sqrt{f'_c b d}
\]

Equation 4

where \( k = 1 \) for displacement ductility less than 2, otherwise \( k = 0 \).

The transverse steel contribution is calculated using Equation 3, except in yielding regions of columns where the transverse reinforcement is considered effective only if \( s \leq d/2 \) and hoops have hooks embedded in the core.

Figure 7 includes ratios of measured shear strength to the shear strength calculated according to the FEMA 273 procedure, as well as similar ratios for ACI 318-99. FEMA 273 tends to be excessively conservative, especially for cases where displacement ductility demand exceeds 2, because it sets a portion of \( V_c \) equal to zero. The mean ratio of test to calculated strength is 4.73; the standard deviation of that ratio is 2.77.

**Alternative Shear Strength Model**

An alternative shear strength model was developed. As with FEMA 273, the model assumes Equation 1 can represent the strength. The concrete contribution
is assumed to be related to the calculated nominal principal tension stress in the column. Principal tension stress capacity was set equal to \( f_{tc} = 6\sqrt{f'_c} \) psi. According to traditional stress transformation relations, the shear stress at which the principal tension stress capacity is reached is given by Equation 5.

\[
\tau_{xy} = 6\sqrt{f'_c} \sqrt{1 + \frac{P}{6A_g f'_c}} \tag{Equation 5}
\]

In a concrete column with flexure, this shear strength is reduced because of interaction with flexural stress and redistribution of internal actions as cracking occurs. This effect can be represented approximately by introducing an aspect ratio term, \( a/d \), where \( a = \) distance from maximum moment to inflection point. Multiplying by the cross-sectional area, \( A_g \), results in

\[
V_c = k \left( \frac{6\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6A_g f'_c}} \right) A_g \text{ psi} \tag{Equation 6}
\]

No bounds are placed on the aspect ratio \( a/d \), though it is noted that the range of values was limited to between 2.0 and 3.9 in the database. Some limits may be appropriate for columns having aspect ratios outside this range.

In Equation 6, the term \( k \) is a modifier to account for strength degradation within the flexural plastic hinges. Similar terms have been introduced in other shear strength models (Aschheim, 1993; Priestley, 1994). For this data set, \( k \) was defined as shown in Figure 7. Degradation relations proposed by other researchers [Aschheim, 1993; Priestley, 1994], as developed from data sets including columns with higher quantities of transverse reinforcement, were found to overestimate the rate of degradation for this data set.

The steel contribution \( V_s \) is defined by Equation 7, which is identical to Equation 3 except the shear strength contribution associated with transverse reinforcement is assumed to degrade with increasing ductility using the same coefficient \( k \) (defined by Figure 8) as was used for the concrete contribution in
Equation 6. Studies on the data in Table 1 showed that alternative expressions such as proposed by Aschheim (1993) and Priestley (1994) overestimated the contribution of hoops. It is noteworthy that for the columns considered in this study, the calculated value of $V_s$ typically was half or less of the calculated value of $V_r$. The reduction for ductility is reasonable considering that the truss mechanism associated with steel and concrete is likely to degrade in much the same way as does the concrete mechanism, especially (perhaps) for small quantities of transverse reinforcement.

$$V_s = k \frac{A_w f_y d}{s}$$

Equation 7

Work is currently being conducted by others to justify the choice of values for $k$ applied in the $V_r$ term based on Baysian updating of the shear strength equation using a large database of experimental data (Gardoni et al., 2000).

Figure 9 plots ratios of measured to calculated strengths using the alternative procedure. The correlation is relatively uniform for the range of ductilities shown. The mean ratio of test to calculated strength is 1.01; the mean minus one standard deviation is 0.90.

**AXIAL CAPACITY OF CONCRETE COLUMNS AFTER SHEAR FAILURE**

Experimental Evidence

Most tests of columns have been terminated shortly after loss of lateral load capacity. The resulting data are useful for columns considered as part of the lateral-force-resisting system. Considering traditional notions of safety (that is, once shear failure begins, axial load collapse cannot be far behind), the data also probably define a practical upper-bound displacement capacity even for columns not considered part of the lateral-force-resisting system in new building designs.
For existing buildings, whether being evaluated for seismic resistance or for seismic rehabilitation, a less conservative approach is required by economic and functionality considerations. If a column can reliably carry gravity load after its lateral strength degradation begins, it may be possible to achieve considerable savings by considering the column as a secondary component. It was mainly for this reason that the tests by Lynn (1996) and Sezen (2000) were conducted.

Figure 10 plots drift ratios corresponding to significant events for the twelve columns reported by Lynn and Sezen. For columns having lower axial loads, the tendency is for axial load failure to occur at relatively large drifts, regardless of whether shear failure had just occurred or whether shear failure had occurred at much smaller drift ratios. For columns with larger axial loads, axial load failure tended to occur at smaller drift ratios, and might occur almost immediately after loss of lateral load capacity.

**A Shear-Friction Model**

A shear-friction model can be used to represent the general observation from Figure 10 that the drift at axial load failure is inversely related to the magnitude of axial load. Figure 11 shows a free-body diagram for the upper portion of a column under shear and axial load. The external moment vector at the top of the column is not shown and will not enter the equilibrium equations written here. The external shear force \( V \) will be assumed equal to zero, under the assumption that the column has lost most of its lateral load resistance. The inclined free surface at the bottom of the free-body diagram is assumed to follow a critical inclined crack associated with shear damage. In this presentation, the “critical” crack is one that, according to the idealized model, results in axial load failure as shear-friction demand exceeds the shear-friction resistance along the crack.

Dowel forces from the transverse reinforcement crossing the inclined crack are not shown; instead, the dowel forces are assumed to be included implicitly in the shear-friction force along the inclined plane. Shear resistance due to dowel action of the longitudinal bars depends on the spacing of the transverse
reinforcement, and reasonably can be ignored for the columns considered in this study.

Relative movement across the shear failure plane tends to compress the longitudinal reinforcement. Given the tendency for buckling, especially in the limit, the axial force capacity of the longitudinal reinforcement will be assumed equal to zero.

In light of the above discussion, equilibrium of the forces shown in Figure 11 results in the following equations:

\[ \sum F_y \rightarrow P = N \cos \theta + V_{sf} \sin \theta \]  
\[ \sum F_x \rightarrow N \sin \theta = V_{sf} \cos \theta + \frac{A_{vf} f_y h \tan \theta}{s} \]

The literature documents shear-friction models that relate \( V_{sf} \) and \( N \) (Mattock and Hawkins, 1972; Mau and Hsu, 1988). The classic shear friction model, included in ACI 318 since 1977, idealizes the crack, across which shear must be transferred, as a flat plane with a coefficient of friction, \( \mu \), and computes the shear capacity as:

\[ V_{sf} = N \mu \]

Substitution of equation 10 into equations 8 and 9, and eliminating the case where \( \mu = \tan \theta \), gives the following expression for the axial capacity of the column:

\[ P = \frac{A_{vf} f_y h}{s} \tan \theta \left( \frac{\cos \theta + \mu \sin \theta}{\sin \theta - \mu \cos \theta} \right) \]

The inclination \( \theta \) of the shear failure plane can be estimated by considering the magnitude of the axial load in the column at the time of shear failure. A simplistic approach is to define \( \theta \) as the angle of the nominal principal tension
stress at the instant when it reaches the tensile capacity of concrete under combined shear and axial load, using the same model used to establish Equation 5. This approach, however, invariably results in an angle steeper than that observed in tests. An empirical approach is suggested instead. Figure 12 plots observed average angle of critical shear cracks observed in the tests by Lynn (1996) and Sezen (2000). (The authors estimated the angles subjectively from photographs.) The angle could be approximated as 60 degrees relative to horizontal, or could have the linear variation suggested by the unbroken line in the figure. The straight line has an intercept at 55° for zero axial load and passes through the angle 90° for $P/P_o = 1$. In Figure 12, $P_o$ is the pure axial capacity of the column given by $P_o = 0.85 f_c' (A_g - A_s) + f_y A_s$ where $A_s =$ area of longitudinal steel and the other variables are defined previously. (The outlying datum at $P/P_o \approx 0.21$ was for Column 3CMH18. That column had a critical crack that was somewhat less steep over most of its length, with a vertical segment near column mid-depth, resulting in the relatively large reported critical crack angle.) All of the columns tested by Lynn and Sezen had a height to width ratio greater than 6.0. For columns with low height to width ratio, it is expected that the maximum crack angle will be limited by the aspect ratio of the column (that is, $\theta_{max} = \arctan(\text{height/width})$).

An empirical approach was used to define the shear-friction coefficient in Equation 11, as follows. The critical crack angle $\theta$ for each column was assessed visually (Figure 12). Knowing this angle and all the other quantities in Equation 11, the value of the shear-friction coefficient was calculated. Figure 13 plots the calculated values as a function of the lateral drift ratio at which column collapse occurred. The data apparently follow a trend that can be approximated by a line.

The data of Figure 13 suggest that the apparent shear-friction coefficient is a function of the drift angle at failure. This relation is plausible considering that increased deformation (and increased sliding along the critical shear plane) degrades the roughness of the shear plane and reduces the effective friction. It is
worth recalling that the increased deformation capacities are associated with reduced axial loads (Figure 10).

The relation between axial load and crack angle (Figure 12), the relation between drift and friction coefficient (Figure 13), and the relation among axial load, transverse reinforcement, crack angle, and friction coefficient (Equation 11) can be combined to produce relations among column axial load, column transverse reinforcement, and drift ratio at loss of axial load capacity. The relations are plotted in Figure 14. The plotted relations suggest the intuitive result that drift capacity increases with increasing transverse reinforcement and decreasing axial load.

To convey a sense of the accuracy implicit in the relations of Figure 14, those relations were used to estimate the drift capacity of ten of the twelve columns reported by Lynn and Sezen. Of the remaining two columns, Column 2CVD12 was subjected to varying axial load and Column 2SLH18 sustained lap-splice failure, so they were not included in the comparison. The results are plotted in Figure 15. The mean ratio of calculated to measured drift at collapse is 0.92; the standard deviation is 0.36. The results of Figure 15, if applied, should be used only with full understanding that a significant number of columns are likely to fail at drifts below the calculated quantities. The relatively large scatter may be a product of inherent randomness associated with the complicated failure mechanism. Additional data and analyses may well improve our ability to predict the onset of gravity load failure of columns.

CONCLUSIONS

Shear failure of columns is identified as a primary cause of collapse of older reinforced concrete building frames in earthquakes. Test data were gathered to understand the effects of materials, geometry, and loading on failure mechanisms. Measured shear strengths are compared with calculations using the
1999 ACI Building Code and the FEMA 273 Guidelines for Seismic Rehabilitation of Buildings. An alternative procedure for calculating shear strengths is found to produce more accurate estimates. Subsequent collapse of columns is examined using a shear-friction model. The model identifies some variables controlling column collapse. Results of the model are compared with test results. Significant scatter between calculated and measured results suggests that additional study may be fruitful.

ACKNOWLEDGMENT

The authors gratefully acknowledge the significant contribution of Professor Abe Lynn, California Polytechnic State University, in the development of the experimental database and the alternative shear strength equation.

This work was supported in part by the Pacific Earthquake Engineering Research Center through the Earthquake Engineering Research Centers Program of the National Science Foundation under Award number EEC-9701568 and by the National Science Foundation under Grant No. BCS-9120214. The experimental studies by Lynn and Sezen were conducted in the research laboratories of PEER at the University of California, Berkeley. Photographs in Figures 1 and 2 are used courtesy of the National Information Service for Earthquake Engineering.

REFERENCES

ACI Committee 318 (1999) Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99), American Concrete Institute, Farmington Hills, Michigan.


Mattock, A.H., and Hawkins, N., (1972) Shear Transfer in Reinforced Concrete - Recent Research, Journal of Prestressed Concrete Institute, V. 17, No. 2.


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Table 1 Test specimen materials and details
Specimen

b
(in.)

d
(in.)

a

Isplice

(in.)

(in.)

No.
bars

Plong.

A.w
(in. 2)

(no.)

s

Tie
type

(in.)

f'c

fy.

fy.

p

fail.
mode

long. tran.

(ksi)

(ksi) (ksi) (kips)

Yu

lly

Ou

(kips)

(in.)

(in.)

JlO

Lynn and Moehle, 1996
3CLH18

18.00

15.00

58.00

none

8

0.03

0.22

18.0 r90

3.71

48

58

113 SCF

61.00 0.76 1.20

1.58

3SLH18

18.00

15.00

58.00

25

8

0.03

0.22

18.0 r90

3.71

48

58

113 SCF

60.00 0.68 1.15

1.69

2CLH18

18.00

15.00

58.00

none

8

0.02

0.22

18.0 r90

4.80

48

58

113 FSCF

54.00 0.72 3.00

4.17

2SLH18

18.00

15.00

58.00

20

8

0.02

0.22

18.0 r90

4.80

48

58

113 FSCF

52.00 0.68 1.80

2.65

2CMH18

18.00

15.00

58.00 None

8

0.02

0.22

18.0 r90

3.73

48

58

340 STF

71.00 0.31 0.60

1.94

3CMH18

18.00

15.00

58.00 None

8

0.03

0.22

18.0 r90

4.01

48

58

340 SCF

76.00 0.28 0.60

2.14

3CMD12

18.00

15.00

58.00 None

8

0.03

0.38

12.0 d90

4.01

48

58

340 SCF

80.00 0.36 0.90

2.50

3SMD12

18.00

15.00

58.00

25

8

0.03

0.38

12.0 d90

3.73

48

58

340 SCF

85.00 0.33 0.90

2.73

2.91

Sezen and Moehle, 2000
2CLD12

18.00

15.50

58.00 None

8 0.025

0.38

12.0 d90

3.06

64

68

150 SCF

72.60 1.06 3.08

2CHD12

18.00

15.50

58.00 None

8 0.025

0.38

12.0 d90

3.06

64

68

600 SCF

78.04 0.53 2.03

3.86

2CVD12

18.00

15.50

58.00 None

8 0.025

0.38

12.0 d90

3.03

64

68

Var. SCF

69.90 0.75 2.19

2.92

2CLD12M

18.00

15.50

58.00 None

8 0.025

0.38

12.0 d90

3.16

64

68

150 SCF

67.35 1.20 3.59

2.99

60

47.00 0.19 0.57

3.00

Belt, Klingner and Jirsa, 1985
12.00

10.38

18.00 None

8

0.02

0.20

8.0 d135

4.33

67

43

7.87

6.81

19.69 None

6

0.02

0.09

3.9 r135

2.84

63

81

18 FSCF

16.61 0.12 0.59

4.84

44

7.87

6.81

19.69 None

6

0.02

0.09

3.9 r135

2.84

63

81

18 FSCF

17.16 0.12 0.59

5.00
4.87

1-1

65

Ikeda, 1968

45

7.87

6.81

19.69 None

6

0.02

0.09

3.9 r135

2.84

63

81

35 FSCF

18.48 0.12 0.59

62

7.87

6.81

19.69 None

10

0.02

0.09

3.9 r135

2.84

50

69

18 FSCF

12.98 0.10 0.52

5.28

63

7.87

6.81

19.69 None

10

0.02

0.09

3.9 r135

2.84

50

69

35 FSCF

15.40 0.09 0.55

5.79

64

7.87

6.81

19.69 None

10

0.02

0.09

3.9 r135

2.84

50

69

35 FSCF

15.40 0.08 0.66

8.00

3.07

Umemura and Endo, 1970
205

7.87

7.09

23.62 None

6

0.02

0.09

3.9 r135

2.55

67

47

35 STF

16.02 0.17 0.51

207

7.87

7.09

15.75 None

6

0.02

0.09

3.9 r135

2.55

67

47

35 STF

23.80 0.13 0.25

1.88

208

7.87

7.09

15.75 None

6

0.02

0.09

3.9 r135

2.55

67

47

88 FSCF

30.36 0.10 0.31

3.20

214

7.87

7.09

23.62 None

6

0.02

0.09

7.9 r135

2.55

67

47

88 SCF

18.59 0.15 0.42

2.86

220

7.87

7.09

15.75 None

6

0.01

0.04

4.7 r135

4.77

55

94

35 FSCF

17.60 0.09 0.94 10.00

231

7.87

7.09

15.75 None

6

0.01

0.04

3.9 r135

2.14

47

76

35 FSCF

11.44 0.07 0.84

9.00

232

7.87

7.09

15.75 None

6

O.D1

0.04

3.9 r135

1.90

47

76

35 FSCF

13.09 0.11 0.94

8.89

233

7.87

7.09

15.75 None

6

0.01

0.04

3.9 r135

2.02

54

76

35 FSCF

15.53 0.11 0.54

4.93

234

7.87

7.09

15.75 None

6

0.01

0.04

3.9 r135

1.90

54

76

35 FSCF

15.07 0.11 0.63

5.71

372

7.87

6.69

19.69 None

4

0.01

0.10

3.9 r135

2.88

76

51

35 FSCF

16.72 0.10 0.42

4.12

373

7.87

6.69

19.69 None

4

0.02

0.10

3.9 r135

2.96

76

51

35 FSCF

19.80 0.14 0.39

2.78

Kokusho, 1964

Kokusho and Fukuhara, 1965
452

7.87

6.69

19.69 None

4

0.03

0.10

3.9 r135

3.18

52

46

88 FSCF

24.75 0.12 0.30

2.53

454

7.87

6.69

19.69 None

4

0.04

0.10

3.9 r135

3.18

52

46

88 FSCF

24.75 0.09 0.20

2.32

Nnt.ttion: A,w = an.:.1 of tie .<.h::cl; ,t =~bear ~p.m; b =square column dimension; d =depth to centerlinL" ofll:usion H:infon:en"lenl; f,. 101111= lo11g. rcinf. ykld \lrength;
fy.tr:u 1 =trans. reinf. yield strength; l~ = l.tp splice length; P =axial load (Var =varying .txi.d load); s =hoop sp.tcing; Yu =peak shear; Oy =yield di.<.placem:nt; &., =
db,pl.tccrll!nt when 20 pcn::cnt of peak shear is lost; P~t 111 ~ = total long. steel ratio;~~= 5./liy. Tie types are: r90 • rcct w/90° hooks; rl35 • rect. w/ 13511 hooks;
d90 ·reel. and diamoud w/90 11 hooks; dl35 · rcct. and diamond w/135° hooks. Failure modes are: FSCF • flexur.d shear compressiou f.tilure, several inclined
cr;Kks; SCF ·~hear comprl'ssion f;Jilun.:, many inclined cracks; STF- shear tension failure, very large inclined crack.


Figure 1  Column shear failures from (a) 1999 Kocaeli and (b) 1994 Northridge Earthquakes

Figure 2  Apparent loss of column axial load capacity without collapse of building frame from (a) 1999 Chi-Chi and (b) 1999 Kocaeli Earthquakes
Figure 3  Gravity load collapse from 1995 Kobe Earthquake (AIJ, 1997)

Figure 4  Typical column test specimen (Lynn, et al. 1996)
Figure 5  Measured relations between lateral load and displacement

Figure 6  Comparison of measured strengths with strengths calculated by ACI 318-99
Figure 7 Comparison of measured strengths with strengths calculated by FEMA 273 and ACI 318-99

Figure 8 Parameter $k$
Figure 9 Comparison of measured strengths with strengths calculated by alternative shear strength model

Figure 10 Measured column drift ratios as a function of axial load
Figure 11 Free body diagram of column after shear failure

Figure 12 Observed angles of critical cracks near axial load failure of columns
Figure 13 Relation between inferred shear-friction coefficient, $\mu$, and drift ratio at axial load failure. The straight line is a least-squares linear fit to the data.

Figure 14 Derived relation among axial load, transverse reinforcement, and drift capacity at axial load failure.
Figure 15 Comparison of calculated and measured drift capacities at axial load failure for test columns
The Influence of Design Ductility Capacity on the Seismic Performance of Medium Height Reinforced Concrete Frame Buildings
by A. C. Heidebrecht and N. Naumoski

Synopsis: This paper describes an investigation into the seismic performance of a six storey moment resisting frame structure located in Vancouver and designed and detailed in accordance with the seismic provisions of the National Building Code of Canada (1995). Both pushover and dynamic analyses are conducted using an inelastic model of the structure as designed and detailed. The structural performance of frames designed with different ductility capacities is evaluated using interstorey drift and member curvature ductility response as performance measures. All frames studied are expected to perform at an operational level when subjected to design level seismic excitations and to meet life safe performance criteria at excitations of twice the design level.

Keywords: building; code; design; drift; ductility; frame; inelastic; performance; seismic
INTRODUCTION

Objective of Paper

The design of buildings to resist earthquakes has benefitted considerably from experience gained by observing damage during previous earthquakes. Most often, that experience has enabled engineers to identify design and detailing practices which result in excessive damage or collapse. However, experience during some of the most recent earthquakes, e.g. the 1994 Northridge earthquake, has heightened concerns about the overall performance of buildings designed in accordance with modern building codes. The surprising extent of damage and collapses occurring in newer structures during the 1994 Northridge earthquake (1) resulted in a call for the building construction community to reexamine the performance criteria on which code provisions are based.

In Canada, seismic loading provisions have been included in the National Building Code of Canada (NBCC) since 1953. These have been revised and updated regularly since 1965 to reflect changes in the state-of-the-art of earthquake design based on research and experience of building performance during earthquakes in other countries. While each set of changes has been made on a rational basis, there has not been a comprehensive evaluation of the level of protection of building structures against strong seismic ground motions. Heidebrecht (2) proposed a research framework for the evaluation of the seismic level of protection which includes the facets of seismic hazard, seismic design, response/damage and vulnerability. That paper describes the application of the research framework to the evaluation of the NBCC level of protection, which is illustrated with a brief description of a pilot project on a six-storey reinforced concrete frame structure.
The objective of this paper is to evaluate the seismic performance of a specific building structure which is designed and detailed in accordance with the provisions of the NBCC. The building is a six storey moment-resisting reinforced concrete frame structure having the same geometric configuration as that in the pilot project referred to above. Both pushover and dynamic analyses are conducted using an inelastic model of the structure as designed and detailed. The structural performance of frames designed with different ductility capacities (fully ductile, nominally ductile and non-ductile) is evaluated using interstorey drift and member curvature ductility response as performance measures. The details of the design, analysis, behaviour and performance evaluation for these frames are presented below.

Description of Building and Design Parameters

The plan of the building is shown in Fig. 1. The storey heights are 4.0 m, with the exception of the first storey which has a height of 5.2 m. For the transverse direction, the two end moment-resisting frames (marked T on the figure) provide all of the earthquake resistance, with the other columns in the building carrying only gravity loads. The design of the transverse frames is dominated by lateral loads, since each frame carries one-half of the lateral load of the entire building while carrying only the gravity load of the adjacent half-bay. For the longitudinal direction, the earthquake resistance is shared among six moment-resisting frames (marked L1, L2 and L3 on the figure). With this configuration of structural systems, the design of the longitudinal frames is governed by both gravity and lateral loads. The floor system consists of a one-way slab spanning in the transverse direction, supported by the beams in the longitudinal frames; the slab is cast integrally with the beams.

The building is located in Vancouver, B.C. for which the NBCC 1995 (3) design peak ground velocity is 0.2 m/s. As an office building, the importance factor I = 1; the building foundation is on rock so that the foundation factor F = 1. Force reduction factors R = 4, 2 and 1.5 are used for ductile, nominally ductile and non-ductile moment-resisting frames respectively, as specified in Table 4.1.9.1.B. of NBCC 1995. The seismic response factor S is a function of the design period T; this is calculated using the formula $T = 0.1N = 0.6s$, (in which N is the number of storeys) in accordance with NBCC 1995 sentence 4.1.9.1.7a). The specified strengths of the concrete and reinforcing steel are, respectively, $f'_c = 30$ MPa and $f_y = 400$ MPa, except that $f_y = 300$ MPa for the slabs. The concrete modulus $E_c = 27,000$ MPa and the steel modulus $E_s = 200,000$ MPa. The CSA A23.3-94 materials standard (4) specifications are used for reducing moments of inertia of beams and columns due to cracking.
Design of Building

The details of the design of both the longitudinal and transverse frames are given by Naumoski and Heidebrecht (5). Only the overall process and some significant aspects are discussed here. Gravity and lateral loads are determined in accordance with NBCC 1995. While the building is symmetrical, the lateral forces carried by the frames are amplified to take into account the accidental eccentricity. The effective increase in lateral forces is approximately 14% for the transverse frames and 11% for the longitudinal frames; it is assumed that the L1 frames make no contribution to torsional resistance and that the L2 and L3 frames share equally in resisting torsion.

Member forces used for strength design are determined by an elastic analysis of the frames subjected to the combinations of factored loads as specified in section 4.1.3.2. of NBCC 1995. The model used for analysis includes rigid zones at the ends of columns and beams, except for the bottom of the first storey columns. The lengths of the rigid zones are selected to be approximately 2/3 of the depth of the columns and beams. The effects of cracking are included by using reduced member stiffnesses as recommended in the commentary of CSA A23.3-94, i.e. 40% and 70% of the gross EI for beams and columns respectively; the gross EI for the beams includes the slab with flange width as specified in clauses 10.3.3 and 10.3.4 of CSA A23.3-94. P-Δ effects are included using the approach specified in clause 10.16.3.3 of CSA A23.3-94. Moment magnification factors to account for P-Δ effects range from 1.02 to 1.20 for the transverse frames and from 1.03 to 1.22 for the longitudinal frames.

The elastic fundamental periods (based on uncracked sections) in each direction are the same for all three values of R because the member section sizes are kept the same and the different strengths are obtained by changing the reinforcing ratios; the periods for the transverse and longitudinal frames are 1.16s and 1.04s respectively. Both of these are substantially longer than the design period of 0.6s. In accordance with the approach specified in NBCC 1995, maximum inelastic interstorey drifts are calculated as R times the drifts obtained from the elastic analyses. The maximum calculated drift for the transverse and longitudinal frames is 1.22% and 1.06% respectively. Both of these are well below the NBCC 1995 drift limit of 2%, primarily because the design of both the transverse and longitudinal frames is controlled by member strength rather than drift.

The design of member reinforcement is done in accordance with CSA A23.3-94 using resistance factors of 0.6 and 0.85 for concrete and reinforcement respectively. Compression reinforcement in the beams is included in the calculation of moment resistance; a rectangular section is assumed for designing the beams, even though the actual beams in the transverse frames have Γ-shapes
and those in the longitudinal frames have T-shapes. The column design differs for the frames with R = 4, 2 and 1.5.

The columns for the ductile frames (R=4) are designed in accordance with the capacity design method as specified in Clause 21 of CSA A23.3-94. This method requires that the sum of the factored moment resistances of the columns framing into each beam-column joint is at least 10% greater than the sum of the nominal moment resistances of the beams framing into the same joint. The capacity design method is applied at the joints at all levels from the first floor to the roof. While Clause 21 of CSA A23.3-94 gives detailed recommendations for the design of the columns at the upper levels of ductile moment resisting frames, there is no mention of the design of column sections at the base of such frames. However, the design of these sections is very important in achieving the preferred mechanism (i.e. formation of plastic hinges first at the beam-ends followed by hinging at the base of the columns) when the frame is subjected to strong earthquake motions, which is the basis of the capacity design philosophy. After discussions with a Vancouver structural designer (6) it was decided to design the moment capacities of the columns at the base of the first storey to be in the same proportion to those at the top of the first storey as computed in the elastic analysis when the structure is subjected to the design seismic load V. In addition to the capacity design requirements for longitudinal reinforcement, the transverse reinforcement is designed to satisfy both the confinement and shear strength requirements for ductile frames, as specified in Chapter 21 of CSA A23.3-94.

The nominally ductile frames (R=2) are designed in accordance with the specifications of Clause 21.9.2 of CSA A23.3-94. Since capacity design is not required for these frames, the longitudinal reinforcement in both the columns and beams is determined using the member forces obtained from the design load combinations. However, for frames with R=2, CSA A23.3-94 provides detailed specifications for design of transverse reinforcement in order to prevent brittle shear failure, i.e. by providing shear capacity which is larger than the shear demand, as specified in Clause 21.9.2.3 of CSA A23.3-94.

The frames with R=1.5 are designed without any consideration for ductile behaviour. The longitudinal beam and column reinforcement is determined using the member forces obtained from the elastic analysis of the frames subjected to the combined design loads. Minimum transverse reinforcement is assumed for the members of these frames in accordance with Clauses 11.2.8 and 11.2.11 of CSA A23.3-94.

The transverse frames designed for R = 4, 2, and 1.5 are designated A2T, B2T and C2T respectively; the corresponding interior longitudinal frames resisting torsion (L2 in Fig. 1) designed as described above are designated as A2L, B2L and C2L. Member sizes and percentages of longitudinal reinforcement for all frames
are given in Table 1; details of transverse reinforcement are given by Naumoski and Heidebrecht (5). The exterior column steel ratios in the longitudinal frames are identical to the interior column steel ratios of the corresponding transverse frames because they are the same columns (see Fig. 1).

ANALYSIS OF FRAMES

Modelling of Frame Members

For the purpose of determining the performance of the three frames when subjected to earthquake ground motions, inelastic models of each frame are developed for use in an inelastic dynamic analysis program, a McMaster enhanced version of IDARC (7)(8). Moment-curvature relationships for the end sections of each beam and column are determined using a fibre analysis of the cross-section. The concrete stress-strain relations for the members of the ductile (R=4) and nominally ductile (R=2) frames include the effect of confinement, based on the model proposed by Mander et al. (9); the relations for unconfined concrete are used for the members of the non-ductile (R=1.5) frames. The computed moment-curvature relationships are simplified into a tri-linear model with the first segment corresponding to the uncracked stiffness, the second segment corresponding to the region between cracking and yielding, and the third segment to the post-yielding range. Stiffness degradation and pinching effects are taken into account in the analyses using a hysteretic model specifically developed for reinforced concrete, which closely approximates experimentally observed behaviour. The parameters used in this hysteretic model are based on published results (10)(11)(12) from experimental investigations of reinforced concrete structures designed in accordance with NBCC requirements. A detailed description of the modelling process and the selection of the model parameters is given by Naumoski and Heidebrecht (8)(13).

Nominal member moment capacities (i.e. using \( \phi_c = \phi_s = 1 \)) are used for performance evaluation, rather than the factored capacities used in design. This is deemed appropriate because the actual material strengths which exist when an actual structure is subjected to an earthquake are likely to be considerably higher than the factored strengths.

P-\( \Delta \) effects, i.e. the additional member forces and deformations associated with moments developed by gravity loads acting on the deflected structure, are included in the analyses. In the analysis of the transverse frames it is assumed that the interior columns of the building have no lateral capacity in the transverse direction and the P-\( \Delta \) effects for the full structure are taken by the two end lateral load resisting frames.
Static Pushover Analysis and Results

Pushover analysis consists of applying a monotonically increasing static lateral load to the structure, with the same force distribution as specified in NBCC 1995. Loading is steadily increased to either a predetermined level of base shear or to a predefined value of lateral deflection. Pushover analysis is an efficient tool used to analyse the behaviour of the structure, highlighting the sequence of member cracking and yielding as a function of the level of base shear. This enables the locations which are likely to be subjected to large inelastic deformation to be identified; this information can be used for both the design of member detailing as well as in the evaluation of the performance of the structure. The load-displacement relationship determined from the pushover analysis is an indication of the global response to lateral loading, including the extent of overstrength and the deflections at which various degrees of damage are likely to occur.

The pushover base shear vs. roof displacement relationships for all the transverse and longitudinal frames are shown in Fig. 2. This figure also shows the design base shear for each frame and identifies the loads/displacements at maximum interstorey drifts ranging from 0.5% to 2.5% at increments of 0.5%. For each relationship, the onset of significant member yielding can be seen clearly by the sharp reduction in stiffness.

The differences in strength among the transverse frames with different R values is readily apparent. The overstrength at the first significant member yielding is approximately 1.5 for the ductile frame (A2T) but is only about 1.25 for the other two frames. However, the differences in deformation behaviour are equally significant. As discussed later, the near-failure displacement ductility ratio can be approximated as the ratio of displacement at 2.5% maximum drift to the displacement at first significant member yielding. These ratios are approximately 4.5, 3 and 2.5 for frames A2T, B2T and C2T respectively. The ductile frame yields at a lower maximum drift and has more deformation capacity prior to failure than the other frames. The superiority of the ductile frame is apparent both from overstrength and deformation capacity considerations.

The behaviour of the longitudinal frames is similar to that of the transverse frames. The overstrength values at first significant member yielding are approximately 1.65 for the ductile frame (A2L) and 1.5 for the nominally ductile (B2L) and non-ductile (C2L) frames; the near-failure displacement ductility ratios are approximately 5, 3 and 2.5 for frames A2L, B2L and C2L respectively. While this comparison is relatively approximate, it shows that the longitudinal frames have slightly larger overstrengths than the corresponding transverse frames. This is likely due to the fact that the design of the longitudinal frames is gravity-load dominated so that their behaviour under very large lateral loads can be expected to be slightly better than that of lateral-load dominated frames.
Each structure was subjected to an ensemble of 15 acceleration time-histories having spectral shapes similar to those of design level seismic ground motions expected in Vancouver. Spectral shapes are related to the $a/v$ ratio, in which $a$ is the peak ground acceleration, in units of "g", and $v$ is the peak ground velocity, in units of m/s. The values of $a$ and $v$ for Vancouver, as shown in the NBCC 1995 seismic zoning maps, are both 0.20, so that $a/v = 1$. The selected ensemble (14) has an average $a/v$ of 1.02, with values for individual records ranging from 0.82 to 1.21. Each of these time-histories was scaled in terms of its peak horizontal velocity, on the basis that the design is velocity-dependent and that the response of structures with periods ranging from 0.5 to 2.5s is related primarily to the peak ground velocity rather than to the peak ground acceleration of the earthquake motion. In order to determine the performance for a full range of excitations which could be expected during a structure's lifetime, excitations ranged from 0.1 m/s to 0.6 m/s. While the highest excitation level corresponds to 3 times the design level, the uncertainty in estimating peak ground motions is such that values ranging from 2 to 3 times the expected value can easily occur (15).

The maximum transient drift in each storey and the maximum curvature ductility factors at the end of each beam and each column are determined from the dynamic response due to each seismic excitation. The maximum values for all the time-histories at each excitation level are analysed statistically in order to determine the mean (M) and mean plus one standard deviation (M+SD) values for each response parameter. The M+SD values are used in the subsequent comparisons because of the requirement that there be a high level of confidence that damage will be less than some specified value; the M+SD level represents approximately an 84% level of confidence.

**PERFORMANCE CRITERIA**

SEOAC (16) has proposed a performance-based approach to seismic design. Performance level expresses maximum permissible damage to a building when subjected to specific earthquake design ground motions (i.e. frequent, occasional, rare and very rare). Each performance level (fully operational, operational, life safe, near collapse and collapse) has an associated damage state, ranging from negligible to complete. Design performance objectives are defined for facilities of varying significance, i.e basic, safety critical and essential/hazardous, by specifying the minimum performance level associated with each specified earthquake design level. For example, basic facilities are expected to perform at a life safe level when subjected to ground motions associated with a rare earthquake level, i.e. having a probability of exceedance of 10% in 50 years.
Of interest in the evaluation of performance is the fact that SEAOC has specified a direct link between the performance level and the maximum permissible transient interstorey drift. The maximum permissible transient drifts, expressed as a percentage of the storey height, and qualitative statements about expected damage for the three intermediate performance levels are:

**operational performance:** 0.5% drift, light overall building damage, negligible damage to vertical load carrying elements, original strength and stiffness retained in lateral load carrying elements with minor cracking/yielding of structural elements; for primary concrete frame members: minor hairline cracking and limited yielding at a few locations

**life safe performance:** 1.5% drift, moderate overall building damage, light to moderate damage of vertical load carrying elements with substantial remaining capacity to carry gravity loads, some reduction of residual strength and stiffness in lateral load carrying elements with lateral system remaining functional; for primary concrete frame members: extensive damage to beams, spalling of cover and shear cracking for ductile columns, minor spalling in non-ductile columns

**near collapse performance:** 2.5% drift, severe overall damage, moderate to heavy damage of vertical load carrying elements which continue to support gravity loads, negligible residual strength and stiffness in lateral load carrying elements; for primary concrete frame members: extensive cracking and hinge formation in ductile elements, limited cracking and/or splice failure in some non-ductile columns, severe damage in short columns

In this paper, the evaluation of performance will be based primarily on comparing interstorey drifts with the drift criteria. As there is little quantitative information concerning the relationship between local damage measures (i.e. member curvature ductilities) and overall building performance, the qualitative descriptions given by SEAOC as summarized above will also be used to evaluate performance.

**DRIFT PERFORMANCE OF FRAMES**

Drifts

Figs. 3a and 3b show the relationships between excitation velocity and the M+SD level transient drift for the transverse and longitudinal frames respectively; these relationships are often referred to as fragility curves. These figures also include the drift limits associated with SEAOC operational, life safe and near collapse performance.
Considering first the transverse frames, there is essentially no difference among the three frames for velocities at or below the design level, i.e. 0.2 m/s. At the design excitation, the M+SD transient drift is 0.5%, i.e. corresponding to the SEAOC operational performance level.

While there are some differences among the three curves for excitations greater than the design level, these differences are not substantial. For all three transverse frames, the life safe performance level drift limit of 1.5% is not exceeded until the excitation reaches twice the design level; the near collapse performance drift limit of 2.5% is reached at an excitation of nearly three times the design level.

The fragility curves for the three longitudinal frames are similar, although the maximum drifts for the higher excitation levels are slightly less than those for the transverse frames. Drift performance at the design excitation is approximately the same as for the transverse frames, i.e. operational performance. The SEAOC life safe limit is reached when the excitation is 2 to 2.5 times the design level; the M+SD drift at 0.6 m/s excitation ranges from 1.5 to 2%, well below the near collapse performance drift limit of 2.5%.

Acceptability of Frame Performance on Basis of Drift

Commentary J of NBCC 1995 states that “structures designed in conformance with these provisions should be able to resist moderate earthquakes without significant damage and major earthquakes without collapse.” However, moderate and major earthquakes are not defined. Consequently, it is not possible to make explicit reference to NBCC 1995 performance expectations in evaluating the performance of these frames. However, because ductility-based force reduction factors are used in calculating seismic design forces, it is implicit that some inelastic behaviour and minor damage would be expected (and be acceptable) at design level excitations. Drifts of 0.5% or less imply little or no damage so that, on the basis of M+SD drift levels, the performance of all frames would be considered acceptable, and probably better than expected.

It is more helpful to make reference to the SEAOC performance criteria mentioned previously. SEAOC makes a direct linkage between performance levels and earthquake design level. For basic facilities, i.e. normal buildings, operational performance is associated with occasional earthquake motions (50% in 50 years probability of exceedance), life safe performance with rare earthquake motions (10% in 50 years), and near collapse performance with very rare earthquake ground motions (10% in 100 years). On this basis, the achievement of operational performance for 10% in 50 year ground motions indicates that these frames exceed the SEAOC performance objectives.
It is also of interest to note that 1997 edition of the NEHRP seismic provisions (17) is using ground motions with a 2% in 50 years probability of exceedance which are defined as maximum considered earthquake ground motions. The stated intent in the 1997 NEHRP Commentary (18) is “to provide a uniform margin against collapse at the design ground motion”. The implication is not that structures are expected to fail at this ground motion but that these ground motions are a lower bound estimate of the margin against collapse. Using this lower bound concept, it would be expected that the response of structures subjected to 2% in 50 year ground motions could be heavily damaged, but not expected to collapse. This conforms to the SEAOC near collapse performance level. The most recent estimates of seismic hazard in Canada (19) show that ground motions at the 2% in 50 year probability level are approximately twice the 10% in 50 year values. On this basis, one could expect the 2% in 50 year maximum horizontal velocity in Vancouver to be approximately 0.4 m/s. The fragility curves in Figs. 3a and 3b show that maximum drifts at this velocity are less than 1.5% (at the M+SD response level), or only about 60% of the near collapse drift limit.

All of these evaluation approaches indicate that the drift performance of these six storey frames designed in accordance with the seismic provisions of NBCC 1995 exceeds the expected minimum. It is notable that this observation can be made for the non-ductile and limited ductility frames as well as the ductile frames, which would be expected to have very good performance when subjected to strong seismic ground motions. However, a cautionary note must be struck here; the modelling of these frames assumes that the more brittle non-flexural modes of failure (e.g. shear) will be prevented by appropriate detailing. While such detailing is expected in ductile frames, it is not clear that the normal detailing and construction methods for non-ductile frames would be sufficient to prevent such non-ductile failure modes, especially at large deformations.

**CURVATURE DUCTILITY PERFORMANCE OF FRAMES**

**Member Curvature Capacities**

The curvature capacity of a member depends upon a number of parameters including cross-sectional geometry, amount and distribution of longitudinal and transverse reinforcement, and the stress-strain properties of the concrete and reinforcement. Assessment of the curvature ductility capacity for frame members subjected to reversing cyclic loading is very important for the investigation of the seismic performance of the frames. Unfortunately, however, curvature capacity results are available from only a few experimental studies. Of special use for ductile and nominally ductile frames are the experimental results reported by Watson and Park (20) and Park et al. (21) from investigations of the performance
of well designed columns subjected to cyclic loading. The test results from these studies show that curvature ductilities of 10 to 20 can be achieved under cyclic loading of well confined columns subjected to axial load of up to 30% of the column axial crushing load.

Results from experimental studies on the performance of columns designed according to earlier codes were examined to assess the capacities of non-ductile frame members. For purposes of comparison with retrofitted columns, Daudeau and Filiatrault (22), Mes (23) and Priestley et al. (24) conducted quasi-static cyclic testing of square and rectangular columns designed in accordance with pre-1970's codes. The curvature ductility capacities of these columns range from 2 to 5.

Curvature capacities for the members of the frames in this study were computed by fibre analysis using actual member cross-sections and the designed longitudinal and transverse reinforcement. The ranges of computed capacities for ductile and non-ductile members are almost twice the ranges of the corresponding experimental results mentioned above. However, the experimental results are normally considered more reliable, particularly because they include deterioration due to cyclic loading, and these are used for the performance evaluation of the frames, rather than the fibre analysis capacities.

**Member Curvature Ductility Demand**

Figs. 4a and 4b show the M+SD level fragility curves for all frames for column and beam curvature ductilities respectively. While the scatter of values is not captured in these diagrams, the coefficients of variation (for curvature at a particular location among the 15 time-history excitations) range from about 0.1 to 0.4 for longitudinal frames and 0.1 to 0.8 for transverse frames. The larger coefficients of variation occur at the higher excitations and are typically at locations with larger curvatures. This is quite reasonable as one would expect more variability for different excitations when the response involves more inelastic deformation.

Consider first the longitudinal frame columns (Fig. 4a); the curvature ductilities in these columns increase very slightly with increasing excitations. The column response of the ductile (A2L) and nominally ductile (B2L) is below yielding even at an excitation of 0.6 m/s. At that excitation, no yielding occurs in any of the columns of the ductile frame A2L and only 1 of the 15 time-histories in the ensemble produces a very small amount of yielding at one joint in the fourth storey of frame B2L. The columns of frame C2L remain elastic for excitation velocities up to and including 0.4m/s but there is significant column yielding when the velocity reaches 0.6 m/s. However, this is of little concern since this yielding takes place almost entirely at the top of the 6th storey interior columns.
Fig. 4b shows that the beams in the ductile longitudinal frame (A2L) suffer a small amount of inelastic deformation (maximum curvature ductility of about 1.5) at the design excitation level but that the beams in the nominally ductile and non-ductile frames (B2L and C2L) remain elastic at this excitation. For excitations at twice the design level (0.4 m/s), the maximum beam curvature ductility in A2L reaches 5 but the maximum for the other two frames is only about 2.5. The pattern changes for $v = 0.6$ m/s, but the maximum values of curvature ductility are still only about 7.5 and 4 for the beams in the ductile and nominally ductile frames respectively, which is well below the deformation capacities of such beams. At this velocity, the maximum beam curvature ductility for the non-ductile frame is about the same as that of the ductile frame, i.e. 7.5. Given that the maximum expected curvature capacity of non-ductile frame members is about 5, failure of some members of the non-ductile longitudinal frame could be expected when the velocity reaches 0.5 m/s. However, these maximum values occur in the beams of the top floor so such failures are not likely to lead to collapse of the structure.

Turning now to the transverse frames, the columns in all these frames remain elastic at the design excitation (0.2 m/s) but deformations increase significantly with higher excitations. As would be expected, the largest ductility demand occurs in the columns of the ductile frame (A2T), with the demands in the other two frames considerably lower. At excitations of 0.4 m/s and higher, there is a very clear hierarchy of increasing demand with increasing member ductility capacity. A maximum $M+SD$ curvature ductility demand of about 7 in the columns of the ductile frame at an excitation of 0.6 m/s is quite acceptable since it is well below the capacity of well designed columns. While one can expect that less stringent detailing in the members of the other frames would result in lower deformation capacities, maximum curvature ductilities of 4.5 and 3.5 at $v = 0.6$ m/s for nominally ductile and non-ductile frames should be sustainable.

The pattern of deformation in the beams of the transverse frames at the design excitation level is essentially identical to that for the beams in the longitudinal frames, i.e. minor inelastic behaviour in the beams of the ductile frame and essentially no yielding in the beams of the other two frames. However, at higher excitations the beams in the transverse frames sustain more ductility demand than their counterparts in the longitudinal frames. At twice the design excitation level, i.e. for which maximum $M+SD$ drift is at the SEAOC life safe performance level, the maximum $M+SD$ beam curvature ductilities are approximately 7.5 in the ductile and nominally ductile frames. These beams can be expected to retain most of their original strength and stiffness at this level of deformation. A maximum ductility demand of approximately 5 in the beams of the non-ductile frame would cause significant distress but not collapse, because this maximum is in the top floor beams.

When excitation is three times the design level (0.6 m/s), the maximum $M+SD$ curvature ductilities in the ductile frame are approximately 13, which is in
the range of experimentally determined ductility capacities (10 to 20) for well
detailed members, as mentioned earlier in this paper. However, the maximum
demand in the beams of the nominally ductile frame is approximately 18, which
approaches the experimentally observed upper value of 20. Consequently, it is
expected that the nominally ductile frame excited at 0.6 m/s would suffer
significant damage in a few of its beams. However, since the largest ductilities
are in the top floor beams, this damage is likely to occur only in these beams so it
would not significantly hamper the structure’s ability to survive such motions
without collapse. The rapid rise of ductility demand (as velocity increases) in the
beams of the non-ductile frame indicates that this frame would be in serious
distress for excitations of 0.4 m/s and larger, with collapse likely if the excitation
velocity were to reach 0.6 m/s.

Relationship of Maximum Member Curvatures to Drifts

It is of interest to evaluate the relationship between interstorey drifts and
member curvature ductilities obtained from dynamic results in order to link the
drift limits for the various performance objectives with specific levels of member
ductility demand. Figs. 5a and 5b show these relationships for the transverse
frames at maximum M+SD response levels for both columns and beams
respectively; these figures contain the same information as Figs. 3 and 4.

Consider first the drift/column curvature relationships (Fig. 5a) for the
transverse frames. There is a near linear relationship for frame A2T, with
maximum curvature ductilities of 4 and 7 associated with the life safe and near
collapse drift limits of 1.5% and 2.5% respectively. For the other two transverse
frames, the gradient of column ductility demand increase with increasing drift
decreases markedly for maximum drifts above 1%. Of the longitudinal frames,
there is also a near linear relationship for frame C2L. Because the columns in
frames A2L and B2L remain elastic at all excitations, there is no apparent
relationship between increasing drift and increasing column ductility demand.

The picture for the beams (Fig. 5b) is considerably different. There is a near
linear relationship for all frames after beam yielding has occurred. The gradients
for all three transverse frames are similar; there is a slightly lower gradient for
longitudinal frames A2L and B2L but the gradient for frame C2L is similar to that
for the transverse frames.

From these figures, it is clear that the maximum inelastic deformations in the
columns and beams of the ductile transverse frame both increase proportionately
as maximum drift increases. There is a similar relationship, with smaller
gradients for the ductile longitudinal frame. However, for the nominally ductile
and non-ductile frames in both directions, as maximum drift increases the column
curvatures increases are limited because beam curvatures continue to increase significantly. This behaviour is quite consistent with design objectives for these frames, i.e. to limit deformations in the columns because of their lower ductility capacities (in comparison with the ductile frames).

Acceptability of Frame Performance on Basis of Ductility Demand

The primary quantitative reference for ductility-based performance is the information on member capacities which is given earlier in this paper; SEAOC’s qualitative descriptions of performance expectations in structural systems and concrete frame members is also of some use.

For excitations at or below the design level, it can be expected that the behaviour of the nominally ductile and non-ductile frame members will be entirely elastic. The columns in the ductile frames can also be expected to remain elastic but there is a very small amount of yielding in a few beams of the ductile frames. This behaviour, for all frames, is consistent with the SEAOC expectations for operational performance. It is also consistent with the observations made on the basis of maximum drifts, i.e. maximum values of 0.5% or less.

Discussion in an earlier part of this paper indicated that ground motions of twice the 10% in 50 year design level correspond approximately to 2% in 50 year values, which are expected to be a lower bound estimate of the margin against collapse. At this excitation, (i.e. \( v = 0.4 \) m/s) the maximum curvature ductilities in the members of the transverse frames are quite modest (maxima of 7.5 and 3.5 in the ductile frame for beams and columns respectively) and these occur at only a few locations rather than being widespread. Not only are these values well below the expected member deformation capacities, but the extent and distribution of damage is far less than stated by SEAOC for the near collapse performance level. Consequently, the performance of the transverse frames is certainly acceptable and actually considerably better than the stated expectations.

The behaviour of the members in the longitudinal frames at 0.4 m/s excitation is even better than that of the transverse frame members. There is no yielding in the columns of any of the frames and the maximum beam ductility demand is 5 (in the ductile longitudinal frame). The performance of the longitudinal frames is also much better than expected.

While there is no explicit performance objective for ground motions at three times the design level, it is clear that these frames could expect to sustain such ground motions without collapse. While the maximum curvature ductility demand at a few locations in the nominally ductile and non-ductile transverse
frames would result in significant loss of capacity in those members, the members of all the other frames could be expected to retain essentially all of their load carrying capacities at this level of excitation. This is important because of the significant variability of ground motions, i.e. it is not unexpected that actual values would be 2 to 3 times the design value. For example, measured values of peak acceleration at several locations during the 1995 Northridge earthquake were well in excess of $1g$ (25) while the design ground acceleration is $0.4g$ (26).

**DISCUSSION AND CONCLUSIONS**

While the frames considered in this investigation have been designed to take into account accidental torsion (loads in transverse and longitudinal frames increased by 14% and 11% respectively, as mentioned previously), the performance evaluation presented in this paper is based entirely on a two-dimensional analysis, i.e. without considering the torsional dynamic response of the structure. This type of structure does not have a strong lateral-torsional dynamic coupling, i.e. torsional periods are well separated from lateral periods. Consequently, the code requirements for increases in response are expected to be conservative; the increases in maximum response parameters would be less than 15%. Given the uncertainties in determining seismic hazard and the use of M+SD level responses, the conclusions reached from the results of a two-dimensional analysis are considered to be valid for medium rise (10 storeys or less) moment-resisting frame structures with configurations (storey heights, column spacings and numbers of bays) similar to the structure considered in this investigation.

As mentioned earlier, SEOAC (Vision 2000 Committee, 1995) has defined as a performance objective that structures of normal importance meet the life safe performance level when subjected to earthquake ground motions having a 10% in 50 yr. probability of exceedance. Since the design velocity used in NBCC 1995 is specified to have that probability, the performance of the frames can be evaluated using this SEAOC performance objective.

Using the M+SD response level to provide a reasonable degree of confidence that the response parameter will not be exceeded, the discussions of both drift and member ductility response given earlier in this paper show that the transverse frames all meet life safe performance criteria for excitations up to twice the design level and that the longitudinal frames meet those criteria for excitations up to 2.5 times the design level. All the frames meet operational performance criteria for excitations up to the design level. Performance at both the operational and life safe levels exceed minimum expectations. The near collapse performance limits are not exceeded in the transverse frames until excitation reaches three times the design level excitation.
The comparison between the performance of the longitudinal and transverse frames shows that the gravity dominated longitudinal frames suffer less damage than the earthquake dominated transverse frames. This is an understandable observation given that the proportionately higher gravity loads in the longitudinal frames result in higher overstrength and a higher overall displacement ductility capacity.

ACKNOWLEDGEMENTS

The authors acknowledge the financial support of the Natural Sciences and Engineering Research Council of Canada and McMaster University, in the form of research grants to the first author. Thanks and appreciation are due to Ron DeVall of Read Jones Christoffersen Ltd. for advice on the configuration and design of the reinforced concrete frames discussed in this paper.

REFERENCES


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NOTES: 1. Percentages in **bold** are cases for which values based on strength are below minimum values required by the materials code. 2. Percentages shown in this table are expressed in terms of gross cross-sectional dimensions.
Figure 1 Plan of a six-story reinforced concrete office building
Figure 2  Base shear vs. roof displacement from pushover analyses
Response of Transverse Frames
Mean + One Standard Deviation (M+SD)

Maximum Excitation Velocity (m/s)

- Near Collapse Limit
- Life Safe Limit
- Operational Limit

Figure 3a Dynamic fragility of transient drifts with excitation velocity: transverse frames
Figure 3b Dynamic fragility of transient drifts with excitation velocity: longitudinal frames
**Figure 4a** Dynamic fragility of member curvature ductilities with excitation velocity: columns

**Figure 4b** Dynamic fragility of member curvature ductilities with excitation velocity: beams
Dynamic Response Relationships
Column Curvatures and Drift

Figure 5a Dynamic member curvature vs. interstorey drift: columns

Dynamic Response Relationships
Beam Curvatures and Drift

Figure 5b Dynamic member curvature vs. interstorey drift: beams