The Paulay Years

by R. Park

Synopsis: An outline is given of the many significant and pioneering contributions made by Emeritus Professor Tom Paulay to the understanding of the behaviour of reinforced concrete and to the design of reinforced concrete structures for earthquake resistance. Particularly innovative has been his research into the design of structural walls for earthquake resistance, including the concept of the use of diagonal reinforcement in coupling beams. Other internationally recognised research described are his outstanding investigations into the mechanisms of shear resistance of reinforced concrete, aggregate interlock across cracks, behaviour of beam-column joints, and the capacity design and detailing procedures for structural walls and frames.
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FROM CAVALRY OFFICER TO UNIVERSITY PROFESSOR

Born in Sopron, Hungary on 26 May 1923, Tom Paulay was initially destined for a life in the Royal Hungarian Army. After attending a boarding school for military cadets in Sopron he entered the Royal Hungarian Military Academy in Budapest. On graduating he was posted as a second lieutenant to the same cavalry regiment in which his father served for many years.

One year later, in 1944, he faced the advancing Russian army in the Prypet Marches of the then Eastern Poland. At the age of 21, after mounting casualties, he found himself in command of a cavalry squadron consisting of 278 men and 308 horses. Action in Poland and later in Hungary, and months spent in various military hospitals, left him somewhat deaf, a feature remembered by his later colleagues and students.

After discharge from the army in 1946, with many other returned servicemen he joined the first year civil engineering class of 360 at the Technical University of Budapest. Describing this stage of his life, Tom Paulay has written that:

"The Technical University of Budapest after 52 days siege of the city was barely habitable. The fact that during the winter snow fell through large holes in the ceiling of the largest lecture room, did not interfere with the attraction with which brilliant lectures in engineering mathematics were followed. The professor wore two raincoats (his winter coat was buried under his house) and he wrote his equations on the blackboard wearing knitted gloves. Dozens of shallow graves all over the campus, where German, Russian and Hungarian soldiers had been hastily buried during the battle of Budapest, were daily reminders. They stifled any temptation to grumble about physical deprivations. Reliance by students on fellowship was a prerequisite to preserve sanity in the process of coping with hunger, the cold and the immense academic pressure."
The compromise between preserving free entry to the university and the greatly diminished immediate need for engineers in a totally collapsed economic system, resulted often in 75% failure rates at the end of the first year. While physical conditions improved slowly the political scene deteriorated dramatically. By 1948 Joseph Stalin and the Red Army imposed virtually full control over society by means of channelling the power into the hands of the Communist Party and its tool, the political police. For them to subdue within the campus an idealistic and hopeful but largely apolitical student body, was a formidable task. However, the outcome was inevitable. In 1948 Tom Paulay was one of the few who escaped from the Budapest equivalent of Rochester Hall (a Catholic hall of residence at the University of Canterbury), dissolved overnight by government orders. He made it across closely guarded forests to Austria and West Germany. Most of his friends, including his roommate, did not succeed. They spent some five years in a concentration camp.

In West Germany he enrolled at the Technical University of Munich but lack of financial resources soon terminated his attempt to continue civil engineering studies. For three years he occupied himself with international student relief activities, sustained by charitable organisations, in his new status as a stateless person. At this stage he began to teach himself English.

His next turn of fate favoured New Zealand. He was offered a scholarship by a small group of Catholic students from Victoria University of Wellington, New Zealand. As a result, in 1951 the International Refugee Organisation delivered Tom Paulay, his wife Herta and baby daughter to New Zealand. Two years later he completed a Bachelor of Engineering (Civil) degree at Canterbury University College. Before graduating he had brief periods of work experience as a maintenance labourer with New Zealand railways and as a labourer in woolstores. After completing BE (Civil) he worked for eight years as a structural engineer with a firm of consulting engineers in Wellington, where his ability and instinct for structural design became clearly evident.

In 1961 he joined the Department of Civil Engineering of the University of Canterbury as a Lecturer. There his main teaching interest was the application of engineering fundamentals to creative structural design. He proved to be a gifted and popular teacher. Encounters with students in the classroom were a prime source of joy to him. The students responded with enthusiasm, in spite of the high demands placed on them, and profited greatly from the experience - a very fortunate generation of students indeed. At the urging of the then head of department he embarked on research work in 1964 which led to a PhD degree in 1969. Progressing through the steps in the academic ladder, in 1975 he was appointed to a person chair (professorship) in civil engineering at the University of Canterbury.
He has maintained a continued interest and intense involvement in research at the University of Canterbury during the last thirty years. Although his first technical paper was published in 1967 when he was age 44, he has published 100 publications since that date, comprising 3 books, 9 book chapters and parts of seminar volumes, 58 papers in refereed journals and 30 papers in conference proceedings (see the attached list of publications by T Paulay). His publications have had a major impact on the seismic design of concrete structures and have been recognised by numerous awards and prestigious appointments both in New Zealand and overseas.

In 1983 he was elected to Fellowship of the Royal Society of New Zealand and in 1987 to Honorary Membership of the American Concrete Institute, the 23rd non-American so honoured since 1926. His services to civil engineering were marked by the Professional Commitment Award of the Institution of Professional Engineers New Zealand in 1985, and by being made an Officer of the Order of the British Empire in 1986. He has also received honorary doctorates from the Swiss Federal Institute of Technology and the Technical University of Budapest. He retired from the University of Canterbury in 1989 after 28 years of extraordinary service and achievement. Although retired he has maintained strong ties with his colleagues, attending most days to work in his study and to talk with staff, students and visitors at the University of Canterbury. He has also kept a high international profile, becoming the President of the International Association for Earthquake Engineering in 1992.

RESEARCH

Tom Paulay's research during the last thirty years has had a profound effect on current understanding of aspects of the behaviour and seismic design of reinforced concrete structures. His many publications are highly regarded internationally for their deep and significant contributions. Indeed many of his publications have become classics. This research work has built on his uncanny ability to appreciate the mechanisms of behaviour of reinforced concrete which has led to a deep understanding of the behaviour of reinforced concrete from the level of elements of members and connections to complete structural systems. He has had the ability to extend this theoretical understanding of reinforced concrete into logical procedures for design, as demonstrated for example by his contributions to capacity design. His work has been characterised by a concern for practical application of theoretical knowledge. His papers have had a decisive influence on the development of building codes, especially in the areas of the earthquake resistance of reinforced concrete structures, both in New Zealand and internationally. His philosophical approach to design has placed him at the forefront of code developments.
His many significant original contributions to the theory of reinforced concrete and to design for earthquake resistance, made either independently or in collaboration with his postgraduate students and his colleagues at the University of Canterbury, have included the mechanisms of the shear resistance of reinforced concrete beams, the transfer of shear across cracks in reinforced concrete by aggregate interlock, the shear and bond transfer mechanisms in beam-column joints, the behaviour of diagonally reinforced coupling beams of structural walls, and the capacity design and detailing for ductility of reinforced concrete moment resisting frames and structural walls. Some highlights of this research work are summarised in the following.

Mechanisms of Shear Resistance of Reinforced Concrete Beams

Pioneering experimental research and deduction by Paulay and Fenwick, first published in 1967(J1) and 1968(J2), brought a new understanding to the mechanisms of shear resistance of reinforced concrete members. For each of the concrete cantilevers between the diagonal tension cracks of the beam in Fig. 1, for beam action the bond force $T_1-T_2$ between adjacent cracks in Fig. 2 must be resisted by, and be in equilibrium with, the axial force, shear force and moment at the fixed end of the cantilever, the dowel forces at the two cracks and the aggregate interlock at the faces of the two cracks. Their experimental work showed that in beams of normal dimensions and without shear reinforcement not more than 20% of the bond force could be resisted by flexure at the fixed end of the cantilever and not more than 20% by dowel action. Aggregate interlock, arising when the two faces of a crack are given a shear displacement relative to each other, was found to resist about 60% of the bond force. Thus the importance of the shear force resisted by aggregate interlock, ignored as a shear resisting mechanism in members until that time, was identified. It is now commonly accepted that the shear "carried by concrete" in reinforced concrete members comprises shear carried by the compression zone, shear carried by dowel action and shear carried by aggregate interlock, of which aggregate interlock resists the greatest share.

Shear Transfer Across Cracks

The shear which could be transferred across cracks by aggregate interlock was further investigated experimentally by Paulay and Loeber and published in 1974(J11). They determined that the shear displacement required to transfer a given shear stress across two rough interlocking faces in the plane of the shear increases with increase in crack width. Typical shear stress-shear displacement relations were measured by Paulay and Loeber for various crack widths.
Shear and Bond Transfer Mechanisms of Beam-Column Joints

The effects of shear and bond in beam-column joints of moment resisting frames subjected to seismic forces were largely ignored by designers up to the late 1960s. Pioneering research work on the shear strength of beam-column joints after diagonal tension cracking of the joint core due to joint shears, and on the bond performance of longitudinal beam and column bars in joint cores, was conducted by Paulay and Park, first published in 1969(C1), 1973(P4) and 1975(B1). This research work has continued during the last 20 years and has resulted in many further publications. Fig. 3 shows a figure from the 1969 publication (C1), drawn by Paulay, which clearly illustrates the problem of shear (resulting in diagonal tension) and anchorage of bars in exterior beam-column joints. The basic model proposed by Paulay for an interior beam-column joint, published in 1975(B1), is shown in Fig. 4. This model indicates that the forces exerted by the beams and columns at the faces of the joint core are transferred across the joint core by two mechanisms:

(a) A diagonal compression concrete strut [Fig. 4(b)] transferring the concrete compression forces.

(b) A truss mechanism [Fig. 4(d)], consisting of a diagonal compression field of concrete struts and well anchored vertical and horizontal reinforcing bars, transferring the bond forces of the longitudinal beam and column bars.

More recent modifications to the model indicate that some of the bond forces are in fact transferred to the ends of the diagonal compression strut of Fig. 4(b), thus reducing the joint shear required to be transferred by the truss mechanism. Fig. 5 illustrates bond forces near the corners of the joint core being transferred to the strut and the remaining bond forces to the truss.

The design of beam-column joints for shear according to the New Zealand concrete design code* is based on the model shown in Fig. 4. Also, restrictions on the ratio of the longitudinal bar diameter to joint core dimension are imposed to reduce bar slip to an acceptable level. The design of beam-column joint cores is still the subject of international controversy, but during the controversy the Paulay model has remained the main basis of the analytical approach for the calculation of the area of shear reinforcement required in joint cores.

Indeed the model is an early innovative example of the application of strut and tie models to a highly discontinuous or disturbed region of reinforced concrete.

Coupling Beams of Reinforced Concrete Structural Walls

Fascinated by the damage to reinforced concrete coupling beams of the structural walls of the Mount McKinley building in Anchorage, Alaska during the 1964 earthquake (see Fig. 6), Tom Paulay embarked on research into reinforced concrete structural walls which led to his PhD in 1969(T1). His first publication(J4) in the technical literature on walls also appeared in 1969 and has been followed by many very significant contributions since. Paulay’s careful experimental study of the behaviour of deep coupling beams, conducted during his PhD research, indicated that conventional longitudinal and transverse reinforcement was inadequate to prevent rapid strength degradation of those beams during cyclic loading which simulated the effects of severe earthquakes (see Fig. 7). This degradation occurs because when the clear span/depth ratio is less than about 1.5 there is a radical redistribution of stresses in the beam due to diagonal tension cracking which results in a spread of tension along the longitudinal top and bottom bars over the whole length of beam leading to significant degradation of strength. With large quantities of conventional (vertical) shear reinforcement, deep coupling beams with aspect ratio of clear span to depth = 1.29 were observed to fail in sliding shear along a vertical section at the face of the wall after cyclic loading, due to a breakdown of the aggregate interlock mechanism. Subsequent studies by Paulay and Binney(J10) revealed that the ductility and useful strength of deep coupling beams can be considerably improved if, instead of using conventional longitudinal and transverse reinforcement, the principle reinforcement is placed diagonally in the beam. Figs. 8 and 9 compare the behaviour of conventionally and diagonally reinforced coupling beams under cyclic loading simulating severe seismic loading well into the inelastic range. For the conventionally reinforced beam (see Fig. 8) no yielding of the vertical reinforcement across the diagonal tension cracks was observed during the cycles of loading. The beam failed by sliding shear without reaching its theoretical flexural strength after limited ductility. The behaviour of the diagonally reinforced beam (see Fig. 9) was excellent, demonstrating extremely ductile behaviour. The model of behaviour of a coupling beam with diagonal reinforcement shown in Fig. 10, proposed by Paulay, leads to extremely simple design equations. The model assumes that after reversed loading into the yield range and diagonal tension cracking in both directions the diagonal bars yield in both tension and compression. A diagonally reinforced beam will only undergo strength degradation if buckling of compression bars occurs. In design it is important to have ties around the
bars of a diagonal band to retain the concrete, thus ensuring some lateral
rigidity and enabling compression yielding of the diagonal bars to be
maintained. Diagonally reinforced coupling beams have now had wide
application in New Zealand and other countries. For example, Fig. 11
illustrates the use of such reinforcement in the coupling beams of the
structural walls of the New Zealand Parliament buildings in Wellington.

Capacity Design and the Detailing for Ductility of Reinforced Concrete
Buildings

Until the late 1960s it was considered in New Zealand that there
were too many uncertainties concerning the behaviour of tall reinforced
concrete buildings during severe earthquakes to permit their construction.
The 1965 New Zealand code for basic design loads required that "All
elements of the structure which resist seismic forces or movements and the
building as a whole shall be designed with consideration for adequate
ductility". No guidelines were given in the code as to how "adequate
ductility" was to be achieved. The commentary to the code stated that a
safeguard is to limit "the use of reinforced masonry buildings to low
structures of minor importance and by building in reinforced concrete in the
intermediate field and in structural steel of adequate ductility for taller
structures and for those of importance to the community".

Significant research in New Zealand at the universities and
elsewhere, and extensive activities of study groups organised by the New
Zealand National Society for Earthquake Engineering, in the late 1960s and
in the 1970s, resulted in significant strides being made in the development
of the capacity design approach and of design provisions for the detailing for
ductility to be used in the seismic design of reinforced concrete structures.
This activity culminated in the publication of the book on reinforced
cement structures by Park and Paulay in 1975(B1) and in the publication
of a greatly improved New Zealand concrete design code NZS 3101 in 1982.
As a result, the use of reinforced concrete for the structure of buildings of
all heights is now commonplace in New Zealand. The general design
provisions of NZS 3101:1982 were based mainly on the 1977 building code
of the American Concrete Institute, but many of the seismic provisions had
their origins in New Zealand. NZS 3101:1982 has been regarded as a
milestone code by many earthquake-prone countries and many of its seismic
provisions have been adopted in seismic codes in Europe, South East Asia,
North America and South America.

The development of the capacity design procedure specified in NZS
3101:1982 was a significant New Zealand innovation. Capacity design was
introduced because of the realisation that the exact characteristics of the
earthquake ground motions that may occur at a given site cannot be
predicted with certainty and the analytical modelling of some aspects of the behaviour of complete structures is still open to question. Nevertheless it is possible to impart to the structure features that will ensure the most desirable behaviour. In capacity design a mechanism of inelastic deformation is chosen (for example, for moment resisting frames flexural plastic hinges in the beams and columns bases) and the chosen regions of yielding are designed for adequate strength and ductility to resist the design seismic actions. The remainder of the structure is then designed for appropriately amplified actions to ensure that flexural yielding does not occur elsewhere, nor shear failure anywhere, and hence that the chosen mechanism of inelastic deformations will be maintained during the cycles of inelastic deformation imposed by a severe earthquake.

The beginning of capacity design in New Zealand was a logical step by step procedure proposed by Hollings for achieving adequate ductility in reinforced concrete building structures by ensuring that yielding occurred only in chosen ductile regions. The procedure proposed by Hollings foreshadowed a number of later developments. Paulay has been a leading light in those developments of the capacity design procedures and detailing provisions.

**Capacity Design and Detailing of Moment Resisting Frames** - Damage to columns of buildings during severe earthquakes has often been irreparable or led to catastrophic collapse. The aim of the capacity design procedure for columns of moment resisting frames is to provide the columns with sufficient flexural and shear strength to ensure that the inelastic deformations of the frame occur mainly by flexural yielding of the beams, rather than by flexural yielding of the columns. That is, soft stories due to sidesway mechanisms with plastic hinges only in columns of one storey are avoided. Fig. 12 shows the bending moments in the column of a 12 storey building as obtained from the code equivalent static earthquake design forces compared with the column bending moments induced at various instants during a severe earthquake as obtained by dynamic analysis. The differences between the static and dynamic results are caused mainly by the effects of higher modes of vibration. An innovative contribution by Paulay first published in 1977(J21) was to recommend multipliers whereby the design flexural, axial load and shear actions found in columns due to the equivalent static earthquake design forces could be amplified to take into account the beams reaching their flexural overstrength, the higher modes of vibration of frames and concurrent earthquake loading. The latter two

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effects were derived from the results of non-linear dynamic analysis. The total amplification factor approached two or more (see Fig. 13). The procedure has been widely used in New Zealand and has been much discussed overseas. The logical steps of capacity design and the detailing of reinforcement for adequate ductility of moment resisting frames has been the subject of many papers published by Paulay before NZS 3101:1982 was issued, for example (J15, J19, J21, J22, J23, J24, J26, J28).

**Capacity Design and Detailing of Structural Walls** - A common conception that was held by many designers is that if walls fail during a severe earthquake it will be by a brittle shear failure. Paulay has shown remarkable insight into wall behaviour and has made a major contribution by demonstrating that, using capacity design, most walls can be designed so as not to fail in a brittle manner. Indeed it was he who insisted that "shear walls" should be referred to as "structural walls", since most structural walls could be designed to deform in a ductile flexure mode if loaded by a severe earthquake into the inelastic range. Paulay was able to illustrate the possible failure modes of structural walls with great clarity. For example, Fig. 14 shows the possible failure modes of cantilever walls. In 1970(T1) and in later publications he also analysed the behaviour of coupled structural walls subjected to seismic loading. He showed that when the strength of the coupling beams is large (that is, $T > M_1 + M_2$ in Fig. 15(b) and (c), noting that the total overturning moment on the wall is resisted by $M_1 + M_2 + T$) the major means of dissipating seismic energy in a well proportioned wall will be by ductile inelastic behaviour of the coupling beams before the walls become inelastic. He also saw the merits of utilising moment redistribution when determining design actions in coupled structural walls (see Fig. 16). In New Zealand ductile coupled walls are now regarded as providing the best means of seismic resistance available for building structures. The stiffness of the walls gives good protection against damage to the non-structural elements and contents of the building. The ductile coupling beams are not part of the gravity load carrying structure and can be easily repaired in the event of damage from an extreme earthquake event. Paulay's research enabled significant strides to be made in formulating the design rules for structural walls in NZS 3101:1982. These rules aim to achieve adequate strength and ductility by ensuring that lateral instability does not occur, buckling of longitudinal compression reinforcement is prevented, the compressed concrete in potential plastic hinge regions is confined, and shear failure is prevented. The capacity design and detailing rules for structural walls, devised single handed by Paulay, were published in many papers by him before NZS 3101:1982 was published, for example (J14, J16, J29, J30, J31), and indeed represent a most noteworthy achievement.

For many years Paulay has conveyed his innovative ideas on
structural design to students and structural engineers. The recent publication of two books, in 1990(B2) and 1992(B3), brings much of Paulay's writings and teaching together. The most recent book(B3), on the seismic design of reinforced concrete and masonry buildings, co-authored with Priestley, will be widely acclaimed.

CONCLUSIONS

The research and teaching of Tom Paulay have had a major impact on the understanding of the behaviour and the design of reinforced concrete for resistance to earthquakes. His pioneering research into the shear resistance of reinforced concrete, aggregate interlock across cracks, the behaviour of beam-column joints, and the capacity design and detailing procedures of moment resisting frames and structural walls, has had a decisive influence on reinforced concrete theory and the development of building codes. His philosophical approach to design has placed him at the forefront of developments of seismic codes.

ACKNOWLEDGEMENTS

It has been a pleasure to know and to work with Tom Paulay for the last 28 years at the University of Canterbury. New Zealand has been fortunate to have the privilege of providing a working environment for a calvary officer who has become a most eminent university professor.

PUBLICATIONS BY T PAULAY

PhD Thesis


Books


Book Chapters and Parts of Seminar Volumes


Lateral Force Transfer in Buildings


Papers in Reference Journals and Special Publications


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Papers in Conference Proceedings


Fig. 1—Equilibrium condition for the shear span of a beam. (a) Free body; (b) and (c) force equilibrium requirements (J1)

Fig. 2—Action on a concrete cantilever. (a) The forces; (b) force equilibrium requirements (J1)

Fig. 3—Force transfer across an exterior beam-column joint. (a) Diagonal tension and compression induced by stress resultants in beam and columns; (b) diagonal tension cracking and bond forces (C1)
Fig. 4—Idealized behavior of interior beam-column joints (a) Internal actions and crack pattern; (b) shear transfer by diagonal compression strut mechanism; (c) forces in the reinforcement only; and (d) shear transfer by truss mechanism (B1)
Fig. 5—Principal mechanisms of shear transfer in interior beam-column joint (B3)

Fig. 6—Damage to coupling beams of structural walls of Mt. McKinley building, Anchorage, Alaska, 1964
Fig. 7—Three-quarter scale reinforced concrete coupling beam in loading frame after testing illustrating diagonal tension cracking (T1)

Fig. 8—Load-rotation relationship for coupling beam containing shear reinforcement in excess of that required by truss action and the sliding shear failure that occurred at the end of the test (T1, J7, B1)
Fig. 9—Load-rotation relationship for diagonally reinforced coupling beam and diagonal tension cracking at end of test (J10, B1)

Fig. 10—Model of diagonally reinforced coupling beam. (a) Geometry of reinforcement; (b) external actions; and (c) internal forces (B1)
Fig. 11—Diagonal reinforcement for a coupling beam of a structural wall (photo courtesy of New Zealand Ministry of Works)

Fig. 12—Comparison of column moments due to horizontal equivalent static and dynamic forces (P13, B3)

Fig. 13—Moment magnifications for column in lower stories of 15-story building (C6, B3)
Fig. 14—Failure modes of cantilever structural walls (P6, B3)

Fig. 15—Flexural resisting mechanisms of structural walls (J29, B3)

Fig. 16—Ductile response of coupled structural wall (P23, B3)
Empirical versus Rational Approach in Structural Engineering — What We Learned from New Zealand in the Trilateral Cooperative Research on Beam-Column Joints

by H. Aoyama

Synopsis: Summarized in this paper are the background state-of-the-art in reinforced concrete beam-column joint design leading to the U.S.-N.Z.-Japan trilateral cooperative research, outline of the trilateral research and its conclusions affecting the design practice in each country. Particular emphasis is placed on the transition of structural engineering research from empirical approach to rational approach which became apparent in the course of trilateral research and discussions.

Keywords: beam-column joints; bond stress; confined concrete; earthquake-resistant structures; lateral reinforcement; slippage; structural design
Appreciation

I have a vague memory of meeting Tom Paulay in early 1960's, at the University of Tokyo, when he visited the Structural Testing Laboratory housing a 2000 ton universal testing machine. I explained the testing facilities in the laboratory, and tried to answer many questions that Tom made. I remember this because he was standing just beside me and these questions came almost from above my head down. His six-feet strong height was so impressive, though I regard myself not a short man among Japanese.

Tom and I met in several conferences since then, but a definite occasion for me to deepen our friendship was the Pacific Conference on Earthquake Engineering in 1979, held in Wellington, New Zealand. After the conference I flew to Christchurch, to visit Tom Paulay and Bob Park at the campus of the University of Canterbury. I was fascinated with the campus, the activities, and the people. Immediately I decided to come and stay in Christchurch for an extended period in the next year.

Together with my wife and two children, I came back to Christchurch in July 1980. Bob Park and his family was in Bristol, U. K., at that time and he offered his house on the Waimairi Road for our family. Tom and Herta Paulay was our host family, and guided all of us in all sorts of living affairs. Aoyama family stayed in Christchurch until March 1981, and all of us had a wonderful time in each one's direction, owing to the extraordinary kindness of Tom and Herta, and of Bob and Kathleen Park after they came back in October.

Since then, the earth shrunk considerably. We could revisit New Zealand several times. Tom and Herta came to Tokyo to stay for two months. Adam and Dorothy Herterendy-Paulay also stayed in Tokyo. But above all, I feel our stay in Christchurch in 1980-81 was the brightest highlight for the friendship between Paulays and Aoyamas.

There were two bright highlights for the technical exchange between Tom and I, or rather, Tom and Japan. One was the impact of the capacity design concept which I received during the above-mentioned stay at the University of Canterbury. A committee, formed in the Architectural Institute of Japan, formulated a design guideline for earthquake resistant reinforced concrete buildings based on ultimate strength concept, which is essentially the capacity design concept. One can immediately notice deep influence of Tom everywhere in this guideline.

Another highlight for the technical exchange was the Trilateral cooperative research on the beam-column joints, on which I wrote the paper for this symposium. In fact, the whole paper was written with the appreciation of Tom in mind. The subtitle of my paper was "what we learned from New Zealand", but it really means "what we learned from Tom Paulay".

Hiroyuki Aoyama
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INTRODUCTION

An international dispute broke out in 1980's between researchers in the United States and New Zealand on the seismic design of reinforced concrete beam-column joints. As an attempt to enhance further discussions and lead to a common conclusion, a trilateral cooperative research was conducted by researchers in US, NZ, and Japan. Meetings and seminars were organized, two technical sessions were sponsored by ACI 352 at the Fall 1989 ACI Convention in San Diego, and a book was published (1).

Beam column joints in moment resisting frames are subjected to very high shear when the frame is loaded horizontally. The art of evaluating the shear force and of designing the joint accordingly was first established for steel frames in 1950's in Japan. As to reinforced concrete, the same problem was recognized about the same time, and experimental studies were initiated in Japan, but they were not publicly known in other countries, nor were they incorporated into building codes in Japan.

The problems as related to reinforced concrete interior beam-column joints may be summarized as follows.

1) Beam bars passing through the joint must develop bond sufficient to transfer the flexural stress to the joint concrete. Same condition applies to column bars, but the stress condition is generally more severe for beam bars. The limitation on bond development results in limiting the bar size relative to the column dimension.

2) Concrete in the beam-column joint is subjected to very high shear stress. In order to avoid premature shear failure, either the shear stress be limited, or the shear stress be resisted by transverse reinforcement. This leads to the problem of how to determine the amount of transverse reinforcement (joint hoops).

While Japanese were studying these problems mainly from academic interest, US and NZ researchers started their experimental effort in 1970's, and proposed seismic design provisions of
beam-column joints. Unfortunately, the proposals in US and NZ were widely different. In terms of two items above, NZ proposal was much more severe than US.

This aroused much concern to US practitioners, and this was the major motive for the trilateral cooperative research conducted in US, NZ and Japan (with a partial participation of China), in the period of 1984-1989. At the final seminar held in 1989 in Honolulu, Hawaii, and subsequent Fall 1989 ACI Convention, American practice in beam-column joint design was generally confirmed to be safe by test results conducted in all participating countries.

Notwithstanding the apparent "victory" of US side on the international dispute, the writer feels that New Zealanders, particularly Professor Thomas Paulay, proposed something more important during the course of this cooperative research. This paper is an attempt to clarify that his point was a transition of structural engineering research from empirical approach to rational approach.

**RESEARCH AND DESIGN PRACTICE**

In the following three sections, research and design practice for beam-column joints in Japan, New Zealand and the United States prior to the trilateral cooperative research are summarized. A major emphasis is placed to distinguish whether research and design practice were approached empirically or rationally.

**Japanese Research and Design**

Probably Japanese were the first in the world to realize the importance of beam-column joint strength in the seismic resistance of concrete frames. The first reports on reinforced concrete (RC) joint (2) and on composite steel and reinforced concrete (SRC) joint (3) both appeared in 1955. A full scale SRC joint was tested in 1956 (4). RC and SRC joints were subjected to reversal of loading in 1963 (5). Unfortunately these early tests were not known to overseas engineers.

In 1964, Wakabayashi and Suenaga tested many steel beam-column joints where joint panel plates were replaced by filled-in concrete, and formulated the shear strength of concrete in the following form (6) (rewritten into SI unit throughout this paper),

\[
\nu_c' = (1.08 - 0.0242f_c')f_c' \\
\]

where \(\nu_c\) is the ultimate shear strength of joint concrete and \(f_c'\) is the concrete compressive strength in MPa.
In the following year, Endo collected then available test data of RC beam-column joints, and formulated the cracking and ultimate strength of joint concrete (7). The latter assumed the same form as eq. (1), as follows.

\[ v_c' = (0.65 - 0.014f_c')f_c' \]  \hspace{1cm} (2)

In formulating eq. (2), effect of lateral reinforcement in the joint was not considered.

In 1971, Koreishi tested several large scale beam-column joints, and improved eq. (2) by expanding Endo's data bank and considering the effect of joint lateral reinforcement, as follows (8).

\[ v_c = (0.5 - 0.01f_c')f_c' + 0.86\sqrt{p_wf_y} \]  \hspace{1cm} (3)

The second term in the right-hand side was exactly same as the second term of well-known Arakawa equation for ultimate shear strength of RC beams (9), published in 1960. Arakawa assumed that the effect of lateral shear reinforcement is not expressed by the product of shear reinforcement ratio \( p_w \) and yield strength \( f_y \) as implied by the truss analogy, but is proportional to the square root of the product. The first term in eq. (3) was lowered from eq. (2) by the effect of the second term.

In 1975, Kamimura (10) again expanded the data bank and reformulated eq. (3). Thereby he found that the effect of joint lateral reinforcement can better be expressed by halving the product of \( p_w \) and \( f_y \). He also found that the level of axial load on the column has little effect on the ultimate strength of the joint. As to the data reduction method, he adopted the average width of beam and column as the effective width of joint, and eliminated test data which were believed to be governed by the beam flexural yielding. Thus he arrived at what is later known as Kamimura equation.

\[ v_c = (0.78 - 0.016f_c')f_c' + 0.5p_wf_y \]  \hspace{1cm} (4)

Thus all the Japanese research on beam-column joints were aiming at the empirical formulation of test data. This type of research works were quite popular in 1960's in Japan. In fact we might call the decade the age of empirical formulas. The most typical achievement of this age in RC was the abovementioned Arakawa equation for shear strength of beams, the application of which was later expanded to columns and shear walls by M. Hirosawa, and is widely used in practice (11). Another typical achievement was the Sugano equation (12) for yield stiffness reduction factor of beams and columns, which is also widely used in practical design and analysis.

Related to the bond behavior of beam bars passing through the joint, little systematic works have been done in this period. One
possible reason was that steel and SRC joints were studied first, and that RC joints followed them. The only notable result was reported in 1976 (13) in the Proposed Design Guideline for D51 Bars by a committee (Chairman Prof. K. Ogura). The commentary for beam-column joint design states the following: the average bond stress at beam yielding (compressive stress being assumed to be 0.5f_y) for available test specimens including those using D51 bars, can be expressed approximately as

$$\tau_{\text{max}} = 1.3 \sqrt{f_c}$$

Research works in this period have so far not been incorporated into the Architectural Institute of Japan (AIJ) Structural Standard (14) at all. Japanese reinforced concrete buildings designed according to the Standard have received no check on the beam-column joints. The largest reasons for this negligence would be that lowrise buildings for which the Standard is mainly oriented are seldom subjected to immature joint failure due to ample column size, and in fact there are little evidence of joint failure in earthquake damage experience. In case of highrise RC buildings where joints are more likely to be a weak link, joints are evaluated for safety by means of Kamimura equation or others, and the design is reviewed at the Building Center of Japan.

The AIJ Structural Standard for SRC (15) includes a section for beam-column joint since 1975. It can be adapted to RC just by eliminating steel sections. Thus it can be and actually is used for the design of highrise RC frames.

New Zealand Research and Design

Paulay, Park and others in New Zealand started experiments on beam-column joints in 1974, and discussed the practice in seismic design (16). Since their points are well known, it is not necessary to quote their discussions in detail. Only the essential portion of their discussions will be introduced here.

When beams around an interior beam-column joint yield due to horizontal loading, forces around the joint are as shown in Fig.1. Axial load on the column is omitted for simplicity. The coefficient $\alpha$ for yield stress $f_y$ is to reflect the effects of both the higher yield stress over the specified value and the strain hardening, and is usually taken to be 1.25 for Grade 275 steel and 1.40 for Grade 380 steel. With the notation shown in Fig.1, the horizontal shear force in the joint $V_{jh}$ is expressed by

$$V_{jh} = (A_{s1} + A_{s2}) \alpha f_y - V_c$$

To carry this shear, Paulay et al. postulated two kinds of resisting mechanism, as shown in Fig.2. One is the concrete strut mechanism, in which forces shown in Fig.2(a) are in equilibrium,
and the horizontal component \( V_{ch} \) of the strut force \( D_c \) can be expressed as

\[
V_{ch} = D_c \cos \beta = C_c + \Delta T_c - V_c' \tag{7}
\]

where \( \Delta T_c \) is the force in the concrete strut transferred from the beam bars by bond.

Another mechanism is the truss mechanism, shown in Fig. 2 (b), by which the beam bar force \( T' + C_s \) or \( T + C_s \) minus the abovementioned \( \Delta T_c \), or \( \Delta T_c' \), must be transferred by bond to the joint concrete, and then resisted by truss action as shown. The horizontal component \( V_{sh} \) of the strut force \( D_s \) can be expressed as

\[
V_{sh} = D_s \cos \beta = T' + C_s - \Delta T_c \tag{8}
\]

Needless to say, the total of eqs. (7) and (8) is equal to \( V_{jh} \) in eq. (6).

\[
V_{jh} = V_{ch} + V_{sh} = T' + C_c + C_s - V_c' \tag{9}
\]

Thus it should be pointed out that the proportion of forces carried by two mechanisms depends on the beam concrete compression \( C_c \) and bond force \( \Delta T_c' \). The larger these forces are, the more horizontal joint shear is carried by the concrete strut mechanism.

Under the action of repeated reversal of horizontal load, it can be shown both \( C_c \) and \( \Delta T_c \) decrease. As long as we consider beams and columns as isolated flexural members, cracks due to tension bar yielding do not close until bars are yielded in compression. Thus \( C_s \) becomes larger, and \( C_c \) smaller, than those under monotonic loading. If beams are provided with equal top and bottom reinforcement, \( C_s \) equals \( T \), and \( C_c \) becomes zero.

The stress distribution in beam bars in the joint may be as shown in Fig. 3 (a) at the first yielding with the average bond stress \( u_1 \). But it will be as shown in Fig. 3 (b) after one or two reversals, where the average bond stress \( u_2 \) becomes larger than \( u_1 \). When the yield penetration occurs as shown in Fig. 3 (c), the bond stress \( u_3 \) will become even larger. The bond force in the concrete strut \( \Delta T_c \) must be that portion of the bond within the neutral axis of column section, denoted by \( c \) in Fig. 3, which becomes smaller as bond stress distribution changes from (a) to (c).

With the decrease in forces \( C_c \) and \( \Delta T_c' \), it is clear that \( V_{ch} \) in eq. (7) becomes smaller, and eventually it will become zero. Thus the strut mechanism will disappear and all the joint horizontal shear must be carried by the truss mechanism. The reasoning by Paulay et al. was entirely rational, and their concept was fully incorporated in the New Zealand Code of Practice (17), which was proposed in 1978 as a draft and was adopted officially in 1982.
American Research and Design Approach

In the United States, beam-column joint tests were first performed in 1967 (18). ACI-ASCE Committee 352 was formed subsequently, and the first and revised reports were published in 1976 (19) and 1985 (20).

The first report in 1976 recommended essentially the following two items for beam-column joints, both of which had been originally stipulated for columns.

(1) When the column axial load is greater than 40 percent of balanced load, transverse reinforcement of same amount as confinement (special transverse reinforcement) shall be provided in the joint.

(2) The horizontal shear stress in the joint may be resisted by concrete and steel. The permissible concrete shear stress is essentially equal to that of columns with very small shear span ratio. When the column is subjected to tension, the permissible concrete shear stress must be zero. Transverse steel for confinement may be counted for the shear reinforcement.

The concept of confinement in the first item has a long tradition in the American practice of column design, being adapted to ACI Building Code in 1936. The second item was the direct application of shear provisions of ACI Building Code since 1963, which had been developed based on the ACI Committee 326 report (21). In this report, the famous Hogsted equation for ultimate shear strength was introduced. It is interesting to note that the decade of 1960's in U.S. was also the age of empirical formulas.

Thus, the first Committee 352 report in 1976 intended to adapt two traditional design concepts to beam-column joints. It said nothing about the bond behavior of beam bars passing through the joint.

The abovementioned New Zealand paper (16) was actually written as a discussion to 1976 report. Namely, without securing bond resistance to beam bars, the frame is susceptible to excessive lateral deformation due to additional pull-out deformation at beam yield hinges. Once the bond resistance is secured, then, the consideration of shear resisting mechanism in the joint, particularly under reversal of loading, would lead to the conclusion that concrete strut mechanism disappears, and truss mechanism must carry all the joint shear, requiring much larger amount of transverse steel than recommended in the report.

Receiving this challenge, however, the Committee 352 proceeded to a direction completely opposite to New Zealanders. The revised report in 1985, and the ACI Building Code Appendix A in 1983 which took the revised report in advance, stipulated the
following three items for beam-column joints.

(1) Unless the joint is confined by beams framing into the joint from four sides, transverse reinforcement of same amount as confinement shall be provided in the joint. For the confined joint which is a joint confined on all sides by beams, amount of transverse reinforcement shall not be less than half that required above.

(2) The shear stress in the joint shall be resisted by concrete only. The shear strength for confined and unconfined joints shall be $1.7\sqrt{f_c'}$ and $1.3\sqrt{f_c'}$, respectively, where $f_c'$ is concrete compressive strength in MPa.

(3) The diameter of beam bars passing through a joint shall be not greater than one twentieth of the joint length. This was recommended in the Committee report only and not in Building Code Appendix A.

Among those three points, the second item was developed based on the review by J. O. Jirsa and others of beam-column joint test data in US, NZ and Japan. They found that the joint shear strength is not affected by the amount of transverse reinforcement, provided that a reasonable amount of transverse steel was provided and for the range of lateral drift up to two percent. This empirical finding was essentially in accordance with that of Koreishi and Kamimura, who found that transverse reinforcement was not fully effective as stipulated by truss analogy. The third item above was also based on an empirical finding.

TRILATERAL COOPERATIVE RESEARCH

Outline of Research and Seminars

The significant difference in the ACI Building Code and New Zealand Code of Practice in the beam-column joint provisions aroused much concern, particularly to U.S. practitioners. This led to the U.S. - New Zealand - Japan trilateral cooperative research in 1984-1989. The first seminar was held in July 1984 in Monterey, California, and a framework of cooperative research was discussed and agreed upon. Jirsa, Park, and Aoyama were named as the principal investigator in each country.

A series of coordinated tests were carried out in three countries (later China joined to form a quadrilateral cooperative research). The tests consisted of full scale column-beam-slab subassemblages subjected to bidirectional reversal of horizontal loading. Each country tested three specimens, namely one interior joint of one-way frame (1D-I), one interior joint of two-way frame (2D-I), one exterior joint of two-way frame (2D-E). Design of
these specimens were to be carried out in accordance with the Building Code and prevalent practice in each country including the detail of joint hoops. They were subjected to bidirectional reversal except for 1D-I, and the loading sequence followed the common rule set up in the first seminar.

In the implementation of test program some modifications were introduced, particularly to Japanese experiments. Due to limitation of loading equipment, specimens were made to half scale. The 1D-I specimen was omitted, and two 2D-I specimens were fabricated with the different amount of beam bars.

Following the abovementioned first seminar, three more seminars were organized. The second and the third seminars were held in May 1985 in Tokyo, Japan, and in August 1987 in Christchurch, New Zealand, respectively. In the fourth seminar held in May 1989 in Honolulu, Hawaii, about thirty papers were presented including reports on test results and discussions on design practices. Based on this achievement a special session was organized at the ACI Fall Meeting in November 1989 in San Diego, California, and a proceedings volume was published from ACI (1).

Comparison of Design in Three Countries

In the early phase of the trilateral cooperative research, some comparative studies were undertaken to demonstrate the difference of design in three countries (22). Fig. 4 shows the amount of transverse reinforcement required for the beam-column joint shown at the upper portion of the figure with respect to shear stress in the joint. NZ Code of Practice of 1982 and US Building Code of 1983 were used. For Japan, SRC Standard and Kamimura equations were used. As the joint shear increases due to increase of beam flexural strength, amount of joint hoops in NZ increases proportionally, because all the joint shear force must be resisted by joint hoops. According to Japanese design formula in SRC Standard, shear is resisted by concrete and steel, hence joint hoops are not required up to certain level of joint shear stress. Kamimura equation gives lower evaluation to the joint hoop effectiveness, hence the required amount increases more rapidly. On the other hand, amount of joint hoops in US does not vary according to the joint shear stress level, because the joint shear is to be resisted by concrete only, and joint hoops are required as confinement regardless of the joint shear stress level.

Figs. 5 and 6 show typical design examples of RC frame in US and Japan, respectively. For Japanese design example, an RC highrise building with thirty stories was referred to. In each figure, beams shown in (b) and (c) frame into a column shown in (d), and the resulting design of beam-column joint is shown in (e).
The beam-column joints in these design examples were redesigned by the design methods of three countries, and the resulting bar arrangement is summarized in Fig. 7. Largest amount of joint hoops is required by NZ Code, followed by US Code, and the least amount by Japanese Code.

Outline of Experiments

A summary of tests performed in three countries made by Kurose (23) is reproduced in Figs. 8 and 9. Detail of each test may be found in the literature (1).

In Fig. 8, measured maximum joint shear stress was plotted against concrete strength. In the figure, J1, J2 and J3 are US specimens, and K1, K2 and K3 are Japanese specimens. NZ specimens use common notation for specimens as mentioned before (i.e. 1D-I, 2D-I and 2D-E). All specimens in Japan and NZ failed by beam yielding, and the beam-column joint did not fail until the last stage. This mode of failure is denoted as B. Specimens in US first failed by beam yielding, but the joint failure occurred subsequently at larger deformations. This mode of failure is denoted as BJ. Two curves in Fig. 8 correspond to permissible concrete shear stress of confined and unconfined joint, respectively, as specified in 1983 ACI Code. J2 is a confined joint and corresponds to the upper full line. J1 and J3 are unconfined joints and correspond to the lower broken line. All of them showed higher strength than the permissible stress.

In Fig. 9 the ordinate shows the ratio of measured maximum joint shear stress to the calculated beam flexural strength, and the abscissa shows the ratio of abovementioned permissible shear stress by ACI to the calculated beam flexural strength. Four specimens (J1, J2, J3, and K1) fell above the 45 degrees line, indicating that the measured maximum shear stress exceeded the permissible stress. However the maximum stress was dictated by the beam yielding, as the value on abscissa was closed to or greater than 1. The value on the ordinate was greater than 1 for all specimens. The difference in mode of failure is also clear. Three specimens failing in BJ mode had considerably smaller value on the abscissa.

Outline of Discussions

In the four seminars held during the course of the trilateral cooperative research, discussions covered wide range of topics including effect of floor slabs and behavior of exterior joints. In the following, however, discussions pertaining to the behavior and design of interior joints, to which most of the discussion time was devoted, will be summarized.
US side had the simplest and clearest insistence. According to them, all specimens were designed in conformance to ACI Code, and they showed satisfactory behavior, even under severe loading condition of bidirectional reversal. This indicates the soundness of American provisions.

Japan side generally followed US, having no intention to persist using existing empirical equations such as Kamimura equation. Experimental results were taken to indicate the correctness of the American conclusion. At the Architectural Institute of Japan, Design Guidelines Based on Ultimate Strength Concept (24) were being compiled in the period of trilateral cooperative research. A design method close to US practice was proposed and adopted in the Guidelines. That is, joint shear force is to be resisted by concrete only, and the amount of joint hoops is to be specified separately regardless of joint shear stress level.

Another point that Japan raised was a quantitative evaluation of beam bar bond characteristics (25). Energy absorption capacity, shown by hysteresis loop area, is in general affected by beam bar bond; larger the bar diameter, poorer the bond, and hence smaller the energy absorption. However, first, bar diameter column size ratio vs. equivalent viscous damping ratio (a measure of energy absorption) at a particular drift angle was formulated empirically. Then building models ranging from four to sixteen stories with various hysteretic energy absorbing capacity were subjected to earthquake response analysis. Comparing them in terms of viscous damping ratio, it was concluded that excessive earthquake response would not occur even the beam bar bond has deteriorated, provided that the bar size remains within certain limit. This work was a counterargument to NZ point that beam bar bond deterioration will always result in loss of stiffness and excessive earthquake response displacement.

New Zealanders found themselves in the most difficult standpoint. They confirmed that their specimens, which conformed to their code, showed satisfactory behavior under severe loading condition of bidirectional reversal, set forth as a common rule to the trilateral cooperative research. However, at the same time, they had to admit that the specimens of other countries which do not necessarily conform to NZ Code also showed almost satisfactory behavior. Parallel with the trilateral testing, Park and Dai (26) tested specimens not conforming to NZ Code, and initiated work towards the liberalization of the Code.

So far it can be superficially viewed that US and Japan defeated NZ in the dispute. However it appears to the writer that the situation is not that simple, as we see the recent development in NZ which will be described in the following part.
Cheung Dissertation

In New Zealand, experimental works for the trilateral research were conducted by P. C. Cheung, a graduate student of the University of Canterbury, under the direction of Professor T. Paulay and Professor R. Park. His works were compiled into the doctoral dissertation in 1991 (27).

A unique elaboration in Cheung's dissertation was the reexamination of beam bar bond behavior including previous testing and Dai's testing (26). As a result, he found that stress distribution shown in Fig. 3 (b) or (c) where the compressive peak stress reaches strain hardening range afy did not actually happen even under large reversal, and beam concrete in compression zone always carried some compressive stress. As the reversal amplitude became larger, bond stress along beam bars in the joint became more enhanced. However, as long as the drift angle remained within certain limit (say two percent), the compression transmission through concrete was always quite effective.

This is a consequence of soft bond behavior: that is, bond stress-slip relation has a certain level of flexibility, and the bond stress reaches its maximum when some amount of slip has taken place. Fig. 10 shows estimated stress distribution in a joint after large reversal of loading. The stress distribution of top bar in Fig. 10 (a) will be as shown in Fig. 10 (b), where the stress at right end is in tension and reaches the strain hardening stress afy, but the stress at left end is in compression and not greater than yield point, because the crack in the previous cycle has already closed due to slipping of the bar. The associated bond stress distribution is shown in Fig. 10 (c). It is possible that cover concrete of the joint portion of column is not effective in carrying bond stress. If this is considered, steel stress will distribute as in Fig. 10 (d), with the resulting bond stress distribution as shown by the full line in Fig. 10 (e), which will be further idealized as shown by the broken line in Fig. 10(e).

The bond force transmitted to the strut mechanism $\Delta T_C$ may be determined by the bond within the column compression zone $0.8c$. The bond stress, $\sigma_0'$ in Fig. 10 (e) may be obtained by dividing the total steel force $(T_1 + C_{S2})$ by $0.6h_c$. Thus $\Delta T_C$ will be found by

$$\Delta T_C = 0.8c (T_1 + C_{S2}) / 0.6h_c \tag{10}$$

where $c$, the neutral axis of the column, can be approximated by considering column axial load $P_e$. 


Using equation (7) with \( \Delta T_e \) above, the joint shear force of strut mechanism can be determined. With this amount of shear being carried by joint core concrete, the shear in truss mechanism becomes smaller, and the amount of joint hoops is reduced.

Proposal for Code Revision

Cheung, Pau and Park proposed a Code revision in beam-column joint provisions as follows (28).

(1) The ratio of beam bar diameter passing through joint \( d_b \) and column depth \( h_c \) is limited as follows.

\[
\frac{d_b}{h_c} \leq 1.9a_1a_2a_3a_4 \frac{\sqrt{f_c}}{f_y}
\]

(12)

where

- \( a_1 \): a coefficient corresponding to level of compressive stress \( (a_1 = 1.1-1.3) \). The compressive stress is at most equal to \( f_y \).
- \( a_2 \): a coefficient corresponding to level of column axial load \( (a_2 = 1.0-1.25) \).
- \( a_3 \): a coefficient to consider difference between one-way frame and two-way frame \( (a_3 = 1.0-1.2) \).
- \( a_4 \): a coefficient to consider difference of upper and lower beam bar bond \( (a_4 = 1.0-1.1) \).

(2) The ratio of joint shear carried by concrete \( V_{ch} \) to the total shear \( V_{jh} \) is determined as follows.

\[
\frac{V_{ch}}{V_{jh}} = 0.3(1 + 3.5 \frac{P_e}{f_yA_g})
\]

(13)

With the introduction of item (1), considerably larger size beam bars can be used than the current NZ Code. Compared to US provisions where \( d_b/h_c \) is uniquely less than 1/20, the limitation is a function of numerous variables. There may be further discussions before it is finally adopted in the NZ Code.

In the item (2), the effect of concrete is not fully incorporated as in US, but is not fully ignored as in the current NZ Code. With this revision, the joint hoop can be reduced by 30 percent even when the column axial load is negligible.

To the writer, however, the process of concept development was more interesting than the proposed revision itself. 

In deriv-
ing the proposed revision, New Zealanders persisted in the long cherished idea of strut and truss models shown in Fig. 2. By modifying steel stress in Fig. 3 as shown in Fig. 10 based on the test observation, they derived at their proposed revision. The macromodel shown in Fig. 2 was unchanged. They insisted throughout that the resisting mechanism in the beam-column joint shall be as shown in Fig. 2.

CONCLUSION -- EMPIRICAL VS. RATIONAL APPROACH

New Zealanders changed their direction to come closer to US and Japanese practice, not by following others blindly but re-claiming the new way by their own effort. The most remarkable in their effort was that they persisted in their macromodel.

As was stated in the first part of this paper, the research works in 1960's and 1970’s were mostly devoted to development of empirical formulas. In contrast, rational approach was proposed more frequently in 1980’s (29, 30, 31, 32). It is felt that the age of empirical formula already has come to an end, and the new age of rational formula, or the age of macromodel has started. In Japan, a new shear design equation was developed on the rational basis and incorporated into the new Guidelines (24), instead of empirical equations such as Arakawa equation.

It is reminded with regret that the discussions during the trilateral cooperative research on beam-column joint did not necessarily concentrate on the macromodels. US and Japanese researchers reported on their tests, but they did not try to explain the test results by NZ model, nor did they propose new model. US and Japanese researchers believe joint concrete can resist imposed joint shear sufficiently, but how the imposed force is transmitted to the joint concrete was not explained rationally. In this connection, the writer feels that the trilateral cooperative research is not yet concluded.

In any case it is now quite clear that future structural engineering researchers should bear in mind the importance of rational approach, and more specifically, the use of macromodels in deriving new ideas and new equations. Empirical approach is sometimes easier, when sufficient amount of test data is available. Furthermore, empirical equations are often more useful in practical applications. In fact the significance in practice of Arakawa and Hirosawa equations for shear strength, and Sugano equation for yield stiffness reduction, cannot be overemphasized even with the finest words. Nevertheless, the author would like to confirm the trend to rational approach in the future research works. This is the most significant lessen given to us by New Zealanders, particularly by Professor T. Paulay.
REFERENCES


Lateral Force Transfer in Buildings 47


$T = C_c' + C_s' \quad \{ \begin{align*}
V_c' & \\
V_b' & \quad T = A_s \alpha y
\end{align*} \} \quad T = A_s \alpha y$

$T = C_c + C_s \quad \{ \begin{align*}
V_c & \\
V_b & \quad T = C_s + C_c
\end{align*} \}$

Fig. 1—Forces around joint and shear in the joint
a) Concrete strut mechanism

b) Truss mechanism

Fig. 2—Shear resisting mechanisms of joint
Lateral Force Transfer in Buildings

Fig. 3—Probable steel and bond stress distributions in joint core

- a) At first yielding
- b) After reversed inelastic loading
- c) After reversal cycles of reversed inelastic loading
Compressive strength of concrete
\[ f_{c'} = 28 \text{ MPa (4 ksi)} \]
Yield strength of joint hoops
\[ f_{yh} = 420 \text{ MPa (60 ksi)} \]

Fig. 4—Comparison of required amount of joint hoops in three countries
Column Normal Beam

Spandrel Beam

Compressive strength of concrete:
\[ f'_c = 4 \text{ ksi (28 Mpa)} \]

Yield strength of reinforcement:
\[ f_y = 60 \text{ ksi (420 Mpa)} \]

Stress multiplier for longitudinal bars:
\[ \alpha = 1.25 \]

Fig. 5—Design example 1 (US practice)
Concrete strength:
\[ f'_{c} = 42 \text{ MPa} \quad (6 \text{ ksi}) \]
Beam and column bars:
\[ f_{y} = 400 \text{ MPa} \quad (57 \text{ ksi}) \]
Slab bars and joint hoops:
\[ f_{y} = 300 \text{ MPa} \quad (43 \text{ ksi}) \]
Stress multiplier for longitudinal bars: \( \alpha = 1.1 \)

Fig. 6—Design example 2 (Japanese practice)
<table>
<thead>
<tr>
<th>Example 1</th>
<th>Example 2</th>
</tr>
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<tr>
<td>$f_c' = 4$ ksi</td>
<td>$f_c' = 6$ ksi</td>
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<tr>
<td>$f_{yh} = 60$ ksi</td>
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<th>Design Examples</th>
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<th>Japanese Design*²</th>
<th>New Zealand Design*²</th>
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<td>#3-@6in. (D10-@15cm)</td>
<td>#5(D16)-7sets ($s_b&lt;10.5cm$)</td>
</tr>
<tr>
<td>(prototype design)</td>
<td>#<a href="mailto:5-@3.5in">5-@3.5in</a>. (D16-@9cm)</td>
<td>2-#5-@4in. (2-D16-@10cm)</td>
<td>#5(D16)-10sets ($s_b&lt;7.5cm$)</td>
</tr>
</tbody>
</table>

*¹ Transverse reinforcement to confine joint concrete
*² Transverse reinforcement to resist joint shear

Fig. 7—Comparison of joint hoops according to design methods in three countries
Fig. 8—Relationship between measured maximum joint shear stress and concrete strength under unidirectional loading.

Fig. 9—Comparison of maximum measured strength/computed beam yield strength and computed joint shear capacity/computed beam yield strength under unidirectional loading.
Fig. 10—Assumed actions in an inelastic interior joint with imperfect anchorage of beam bars (27)
Synopsis: Problems associated with design of beam-column joints for shear has been studied extensively in many countries. Work in New Zealand on the performance of joints in reinforced concrete moment resisting frames in seismic zones served to alert designers all over the world to consider these problems. Fundamental studies conducted by Paulay and his colleagues and students contributed immeasurably to our understanding of the behavior of joints. However, the approaches used in design codes have not always been the same as those used in New Zealand. The reasons for these differences have much to do with design philosophies, research objectives, and code development procedures. Shear problems at locations other than joints and in elements where rehabilitation (repair and strengthening) is needed to improve performance of structures under earthquake generated deformations still lack definition sufficient for developing code provisions.

Keywords: beam-column joints; joints (junctions); shear properties; standards
Appreciation

It is hard to remember the time I first met Tom Paulay. Certainly I knew him from his writings long before we met in person. Shortly after I joined the University of Texas, Tom came to visit and present a paper. One day we were at a salad bar and Tom noticed a bowl of peppers. I warned him that jalapeno peppers were quite hot but he assured me that no peppers were too hot for a Hungarian! After trying a large bite of one, followed by glasses of water and beer, he allowed that they were indeed hot enough for a Hungarian.

Since then we have participated in many mild to peppery technical discussions and have enjoyed many delightful social occasions. Perhaps none as enjoyable as the evenings in Tom and Herta's home to be regaled by Tom with his unflagging hospitality and to partake of Herta's marvelous cuisine. Tom has always maintained the discipline and training learned as a young Hungarian cavalryman. He has also kept up what I suppose is an art that probably started in Central Europe—the gallant gesture of greeting ladies with a kiss of the hand. That coupled with a steady stream of stories, an infectious laugh, and a mischievous sparkle in his eyes, makes him a hit with our wives but causes us a great deal of misery. It is truly annoying to be asked "Why don't you ever kiss my hand like Tom does?"

Our professional paths (and swords) have crossed in the matter of the design and behavior of beam-column joints. We have not always agreed on the interpretation of test results and on the application of those results to design and practice. In fact, colleagues have asked if we ever agreed on the issues. I think we always agreed on the issues but not necessarily on the solutions. Our colleagues did not know Tom or me very well if they thought something as mundane as beam-column joints could interfere with our friendship. It never occurred to me that we had anything but frank exchanges on technical issues dear to our hearts but that was simply part of our relationship as professional colleagues. Tom has always listened to other opinions, accepted some and rejected others, but has never lost his sense of humor or his genuine enjoyment and delight in good friends and good times with them.

I am honored to share some of my thoughts on a subject about which both of us have expressed strong and sometimes divergent opinions. But I know that our discussions have always been carried out in the same spirit as has all of Professor Paulay's work; to understand how concrete structures behave and to translate that knowledge to those who can best use it. We are fortunate to have been his students and we are richer for the experience of knowing Tom Paulay.
INTRODUCTION

Shear problems continue to vex designers and researchers. One major source of difficulty is the lack of a "theory" that will explain the many different situations where shear is involved. In the case of frame structures containing beam-column joints, shear was not even considered in the design of the joints until research reported in the 1960's and 70's demonstrated the importance of the joint to the performance of the frame—especially in structures subjected to earthquake forces. The early work done at Canterbury University in New Zealand by Park and Paulay and their students influenced practice worldwide. In 1976, a joint committee of the American Concrete Institute and the American Society of Civil Engineers (ACI 352) published design guidelines (1) for beam-column joints. That document was the first to indicate the need for detailing the joint as well as detailing the members to meet shear and anchorage requirements. Similar documents have been developed in other countries.

In 1983, the Applied Technology Council published a report (2) on "Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers." Several comments from that report are important to note—"The methods and approaches recommended in various publications by various groups for design of concrete frame joints differ, in some cases significantly. In the United States the current design procedure for concrete frame joints subject to cyclic loading follows the 1976 ACI-ASCE Committee 352 report, which is based on the beam shear model. The 1982 NZS 3101 of the Standards Association of New Zealand, on the other hand, utilizes compression strut and joint truss mechanisms to determine the joint shear capacity." The ATC report discussed the disagreements between NZ and US researchers and described possible reasons for those differences. In 1981, Jirsa (3) published a paper in which the differences in design approaches were attributed to the variety of loading histories applied to test specimens, to geometry of the specimen, and to interpretation of the test results. Both Refs. 1 and 2 suggested the need to test specimens which better reflected real structures (beams in two directions with floor slabs) and two-directional loadings. Some of these concerns were addressed as part of a US/Japan study on large-scale structures (4).
The 1983 ATC report (2) concluded that "A consensus needs to be reached on the most appropriate mechanism to represent the behavior of reinforced concrete frame joints under anticipated deformation reversals." Following the 1984 World Conference on Earthquake Engineering in San Francisco, a group of designers and engineers met to discuss the development of a collaborative research project to address the issue of "consensus" regarding joint behavior and design requirements. Eventually, the project involved engineers from New Zealand, Japan, China, and the United States. The results of that effort were presented at technical sessions at the Fall 1989 ACI Convention in San Diego and reported in ACI SP-123 (5). The cooperation among the researchers involved in this project was a unique experience and could serve as a model for other research teams. While the project did not resolve all the differences in design guidelines, many aspects of joint detailing were clarified and, at the very least, the differences were reduced.

The objective of this paper is to explore in more detail how reasons other than differences between researchers in interpreting test results lead to design approaches in building codes that are not all the same. The elements which drive code development may dictate the form in which design standards are finally published.

BUILDING CODES

Objectives

Coverage Building codes should be governed by one over-riding principle: The code must set the minimum level for design consistent with protecting the public. Rules are given for providing life safety (simply defined as sufficient strength to protect the occupants), durability, and serviceability. There are a variety of ways that may be done. The ACI Building Code (6), and most other codes for that matter, is a mixture of performance requirements and specific rules. The intent, if not always the result, is a code which is

1. Clear, concise, and easy to use
2. Precise
3. Complete

and which results in buildings that are economical and constructible. The difficulty lies in the conflicting nature of these objectives. The ideal code is much like beauty—it depends on the " beholder". Many engineers would prefer a performance code because it allows them to show that their design meets certain general requirements without the need to meet a battery of specific rules. Others designing simple structures would prefer specific rules without
the need for extensive and possibly expensive engineering calculations. It is virtually impossible to be precise and/or complete and concise.

**Format** To meet these conflicting objectives, tiered codes have been proposed. A first tier might consist of a general performance requirements involving equilibrium, compatibility, and serviceability. A second tier might contain some specific rules which include a number of parameters known to be of importance to that aspect of design. A third tier might be a simple, conservative rule to cover a myriad of different situations with a straightforward, easily-applied rule. A designer could then choose the level appropriate to his design situation. Corley (7) has outlined the nature of a multi-tiered code format.

**Reasons for change** Changes in codes are made because our knowledge has changed. Knowledge impacting building codes comes from either research or experience. Strong arguments for making changes to an existing code include:

1. The code is in error, generally not sufficiently restrictive.
2. Understanding of the phenomena involved improves.
3. New processes or technologies gain acceptance and become routine.

There are a few notable cases where code changes were made rapidly in response to demonstrated inadequacies in the code. One such case is the shear failures which occurred in Ohio in 1955 in some rigid frames (8). The changes were written to apply to the type of construction that collapsed and involved the minimum shear reinforcement necessary to prevent such failures in a zone where low shear stresses would be computed.

The most common reason for making a change involves the improvement of our understanding of structural behavior through research and field experience. The growth of the code can be traced directly to the volume of research conducted to improve the basic understanding of all aspects of reinforced concrete response.

As our use of different materials increases and our structures become more complex, the demand on the code to cover all cases increases. There is pressure on code writers to include these developments to "legitimize" the process or technique. While inclusion of all such material will make the code "complete" it will likely lead to an unworkable, cumbersome, and perhaps confusing document. It is sometimes felt that unless the code addresses all issues, it will be difficult to apply innovative design tools and implement new construction practices and techniques. That interpretation would lead to the conclusion that codes inhibit rather than encourage the designer to use new knowledge. There are many avenues in the existing literature and technical society structure for promoting new ideas. In addition there must be a
mechanism for allowing an engineer to incorporate new concepts in his design even if they are not addressed specifically in the governing building code. This is most often done through the use of provisions such as that contained in Sec. 1.4 of the ACI Code (6).

"Sponsors of any system of design or construction within the scope of this code, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by this code, shall have the right to present the data on which their design is based to the Building Official or to a board of examiners appointed by the Building Official. This board shall be composed of competent engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of the code. These rules when approved by the Building Official and promulgated shall be of the same force and effect as the provisions of this code."

The acceptance of designs using this approach depend on a building official or his representatives being sufficiently confident of their ability to evaluate a proposed design and approve it. Many building departments do not have the manpower or budget to go beyond strict interpretation of the codes. Prior to 1990 when ultimate strength based guidelines for earthquake resistant reinforced concrete buildings were introduced by the Architectural Institute of Japan, high-rise building designs in Japan were approved in a manner similar to that described above (6) because there was no guidance in many issues related to moment-resisting frames, particularly detailing of joints.

**Code Writing Process**

Codes are produced in different ways. To the user, the process may seem mysterious and capricious. However, it is taken very seriously by those directly involved and the "protection of the public" is of paramount concern. The ACI Code is produced through a consensus process which involves a group of engineers (35 to 40) selected to represent various segments of the "building" industry. About a third are consultants, a third represent producers (various material suppliers and trade organizations) and about a fourth are researchers and academicians. The remainder represent government officials and contractors. A large group is likely to be more representative of the industry. It is also more likely to produce a document that will be acceptable to the largest number of users but the process of arriving at consensus will certainly be more cumbersome. The group works on a voluntary basis and meets at least twice a year for two-day meetings. To insure that all views are considered, the consensus process requires that all proposals be considered by letter ballot and that negative votes be formally resolved. Such resolution may
involve making changes so that the voter withdraws his negative without incurring the negative vote of another member or it may be done by a vote of the committee to find the negative voter's argument "non-persuasive." Other groups may have different variations of the consensus process but all must insure that the opinions of members are heard and respected.

The code that is produced under these rules reflects a good deal of compromise and experience. Proposals for changes may come from a variety of sources. Each may have justifiable reasons for the change proposed but the interests of all segments of the building industry including the public may result in substantial changes being made by the code committee and the proposal may be rejected. While this may be frustrating to the proposers, it is the result of the process and, as the record indicates, has produced sound documents. Siess (8) has written an excellent overview of the process by which codes are developed. Figure 1 is extracted from his paper and reflects clearly the complex interaction between research, practice, and building codes. In Fig. 1, the thickness of the lines between sources of knowledge may vary depending on the information available.

Implementation

Unlike many countries, there are no national codes in the United States. The codes that are developed are "model" codes and do not have any legal standing until they are adopted by a local jurisdiction or agency. The model codes, such as ACI 318, are written to be applicable to any locale. In most cases the local jurisdiction adopts the model code with little or no change, but changes can and have been made before adoption. As a result, code writing groups try to produce documents that will be acceptable in all parts of the country and will provide guidance to experienced designers in large offices as well as engineers dealing only occasionally with structural design.

DEVELOPMENT OF BEAM-COLUMN JOINT PROVISIONS

When the first provisions for beam-column joint design were proposed, the impetus for developing such provisions came largely from the research community. Tests on joints demonstrated the vulnerability of joints to shear and anchorage failures. However, contractors expressed concern that the requirements for transverse reinforcement in a joint would make it impossible to construct. There was the additional concern that the requirements for joints should be adjusted for the level of seismicity at the site of the structure. Finally, many engineers reacted negatively because it was difficult to point to joints as a cause of failure in structures damaged in earthquakes. All of the
issues cited above impacted the decisions regarding beam-column joint code provisions.

1976 ACI-ASCE Report

The main aspects of the 1976 design provisions were:

1. Two types of joints—one for seismic areas and one for ordinary ultimate strength design.
2. A requirement that the joint be designed for steel stresses in excess of yield to account for strain hardening and differences between nominal and actual yield.
3. Requirements for transverse reinforcement through the joint to carry shear and to provide confinement to the concrete. The calculations for the amount of transverse reinforcement were based on the model used for beam shear. Confinement requirements were nearly the same as those for columns. Some detailing requirements for the transverse reinforcement were given (Fig. 2). Anchorage of tie legs over perimeter hoops was difficult and was subsequently changed.

The 1976 report was used by a number of designers to detail joints in moment resisting frames in seismic regions. Although there was some reluctance initially to the complications for construction of joints with heavy transverse reinforcement requirements, it was not an impossible requirement and research continued with the aim of making the provisions less restrictive by reducing the transverse reinforcing required. However, designers became convinced that the empirical evidence regarding beam-column joint behavior was sufficient to warrant code coverage and in 1983 Appendix A of ACI 318 included specific design rules for joint detailing.

ACI 318-83

The material in the 1983 code was modified from that in the 1976 report. Research results presented to the code committee indicated that when the strength of the joint was reached, forces were transferred through the joint primarily by compression struts in the concrete and the main role of the transverse reinforcement was to confine the concrete to permit it to carry compression (9). However, the concept of a joint shear stress was maintained because it was easy to understand and to calculate. The resulting code provisions maintained the key elements of the 1976 report but the amount of transverse reinforcement was reduced in some cases and the details for the ties were simplified and more consideration was given to constructibility.
The 1983 ACI 318 provisions for joint design reflect the various competing elements which interact to produce a code. First, there had to be a preponderance of evidence that special provisions were needed in the code. This resulted in joint design requirements being included only for structures in seismic zones. Second, simplicity was desired so the concept of a calculation similar to beam shear was maintained even though the mechanism was not like beam shear. In this case the code writers followed one of Siess's admonitions "If you are going to be wrong, be wrong in the simplest possible way". (Siess served as chairman of the committee responsible for ACI 318-83.) Third, the provisions were sensitive to cost and constructibility issues. Fourth, it was felt that inclusion of the provisions would make it easier to obtain approval for designs by building officials.

In many ways these concerns were more important than a precise representation of research findings. There was a realization that the code did not require as much reinforcement as other codes such as the NZ Standard. However, there was considerable skepticism in ACI 318 that the "ailment warranted the medicine." The lack of a definitive connection between inadequate joint detailing and damage to frames in earthquakes was a prime factor. A corollary argument was that many buildings that had been subjected to earthquake motions had not failed even though the joints would have been found woefully inadequate by the proposed code provisions. Many reasons may be (and were) offered to explain the field experience. In the end ACI Committee 318 reached a consensus--it was something all could "live with." While not perfect, it offered its users the opinion of a group of diverse individuals representing a variety of interests as to the minimum requirements for joint details that will provide adequate safety to the public.

**Other Issues**

With time the joint detailing provisions have come to be understood and accepted. New issues have come up and research has been done to address some of these issues. The role of the slab has been widely discussed (5). The problems of bidirectional loading on frame systems has been studied. One approach for explaining the transfer of forces through the joint when beams in both directions are stressed is the use of compression struts (10). Figure 3 shows the difficulties in defining simply the struts forming in joints with different sized beams. Problems of unsymmetrical and eccentric geometries of beams framing into the joint have been reported (5). Following the 1985 Mexico City earthquake the problems of strengthening frame systems including joints (Fig. 4) has been studied (11). However, in all these cases the empirical evidence has not been considered sufficient enough by code writers to warrant adding new code clauses or modifying existing rules.
Shear in Other Places

One of the reasons the beam-column joint problem has been so intractable is that it is so hard to define. The number of parameters that influence the shear strength of joints is large and although each may have only a small influence, taken in aggregate the influence becomes significant. In addition, many failures are attributed to shear when in fact the mechanism triggering shear may be some other action such as a bond failure, bearing, or detailing inadequacy. The recent introduction of strut and tie models offers some hope that a unifying procedure can be developed to cover many shear problems. The strut and tie model have been proposed as an alternative to the current simple calculations for ordinary beam shear. Considering the factors which impact code development discussed previously, it is unlikely that a procedure which is more complex and does not improve economy, constructibility, safety, or simplify the code is likely to be adopted by a code-writing group representative of the building industry.

The merits of the strut and tie model are primarily in providing a means of visualizing the flow of forces through a complex region where members are joined, forces or reactions are applied, reinforcement changes direction, or member dimensions change. The difficulties in applying simple beam shear rules to these cases is obvious. The designer will be able to show conceptually the flow of forces through a system and develop a set of struts (concrete compressive elements) and a set of ties (reinforcement) to satisfy equilibrium. The problem is to define the size and shape of struts and the permissible stresses on the strut. Steel stresses are more easily defined but the manner and location (nodes) at which the forces are transferred between steel and concrete have been extremely difficult to define. An example of the strut and tie model for a dapped beam is shown in Fig. 5. The specimen shown in Fig. 5 was part of a study (12) in which work was also done to define node behavior for typical nodes in the dapped beam. While the work was very promising the extension of the results to a variety of other cases awaits further research.

Conclusions

The development of provisions for guiding designers in detailing beam-column joints illustrates the dilemmas faced by code writers in trying to accommodate the various constituencies that make up the "building industry." The transfer of knowledge from research and experience to code provisions is neither simple nor rapid. A building code must be based on the fundamental need to "protect the public." In the case of beam-column joint provisions for shear, the code writers in the U.S., decided to adopt an approach which met the following constraints:
Lateral Force Transfer in Buildings

a. simple and understandable to the knowledgeable as well as the novice designer,
b. produce a constructible design, and
c. not impact unfavorably on the economy of construction.

The provisions were included in the ACI Building Code only after the members of the code committee were convinced that there was a need to provide the guidance and that there was sufficient information from research and experience to develop reasonably complete, accurate rules consistent with public safety.

Code writers must be responsive to the need for change and open to suggestions from the many segments of the industry they represent but the final decision must be based on their answer to the question "Will the public be better served if the change is made?"

References


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Lateral Force Transfer in Buildings

Fig. 1—Interaction between research, practice, and building codes (Ref. 8)

Note: For single leg tie, the 90° hook may be bent into core in place or alternated along column on opposite faces. If 90° hook is bent to not less than 135°, cover may be reduced and required cover maintained on perimeter hoop. Tie must be placed next to vertical column bars. The tie may be bent around the longitudinal bar.

Fig. 2—Transverse reinforcement details in 1976 report (1)

Fig. 3—Strut mechanism under bidirectional actions on joint (10)
Special structural steel assembly for joint

Reinforcement in place

Fig. 4—Jacketing of beam-column joint regions (11)
Lateral Force Transfer in Buildings

Design model

Reinforcement

Cracking at intermediate load levels
Cracking at failure

Fig. 5—Strut-and-tie application to dapped beam design (12)
Controversial Issues in the Seismic Design of Connections in Reinforced Concrete Frames

by D. Mitchell

Synopsis: This paper discusses aspects of the design of connections in reinforced concrete frame structures which often get overlooked. The need for careful assessment and detailing of slab-column connections, in flat plate structures combined with walls, is addressed. The way in which the strength and stiffness of spandrel beams can significantly alter the expected response of beam-column connections is illustrated by experimental results and observed seismic damage. Detailed analysis of beam-column joint regions using the modified compression field theory demonstrates behavioral features that have important design implications. The use of non-linear finite element modelling of joint regions to design efficient, yet practical, retrofit measures is discussed. An alternate form of construction using ductile steel link beams to connect reinforced concrete walls is presented. The important design features for the connection of these beams to the walls are highlighted.

Keywords: beams (supports); earthquake-resistant structures; frames; joints (junctions); reinforced concrete; slabs; structural design
APPRECIATION

Tom Paulay is truly a remarkable man. His contributions to the general understanding of reinforced concrete behavior are enormous. He has the rare ability to develop rational models which actually get used in practical design applications. I have, over the years, been profoundly affected by guidance from Tom through many discussions.

I thank you for all the help, Tom, and look forward to more discussions in the future.

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INTRODUCTION

The purpose of this paper is to discuss several aspects of seismic behavior of frame joints that are often overlooked by designers. A means of providing an alternate load path to avoid progressive collapse of two-way slabs is discussed. The role of spandrel beams in determining the effective width of slabs is presented. The need to consider the influence of cover spalling at exterior joint regions is examined. Ways of assessing the strength of joint regions both for new construction and for the retrofitting of deficient joints is presented.

An alternate form of construction using ductile steel link beams to connect reinforced concrete walls is presented. Aspects of the design of this structural system are discussed.

STRUCTURAL INTEGRITY OF TWO-WAY SLAB SYSTEMS

A very popular form of structural system in moderate seismic zones is the combination of walls with two-way slab systems. In interpreting the National Building Code of Canada (NBCC) [1] and the Canadian Standard for the Design of Concrete Structures (CSA A23.3) [2] a designer, in many situations, would require that the structural walls be capable of carrying the entire design lateral forces. In addition, the slab system must be capable of resisting gravity load effects in their deflected configuration. The slabs must therefore have an adequate amount of flexural reinforcement, as well as being capable of transferring the necessary unbalanced moments and shears through the slab-column connections. An additional consideration is the need to check bar lengths in the slabs because the bars will typically require longer lengths than that required for gravity loads only.

It is well known that punching shear is the "Achilles heel" of flat plate and flat slab construction, being the primary mode of failure in collapses of two-way slab structures [3]. There were many examples of failures of two-way slab structures in Mexico City due to the 1985 earthquake [4]. Figure 1 shows the typical layout of the interior region of a flat plate structure. Consider a situation where, due to shear and moment transfer, a punching shear failure occurs at column 5. If no alternate
load path is provided, then due to this failure the slab is forced to span between columns 4 and 6 as well as between columns 2 and 8. This doubling of the span causes significant increases in the moments and shears to be transferred through the slab-column connections at columns 2, 4, 6 and 8. Consequently, failure will occur at these four columns and the failure would rapidly progress horizontally. This horizontal progression of the failure would be accompanied by a vertical progression of the failure as the failing slab would fall onto the floor below, causing the lower floor to also fail. Hence the structure would suffer a progressive collapse.

One very effective way of arresting progressive collapse is to provide alternative load paths. Figure 2 illustrates the important role that the reinforcement can play after a punching shear failure has taken place. After shear failure has occurred the top reinforcement rips-out of the top surface of the slab and hence is ineffective. Therefore a slab-column connection without bottom steel adequately anchored into the column region would have negligible post-punching resistance. In contrast, bottom reinforcement which is properly anchored into the column region does not rip-out of the slab and therefore can provide significant dowel action which can arrest the failure. If the bottom bars are well anchored and effectively continuous then they could provide an alternate load path in the form of a tensile membrane once the slab-column connection is severely damaged. Figure 3 shows that if an initial localized failure develops then the slab hangs off of the columns by the effectively continuous bottom bars and the distributed slab reinforcement. The surrounding slab regions provide in-plane horizontal restraint capable of anchoring the tensions in the reinforcement. Punching shear in the interior region of the structure would lead to two-way tensile membrane action while an edge panel would resist the loads by one-way tensile membrane action (see Fig. 3).

The key in achieving this alternate load path is to design and detail the bottom reinforcement to provide the tensile membrane resistance and thus enable the slab to hang off the columns after a punching shear failure. Tensile membrane analysis and experiments on slabs at McGill University [5] resulted in the development of a design expression for the required area of continuous bottom reinforcement which was included in the 1984 Canadian design standard [2]. A modified form of this equation for the required total area of effectively continuous bottom steel protruding from a column is:

$$A_{sbt} = \frac{2 V_s}{\phi f_y}$$

where $A_{sbt}$ is the total area of effectively continuous bottom steel
protruding from all sides of a column, \( V_s \) is the likely service load shear on the slab column joint, \( \phi \) is the capacity reduction factor for tension (0.9) and \( f_y \) is the specified yield strength of the bottom bars.

In order to be considered to be effectively continuous the details of the bottom bars must satisfy one of the following conditions:
(a) bottom bars are lap spliced within a column or support reaction area with bottom bars in adjacent spans using a Class A splice, \( l_d \).
(b) bars are lap spliced outside of column using a minimum lap splice length of \( 2l_d \).
(c) at discontinuous edges the bars must be anchored such that the yield stress can be developed at the face of the support reaction area. Figure 4 illustrates examples [5,6] of methods of providing effectively continuous bottom bars.

The 1989 ACI code [7] requires that at least two slab bottom bars be continuous or spliced at the support with Class A splices or anchored into the support. A more appropriate method would be to use Equation (1) to calculate the residual shear capacity following a punching shear failure. The residual shear capacity, \( 0.5\phi A_{sh}f_y \) assumes that after a punching shear failure the anchored bottom bars resist the full shear and form an angle of 60 degrees with the vertical column axis. The 1984 Canadian Standard requires that the loading in Equation (1) not be taken less than the unfactored service loading but also requires that the loading not be less than twice the dead load, which corresponds to typical load levels during construction.

**ROLE OF SPANDREL BEAMS**

One key aspect of design of ductile frame structures is the need to have a strong-column, weak-beam structural system. It is therefore important to be able to predict the strength of the beams framing into the columns. Figure 5 illustrates the role of the spandrel beam at exterior beam-column connections [8]. The tensions in the slab bars create torsion in the spandrel beams (see Figs. 5a and 5b). Hence the number of slab bars that could be participating to resist the negative moment in the main beam framing into the column will be a function of the torsional stiffness and strength of the spandrel beams. Recent tests at McGill University [9] indicate that the number of effective slab bars, which is related to the effective slab width, can be determined if the size, and reinforcement details of the spandrel beam are known. As the lateral forces are increased on the exterior connection the tensions in the slab bars increase and therefore the spandrel beam twists and bends towards the inside of the building. At some point the spandrel beam will crack due to the torsion and at this point the spandrel’s torsional stiffness decreases significantly. Upon further loading the spandrel beam twists
and bends laterally and a new load resisting mechanism is set up as shown in Fig. 5c. This load resisting mechanism is described by a strut and tie model. In this model, the tensions in the anchored slab bars are resisted by a series of compressive struts with the top longitudinal bars in the spandrel beam serving as tension ties.

The number of effective slab bars is determined by the stronger of the two mechanisms, the torsional resistance of the spandrel beam or the strut and tie resistance of the idealized truss model shown in Fig. 5c. Figure 6 shows the severely damaged exterior frame joints of the Baybridge Office Plaza due to the 1989 Loma Prieta earthquake [10]. The torsional cracking and spalling of the concrete cover in the joint region are evident.

Experiments investigating the influence of different sizes and amounts of reinforcement in spandrel beams indicate that in many situations the slab bars will play a significant role in contributing to the flexural capacity of the main beam. If this important feature is not properly accounted for, then an underestimation of the negative moment strength of the main beam could result in a design with the beams being stronger than the columns. Figure 7 illustrates the significance of determining the effective slab width correctly, by comparing the experimentally determined moment-curvature responses of two specimens with five response predictions [8,9]. The assumptions concerning the effective slab width in making these predictions included the beam alone, effective slab widths of $3h_1$, $4h_1$ and the full slab width of the test specimen. In addition a prediction assuming the full specimen width with a variable strain distribution across the tension flange was made and it gave the most accurate prediction of the response. Figure 7 clearly demonstrates the need to assess the effect of the slab reinforcement on the beam negative moment capacity. Designing the beam alone to carry the negative moment will typically lead to gross underestimation of the beam strength. Figure 7 also demonstrates that the size and reinforcement of the spandrel beam plays an important role. Specimen R4 had a 400 x 600 mm spandrel beam while specimen R4S had a 250 x 600 mm spandrel beam. Specimen R4 resulted in the full slab width being effective, while specimen R4S had an effective width of about $4h_1$.

An additional feature of the response of exterior frame joints is the significant spalling of the concrete cover that occurs on the outside face of the joint region (see Fig. 6). The cover spalling reduces the column strength [8] and therefore should be considered in assessing the relative column-to-beam strength ratio.
ASSESSING RESPONSE OF BEAM-COLUMN JOINTS

Figure 8 compares the predicted joint shear versus joint tie strain responses with those measured by Uzumeri and Seckin [11] and Ehsani and Wight [12] at the peak of each cycle. The predictions were made [8] by analyzing the joint region for combined axial load, shear and moment using a sectional response prediction and the modified compression field theory for shear [13,14]. The response predictions therefore account for the cross-sectional properties of the joint region, the amount of transverse reinforcement in the joint, the amount and distribution of the column vertical steel and the shear to moment ratio in the joint region. The sectional analysis assumes a uniform field of diagonal compressive stresses in the joint. The two predictions shown in Figs. 8a and 8b used different assumptions; one prediction assumed average tensile stresses, \( f_{c1} \), in the cracked concrete consistent with the modified compression field theory and the other assumed that \( f_{c1} \) was zero. As shown in Fig. 8, the tensile stresses quickly reduce with reversed cyclic loading. It is interesting to note that the combined loading in the joint region causes yielding first in the vertical column bars, followed by yielding of the joint hoops.

ANALYSIS OF RETROFIT MEASURES ON JOINTS

There are many examples of retrofit techniques applied to reinforced concrete frame structures. The extensive retrofit measures in Mexico City following the 1985 earthquake provide many different examples of these techniques [15]. Figure 9 shows the details of two specimens tested at McGill University [16,17]. Specimen R2.0C was designed such that the column was weaker than the beam. A companion specimen to R2.0C was retrofitted by sleeving the column with reinforced concrete as shown in Fig. 9 to produce specimen R2.0RC. The retrofit consisted of adding four corner bars and closed hoops to increase the size of the column. The vertical column bars passed through holes in the slab to provide continuous vertical steel. In order to decide on the reinforcement of the joint region a non-linear finite element analysis (computer program FIELDS [18]) was carried out on the joint region before and after retrofit. This analysis enabled the influence of strong hoop bands (2 No. 10 hoops), immediately above and below the joint to be assessed. Figure 10 shows the magnitude of the principal compressive stresses and their directions for the specimen before and after retrofit. Instead of predicting a uniform field of diagonal compressive stresses there are concentrations of compressive stresses. This analysis lead to the very simple joint repair shown in Fig. 9b. The reversed cyclic loading responses of specimens R2.0C and R2.0RC are compared in Fig. 11. The specimen without the retrofit failed by yielding of the joint ties and by yielding of the vertical reinforcement in the column. The specimen with the retrofit displayed significantly improved strength and ductility.
At later stages in the testing of the retrofitted specimen the original joint ties and the added hoops immediately above and below the joint yielded. This indicates the effectiveness of these added hoops in delaying joint yielding and also illustrates the advantages of using a more sophisticated analysis tool for assessing retrofit details.

**USE OF DUCTILE STEEL LINK BEAMS TO COUPLE WALLS**

The use of ductile steel link beams to connect reinforced concrete walls has been found to provide an excellent alternative to reinforced concrete coupling beams. Figure 12a shows the construction of a full-scale specimen tested at McGill University [19]. The reversed cyclic shear loading was provided using the loading frame shown in Fig. 13a. The link beam was designed to yield in shear, while remaining elastic in flexure. In addition, the embedment of the steel beam into the wall was designed to transfer the shear and moment corresponding to shear yielding in the steel beam. A view of specimen 2 after testing is given in Fig. 12b. The hysteretic response (see Fig. 13b) demonstrates the excellent ductility and energy absorbing capacity of this form of construction. The ease of construction, saving on formwork and the excellent reversed cyclic loading behavior are all factors which combine to make this form of construction a promising alternative.

**CONCLUSIONS**

This paper addresses a number of issues in the seismic design of frame connections. A simple, more rational approach to providing structural integrity of slab-column connections is provided. The role of spandrel beams in the seismic response of exterior frame joints is described and the consequences of their behavior on the effective slab width for the design of beams is presented. The participation of slab bars in the negative moment capacity of the beams can increase the beam strength and hence may upset the strong-column, weak beam design philosophy. The influence of concrete cover spalling on the strength of exterior columns further reduces the margin of the column strength to beam strength ratio. The application of the modified compression field theory in predicting the response of joints indicates the important influence of shear on the need for tensile forces in the vertical column bars. A demonstration of the application of non-linear finite element analysis in assessing different retrofit scenarios is presented. This more sophisticated tool provides a more accurate assessment of the disturbed joint region.

The use of steel link beams to connect reinforced concrete walls results in a simple form of construction with excellent ductility and energy
absorbing characteristics.

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Fig. 1—Punching shear failure occurring around an interior column

a) Without continuous bottom bars

b) With continuous bottom bars

Fig. 2—Effectiveness of properly anchored bottom reinforcement in arresting failure
Fig. 3—Formation of tensile membrane action in a two-way slab structure

Fig. 4—Ways of providing effectively continuous bottom reinforcement
a) Flow of forces from slab

b) Torsion in spandrel

c) Strut-and-tie model in slab above spandrel

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b) Specimen 2 after testing under reversed cyclic loading

Fig. 12—Use of ductile steel link beams to connect concrete walls
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a) Test setup for reversed cyclic loading

b) Applied shear versus relative wall displacement

Fig. 13—Loading apparatus and response of specimen 2
Anchorage of Beam Bars with 90-deg Bend in Reinforced Concrete Beam-Column Joints

by O. Joh, Y. Goto, and T. Shibata

Synopsis: Based on the author's previous tests, failure modes of beam bar anchorage with 90-degree bend used in reinforced concrete beam-column joints were classified into three types: a side split failure, a local compression failure and a raking-out failure. In order to clarify the raking-out failure, the least understood of the modes, column type specimens with beam bars with 90-degree bend in the beam-column joint were tested under pull-out loading at the bars. The specimen variables were: development length, column depth, lateral reinforcement ratio, spacing between beam bars, concrete compressive strength. From the test results, influence factors on the raking-out failure mode were discussed and an equation evaluating anchorage strength was proposed.

Keywords: anchorage (structural); axial loads; beam-column joints; beams (supports); compressive strength; cover; failure; pullout tests; reinforced concrete; strength; stresses; tensile strength; tests
Appreciation

I have looked to Professor Thomas Paulay for exactly ten years since I sent him a letter requesting him to accept me as a visiting researcher, introduced by my former boss in 1983. For the first time I saw him at the eighth WCEE held in San Francisco in the following year, I felt as if my small body became shrunk because the great professor personally greeted me just after my presentation.

The US/NZ/Japan Cooperative Science Program on Structural Design of Reinforced Concrete Beam-Column Joints for Strong Earthquakes was carried out from 1983 to 1989. I attended the seminars as my main research work corresponded with the subject of the program. By attending the seminars I found out that nationalities and personalities were reflected in the ways of research and was given a great impact by active discussions of Professor Paulay.

I arrived in New Zealand to carry out research under Professor Paulay at University of Canterbury for eight months from the summer of 1984 (it was winter in New Zealand though.) At that time my family and I received his kind assistance both in public and private. My account in a bank in New Zealand has been under his control since we left there. In all the wide world, maybe there is no one, except me, who has the great professor manage one’s bank account. The reason why everyone highly respects him is that he is warm-hearted.

Tom Paulay taught me during the eight months in the following way: we should theorize first to explain a mechanism of phenomena and then check or complement the theory by experiments. In this way we can comprehend behavior of R/C structures and lead adequate design methods. My research work is still experiment-oriented, but I am making great efforts to master the way after Paulay.

It is widely known that the earth rotation on its axis makes a swirlpool counterclockwise in the Northern Hemisphere and clockwise in the Southern Hemisphere. Seeing New Zealand researchers give a different viewpoint, I have a feeling that the circuits of thinking of people living in the Northern and Southern Hemispheres are influenced by the earth rotation respectively. Tom’s excellent ability of hitting on ideas and solving problems must be led by the twin circuits of thinking developed by living in Hungary and New Zealand.

The seventieth birthday is a special anniversary for Japanese and is called 'koki.' A literal meaning of the word is 'seldom'. People seldom lived until seventy years old in former times when people’s life span was about fifty. Therefore 'koki' includes the meaning with respect and admiration for such a long-lived people. I would like to celebrate the seventieth birthday of Tom who has won the respect and admiration of all for his excellent research works and unusual wonderful personality.

Osamu Joh
INTRODUCTION

In reinforced concrete structures, when a main bar of a member is anchored into an adjacent member, the bar end can be arranged with a straight anchorage in case the adjacent member has a large depth, but the bar end is usually arranged with a 90-degree bend in the opposite case. Resistance of bar anchorage with a 90-degree bend is shared by the straight portion and bent portion of the bar. The resistance at the bent portion increases in accordance with tensile stress level of the bar and thereby the concrete surrounding the bend may possibly fail.

In a cast-in-place reinforced concrete frame structure, beam longitudinal bars are usually anchored with a 90-degree bend in an exterior beam-column joint. In a precast concrete frame structure, the 90-degree bend is sometimes also used even in an interior beam-column joint.

Many structural design codes specify minimum requirements of development length, radius of bend and thickness of side-cover concrete according to strengths of materials and bar diameter in order to avoid anchorage failure. However it becomes difficult to satisfy these requirements in a beam bar anchorage with a 90-degree bend in a beam-column joint because of the following reasons:

1) Beam longitudinal bars in a frame structure are usually required to reach the actual yield stress at the ends of beams in ultimate strength design;
2) In a high-rise building, the depth and width of columns at the lower stories are preferred to be not so large as those at upper stories though the stresses become larger. And also the
width of beams at the lower stories tend to approach the width of columns, so that the thickness of cover concrete at the bar bend is small;

3) The development length of a 90-degree bent bar at the second or third layer of multi-bar-layer arrangement in an exterior joint is shorter than that at the outer layer;

4) In a precast concrete interior joint, the developments of beam bars sticking out from the precast beam ends usually overlap each other and consequently concrete casting becomes difficult. Therefore, construction engineers prefer the use of short horizontal development length with a 90-degree bend in order to avoid the overlapping;

5) Structural designers and construction engineers prefer the small bend radius because they get flexible arrangement of transverse beam bars crossing inside the bend bar.

Reinforced concrete beam-column joints have to be designed so as to avoid not only shear failure but also anchorage failure in order to assure the yield of beam ends. However the anchorage failure has several failure modes and depends on many influencing factors such as the development length, the radius of bend and the thickness of cover concrete. If all anchorage failure modes of 90-degree bend are classified and previous equations evaluating anchorage strengths are reviewed with respect to each failure mode, unknown factors influencing the strength will be clarified.

The purpose of this study is to establish a general evaluation for all anchorage failure modes of 90-degree bar bend in a beam-column joint. Therefore, the anchorage failure modes are classified and experimental studies to elucidate unknown factors influencing the anchorage strengths were carried out.

CLASSIFICATION OF ANCHORAGE FAILURE MODES

Failure modes of beam bar anchorage with a 90-degree bend in a beam-column joint are classified into three types as shown in Fig. 1 based on the author's and others' previous experimental test results [e.g. Refs. 1-4].

Side Split Failure

Thickness of cover concrete expresses a distance, Co, between an outside beam bar and a column face in a beam-column joint as shown in Fig. 1(a). When this thickness is very thin, the cover concrete beside the bend peels off with the shape of a dish because a wedge effect of the bend bar increases split stress working in lateral direction against the beam bar. The anchorage resistance of outside bars decreases drastically after side split failure.

Most previous studies on anchorage strength of 90-degree bend focused mainly on the side split failure [Refs. 2-5]. In 1992 S.Fujii and S.Morita proposed a formula (Eq. A1 in Appendix)
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[Ref. 6] revised from their previous formula [Ref. 2]. Its application was extended from beam-column joints to general members and from normal strength concrete to high strength concrete. The formula resulted from the experimental results which were based on specimens which had failed mainly in side split mode, thereby rendering its applicability in other failure modes impractical. M. Murakami and T. Kubota proposed a formula to be applied in double layer bar arrangements [Ref. 3]. However, it is not yet ascertained whether both the formulas can be applied in a bar bend with a small radius.

**Local Compression Failure**

Even though a bent bar has a thick cover concrete at the both sides so that the cover concrete does not fail in a side split, and the number of beam bars is not so many that the raking-out failure mentioned below does not occur, the anchorage of bar bend fails in local crushing of concrete if the radius of bar bend is too small. Since the local compressive stress may be inversely proportional to the radius of bar bend, the concrete inside the bend with a small radius crushes. After reaching such strength, the anchorage resistance does not reduce so rapidly and each bar pulls out of the beam-column joint gradually. This failure mode appears at each bar bend individually.

Structural design codes generally specify minimum radius of bar bend, which is acquired from relations of material strength and bar diameter, instead of calculating the radius of bar bend in each bar. An example of required minimum radius of a 90-degree bar bend, provided in AIJ Standard [Ref. 7], is shown in Table 1. The smallest radius used in practice at an inner layer of multibar-layer arrangement is about 1.5 times the actual bar diameter and is smaller than the required value applied to normal beam bars.

**Raking-Out Failure**

Even if neither the side split failure is prevented by thick cover concrete nor the local compression failure is prevented by a large bend radius, a concrete block inside a L-shaped bar layer is raked out toward the beam side caused by providing many beam bars and/or short development length in a beam-column joint, and all the beam bars lose their resistance at the same time. This failure mode, named 'raking-out failure' by the authors, is similar to shear failure of a beam-column joint in relation curve of total bar force and bar displacement. However, the shear failure is caused by compression failure of diagonal concrete strut in a joint. In contrast, the raking-out failure is independent of the compression failure. The raking-out failure is different from so-called cone-shaped failure because the crack plane of the raking-out failure runs across the overall joint width and one part of the cracks appears along the bar bend and tail. There are few studies on the behavior of this failure mode [Ref. 8].
EXPERIMENTAL WORK

Test Specimens

Specimens had an exterior beam-column joint form with no beam concrete nor beam longitudinal bar in the compression side to simplify their production as shown in Fig. 2. Beam longitudinal bars in the tension side had identical characteristics; i.e., (1) inside radius of the bar bend was 3 $d_b$ ($d_b$: nominal bar diameter of 19 mm), (2) tail length after the bar bend was 12 $d_b$, (3) the bars were a high-strength threaded deformed bar. These bars were intersected at the beam-column joint portion in the column with a story height of 1750 mm, which was about half scale of an actual structure.

The specimens consisted of ten variables: development length $L_{dh}$ (distance from beam end to tail center); moment arm of beam $j_b$; column depth $D_c$; spacing of bar $S_0$ (distance between bar centers); thickness of cover concrete $C_0$ (distance between column side face and bar center); number of bar layers; lateral reinforcement ratio $P_w$ in joint; column axial stress $\sigma_c$; loading type (cyclic or monotonic) and concrete compressive strength $\sigma_B$. Most variables had two or three variations as shown in Table 2. The standard specimen LA1-1 had a column width $b_c = 300$ mm, $D_c = 400$ mm, $j_b = 328$ mm, $P_w = 0.2\%$, $\sigma_B = 30$ MPa, four beam bars of single layer arranged with $S_0 = 57$ mm, $C_0 = 64.5$ mm and $L_{dh} = D_c/2 = 200$ mm, and was subjected to monotonic lateral loading and no axial force. The thickness of cover concrete was the minimum value obtained when beam bars were passed inside of column corner bars and was calculated so as not to fail in the side split mode by the formula (Eq. A1). The other specimens were different from the standard specimen only in the variable shown in Table 2. Some details are shown in Fig. 2. In Specimen LA6-1, the second bar layer, consisting of four bars, was placed inside the outer layer with a spacing of 3 $d_b$.

Mechanical properties of materials are shown in Table 3. The aggregate was crushed-stone with a maximum size of 13 mm matched with the scale of specimens. High strength beam bars were used in all specimens to avoid their yielding, but for column bars and hoops, high strength reinforcements were used only in Specimen LA10-2 which was made of the highest strength concrete of Grade 70 MPa.

Instrumentation and Loading

The loading arrangement is schematically shown in Fig. 3. Tensile load $P_1$ was supplied horizontally at the beam bars by a 2000kN oil-jack and reactions $R_1$ and $R_2$ were supported at the compression side of imaginary beam cross section by a steel plate, with its height of one-fifth beam depth, and at the bottom of column, respectively. The other load $P_2$ was supplied at the top of column by a 500kN oil-jack controlled so as to generate the same shear force in both columns. The four beam bars (eight
bars in double layers) were controlled so as to distribute the same pull-out displacement in order to simulate actual beam bar conditions, consequently tensile loads were slightly different from each other. Specimen LA9-1 was subjected to the cyclic tensile load increasing the peak deformation at each cycle. Specimens LA8-1 and LA2-2 were subjected to constant axial force vertically by another oil jack using a pair of loading steel beams and four tie rods (not shown in the figure.)

The beam bar displacements at point F located on the column face and at point B located at the beginning point of a bend were measured using a reference frame which was supported at point C located at the center of column depth by a pin condition as shown in Fig. 4. That is, the displacement at point F (abbreviated $\delta_F$ hereafter) was given as the displacement relative to point C after correcting the relative displacement at point $F'$ using the deformation between points F and $F'$ obtained from the measured bar strain. Furthermore, the displacement at point B ($\delta_B$) was measured directly as a bar slippage relative to point C by using high strength fine wires. The strains at selected points on the reinforcements and in the concrete, and the deformations of the main portions of the specimens were measured.

EXPERIMENTAL RESULTS AND DISCUSSION

Behavior of Cracks and Failure

Fig. 5 shows crack patterns at the final loading stage of some specimens. The crack patterns were different in each specimen, but the common failure plane in all specimens consisted of three main cracks: a sloped crack which appeared from the bar bend portion into the lower column with inclinations of 30-degrees to 60-degrees; a vertical crack along the bar tail; and a diagonal crack running toward the compression zone in the joint. In all specimens, the concrete block with a trapezoidal horizontal cross-section formed by these three cracks was raked out and the anchorage failed finally without yielding in beam bars. However, this failure plane approached a shape from a trapezoid to a triangle with the decrease of the strut angle $\theta$, which is defined as the angle between the horizon and a straight line connecting the $R_1$-reaction point and the intersection of center-lines of the horizontal bar and tail bar, as shown in Fig. 6. In Specimens LA7-1 and LA7-2 with the large lateral reinforcement ratio, many cracks appeared inside the bend dispersively as compared with other specimens. In Specimens LA8-1 and LA8-2 subjected to column axial force, the angles of cracks in the upper and lower sides for beam bars were steep.

Comparison between Experimental and Calculated Strengths

Table 4 lists the total bar force $T_u$ defined as the sum of whole beam bar forces in each specimen and the joint shear force $V_{ju}$ at the ultimate stage. Except for LA7-2, the experimental
shear forces \( \exp V_{ju} \) which was the difference between the total bar force \( \exp T_u \) and the column shear force \( P_2 \) were clearly below the values \( \cal V_{ju} \) calculated by Eq. 1 which evaluates shear strength in a exterior beam-column joint estimated by the AIJ Design Guidelines [Ref. 9].

\[
\cal V_{ju} = k \cdot \sigma_B \cdot t_j \cdot \ldh \\
\text{(Eq. 1)}
\]

where, \( k \) : factor dependent on shape of a beam-column joint; 0.30 for interior joint, 0.18 for exterior joint

\( t_j \) : effective joint width

\( \ldh \) : projective horizontal development length

\( L + r + db = \ldh + db/2 \)

\( L \) : development bar length of straight portion

\( r \) : inside radius of bar bend

\( db \) : bar diameter

This shows that the anchorage failure occurred before the shear failure in the joint. However, Specimen LA1-3 with the smallest strut angle may have reached joint shear failure, side split failure and raking-out failure at almost the same time because compressive failure in the joint concrete and side split failure were observed to be a small degree. Since Specimen LA7-2 did not show such failures, though \( \exp V_{ju} \) was nearly equal to \( \cal V_{ju} \) for the specimen, it can be considered that the specimen failed in raking-out by the effect of its large lateral reinforcement ratio on the strengthening of joint shear resistance, which was not considered in Eq. 1.

The foregoing estimation (Eq. A1 shown in the Appendix) for the anchorage strength expresses the tensile force of each beam bar. The strength is provided separately for the bar located at beam face sides and for the bar located around the center side of a beam width. In order to compare with the experimental total bar force \( \exp T_u \), the calculated anchorage strengths \( \cal T_{u1} \) and \( \cal T_{u2} \) were acquired from Eq. 2 and Eq. 3, respectively. The strengths \( \cal T_{u1} \) and \( \cal T_{u2} \) are based on the assumptions that the center side bars also had the strength in the side split failure applied to face side bars and that the center side bars had larger strength than the face side bars, respectively. As for LA1-3 appearing side split failure, \( \cal T_{u1} \) was nearly equal to \( \exp T_u \). The experimental values of the other specimens without the side split failure, however, are remarkably smaller than the both calculated values, except Specimens LA8-1 and LA8-2 which were subjected to axial load.

\[
\cal T_{u1} = (n + m) \ Pu_F \\
\text{(Eq. 2)}
\]

\[
\cal T_{u2} = n \ Pu_F + m \ Pu_C \\
\text{(Eq. 3)}
\]

where, \( Pu_F, Pu_C \) : tensile force of beam bar at face side and center side in beam width, calculated by Eq. A1
Influence Factors on Anchorage Strength

Fig. 6 shows the relations between the experimental total bar force at ultimate strength $\exp T_u$ and four influence factors. Fig. 6(a) indicates that $\exp T_u$ was nearly proportional to the square root of concrete compressive strength $\sigma_B$. Fig. 6(b) indicates that $\exp T_u$ was proportional to the lateral reinforcement ratio $P_w$, and $T_u$ at $P_w = 0$ estimated by the regression line, was the contribution of concrete. Fig. 6(c) indicates that $\exp T_u$ was proportional to $1/\sin \theta$ because the strut angle $\theta$ decreased depending on the increase of horizontal projection length of bar development. The relation between $\exp T_u$ and effective joint width $b_e$ which was obtained by subtracting the total width of beam bars from the column width was not clear in Fig. 6(d) because the range of variation in $b_e$ was small and the value of $\exp T_u$ of Specimen LA5-2 was less than that of LA5-1 although the value of $b_e$ of LA5-2 was larger than that of LA5-1. The reason for the latter behavior has not yet been made clear. When the result of LA5-2 was omitted, $\exp T_u$ seemed to be almost proportional to the effective joint width. The column axial stress of $\sigma_B/6$ increased the anchorage strength, but the strength of specimens subjected to the axial stress of $\sigma_B/6$ and $\sigma_B/3$ was comparable. This shows the existence of limitation of anchorage strengthening by the axial stress.

Evaluation of Anchorage Strength with Raking-Out Failure

All influence factors are considered to yield an anchorage strength $\cal T_u 3$ in raking-out failure:

$$\cal T_u 3 = T_c + T_w$$

(Eq. 4)

where, $T_c = 2 L_{dh} \cdot b_e \cdot \sigma_t (1 + 6.32 \sigma_o / \sigma_B) / \sin \theta$

$T_w = k_w \cdot a_w \cdot \sigma_{wy}$

$L_{dh} = L + r + d_b/2 = L_{dh} - d_b$

$a_w$ : total sectional area of lateral reinforcement crossing failure plane

$b_e$ : effective joint width $= b_b - n d_b$

$k_w$ : coefficient of effective lateral reinforcement $= 0.7$

$\sigma_t$ : concrete tensile strength $= \sqrt{\sigma_B}$

$\sigma_o$ : column axial stress, but not more than $\sigma_B/6$

$\sigma_{wy}$ : yield stress of lateral reinforcement

$\theta$ : strut angle
It could be assumed that the failure plane had appeared along two lines with 45-degree angles of elevation and depression from the intersection of axes of beam bar and tail bar as shown in Fig. 7, and that the anchorage strength consisted of concrete and reinforcement resistances at the failure plane. The concrete resistance could be evaluated as the horizontal contribution of cracking strength along the failure planes, and it was equal to the tensile strength perpendicular to the vertical plane with a height of twice $L_{dh}$. The concrete resistance $T_c$ used in Eq. 4 was divided by $\sin \theta$ because the component of shear resistance transmitted directly to the compression zone through the diagonal concrete strut increased with the decrease in strut angle $\theta$ and consequently the concrete resistance increased proportionally to $1/\sin \theta$. The reinforcement resistance could be evaluated as the total force generated in the lateral reinforcement crossing the failure planes. Measured strain distribution of lateral reinforcement showed that the reinforcement forces near the beam bars were larger than those far from the beam bars and yielded at the ultimate stage. Therefore, effective coefficient $k_w$ was used in order to obtain the resistant force from the sum of yield forces in the reinforcement, and it corresponded to the coefficient expressing the average stress of reinforcement provided within the failure planes. The additional concrete resistance was assumed to increase proportionally to $\sigma_o$ up to $\sigma_o = \sigma_B/6$, and was constant for $\sigma_o$ over $\sigma_B/6$.

The experimental and calculated anchorage strengths are compared in Table 4 and Fig. 8. The average of ratios of the experimental to the calculated, $Y$, was 0.97 for all specimens. $Y = 1.18$ for LA10-1 made of high strength concrete was the largest and $Y = 0.77$ for LA5-2 with large concrete cover thickness was the smallest. A reason for small strength for LA5-2 may be that the failure planes of LA5-2 were formed in three dimensional failure planes at the thick cover concrete and were different from those of other specimens. However the proposed equation can be considered adequate for the evaluation because the errors were almost within $\pm 20\%$ against many kinds of influencing factors.

CONCLUSIONS

The authors' and other researchers' previous experimental results proved that failure modes of beam bar anchorage with 90-degree bends in reinforced concrete beam-column joints can be classified into three types. One is a side-split failure that concrete covers beside bar bends in a joint spall out with a dish shape individually at both sides of the joint. Another is a local compression failure that a small part of concrete just inside a bar bend crushes individually at each beam bar. The other is a raking-out failure that a concrete block inside a L-shape bar layer is raked out toward the beam side and all beam bars lost their resistance at the same time.
Column type specimens with beam bars with 90-degree bends in their beam-column joint portions were prepared to clarify the anchorage behaviors with the raking-out failure. Ten variables: development length; bend radius; spacing between beam bars; lateral reinforcement ratio; concrete compressive strength and so on, and one to three variations in each variable, were used for the specimens. From the experimental results obtained by the loading tests in which all beam bars in each specimen were subjected to the same pull-out displacement, it was clarified that the anchorage strength was proportional to the root of concrete compressive strength and to the reciprocal of \(\sin \theta\) (\(\theta\) is a strut angle), the additional strength was proportional to lateral reinforcement ratios, and so on. After discussing the influence factors on anchorage failure, the equation evaluating anchorage strength with raking-out failure was proposed as Eq. 4.

**ACKNOWLEDGEMENTS**

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APPENDIX

S. Fujii and S. Morita proposed the following formula to calculate anchorage strength of 90-degree bent bars arranged in general reinforced concrete members. The formula resulted from the test results of specimens which had failed mainly in side split mode. The anchorage strength is expressed by the tensile force $P_u$ in each bar.

$$P_u = f_u \cdot d_b \cdot r$$  \hspace{1cm} (Eq. A1)

where

- $f_u$: local compressive strength of concrete at bar bend
- $d_b$: bar diameter
- $r$: radius of bar bend

The local compressive strength of concrete in Eq. A1 consists of seven factors as follow:

$$f_u = k_0 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot f_{bo}$$  \hspace{1cm} (Eq. A2)

where,

- $k_0$: coefficient dependent on concrete strength $f_c$
  - $f_c < 400 \text{ kgf/cm}^2$: $(f_c/400)^{1/2}$
  - $f_c \geq 400 \text{ kgf/cm}^2$: $(f_c/400)^{1/3}$
- $k_1$: coefficient dependent on shape of bend
  - $(r/3d_b)-0.72$
- $k_2$: coefficient dependent on cover concrete thickness $C_0$ and spacing between bars $S_0$
  - $(0.7 + 0.011S_0/d_b)(0.38 + 0.1C_0/d_b)$
k3: coefficient dependent on location of bend and direction of tail (a: distance between bar and support, D: depth of member)

\[ k_3 = k_{3a} \cdot k_{3b} \]

where

- \( k_{3a} = 1.2\left(1 - \frac{a}{1d_h}\right)^2 + 1 \) when \( a/1d_h \leq 1 \)
- \( k_{3a} = 1.0 \) when \( a/1d_h > 1 \)
- \( k_{3b} = 0.85 \) when \( 1d_h/D \leq 0.5 \)
- \( k_{3b} = 1.0 \) when \( 1d_h/D > 0.5 \)

k4: coefficient dependent on projected development length \( l_{dh} \)

\[ k_4 = 0.038 \cdot \frac{l_{dh}}{d_b} + 0.544 \leq 1.15 \]

k5: coefficient dependent on amount of transverse reinforcement (\( A_s \): cross section of reinforcement around bend, \( f_y \): yield strength of reinforcement, \( s \): spacing of reinforcement)

\[ k_5 = 1 + 0.0007(A_s \cdot f_y/s) \]

\( f_{bo} \): basic local compressive concrete strength

\[ f_{bo} = 1910 \text{ kgf/cm}^2 \]

**Table 1 — Minimum radius of reinforcement bar bend with angle of 90 deg and less located at intermediate portion; provided by AIJ recommendation for detailing and placing of concrete reinforcement**

<table>
<thead>
<tr>
<th>Type of Reinforcement</th>
<th>Grade of Reinforcement</th>
<th>Range of Bar Diameter, ( d )</th>
<th>Minimum Radius</th>
</tr>
</thead>
<tbody>
<tr>
<td>i) Hoop Stirrup Spiral</td>
<td>235 MPa</td>
<td>( d \leq 16 \text{(mm)} )</td>
<td>1.5 ( d )</td>
</tr>
<tr>
<td></td>
<td>295 MPa</td>
<td>( 19 \leq d )</td>
<td>2 ( d )</td>
</tr>
<tr>
<td></td>
<td>345 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ii) Reinforcement except i)</td>
<td>235 MPa</td>
<td>( d \leq 16 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>295 MPa</td>
<td>( 19 \leq d \leq 25 )</td>
<td>3 ( d )</td>
</tr>
<tr>
<td></td>
<td>345 MPa</td>
<td>( 29 \leq d \leq 41 )</td>
<td>4 ( d )</td>
</tr>
<tr>
<td></td>
<td>395 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>295,345,395 MPa</td>
<td>( d = 51 )</td>
<td>5 ( d )</td>
</tr>
<tr>
<td>Specimen</td>
<td>Dimensions in Loading Direction</td>
<td>Dimensions in Transverse Direction</td>
<td>Reinforce in Joint</td>
</tr>
<tr>
<td>----------</td>
<td>----------------------------------</td>
<td>------------------------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td></td>
<td>Depth of Column Dc</td>
<td>Width of Beam b_d</td>
<td>Development Length L_d</td>
</tr>
<tr>
<td>LA 1-1</td>
<td>400</td>
<td>300</td>
<td>D_c/2</td>
</tr>
<tr>
<td>LA 1-2</td>
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<td>300</td>
<td>D_c/3</td>
</tr>
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<td>LA 2-2</td>
<td>228</td>
<td>428</td>
<td></td>
</tr>
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<td>LA 3-1</td>
<td>300</td>
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<td></td>
</tr>
<tr>
<td>LA 3-2</td>
<td>500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LA 4-1</td>
<td>273</td>
<td>357</td>
<td></td>
</tr>
<tr>
<td>LA 4-2</td>
<td>273</td>
<td>357</td>
<td></td>
</tr>
<tr>
<td>LA 5-1</td>
<td>4.7d_b</td>
<td>6.0d_b</td>
<td></td>
</tr>
<tr>
<td>LA 5-2</td>
<td>4.7d_b</td>
<td>6.0d_b</td>
<td></td>
</tr>
<tr>
<td>LA 6-1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LA 7-1</td>
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<td></td>
<td></td>
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<tr>
<td>LA 7-2</td>
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<tr>
<td>LA 8-1</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>LA 8-2</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>LA 9-1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LA10-1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LA10-2</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Notes: Unexpressed column's values are the same values as Specimen LA1-1, \( d_b = \) Diameter of beam bar = 19mm
TABLE 3 — MEASURED PROPERTIES OF CONCRETE AND REINFORCEMENT

<table>
<thead>
<tr>
<th>Diameter of Bar</th>
<th>$\sigma_y$ kg/cm²</th>
<th>$\varepsilon_y$</th>
<th>$\sigma_{\text{max}}$ kg/cm²</th>
<th>$\varepsilon_8$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>D 19</td>
<td>6710</td>
<td>3620</td>
<td>9010</td>
<td>12.9</td>
</tr>
<tr>
<td>D 16</td>
<td>3760</td>
<td>2460</td>
<td>5760</td>
<td>24.4</td>
</tr>
<tr>
<td>$\phi$</td>
<td>5800</td>
<td>4460</td>
<td>9830</td>
<td>12.2</td>
</tr>
<tr>
<td>D 6*</td>
<td>3340</td>
<td>1730</td>
<td>4220</td>
<td>25.5</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>$\sigma_b$ kg/cm²</td>
<td>$\varepsilon_b$</td>
<td>$E_{1/3}$ $E_{2/3}$ x10^5 kg/cm²</td>
<td></td>
</tr>
<tr>
<td>30 MPa</td>
<td>315</td>
<td>2750</td>
<td>2.18 1.96</td>
<td></td>
</tr>
<tr>
<td>50 MPa</td>
<td>487</td>
<td>2270</td>
<td>3.37 3.13</td>
<td></td>
</tr>
<tr>
<td>70 MPa</td>
<td>754</td>
<td>2760</td>
<td>2.96 2.90</td>
<td></td>
</tr>
</tbody>
</table>

Notes: $\phi$ = Round bar, D = Deformed bar, * = Indent type bar
$\varepsilon_b$ = Elongation, $\mu = 10^{-6}$ strain
$E_{1/3}, 2/3$ = Young’s Modulus at $\sigma_b/3$ and $2\sigma_b/3$

TABLE 4 — RESULTS OF OBSERVED AND CALCULATED VALUES AT ULTIMATE STAGE

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental Values</th>
<th>Calculated Values</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Force</td>
<td>Beam Bar Disp.</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>$\delta_B$</td>
<td>$\delta_P$</td>
<td>$\delta_B$</td>
</tr>
<tr>
<td>LA 1-1</td>
<td>22.3 17.7</td>
<td>2.08 1.87</td>
<td>34.7 41.2</td>
</tr>
<tr>
<td>LA 1-2</td>
<td>13.0 10.5</td>
<td>2.54 2.30</td>
<td>24.6 31.6</td>
</tr>
<tr>
<td>LA 1-3</td>
<td>62.3 50.5</td>
<td>1.30 1.85</td>
<td>68.0 59.8</td>
</tr>
<tr>
<td>LA 3-1</td>
<td>15.3 12.6</td>
<td>1.85 2.08</td>
<td>27.8 34.5</td>
</tr>
<tr>
<td>LA 3-2</td>
<td>36.7 29.1</td>
<td>1.58 1.72</td>
<td>49.4 50.8</td>
</tr>
<tr>
<td>LA 4-1</td>
<td>24.0 18.5</td>
<td>0.95 1.70</td>
<td>37.1 43.7</td>
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<tr>
<td>LA 4-2</td>
<td>32.0 25.1</td>
<td>2.04 2.23</td>
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</tr>
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<td>LA 5-1</td>
<td>30.1 24.2</td>
<td>1.72 1.75</td>
<td>37.3 50.2</td>
</tr>
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<td>LA 5-2</td>
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<td>39.1 77.6</td>
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<td>0.92 0.91</td>
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<td>LA 7-1</td>
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<td>3.03 2.94</td>
<td>36.1 44.7</td>
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<tr>
<td>LA 7-2</td>
<td>41.4 33.1</td>
<td>2.76 3.32</td>
<td>34.8 49.4</td>
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<tr>
<td>LA 8-1</td>
<td>45.8 37.2</td>
<td>1.56 1.46</td>
<td>41.9 45.2</td>
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<tr>
<td>LA 8-2</td>
<td>45.4 36.8</td>
<td>1.89 1.87</td>
<td>43.9 46.2</td>
</tr>
<tr>
<td>LA 9-1</td>
<td>23.1 17.4</td>
<td>0.65 0.62</td>
<td>34.5 41.0</td>
</tr>
<tr>
<td>LA10-1</td>
<td>35.0 28.1</td>
<td>0.30 0.80</td>
<td>53.7 51.2</td>
</tr>
<tr>
<td>LA10-2</td>
<td>48.9 40.0</td>
<td>3.13 3.30</td>
<td>101.8 107.3</td>
</tr>
</tbody>
</table>

Notes: $T_u$ : Total tensile bar force (tonf)
$V_{ju}$ : Shear force in beam-column joint (tonf)
$\delta_B, \delta_P$ : Displacement of beam bar at point B and point P (mm)
Fig. 1—Classification of anchorage failure modes

Fig. 2—Details of specimens
Fig. 3—Loading arrangement

Fig. 4—Instrumentation for displacement of beam bar
Fig. 5—Examples of crack patterns appearing on side face of column
Fig. 6—Relation of anchorage strength and influencing factors

\[ T_u = 1.50 \sqrt{\sigma_b} \]

\[ T_u = 1.14 \beta \]

\[ T_u = 5.4 + 2.3 \, \text{p}_{\text{w}} \]

\[ T_u = 146 + 146 / \sin \theta \]

\[ b_e = b_c - n \, d_b \]
Bar tail

$\theta$: Radius of bar bend

$\theta$: Strut angle

Fig. 7—Model of failure planes and definition of strut angle

Fig. 8—Relation between experimental and calculated anchorage strength
Dynamic Ductility Demand and Capacity Design of Earthquake-Resistant Reinforced Concrete Walls

by H. Bachmann and P. Linde

Synopsis: Reinforced concrete structural walls may provide efficient earthquake-resistance in multi-storey buildings. In Europe they are commonly combined with gravity load dominated slender columns whereby the entire horizontal action is taken by the walls.

In recent years it became possible to design R/C structural walls in a clear manner according to the capacity design method which is based on an "elastic" equivalent static force reduced by a global displacement ductility factor and by an overstrength reduction factor.

In this paper a nonlinear dynamic performance check of capacity designed walls is carried out. For this purpose a newly developed macro model is used for the modelling of the wall. Nonlinear time history analyses are carried out with a ground motion compatible to the elastic design response spectrum of the Swiss Standard SIA 160 as input.

The major findings of this paper pertain to three important design aspects as follows:

1) the dynamic rotational ductility demand may have a different distribution over various height to length aspect ratios of the wall than previously anticipated by static analysis.

2) the dynamic bending moment demand over the height of the wall may differ from the static assumption depending on the aspect ratio of the wall. This necessitates a modified moment capacity distribution.

3) the dynamic shear force at the base of the wall may exceed the previous assumptions of the capacity design method

Keywords: capacity; ductility; dynamic structural analysis; earthquake-resistant structures; flexural strength; hysteresis; reinforced concrete; standards; stiffness; strength; structural analysis; structural design; walls
Appreciation

The name and personality of Tom Paulay are in many ways related to Switzerland and to the writer. Three areas are worthy of special mention: The Swiss earthquake standard provisions, the Swiss Federal Institute of Technology (ETH) in Zurich, and Switzerland's alpine landscape and culture.

Switzerland is a country with moderate seismicity. Six hundred years ago the city of Basle was completely destroyed by a severe earthquake. The 1855 Visp earthquake was less intense. But if this earthquake were to happen today, damage of up to 20 billion US Dollars could be expected. In 1980 it was decided that in a new Swiss standard for actions on structures a modern and rational concept for the earthquake design of structures should be implemented. This concept was strongly influenced by Tom Paulay.

The first encounter with Tom was in 1968 in Zurich when he visited Europe for the first time since emigrating to New Zealand. Problems such as the influence of shear on the rotational capacity of reinforced concrete plastic hinges just treated in both Tom's and the writer's dissertations led to lively discussions. In 1982 and on several occasions subsequently Tom was a prominent guest professor at the ETH. As a result the Swiss engineering community began to get a deeper insight into the nature and extent of damage earthquakes can cause to structures. Many modern ideas introduced and explained by Tom found their way into the Code Task Group. In 1989 the new Swiss Standard SIA 160 with completely new and modern earthquake provisions became effective.

Everybody who knows Tom knows that his personality creates more than simply professional relationships. To his love of life belong deep friendships. And to the friendships in Switzerland belong the Swiss alpine landscape and its manifold culture. On unforgettable trips and hikes not only capacity design but also questions of European history, religion and philosophy were deeply discussed, often accompanied by good food and wine.

The impact of Tom Paulay on the education of the Swiss engineers in the earthquake design of structures and on the new earthquake standard provisions was honoured in 1988 by the Doctor honoris causa - Dr. hc. - of the ETH. This was a small token of the deep gratitude of many Swiss friends for all the knowledge he has imparted to us. The writer would also like to thank Tom's lovely spouse Herta, who always accompanied Tom and cared for him with great love and devotion. This was not without many sacrifices on her part, without which all the good things we have shared with Tom would not have been possible.

Thank you both, Herta and Tom.

Hugo Bachmann
INTRODUCTION

In multi-storey buildings reinforced concrete structural walls may provide efficient earthquake-resistance. In Europe it is common to design buildings with structural walls combined with gravity load dominated slender columns. This system allows for a clear separation of the design actions on the different members. Whereas the slender columns are only designed for gravity loads, the structural walls are designed to resist the entire horizontal earthquake action.

During the last two decades a clear and logical method for the earthquake design of reinforced concrete buildings has been developed, known as the Capacity Design Method. This method, which is also applicable to structural walls, is based on extensive theoretical and experimental work in the field of reinforced concrete mainly carried out at the University of Canterbury, Christchurch, New Zealand. The method has been published in books in German (1) and in English (2). On the other hand, the nonlinear dynamic behaviour of capacity designed buildings has however not yet been researched extensively.

This paper reports on an attempt to verify and check the nonlinear dynamic behaviour of a capacity designed multi-storey building with structural walls resisting the earthquake action.

First, the design philosophy of R/C structures with walls and gravity load dominated columns is described. In the same section the capacity design of multi-storey structural walls is summarized.

In the next section a numerical model for the simulation of the nonlinear dynamic behaviour of multi-storey structural walls is presented. The model is of the "macro model" type and describes the global behaviour of wall sections. Four nonlinear springs connected via rigid beams make up an element of the model.

In the following section the application of the numerical model is presented for an eight-storey building located in Switzerland. Discretization, modelling of masses, ground motion input, and dynamic analysis method are described.

Based on the numerical results achieved by the nonlinear dynamic analysis, the subsequent two sections deal with the important subjects of the dynamic ductility demand and the dynamic moment demand.
Following this is a section devoted to the presentation of plausible newly developed rules for the flexural capacity design of multi-storey walls. These rules are based on the results of the nonlinear dynamic analyses.

In a further brief section the dynamic shear demand is presented and related to the shear strength and the demand proposed in the capacity design method.

Finally, a summary and conclusions are provided.

**DESIGN PHILOSOPHY OF R/C WALL STRUCTURES WITH GRAVITY LOAD DOMINATED COLUMNS**

In the following the design philosophy behind R/C wall structures with gravity load dominated slender columns is briefly discussed.

In Europe multi-storey buildings are often designed as hybrid frame-wall structures. Thereby, a division of the resistance of gravity load and lateral action is made. The gravity loads arising from self weight, dead and live loads, are carried over reinforced concrete slabs and T-beams to slender gravity load dominated columns. These columns which are often prefabricated and may be frequently found with a square or round reinforced concrete section are thus designed for gravity load only, but will in fact resist a small bending moment at the top and bottom. The entire horizontal action, however, is resisted by reinforced concrete structural walls, which stretch over the entire height of the building. The structural wall is also carrying gravity load from its tributary area. The advantages with this design philosophy are: simple design, clear flow of forces, straightforward construction and detailing.

The influence of the gravity load columns on the earthquake behaviour of the wall is usually very small. This was confirmed by the nonlinear dynamic analyses performed in (3). Consequently, the design procedure described above may be justifiable. Furthermore, the essential earthquake behaviour of a building of this type may be numerically treated by modelling the wall only with the proper building masses.

The design of a multi-storey structural wall of this type will, according to the capacity design method, be carried out as follows. A plastic hinge zone is specified, usually at the base of the wall. Within this zone construction detailing is provided so that large deformations and yielding of the vertical flexural reinforcement may occur. This zone stretches over a height $L_p$ equal to the larger of the horizontal wall length $L_w$ or a sixth of the wall height $H_w$. The rest of the structural wall is intended to behave essentially elastically, and should be protected against yielding, see Fig. 1.

The entire horizontal earthquake action in one direction of the building is calculated, based on an equivalent static force $E$.

$$ E = \frac{a_h W}{g} \frac{1}{\mu_a} C_d $$

(1)

This force is obtained as the building mass $W/g$ times an "elastic" acceleration $a_h$ reduced by a global displacement ductility factor $\mu_a$ and by an overstrength reduction factor $C_d$. The building mass based on the Swiss Standard SIA 160 (4), in the case of earthquake as predominant action, stems from self-weight with a load factor of 1.0 and live load with a load factor of 0.3. The acceleration depends on the location of the building (seismic zone and type of ground) and its
Lateral Force Transfer in Buildings

fundamental period of vibration, and may be taken from an elastic design spectrum, see Fig. 2. Switzerland has four seismic zones, each calibrated by a design ground acceleration which appears on the right side of the spectrum.

A first force reduction is performed by division with a global displacement ductility factor $\mu_A$, which may be chosen within the range of "natural ductility" ($\mu_A$ equal to e.g. the "K-factors" of the structural class I (SCI) of the Swiss Standard SIA 160, that means $\mu_A = 2$ for walls) over "restricted ductility" ($\mu_A = 3$) to full ductility ($\mu_A = 5$), both the latter according to the capacity design method with special attention given to necessary structural detailing.

A second force reduction is performed by means of an overstrength reduction factor $C_d$ ("construction factor"), taken as 0.65. This factor stems from the formulation

$$C_d = \frac{1}{\gamma_R \lambda_o}$$

(2)

where the factors $\gamma_R$ and $\lambda_o$ are defined below.

The horizontal earthquake action in one direction is then distributed to the walls, with a possible torsional effect accounted for. The horizontal action resisted by a particular wall is further distributed over the height of the wall according to a linear inverted triangular pattern, shown in Fig. 1. The equivalent static bending moment and shear force distribution of the wall is thus obtained as $M_E$ and $V_E$. The effective gravity load $P_E$ on the wall arises from its tributary floor area, and is due to the same weight that was used for the calculation of the horizontal action.

The bending moment demand $M_i$, which has to be satisfied by the resistance $M_R$ calculated with design values for strength, is obtained as

$$M_i = \gamma_R M_E$$

(3)

where $\gamma_R$ is the resistance factor (the inverse of the strength reduction factor in the ACI code) usually taken as 1.2. The shear demand $V_w$ is

$$V_w = \omega_v \Phi_{o,w} V_E$$

(4)

where $\Phi_{o,w}$ is the flexural overstrength factor for the wall obtained as

$$\Phi_{o,w} = \lambda_o \frac{M_E}{M_R}$$

(5)

$\lambda_o$ being the overstrength factor for reinforcing steel, usually taken as 1.2 or more. The dynamic magnification factor $\omega_v$ is for $n$ storeys obtained as

for buildings up to six storeys

$$\omega_v = 0.9 + \frac{n}{10}$$

(6)

and for buildings with six or more storeys

$$\omega_v = 1.3 + \frac{n}{30}$$

(7)

For the gravity load no resistance factor is used since it would be on the unsafe side to reckon with a higher gravity load tending to give more bending moment resistance than available ($P_i = P_E$).
MODELLING OF WALLS

Many different numerical models for the simulation of the earthquake behaviour of structural walls have been developed. For an overview see e.g. (5). The major types may be divided into macro models, describing a global behaviour of the wall, and micro models which are based on the description of the local material behaviour.

In this paper a macro model will be used, developed in (5). The reason for the choice of this model is its suitability for design-related problems. The model is capable of delivering section quantities such as rotational ductility and cross sectional forces.

The model consists of four nonlinear springs connected by rigid beams. An element comprising all the springs and rigid beams is shown in Fig. 3. A multi-storey wall should be discretised by several such elements. In the element the outer vertical springs simulate the flexural behaviour and obey hysteretic rules shown in Fig. 4. The horizontal spring simulates the shear behaviour and the central vertical spring supplies additional stiffness in compression only so as to model axial behaviour correctly. A detailed presentation of the model and its hysteretic behaviour is provided in (5).

The hysteretic rules for the outer vertical springs are made up of the skeleton curve and the unloading and reloading curves. As shown in Fig. 4, the skeleton curve contains an uncracked elastic part in compression, a cracked part in tension, and a yielded part in tension.

The uncracked stiffness is obtained from the elastic flexural behaviour of the wall cross section. The cracked stiffness is assumed to be a fraction \( \alpha_{cr} \) of the uncracked stiffness, and is typically chosen in the range of 0.4 to 0.8. The relation between the local reduction factor \( \alpha_{cr} \) for the vertical outer spring, and the global reduction factor \( \alpha_g \) for the cross sectional moment of inertia was derived in (5) and is shown as the upper right portion of the graph in Fig. 5.

The yield stiffness is assumed to be a fraction \( \alpha_y \) of the uncracked stiffness, typically chosen in the range 0.01 to 0.05. The force level at yielding may be determined from an investigation of the moment-curvature relation of the cross section. This may be performed by means of a computer program dividing the cross section into fibres for which the correct amount of reinforcement are considered. The lower left portion of the graph in Fig. 5 represents the relation between the local reduction factor \( \alpha_y \) and the global reduction factor \( \alpha_g \) for the cross sectional moment of inertia.

Obeying material laws for steel and concrete, an incremental-iterative procedure results in a moment vs. curvature relation for the desired cross section. From this relation a yield moment may be chosen and the yield level of the vertical spring may be obtained by dividing this moment by the distance between the outer vertical springs of the model. The unloading and reloading rules are shown in Fig. 4, and are largely empirical. The force level on the elastic compressive branch indicated with \(-\alpha_{el}F_y\) represents the point where flexural cracks are closing.

The horizontal spring which simulates the shear behaviour obeys the hysteretic rules shown in Fig. 6. The skeleton curve is bilinear and the unloading and reloading is origin-oriented. The elastic branch on the skeleton curve represents uncracked shear stiffness, which may be obtained theoretically. The second branch represents the cracked shear stiffness, which together with the crack force essentially must be determined empirically, see (5). Yielding in shear is not
anticipated due to the use of capacity design principles for assessing the design shear force, see equation (4).

Finally, the central vertical spring acts in compression only where its stiffness is derived so as to achieve correct elastic compressive behaviour together with the two outer vertical springs.

NUMERICAL APPLICATION

The model presented in the previous section is used in the following numerical application, which is described in more detail in (5). An eight-storey office building located in Switzerland in the seismic zone 3b is to be analysed. The plan and elevation are shown in Figs. 7 and 8. Earthquake action horizontally perpendicular to the long side of the building is studied. The two structural walls, located in each facade, equally share half of the earthquake action, and in the following we will study one of these walls. The global displacement ductility factor $\mu_A$ is chosen equal to three (restricted ductility, $\mu_A = 3$) and for an additional case equal to five (full ductility, $\mu_A = 5$).

A storey weight of 3.08 MN (2.78 MN for roof) for half of the building with a design fundamental period of around 0.90 Hz gives equivalent static wall section forces at the wall base as $M_E = 16.1$ MNm and $V_E = 0.72$ MN for the restricted ductility level, and $M_E = 9.7$ MNm and $V_E = 0.43$ MN for the full ductility level. This gives a flexural demand of $M_i = 19.3$ MNm and $M_i = 11.6$ MNm, for the restricted and full ductility levels respectively, by use of the resistance factor $Y_R = 1.2$. The effective normal force $P_i$ is taken unchanged, equal to $P_E = 4.15$ MN at the wall base arising from the floor weights tributary to the wall.

The cross section is reinforced according to Figs. 9 and 10 for the two ductility levels, giving a flexural strength $M_R$ of 19.4 MNm and 15.8 MNm for the restricted and full ductility levels, respectively. The relatively large difference compared to the demand for the latter is largely due to nominal minimum reinforcement requirements. With $\lambda_0$ taken as 1.2 the flexural overstrength factor $\phi_{o,w}$ is obtained according to (5) as 1.45 for restricted ductility and 1.95 for full ductility. The dynamic magnification factor $\omega_v$ is obtained as 1.57 according to (7) for both ductility levels. This gives the shear demands $V_w$ equal to 1.59 MN for restricted ductility and $V_w$ equal to 1.32 MN for full ductility.

With the minimum horizontal reinforcement ratio of 0.20 %, the shear strength $V_R$ is obtained as 2.70 MN for both ductility levels, made up of a concrete contribution of 1.31 MN and a steel contribution of 1.39 MN, calculated according to standard methods, see e.g. (1).

The curtailment of flexural reinforcement along the height of the wall above the plastic hinge is performed linearly (until meeting the nominal minimum reinforcement) as proposed in the capacity design method (1) and shown in Fig. 11.

For the numerical calculations one of the lateral walls placed in the short facades is modelled. The discretization of this wall is shown in Fig. 12. The plastic hinge zone stretching into the second storey is discretised by three elements. The rest of the second storey and the subsequent storeys making up the elastic region are discretised by one element each.

The storey masses are modelled as point masses located at each floor level. Two point masses (one at each horizontal end of the wall) accounting for the mass of half the building at that floor level are used.
The earthquake input is modelled by a horizontally acting ground motion. This ground motion is artificially generated and conforms to the SIA elastic design spectrum of Fig. 2. The ground motion in the form of an acceleration history is shown in Fig. 13. The duration is 10 s, with a strong motion phase of about 7 seconds.

Prior to the dynamic analysis, a gravity load step is applied whereby it is assumed that all members behave elastically. The dynamic analysis is performed as a nonlinear time integration carried out according to the implicit $\alpha$-method, see (6). Time increments of 0.01 seconds are used, and equilibrium tolerances of one percent of the maximum element forces are accepted. Slight Rayleigh damping of around one percent in the first and second modes is added in order to dampen higher modes.

**DYNAMIC DUCTILITY DEMAND**

As mentioned above, a plastic hinge zone of a structural wall is deliberately chosen as an area where large deformations are allowed during earthquake action. These large deformations take place without a structural failure when precautions are taken in this zone by means of careful structural detailing. This detailing is described comprehensively in (1) and (2). Important parts of it consist of the control of wall thickness in order to avoid buckling, control of the compressive zone extension, and the proper design and control of reinforcement bar placing, placing of hoops and tie bars, etc.

In order to carry out the abovementioned detailing properly, it is necessary to obtain a good estimate of the expected deformations in this zone. Although it is possible to perform this by hand calculation for an anticipated equivalent static force acting on the wall, the deformations obtained from a nonlinear dynamic analysis will generally look different. We focus here on the curvature ductility demand in the plastic hinge zone. For capacity designed walls this demand is defined as follows. Beyond yielding it is assumed that the entire plastic hinge exhibits a constant curvature. For any given post yield curvature $\phi$ of the plastic hinge zone, the curvature ductility $\mu_\phi$ is defined as

$$\mu_\phi = \frac{\phi}{\phi_y} \tag{8}$$

where $\phi_y$ is the curvature at the onset of yielding. Based on assumptions of a cantilever wall of height $H_w$, length $L_w$, with a plastic hinge zone of length $L_p$, and for different chosen global displacement ductility levels, a hand calculated diagram of the expected curvature ductilities from (1) is shown in Fig. 14. The figure shows the curvature ductility demand versus different wall aspect ratios defined as $r_a = H_w/L_w$. For each chosen global displacement ductility a shaded zone is given which covers a variation of the assumed length $L_p$ of the plastic hinge. Furthermore the results of Fig. 14 base upon an assumption of a horizontal concentrated force placed at the free end of the wall.

For the structural wall of the eight-storey building, the curvature ductility at the wall base is obtained as a result from the nonlinear dynamic analysis. Figs. 15 and 16 show the curvature ductility demand versus time obtained from the wall element closest to the base, for the restricted and the full ductility design,
respectively. The maximum curvature ductilities reached were found to be around 6.5 and 8.5, respectively. This wall has an aspect ratio of $32.0 \, m / 6.0 \, m = 5.33$.

In order to study how the dynamic influence on the curvature ductility demand varies over different aspect ratios, this wall was redesigned using some different wall lengths, resulting in different aspect ratios. The redesign consisted basically of a changed fundamental period and thereby a changed equivalent static force, but all designs had to meet nominal minimum reinforcement requirements. In this manner additional aspect ratios of 10.0, 8.0, and 3.0 were obtained. The resulting curvature ductility demands are shown in Fig. 17 for chosen restricted and full ductility levels. Plastification was allowed over a constructive plastic hinge zone of three elements, together having the height $L_p$ equal to 6.0 m for all walls. The presented results are in all cases taken from the element closest to the base, which exhibits the largest ductility values. For each global displacement ductility a shaded zone is given in the figure, covering different assumptions of the strain hardening ratio $\alpha_y$ in the numerical model (seen in Fig. 4).

It is seen from the nonlinear dynamic results in Fig. 17 that we obtain similar curvature ductility demand for low aspect ratios compared to the static hand calculations of Fig. 14, whereas for high aspect ratios the nonlinear dynamic results indicate lower curvature ductility demand. Physically this finding could be confirmed by an energy balance, discussed in more detail in (5), where the amount of dissipated energy of the plastic hinge zone is compared to the amount of dissipated energy of the upper region intended to remain elastic. It was found that for more slender walls, more energy is dissipated in the upper region of the wall, where considerable cracking and some yielding takes place during the nonlinear dynamic analysis.

**DYNAMIC MOMENT DEMAND**

The next quantity to be discussed is the bending moment demand over the height of the wall. The assumed basic moment distribution is shown as the curved moment line on the right in Fig. 11. This distribution stems from the inverted triangular equivalent static force distribution according to Fig. 1. However, for design purposes the dotted straight line in Fig. 11 is used. The bending moment at the base of the eight-storey wall with aspect ratio 5.33 obtained from the nonlinear dynamic analysis is shown versus time in Fig. 18 for the restricted ductility design. Extracting the maximum bending moment reached for all elements along the height of the wall results in a dynamic moment demand as shown in Fig. 19 (left). For the more slender wall with aspect ratio of 8.0 the corresponding distribution over the height is shown in Fig. 19 (right), also for the restricted ductility design. It is clearly seen that for the slender wall some increase in bending moment over the upper storeys is obtained due to the effect of higher modes. It should be noted that, in general, these maximum moments at a particular storey are not reached simultaneously with the maximum moments at other storeys, which means that a distribution like this does not exist at any one time.
POSSIBLE CAPACITY DESIGN RULES FOR FLEXURAL STRENGTH BASED ON DYNAMIC MODELLING

In view of the results based on the nonlinear dynamic analysis which were presented in the previous sections it may be possible to slightly modify the capacity design rules for multi-storey walls compared to the rules which are largely based on static assumptions, see (1). We will here limit the presentation to a brief suggestion on how the flexural design may be modified. More details are presented in (5).

It is clearly seen that the bending moment distribution over the height of the wall, obtained by nonlinear dynamic analysis and shown in Fig. 19, generally does not agree with the assumptions of Fig. 11.

In Fig. 20 a proposed distribution of flexural strength is shown. It is proposed to design the plastic hinge zone in the same manner as before, with a constant strength over the height \( L_p \). As seen from the figure, an increased strength is suggested immediately above the plastic hinge zone in order to avoid yielding in the upper part of the wall which means that this part always remains elastic. It is proposed that the strength of the elastic region be set at a level corresponding to the overstrength moment which may be transferred from the plastic hinge zone. From Fig. 20 we obtain

\[
R_e = \lambda_o R_p
\]

where \( \lambda_o \) is usually taken as 1.2.

Based on the results from the dynamic analyses which indicate that relatively high bending moments may occur at the mid and upper storeys it will be necessary to have the increased flexural strength set at the bottom of the elastic region over a certain height in order not to allow any yielding to occur. Thus it is proposed that over a height \( L_{ec} \), a constant flexural strength is specified. The necessary height \( L_{ec} \) proved to be dependent on how slender the wall is, and for a given ground motion, the wall's fundamental period \( T_1 = 1/f_1 \) proved to be a possible choice as a proportional input parameter for \( L_{ec} \). In (5) it is proposed that \( L_{ec} \) be taken as a fraction of the total height of the elastic region \( L_e \) according to

\[
L_{ec} = \alpha_{ec} L_e
\]

where \( \alpha_{ec} \) may be taken as

\[
\alpha_{ec} = 0.20/f_1
\]

and where \( f_1 \) is expressed in Hz. The curtailment of flexural reinforcement above the height of \( L_{ec} \) would be carried out essentially as proposed in (1) i.e. linearly until a possible minimum flexural strength \( R_{min} \) is reached due to nominal minimum reinforcement requirements.

Fig. 21 shows the result for the eight-storey wall with an aspect ratio 5.33 and restricted ductility design, when the rules according to (1, 2) are followed. It is seen that the demand from the nonlinear time history analysis reaches the strength at a few locations above the plastic hinge. In Fig. 22, the strength distribution proposed here is applied to the same wall.

Figs. 23 and 24 show the corresponding designs for a more slender eight-storey wall with an aspect ratio of 8.0. In Fig. 23 it is clearly seen that yielding
takes place over several stories if no additional strength is provided above the plastic hinge. In Fig. 24 yielding above the plastic hinge zone is avoided.

For the eight-storey wall with the aspect ratio of 5.33 and restricted ductility (cross section shown in Fig. 9) it was found that over the region $L_{ec}$ four additional bars D20 at each end would be necessary. In Fig. 25 the strengthened cross section above the plastic hinge is shown. In Fig. 26 the transition between the plastic hinge and the strengthened elastic region is shown in elevation (left) and vertical section (right).

It should be noted that by the proposed design rules not only the "elastic" region above the plastic hinge is protected from yielding but also a definite physical limitation of the plastic hinge zone is created. It may now be possible to clearly define this zone. This may have some advantages over making a conservative assumption for the definition of the zone where special detailing for plastic deformations is necessary.

In (5) the proposed rules are shown to cover the moment demand well for different walls and their necessity becomes clear especially for more slender walls. However, the simple expression (11) only appears to be practicable within a limited frequency range of perhaps 0.4 to 2 Hz. For stiffer walls, however, the rules described in (1) appear to be sufficient.

It is rather for relatively slender walls that the question becomes important.

**DYNAMIC SHEAR DEMAND**

In addition to the above discussion on the bending moment demand, the shear demand in the plastic hinge zone will briefly be discussed. Fig. 27 shows the shear force versus time at the wall base for the eight-storey wall with aspect ratio 5.33 and restricted ductility design.

The shear demand $V_w$ according to the rules given in (1, 2) and in expression (2) of this paper is shown in the figure as a dotted line. It has a value of 1.59 MN. Furthermore, it can be seen from the figure that the dynamic shear demand reaches a peak of around 2.4 MN which clearly exceeds $V_w$.

But in this case the shear strength $V_R$, obtained as the sum of the concrete and nominal minimum reinforcement steel contributions lies around 2.7 MN, and is also shown in the figure. Hence, in this case the dynamic shear demand does not exceed the shear capacity.

However, in other cases where the nominal minimum reinforcement does not deliver a large contribution (e.g. walls with smaller cross sectional area) the dynamic shear demand may actually exceed the shear strength obtained from the traditional rules. This problem is discussed further in (5).

**SUMMARY AND CONCLUSIONS**

This paper focuses on the nonlinear dynamic behaviour of multi-storey capacity designed structural wall buildings subjected to earthquake action. Initially the design concept of earthquake-resistant wall buildings with gravity load dominated columns commonly used in Europe is presented, followed by a summary of the rules of the capacity design of multi-storey walls.

A numerical macro model designed to be used for the global modelling and nonlinear time history performance check of multi-storey walls is presented next,
followed by the application of this model to the example of an eight-storey capacity designed structural wall building. The numerical discretization and ground motion input are described. Numerical results pertaining to the dynamic curvature ductility demand and the dynamic moment demand are presented followed by a final suggestion for modified rules for flexural capacity design of multi-storey walls. The dynamic shear demand is shown and related to the traditional capacity design rules.

Based on the results from the nonlinear dynamic analysis using the macro model, the major conclusions of this paper are as follows:
- The dynamic curvature ductility demand in the plastic hinge at the base of the wall may have a different distribution over various height to length aspect ratios of the wall than anticipated from static analyses.
- For low aspect ratios, the dynamic curvature ductility demand corresponds well to the static values, whereas for high aspect ratios the dynamic curvature ductility demand will generally be lower than the static values.
- The dynamic moment demand may have a different distribution over the height of the wall compared to the assumptions from the equivalent static force calculation.
- In the mid and upper storeys, higher moments may develop than anticipated from the static force calculation.
- In order to provide a safe design based on the dynamic moment distribution, a modified flexural strength distribution may be applied which specifically aims at elastic behaviour in areas above the plastic hinge zone.
- By the proposed flexural design rules a clear physical limitation of the plastic hinge zone is achieved where special detailing for plastic deformations is necessary.
- The dynamic shear demand at the wall base may be significantly higher than proposed in previous capacity design provisions, which may have to be modified in the more conservative direction.

REFERENCES


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Fig. 1—Typical multi-story structural wall with definitions

\[ L_p = \max(L_w, H_w/6) \]

Fig. 2—Elastic design response spectra (5 percent damping) for stiff ground (solid line) and for medium stiff ground (dotted line) according to SIA 160 (4)
Fig. 3—Macro model for global simulation of structural walls

Fig. 4—Hysteretic model for outer vertical springs of macro model
Fig. 5—Relation between local (outer vertical spring) and global flexural stiffness reduction of macro model.

Fig. 6—Hysteretic model for horizontal spring of macro model.
Fig. 7—Plan of eight-story building

Fig. 8—Elevation of eight-story building
Lateral Force Transfer in Buildings

Fig. 9—Cross section of structural wall at plastic hinge (restricted ductility design)

Fig. 10—Cross section of structural wall at plastic hinge (full ductility design)

Fig. 11—Distribution of flexural strength proposed in (1, 2)
Fig. 12—Discretization of eight-story wall

Fig. 13—Artificially generated ground motion compatible to SIA 160 design response spectrum, Zone 3b and medium stiff ground of Fig. 2
Lateral Force Transfer in Buildings

Fig. 14—Relation between wall aspect ratio and curvature ductility demand for given design displacement ductility proposed in (1, 2)

Fig. 15—Curvature ductility demand at wall base during nonlinear time history analysis, restricted ductility design ($\mu_\Delta = 3$)
Fig. 16—Curvature ductility demand at wall base during nonlinear time history analysis, full ductility design ($\mu_\Delta = 5$)

Fig. 17—Relation between wall aspect ratio and curvature ductility demand for given design displacement ductility resulting from nonlinear time history analysis of the eight story wall
Fig. 18—Bending moment at wall base during nonlinear time history analysis

Fig. 19—Distribution of maximum bending moments reached from nonlinear dynamic analysis of eight-story wall, restricted ductility design: aspect ratio 5.33 (left), and aspect ratio 8.0 (right)
Design strength with nominal minimum reinforcement and zero normal force

Demand from capacity design rules according to (1, 2)

Demand from nonlinear time history analysis taking into account steel strain hardening

Moment from equivalent static force calculation $M_E$

Design strength $M_R \geq \gamma_R M_E$

Fig. 21—Flexural strength of eight-story wall according to (1, 2), aspect ratio 5.33
Design strength with nominal minimum reinforcement and zero normal force

Proposed design strength in elastic region

Overstrength $M_o$

Demand from nonlinear time history analysis

Moment from equivalent static force calculation $M_E$

Design strength $M_R \geq \gamma_R M_E$

Fig. 22—Proposed flexural strength of eight-story wall, aspect ratio 5.33

Design strength with nominal minimum reinforcement and zero normal force

Unintended yielding caused by moment demand not covered by existing capacity design rules

Demand from capacity design rules according to (1, 2)

Demand from nonlinear time history analysis taking into account steel strain hardening

Moment from equivalent static force calculation $M_E$

Design strength $M_R \geq \gamma_R M_E$

Fig. 23—Flexural strength of eight-story wall according to (1, 2), aspect ratio 8.0
Design strength with nominal minimum reinforcement and zero normal force

Proposed design strength in elastic region

Overstrength $M_o$

Demand from nonlinear time history analysis

Moment from equivalent static force calculation $ME$

Design strength $MR \geq \gamma_R ME$

Fig. 24—Proposed flexural strength of eight-story wall, aspect ratio 8.0

Fig. 25—Proposed strengthened cross section of elastic region above the plastic hinge (four additional bars D20 at each end)
Fig. 26—Proposed transition between plastic hinge and strengthened elastic region showing elevation (left) and vertical section A-A (right)

Fig. 27—Base shear versus time for eight story wall with aspect ratio 5.33 and restricted ductility design
Elongation in Ductile Seismic-Resistant Reinforced Concrete Frames

by R. C. Fenwick and B. J. Davidson

Synopsis: To survive a major earthquake, current practice requires seismic resistant frames to be designed to be ductile. To achieve the required level of ductility in multi-storey frames, the majority of the potential plastic hinge zones are located in the beams. The inelastic rotation, which may develop in these zones, arises predominately from the tensile yielding of the reinforcement. The associated compressive strains are small and as a consequence elongation occurs. Test results show that elongations of the order of 2 to 4 percent of the member depth develop in plastic hinge zones of beams subjected to cyclic loading before strength degradation occurs. The factors influencing elongation are reviewed. The results of a time history analysis, in which elongation effects are modeled, shows that this action, which is neglected in current design practice, has important implications for the detailing of columns and the design of supports for precast components and external cladding.

Keywords: axial loads; beams (supports); ductility; earthquake-resistant structures; frames; hinges (structural); reinforced concrete; shear properties
Appreciation

It was my good fortune, through an unexpected chain of events, to become Tom Paulay's first post graduate student. This occurred about six months after Tom joined the staff at Canterbury and it was before the many commitments, which Tom now has to other organisations, had developed. As a result I had the benefit of much more of Tom's time than could possibly have been the case with later post graduate students. We had numerous lengthy and enjoyable discussions on the topic of my research, shear in reinforced concrete, and on a wide range of other topics. Often these sessions would start about 4.30pm and finish after 6pm, when we would both hurry home. I would offer the explanation for my late arrival to my mother that Tom had kept me talking. Years later I learnt from Herta that Tom used to arrive home with the explanation that Richard kept him talking!

I owe much to Tom. His enthusiasm for structural design is infectious, as is his insistence of finding a simple physical model for the purposes of design or interpreting experimental results. Whenever possible on my visits to Christchurch I call in to see Tom. On these occasions we have had many stimulating and unrestrained discussions which have been of very great value to me. I certainly hope there will be many more of these occasions in the future. He has a very warm interest and concern for the welfare of his former students and their families.

Structural engineering in New Zealand owes much to Tom. Firstly for more than two and a half decades a large portion of our Civil Graduates have been in classes taken by Tom. He is renowned for his ability to get the class to work hard and long on the design projects, his concern for students welfare and the energy and enthusiasm with which he puts into his teaching. Secondly he played a major role in developing the impressive research programme into the seismic resistance of concrete structures at Canterbury. From this has come many important ideas and concepts which have been incorporated into N.Z. codes of practice and in recent years increasingly into a number of overseas codes. Thirdly he is much respected for his generosity with his time. Even when under pressure of impending deadlines he has been very willing to make time to discuss structural problems with students, previous students or practising engineers.

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INTRODUCTION

Current design practice requires multi-storey frame buildings to be designed to perform in a ductile manner in the event of a major earthquake (1,2,3). To achieve the required level of ductility frames are detailed so that a beam sway mode develops in preference to a column sway mode. This objective is achieved in codes of practice by requiring the sum of the flexural strengths of the columns at each beam column joint to exceed by some margin the sum of the combined beam flexural strengths. With this arrangement, in a major earthquake the vast majority of the plastic hinges will be located in the beams. It is the behaviour of these zones that largely determines the dynamic performance of these structures after initial yielding has occurred.

In the paper, the effects of elongation due to the formation of plastic hinges in the beams of ductile frames are discussed. This aspect has received very little attention in the literature and it has been ignored in the vast majority of time history analyses that have been made to develop design procedures. The mechanisms causing elongation are described and the different factors which may influence its magnitude are investigated in beam tests. A model of a potential plastic hinge zone, which allows elongation effects to be predicted, is described, and it is used in the time history analysis of a six storey frame. By repeating the analysis with a different plastic hinge model, which neglects elongation effects, the significance of this action on the seismic performance is demonstrated.

The design concept of providing a hierarchy of strengths in a ductile frame to ensure that a beam sway mode develops in preference to a column sway mode is relatively new. As a relatively small number of structures have been designed and constructed on the basis of this concept, little practical experience has been gained from their actual performance in severe earth-
quakes, in which ductility demands comparable to the design level have been sustained. As the stock of these structures increases this experience will be gained. This lack of practical experience makes it important to examine the performance of individual structural components so that realistic models can be developed for use in time history analyses. Such work is essential in developing satisfactory code rules. Inadequacies in the modeling can lead to incorrect predictions of behaviour and design rules which may not ensure satisfactory performance in a major earthquake.

**PLASTIC HINGES IN BEAMS**

*Plastic Hinge Types*

Two different forms of plastic hinge, namely reversing and unidirectional, can develop in the beams of seismic resistant frames during a severe earthquake. These are illustrated in Figs. 1 and 2, for a beam with uniform flexural strengths along its length.

When the seismic actions on a beam increase to a critical level, two plastic hinges develop, one of which sustains a negative bending moment and the other a positive bending moment. The locations of these plastic hinges depends upon the relative magnitudes of the maximum seismic shear that can be sustained, and the shear arising from the gravity loads supported by the beam. Where the maximum seismic shear is greater than the gravity load shear, the positions of maximum positive and negative bending moment are located against the column faces and the plastic hinge zones also form in these locations, as is illustrated in Fig. 1. With a reversal in the direction of the seismic forces the sign of the bending moment acting on each plastic hinge changes. Hence, positive and negative plastic hinge rotations are imposed on the same hinge zones. These are referred to as reversing plastic hinges.

For the case where the gravity shears exceed the maximum seismic shear force, a point of zero shear force occurs in the span. This defines the location of the maximum positive bending moment and hence the location of the positive moment plastic hinge. As illustrated in Fig. 2, when the structure sways to the right, a negative moment hinge forms against the right hand column face and the positive moment hinge develops in the span on the left hand side of the beam center-line. With the reversal in the seismic actions, the negative moment plastic hinge forms against the left hand column and a further positive moment plastic hinge forms in the span on the right hand side of the center-line. Each of the four plastic hinges in the beam sustains inelastic rotations of one sign only. They are referred to as unidirectional plastic hinges. This form of plastic hinge can be expected to develop in many medium to low rise frame buildings.
In severe earthquakes, where the frames provide lateral resistance and the beams support gravity loads.

In beams which develop unidirectional plastic hinges in a severe earthquake, the negative moment inelastic rotations are sustained in the plastic hinges adjacent to the column faces and the positive moment inelastic rotations are sustained in the spans. Each inelastic lateral displacement causes additional inelastic rotations to be sustained by two of the plastic hinges. As the earthquake progresses, so the rotation in each hinge progressively increases together with the beam deflection, as is illustrated in Fig. 2(d).

During the passage of an earthquake the locations of positive moment plastic hinges can be expected to vary along the span, due to variation of the vertical seismic forces acting on the beam and the magnitude of the strain hardening sustained in the negative moment plastic hinges. Furthermore, as the maximum positive bending moments occur in locations of low shear, the associated yielding may be expected to spread along an appreciable length of beam, generating only small strains in the reinforcement and relatively little strain hardening.

The situation is very different for the negative moment plastic hinges. These are confined to short lengths in high shear zones at the ends of the beam and consequently high curvatures and significant strain hardening occurs in these plastic hinges.

ELONGATION IN PLASTIC HINGES

Previous Research

The behaviour of reversing plastic hinge zones in concrete beams has been extensively studied. However, the elongation effects in these have received little attention. The possibility of unidirectional plastic hinges forming in seismic resistant frames has been largely neglected in structural testing. Frequently where models have been developed for time history analyses the possibility of unidirectional hinges forming in the beam spans has been overlooked and as a result, frame strengths have been over-estimated and plastic hinge rotations under-estimated.

Elongation effects are generally not apparent in structural tests on statically determinate units, such as beams or beam-column sub-assemblies. In these situations no reaction is induced due to the elongation and as a consequence it is easily overlooked. However, the situation changes when indeterminate sub-assemblies are tested. A clear example of this occurred in
the test of a seven storey building (4) where the lateral force resistance was provided by a combined frame-wall system. Elongation occurred in the wall due to the formation of a plastic hinge at its base. This increase in length was restrained by the surrounding columns, which went into axial tension, with an axial compression force being induced in the wall. As a result the lateral strength of the structure was very significantly increased, though there was a loss of ductility. The foundation forces were very different from those predicted by a conventional analysis, and if the structure had not been constructed on a strong floor, a premature non-ductile failure could have been anticipated in the foundations.

Zerbe and Durrani (5,6,7) found in simulated seismic lateral load tests on a number of indeterminate two bay beam-column and column-slab sub-assemblies that the behaviour was appreciably different from that which could be expected from test results obtained from individual statically determinate elements. In their tests a lateral load was applied to a stiff distribution beam, which was connected by pin joints to the tops of the columns. The bottom of each column was pinned to a rigid base. The test arrangement is shown diagramatically in Fig. 3. With this test arrangement, elongation of the beam due to plastic hinging was restrained by the flexural stiffness of the short columns. The axial forces induced in the beams improved their flexural strength together with the performance of the beam-column and column-slab joints. Such advantageous restraint can not be expected in frames, except to a very limited extent in the first floor beams where the foundation provides some restraint.

For a number of years elongation measurements have been made in structural tests carried out at Auckland and Canterbury Universities (8,9,10,11). These show that elongations of the order of 2 to 4 percent of the member depth can be expected in reversing plastic hinge zones before strength degradation occurs.

Beam Tests at University of Auckland

In this section elongation measurements obtained in several series of beam tests are described. In all cases the beams were tested as simple cantilevers springing from an anchorage block, which was prestressed to the floor. The testing arrangement, which is shown in Fig.4 for the beams M1 and M2U, was typical of that used in all tests. Displacement transducers, which were attached via studs to the longitudinal reinforcement, enabled shear, flexural and elongation measurements to be obtained along the length of the member.
The cross-section of all the beams was 500 mm by 200 mm, though in one case (T3T) a composite slab was added. Additional 10 or 12 mm bars were welded to all the beam longitudinal reinforcement where it entered the springing block, to ensure that the plastic hinge was confined to the beams. Details of the beams are given in Figs. 4 and 5 and in Table 1. In all cases the reinforcement details in the plastic hinge zones complied with the requirements contained in the UBC-91 and NZS 3101-82 concrete codes (1, 12).

At the start of all tests two "elastic" load cycles were applied to the beam. In these the maximum load in each direction was taken to three quarters of the value which would generate the theoretical ultimate flexural strength at the critical section of the beam. The average of the four maximum displacements was found and the ductility one displacement was defined as this value divided by three quarters. In the standard loading sequence, which was followed unless noted otherwise in Table 1, the elastic load cycles were followed by two complete load cycles, in which for each cycle a displacement ductility of 2 was applied in each direction. From here pairs of load cycles to displacement ductilities of 4 and 6, with further cycles to ductilities of 6 or 8 and 10 were applied.

The loading cycle for beam M2U was modified so that a unidirectional hinge was formed, in which inelastic deformation was sustained in one direction only. In this case the elastic load cycles were followed by pairs of load cycles, in which the downward deflections were taken to 2, 4, 6, 8, 10 and 12 displacement ductilities, but the upward load was limited so that the bending moment at the critical section just reached three quarters of the positive theoretical flexural strength.

Two of the beams were tested dynamically, so that the strain rates were comparable to those that could be expected in a major earthquake. The loading cycles used in these were a little different from the standard cycles in that after each peak displacement in each direction a few additional load cycles to smaller displacements were added before the next peak displacement was applied in the opposite direction (10).

**Strains in Unidirectional and Reversing Plastic Hinges**

Two identical beams were build and tested. The details are shown in Fig. 4 and Table 1. The first of these, M1, was tested with the standard load history to form a reversing plastic hinge, while in the second one, M2U, was tested with the modified load history, to form a unidirectional hinge (see previous section). In the beam with the reversing hinge, failure occurred in the first load cycle at a displacement ductility of 10. With the unidirectional plastic
hinge failure occurred in the first cycle of the displacement ductility 14. Some of the displacement measurements made on the longitudinal flexural reinforcement in the plastic hinge zones of these two beams are shown in Fig. 6. It can be seen that the behaviour of the two plastic hinge zones was very different. With the unidirectional plastic hinge, the strains in the bottom bars which were not yielded, were negligible compared with those in the top bars. In this case the elongation, measured at the mid-depth of the beam can be assessed in terms of the rotation, \( \theta \), the plastic hinge sustains, by the expression -

\[
\text{elongation} = \theta \left( d - d' \right)/2 \quad \text{(Eq. 1)}
\]

where \( d - d' \) is the distance between the centroids of the top and bottom reinforcement.

With the reversing plastic hinge, in the first inelastic displacement, the compression reinforcement sustained a small compressive strain. With the reversal of the loading direction, the reinforcement in the new compression zone, which had yielded in tension in the previous half cycle, did not yield back to allow the cracks to close. With each subsequent load cycle the reinforcement in the compression zone increased in length until close to the failure when the bars buckled in compression. The elongation in the reversing plastic hinge zone arises from two causes; namely the extension of the longitudinal reinforcement in the compression zone, \( e \), and the rotation sustained by the zone. It is given by the expression -

\[
\text{elongation} = e + \theta \left( d - d' \right)/2 \quad \text{(Eq. 2)}
\]

From the test results shown in Fig. 6 it can be seen that the elongation associated with the extension of the compression zone reinforcement was approximately twice that associated with rotation in the ductility 4, 6 and 8 load cycles.

There are two reasons why the magnitude of \( e \) increases with load cycling in a reversing plastic hinge zone. Firstly, when a deformed reinforcing bar yields in tension, extensive cracking occurs in the surrounding concrete. This causes the concrete to dilate. In addition aggregate particles become wedged in the cracks. To close the cracks an appreciable force has to be applied to the concrete. Secondly, as the direction of loading reverses sets of intersecting diagonal cracks develop right through the beam. The only viable shear resisting mechanism in this situation is provided by a truss like action, with the stirrups going into tension and diagonal compression forces being sustained in the web of the beam, as illustrated in Fig. 7. The equilibrium requirements at a normal section show that the flexural compression force, \( C \), is always smaller than the corresponding tension force, \( T \), due to the longitudinal component of the diagonal compression forces. Both these actions
lead to the compression force in the compression zone reinforcement being less than the flexural tension force. As a result inelastic rotation occurs more by the tensile reinforcement extending, than contraction of the reinforcement in the compression zone. Thus the value of \( e \) increases with each cycle until buckling of the reinforcement occurs.

The elongation measurements made on the two beams, M1 and M2U, at the peak displacements in the load cycles, are shown in Fig. 8, together with the values predicted by Eq. 1. It can be seen that the elongation in the unidirectional plastic hinge is accurately predicted. However, for the reversing hinge the values are greatly underestimated by this equation. In this case approximately two thirds of the elongation arises from the extension, \( e \), of the reinforcement in the compression zone. The shear deformation in the reversing hinge is greater than that in the unidirectional hinge. This resulted in much smaller rotations being sustained in the former beam than latter beam at comparable load stages (13).

Influence of Moment to Shear Ratio, Beam Details and Axial Load on Elongation

If the monotonic stress-strain relationships are known for the reinforcement and the concrete, conventional flexural theory can be used to predict the moment rotation and elongation characteristics of unidirectional plastic hinges. However, this does not hold for reversing plastic hinges. The cyclic yielding of the reinforcement changes its stress-strain characteristics, and the extensive cracking associated with this cyclic yielding modifies the stress-strain behaviour of the concrete. In addition the shear resisting mechanism, which is illustrated in Fig. 7, has an appreciable influence on the response of reversing hinges. In the remainder of this section the influence of different factors on elongation in reversing plastic hinge zones is investigated by reviewing the results of beam tests.

To investigate the effect of the moment over the shear force times effective depth ratio (M/Vd) on elongation in reversing plastic hinges, the results of tests on beams F1 to F4 (see Table 1) were examined. These beams had equal top and bottom steel areas. To vary the shear stress level the length of the shear span was changed between tests. Beam F1 had the highest shear stress level, with a M/Vd ratio of 2.1. The corresponding values for beams F2 to F4 were 3.0, 3.9 and 4.8 respectively. To enable the results to be compared in a consistent manner the displacement ductilities were related to the reference point, which was located 1100 mm from the springing. The ductility one displacements were 5, 6, 4.5 and 5 mm respectively for the beams F1 to F4. From the results in Fig. 9 it can be seen that the M/Vd ratio had little effect on
the elongation. The high shear stresses sustained in the shortest beam \( \frac{M}{V_d} = 2.1 \) caused this member to degrade prematurely in strength and stiffness, which resulted in smaller elongations in the ductility 8 cycles.

In Fig. 10 the effect of varying the section shape and of having different longitudinal reinforcement areas on each face is investigated. Beam T,3T was a tee beam, with each outstanding flange being reinforced with 5 deformed 10 mm bars. Measurements indicated that all bars exceeded the yield strain during the test. The average elongation obtained in the upward and downward displacements was not appreciably influenced by the ratio of the longitudinal reinforcement areas on each side of the beam or the addition of the composite flanges. The elongation did increase in half loading cycles where the smaller area of reinforcement was in tension and it reduced when this area was in compression. Overall, however, the slab had no significant effect on elongation. The slab reinforcement, which yields in tension on the downward loading acts to prop the cracks open when the loading direction reverses. Recent tests on three on three bay frames with and without slabs have confirmed that elongation is not restrained by the presence of a cast in situ slab(14).

In Fig. 11 the effect on elongation of applying axial loads to plastic hinges is illustrated. In these tests the ratio of the axial load level to gross cross-sectional area times the concrete strength \( \frac{P}{A_c f_c} \) varied from zero to 0.145. It can be seen that applying the highest axial load level reduced the elongation to approximately one third of the value which would be expected in a beam without axial load.

**PLASTIC HINGE MODELS**

The sub-assembly for modeling the potential plastic hinge zones in the beams is illustrated in Fig. 12(a). At the potential plastic hinge position two rigid flexural members are mounted on the beam. These members, which are 20 mm apart, are joined by "c" and "r" truss members and the "s" beam member. The r and c members represent the concrete and longitudinal reinforcement on each side of the beam. The assumed stress strain characteristics of these are shown in Fig. 12(b). When the concrete is subjected to tension a crack forms and its load carrying capacity is lost until the crack closes. The reinforcement is assumed to behave in a bi-linear relationship. The function of the s member is to transmit shear between the rigid flexural members. It is held in position at one end, with an axial load release. This particular method of modeling unidirectional plastic hinges has been found to give realistic predictions of elongation, and a reasonable prediction of the moment rotation characteristics (15). It also provides a realistic way of modeling the effect of axial load on the flexural strength of the beam for axial
forces in the range of \(0.05 \frac{A_f}{t_c}\) in tension and \(0.15 \frac{A_f}{t_c}\) in compression.

To assess the significance of elongation on the seismic performance of a structure, by means of time history analyses, it is necessary to duplicate the analyses both with and without elongation effects being modeled. The non­elongating sub-assembly for a plastic hinge is obtained by making a number of changes to the elongating model. The first of these is to give the "s" member a high axial stiffness and replace the axial load release at one end by a pin. With this arrangement the member resists the axial load. The second change is to reduce the stiffness of the "c" truss elements so that they carry negligible axial forces. A bending moment acting on this model is resisted by equal but opposite forces in the two "r" members. The result is that no elongation occurs and there is no interaction of the flexural strength with axial load. The behaviour with this arrangement is very similar to the plastic hinge model used in the beams of most dynamic analyses, in which the positive and negative bending moment flexural strengths are specified at a node or column face in a beam.

**SEISMIC ANALYSIS OF A MULTI-STOREY FRAME**

*Frame Description*

To investigate the significance of elongation of the seismic performance of a structure a frame was sized. Two computer models were developed for this frame and both were used in time history analyses. In the first computer model the potential plastic hinge zones in the beams were represented by the sub-assembly of elements which allowed elongation effects to be incorporated, while the second model these zones were represented by the non-elongating sub-assembly. In the remainder of this paper these computer models are referred to as the elongating and non-elongating models.

The frame was designed to provide part of the gravity and lateral load resistance for the idealised six storey building with the idealised floor plan shown in Fig. 13. It was assumed that this structure is to be located in the most seismically active region in New Zealand (seismicity approximately equivalent to Zone 4 in UBC-91 code), and on soils of intermediate flexibility (approximately equivalent to \(S = 1.25\) in UBC - 91). The lateral force resistance in the \(x\) direction is provided by the frames located on lines 1, 3 and 5. The frames on lines 2 and 4 were assumed to carry part of the gravity loading from the floors but be flexible with regard to the lateral forces. The frame on line 3 is the one which is analysed. As shown in the figure the precast flooring units are supported directly by the beams. The resultant gravity loading, of 53.7 kN/m acting on the beams at each level, is sufficient to ensure that unidirectional plastic hinge zones develop in the event of a major
earthquake. The member sizes were proportioned to comply with the requirements of the New Zealand loadings code (3).

The structural walls on the lines A and D, in the idealised floor plan, resist the torsional actions and the seismic forces in the y direction. The seismic mass associated with the frame on line 3 is 232 tonnes at each level. This is distributed in the ratio of 1 to 2 to the external and internal columns at each level respectively.

To determine the required beam strengths a gravity load analysis was made together with a modal response spectrum analysis for seismic actions. The seismic analysis was based on a structural ductility factor of 6 (approximately equivalent to an $R_w$ factor of 12.6 in the UBC code). The design flexural strengths of the beams at the potential negative moment plastic hinges were taken as the greater of, 1.4 times the dead load bending moments, or the sum of the dead and live load bending moments plus or minus the seismic bending moments. A very limited amount of moment redistribution was applied to equalise the required negative moment flexural strengths in the beams at each individual level.

Assuming that the actual initial negative bending moment yield strengths were equal to the design values, described in the previous paragraph, the locations and magnitudes of the corresponding positive moment plastic hinge zones were found. This procedure gave the minimum beam flexural (dependable) strengths which would satisfy the requirements of the NZ loadings code (3).

The initial flexural yield strengths of the column were found from the method recommended in the commentary to the NZ concrete code (12). This approach is described in detail in reference (16). The actual column yield strengths used in the analyses were on the conservative side of these values, as it was felt to be unrealistic to have too many changes in the height of the column.

The principal results of the gravity and modal analyses are summarised in Table 2 together with the initial flexural yield strengths of the potential plastic hinge zones. For this particular structure the base shear required by the NZ loadings code was close to 60 percent of the corresponding value from the UBC-91 code. The main reason for this difference lies in the restrictions that the UBC code places on the base shear when the fundamental period is determined by calculation rather than the empirical equation.
Strain Hardening Characteristics, Damping and the Earthquake Ground Motion.

An analysis of the results of a series of reinforced concrete beam tests (9) showed that the increase in strength, $\Delta M$, in reversing plastic hinges above the first yield bending moment, $M_i$, as a result of strain hardening, could be assessed from the expression:

$$\Delta M = 3.75 \theta M_i$$

(Eq. 3)

where $\theta$ is the angle expressed in radians sustained by the plastic hinge. This expression was used to determine the strain hardening characteristics of the potential plastic hinges in the columns and the negative moment plastic hinges in the beams. The coefficient of 3.75 was replaced by 1.0 to give the corresponding strain hardening characteristics in the positive moment plastic hinges.

In all the models the mass and stiffness damping coefficients were selected to give an equivalent of 5 percent viscous damping in the first and second modes.

The analyses were based on the El Centro 1940 N00E ground motion. This record was scaled so that the elastic response obtained from a single degree of freedom oscillator, with a period equal to the fundamental period of the structure, was equal to the design response spectrum value. This gave a scale factor of 1.2.

Results of Time History Analyses

The lateral deflection envelopes for the external columns in the frames with the elongating and non-elongating models are shown in Fig. 14. With the non-elongating model the beams act as stiff ties, which ensures that the deflection profiles of all columns are essentially the same. However, with the elongating model the growth in length of the beams causes the columns to be pushed outwards. From the two analyses it can be seen that the effect of elongation is to increase the interstorey deflection sustained in the lower stories at the external column lines. In addition to this, the maximum plastic hinge rotation at the base of these columns is approximately doubled and an additional plastic hinge is induced just below the first floor beams. Clearly the increase in rotation above that sustained by the non-elongating model would increase dramatically with the number of bays in the frame.

The bowed shape induced in the columns resulting from the elongation of the beams increases the bending moments which induce flexural tension stresses on the outside faces of the external columns. The maximum bending
moments in these columns at the beam faces at each level are given in Table 3. The negative sign is assigned to the bending moment which induces flexural tension on the external face of the column. The gravity load action adds to the positive bending moments in the lower levels and the negative bending moments in the upper levels of the columns in each storey.

The elongations which develop at each level during the passage of the earthquake are shown in Fig. 15. At levels 3 and 4 values of close to 175 mm are sustained, while in levels 1 and 6 the corresponding elongation is of the order of 95 mm. For level 6 the portal type action associated with the gravity loads induces appreciable axial compression in the beam. This increases the flexural strength and reduces the elongation. At level 1 the proximity of the ground and the stiffness of the columns act to partially restrain the elongation. The elongation predicted in this analysis are reasonably consistent with values measured experimentally. In a test of a single bay portal frame, with a span of close to 5000 mm and a beam depth of 500 mm, which was loaded so that it formed predominantly uni-directional plastic hinges, elongations of the order of 40 mm were observed when the frame was displaced under cyclic loading conditions by about ± 50 mm. With three bays, this would have given an elongation of the order of 120 mm. Thus these experimentally measured values [17] are comparable with the results of the analysis.

The maximum values of the axial forces in the beams, as predicted from the analyses with the two models, are reproduced in Fig. 16. As noted in the previous paragraph the gravity loads induce axial compression in the level 6 beams and the reaction to this generates some axial tension at level 5. The predicted magnitudes of these actions at these levels are similar for both models. In the other levels the difference in the predictions of the two analyses shows the effects of elongation. Axial compression is induced in the beams at level 1, and a consequence of this and the resultant bowed shape of the columns, is that significant axial tensile forces are induced in the beams at levels two, three and four. The corresponding axial forces in the non-elongating model are small.

The maximum negative and positive bending moments acting in the beams are compared with the first yield bending moments (neglecting axial load) in Fig. 17(a) and the negative moment plastic hinge rotations in Fig. 17(b). For the negative bending moments the strain hardening causes approximately a 20 percent increase in the maximum moment that is sustained. This value is consistent with the high inelastic rotation demand, which develops in unidirectional plastic hinges. In this case the plastic hinge rotation demands reached 3.2°; a value which is close to the limit that can be sustained by a beam detailed to satisfy the requirements contained in the UBC-91 or the NZ Concrete Code-82 (1,12). In Fig. 17(c) the maximum vertical deflection sustained by the beams mid points at each level are shown. It can be seen that these are of the order of 200 mm over the height of the frame.
The general order of structural actions predicted to arise in this frame as a result of elongation are in agreement with values obtained from previous analyses on a three and a different six storey frames using a number of different earthquake ground motions(15).

**DISCUSSION AND CONCLUSIONS.**

1. Elongation occurs when a plastic hinge forms in a beam. Experimental and analytical studies show that this action, which is neglected in current design practice, has important implications for the seismic performance of ductile reinforced concrete frame structures.

2. Two forms of plastic hinge may develop in the beams of ductile frame structures in a severe earthquake, namely reversing plastic hinges, where both positive and negative inelastic rotations are successively imposed on the same zone, and unidirectional plastic hinges where the negative moment inelastic rotations accumulate close to the column faces and positive moment rotations at regions in the span of the beam. With the unidirectional plastic hinges each inelastic displacement of the structure causes the plastic hinge rotations to increase in magnitude. A consequence of this is that unidirectional plastic hinges are required to sustain substantially greater inelastic rotation demands than reversing plastic hinges.

3. Measurements made on several series of reinforced concrete beams, which were detailed to satisfy the seismic requirements in the UBC and NZ concrete design codes (1,12) showed that plastic hinges elongate by 2 to 4 percent of the member depth before strength degradation occurs.

4. In unidirectional plastic hinges, the moment rotation and elongation characteristics can be predicted from conventional flexural theory using the stress-strain characteristics of the concrete and reinforcement. However, with reversing plastic hinge zones the situation is more complex. In this case allowance has to be made for the Bauschinger effect in the reinforcement, the contact effect in the concrete associated with the closure of the cracks, the effect of shear and the change in the stress-strain characteristics of the concrete with cyclic loading.

5. Measurements on test beams show that if inelastic rotation is imposed in one direction and this direction is reversed, the cracks in the compression zone do not close unless more than a critical level of axial load is applied to the beam. The magnitude of the critical axial load depends upon the shear force being sustained by the plastic hinge and
the dislocation of the aggregate particles at the cracks (contact effect). The extension of the reinforcement in the compression zone, from one cycle to the next, provides the major contribution to the elongation which occurs in reversing plastic hinges.

(6) Changing the ratio of the top to bottom longitudinal reinforcement, or adding a composite slab to the beam, was found to have very little effect on the elongation which develops with cyclic loading.

(7) A model of a potential unidirectional plastic hinge zone, which is suitable for use in time history analyses and which allows elongation effects to be modeled, is described.

(8) A six storey three bay frame, which was required to provide lateral resistance for earthquake actions and resist gravity loading, was designed to comply with the New Zealand loadings code (12). This frame was modeled in two ways. In the first, elongation in the beam plastic hinges was included and in the second it was neglected. Comparing the results from time history analyses of these two models enabled the structural effect of elongation to be assessed.

The analyses indicated that for an earthquake, which induces the design level of ductility demand, elongation has important implications. In particular, elongation caused the maximum interstorey deflection and the plastic hinge rotations of the column base in the first storey to be doubled. The elongation predicted in the different levels varied from 85 to 175 mm or between 1 and 2.2 % of the beam depth per plastic hinge. Such elongations are consistent with experimentally measured values. Such movements have important implications for the detailing of the supports for precast floor components and the external cladding.

REFERENCES


4. Wight, J.K. (editor), "Earthquake effects on reinforced concrete structures", American Concrete Institute, Special Publication SP84, 1985, 428 p.


16. Paulay, T., "Deterministic design procedure for ductile frames in seismic areas", in Reinforced Concrete Structures Subjected to Wind and Earthquake Forces, American Concrete Institute, Special Publication SP-63, 1980, pp. 357-382.

### TABLE 1 — DETAILS OF BEAM TESTS

<table>
<thead>
<tr>
<th>Ref</th>
<th>Beam</th>
<th>Shear span (m)</th>
<th>( A_{\text{top}} ) (mm)</th>
<th>( f_y ) (MPa)</th>
<th>( A_{\text{top}} ) (mm)</th>
<th>( f_y' ) (MPa)</th>
<th>( P/A_y f_y' )</th>
<th>( M_i ) (kN.m)</th>
<th>( M_i/\ell ) (kN)</th>
<th>( V_i ) (kN)</th>
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<td>10</td>
<td>F1</td>
<td>0.923</td>
<td>5-D20</td>
<td>290</td>
<td>5-D20</td>
<td>0</td>
<td>30.0</td>
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<td>199</td>
<td>321</td>
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<td>-</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>138</td>
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<td>-</td>
<td>-</td>
<td>83</td>
<td>188</td>
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<td>298</td>
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<td>132,92</td>
<td>254</td>
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<td>11</td>
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<td>5-D20</td>
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<td>42.1</td>
<td>200</td>
<td>133</td>
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<tr>
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<td>T2,2</td>
<td>-</td>
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<td>3-D20</td>
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<td>0</td>
<td>37.6</td>
<td>196,127</td>
<td>131,85</td>
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<td>11</td>
<td>T3T (8)</td>
<td>5-D20 + 10-D10</td>
<td>312</td>
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<td>312</td>
<td>0</td>
<td>33.4</td>
<td>291,217</td>
<td>194,145</td>
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<td>235</td>
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<td>T3D (6)</td>
<td>-</td>
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<td>-</td>
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<td>233</td>
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<td>S2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.145</td>
<td>-</td>
<td>295</td>
<td>197</td>
<td>*</td>
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<tr>
<td>15</td>
<td>S3</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>0.039</td>
<td>36.8</td>
<td>232</td>
<td>154</td>
<td>*</td>
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<td>15</td>
<td>M1</td>
<td>1.300</td>
<td>2-D28</td>
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<td>2-D28</td>
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<td>0</td>
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<td>129</td>
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<tr>
<td>15</td>
<td>M2U(7)</td>
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<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>*</td>
</tr>
</tbody>
</table>

(1) Top and bottom longitudinal steel.

(2) \( P \) is the axial load applied during the test.

(3) \( f_y \) is the yield stress of longitudinal reinforcement.

(4) Theoretical flexural strength based on Whitney stress block.

(5) Shear strength provided by stirrups as per ACI 318 Code \( V_i = A_y f_y d/s \).

(6) Dynamic tests carried out at a rate comparable to major earthquake.

(7) Unidirectional plastic hinge test

(8) Tee beam, see Fig.5.
TABLE 2 — PRINCIPAL RESULTS OF GRAVITY LOAD AND MODAL ANALYSIS OF FRAME

<table>
<thead>
<tr>
<th>Structural periods</th>
<th>Proportion of mass participating in mode</th>
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<tbody>
<tr>
<td>$T_1 = 1.65s$</td>
<td>$M_1 = 0.824$</td>
</tr>
<tr>
<td>$T_2 = 0.51s$</td>
<td>$M_2 = 0.104$</td>
</tr>
<tr>
<td>$T_3 = 0.27s$</td>
<td>$M_3 = 0.041$</td>
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Deflection of top level
Deflection at top level times structural ductility factor
Maximum interstorey deflection
Maximum interstorey deflection times structural ductility factor

<table>
<thead>
<tr>
<th>Deflection of top level</th>
<th>44.3 mm.</th>
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</thead>
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<tr>
<td>Deflection at top level times structural ductility factor</td>
<td>266 mm</td>
</tr>
<tr>
<td>Maximum interstorey deflection</td>
<td>10.2 mm</td>
</tr>
<tr>
<td>Maximum interstorey deflection times structural ductility factor</td>
<td>61.3 mm</td>
</tr>
</tbody>
</table>

| (0.018 interstorey ht.) |

<table>
<thead>
<tr>
<th>Beam flexural strengths (kNm)</th>
<th>Column flexural strengths (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>+ve</td>
</tr>
<tr>
<td>-------</td>
<td>-----</td>
</tr>
<tr>
<td>6</td>
<td>-423</td>
</tr>
<tr>
<td>5</td>
<td>-468</td>
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<tr>
<td>4</td>
<td>-545</td>
</tr>
<tr>
<td>3</td>
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<tr>
<td>2</td>
<td>-567</td>
</tr>
<tr>
<td>1</td>
<td>.</td>
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<tr>
<td>G</td>
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TABLE 3 — MAXIMUM BENDING MOMENTS (kNm) IN EXTERNAL COLUMNS

<table>
<thead>
<tr>
<th>Storey</th>
<th>Level</th>
<th>Non-elongation Model</th>
<th>Ratio of Elongating Non-elongating values</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>-ve</td>
<td>+ve</td>
</tr>
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<td></td>
<td>4</td>
<td>-113</td>
<td>463</td>
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<tr>
<td>4</td>
<td>4</td>
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<td>116</td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>-423</td>
<td>429</td>
</tr>
</tbody>
</table>
a) Sway to right

b) Sway to left

c) Bending moments

End of 1st, 2nd etc. cycles?

d) Deflected shape

Fig. 1—Reversing plastic hinges

---

a) Sway to right

b) Sway to left

c) Bending moments

End 1st cycle

2nd cycle

d) Deflected shape

Fig. 2—Unidirectional plastic hinges
Fig. 3—Test arrangement used by Zerbe and Durrani\(^5\)\(^7\)

Fig. 4—Details and instrumentation for beams M1 and M2U

---

**a)** Beam dimensions and reinforcement details

- 2-12mm bars welded to each 28mm bar
- Reversing jack
- 2-D28 (\(f_y = 317\)MPa)
- 10mm stirrups (\(f_y = 306\)MPa)

**b)** Instrumentation on beam

- Displacement transducers
- Displacement transducer mounted between beam and independent frame
- Deflection at load point
- Rod to operate transducer
- Stud welded to bar, gap left around stud

SECTION A-A
All beams except $T_1$, $3T$, $M1$ and $M2U$

Beam $T_3T$

Fig. 5—Reinforcement details for beams

Fig. 6—Longitudinal strains in unidirectional and reversing plastic hinges
Fig. 7—Shear actions in beam

Fig. 8—Elongation in reversing and unidirectional plastic hinges in beam M1

Fig. 9—Effect of varying $M/Vd$ ratio on elongation and $M2U$
Fig. 10—Effect of differing \( A_s / A_s' \) ratios on elongation

Fig. 11—Effect of differing axial load levels in elongation
a) Plastic hinge model

b) Stress-strain properties of truss elements

Fig. 12—Subassembly used to represent a unidirectional plastic hinge

Fig. 13—Idealized plan on six-story building
Fig. 14—Deflected shape envelopes for external columns in frame models

Fig. 15—Development of elongation with time at different levels
Fig. 16—Maximum axial forces sustained by beams in two frame models

Fig. 17—Maximum bending moments, plastic hinge rotations, and vertical deflections in beams of two frame models
Potential Problems in Design for Maximum Flexibility

by A. J. Carr and M. Tabuchi

Synopsis: The New Zealand Standard for design loadings for buildings (NZS4203) was revised in 1992 superseding the earlier standard NZS4203(1984). Some of the most significant changes in the new code are a considerable increase in the allowable interstorey drifts and a marked reduction in the seismic lateral forces for structures with longer natural periods. Designers may now be encouraged to design buildings to the maximum allowable drifts as the resulting buildings will attract smaller lateral loads. Reinforced concrete buildings designed with the new loadings code may be constrained by the minimum reinforcement requirements rather than strength requirement of the loadings code and, as a result, they may have a different distribution of strength capacity from that assumed in the code design. As a result, buildings designed using the capacity design principles may not have the strength distribution that the designer intended. The reasons for this problem are discussed and the effects of the irregular distribution of strength capacity are investigated using inelastic response analysis. It was found that the large reduction of the design lateral forces resulting from the large allowable inter-storey drifts may lead to the problem. The design lateral forces or the deflection limits defined in the new code, NZS4203(1992), may need to be reconsidered.

Keywords: deflection; flexibility methods; loads (forces); shear properties; standards; stiffness; structural design
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INTRODUCTION

The New Zealand loadings code for earthquake design has two limit states, an ultimate limit state where the structure is to survive a major earthquake with some degree of inelastic response and a serviceability limit state where the structure should remain essentially elastic in smaller but more frequent earthquakes.

In New Zealand the design of reinforced concrete framed buildings for earthquake resistance follows the principles of capacity design (Paulay and Priestley 1992). In the ultimate limit state a ductile failure mechanism is selected where plastic hinges are usually assumed to occur at the ends of the beams and at the bases of the ground floor columns. The beams are designed to have a flexural strength required to resist the code lateral forces at these hinge locations and the beam overstrength moments are computed. These overstrength moments are factored up to account for the effects of the higher modes of free vibration on the distributions of moments to the columns above and below the joint. These moments are also factored up to account for possible column understrengths. The flexural and shear strengths of the columns, the shear strengths of the beams and the strengths of the beam-column joints are designed so that the only yield mechanisms that can occur under the design level, or larger, earthquakes is in the selected plastic hinge locations. The frames are then checked to ensure that the displacements and the interstorey drifts are within the code prescribed limits and that the serviceability limit state requirements are satisfied.

The New Zealand Standard for design loadings for buildings (NZS4203) was revised in 1992 superseding the earlier 1984 standard. The new code allows for considerably larger displacements and interstorey drifts and the design lateral force coefficients are considerably reduced for structures with longer natural periods of free vibration.

The changes to the code have meant that there is now a temptation to design building frames to meet the larger allowable drift limits. As these frames are more flexible their natural periods of free vibration are longer and they will attract much smaller lateral forces under earthquake excitation. In some cases the required strengths are such that the necessary reinforcement is governed not by the forces derived from the loadings code but by the minimum allowable steel areas of the Reinforced Concrete Code (NZS3101). The danger is that the resulting strength distribution in the structure may be very different from that
assumed in the design and that unless the capacity design is repeated using the minimum strengths then the desired mechanism of failure may not be ensured. This may result in a non-ductile and possibly catastrophic collapse of the structure in a severe earthquake.

**DESIGN LATERAL FORCE AND DEFLECTION LIMIT**

If the structural design is dominated by the earthquake loading, the structure is required to satisfy the appropriate lateral strength and deflection limits. The design lateral force is, in most cases, defined by the design earthquake spectra and the fundamental period of the structure. In the new seismic loadings code, NZS4203(1992)[1], the concept of equal displacements is adopted to take into account the inelastic behaviour of the structure. The deflection limits which are defined in NZS4203(1992) are used to control the inter-storey drifts.

**Design Lateral Force**

The design lateral force at floor $i$ of a building is defined in NZS4203(1992) as follows:

$$ F_i = 0.92V \frac{w_i h_i}{\sum (w_i h_i)} $$

$$ F_{top} = 0.08V + 0.92V \frac{w_{top} h_{top}}{\sum (w_i h_i)} $$

where

$V = CW$: Base shear force
$C$: Lateral force coefficient
$W$: Total weight
$w_i$: Weight of $i$th floor
$h_i$: Height of $i$th floor

$\sum$: Implies summation from $i=1$ to $n$

The lateral force coefficient, $C$, is given by:

$$ C = C_h(T_1, \mu) S_p R Z L \geq 0.03 $$

where

$C_h(T_1, \mu)$: Basic seismic hazard acceleration coefficient
$T_1$: Fundamental period of free vibration of the structure
$\mu$: Structural ductility factor
$S_p$: Structural performance factor $= 2/3$
$R$: Risk factor varies from 0.6 to 1.3
$Z$: Zone factor varies from 0.6 to 1.2
$L$: Limit state factor 1.0 (ultimate) or 1/6 (serviceability)
The basic seismic hazard acceleration coefficient $C_h(T_1, \mu)$ is determined by the normalised shapes of the uniform hazard spectra which is established by the SANZ Seismic Risk Sub-committee for New Zealand using modified Katayama attenuation relationships[2]. There are three subsoil categories, (a) rock or very stiff soil sites, (b) intermediate subsoil sites and (c) flexible or deep soil sites. Fig.1 shows the basic seismic hazard acceleration coefficient for the intermediate subsoil sites (category (b)). As shown in Fig.1, the basic seismic hazard acceleration coefficient is a function of structural ductility factor $\mu$, which is determined by the deformation capacity of the seismic resisting system. There are upper limits for structural ductility factors for several typical structural systems as summarised in Table 1. The structural performance factor, $S_P$, is a reduction factor based on the fact that in past earthquakes, buildings, on average, perform better than can be predicted by calculation using simplified analyses. The values of $Z$ in Fig.2 are equivalent to the 450 year return period, elastic 5% damped uniform hazard contours for a structural period of 0.2s calculated by the SANZ Seismic Risk Sub-committee. The probability of exceedance of the hazard spectra is varied using a risk factor $R$ and is 1.0 for the buildings of normal occupancy and usage.

In this report, the fundamental periods of all prototype structures were more than 0.7 seconds, and all structures were designed as ductile reinforced concrete frames, which means that the maximum ductility factor $\mu$ is 6. The risk factor $R$ was 1.0, the zone factor $Z$ was 1.2 and limit state factor $L$ used the ultimate limit state value of 1.0. The soil condition was assumed to be an intermediate soil.

**Deflection Limits**

According to NZS4203(1992), where the equivalent static analysis method is used, the inter-storey drifts shall not exceed the following fractions of the corresponding storey height:

\[
\begin{align*}
0.020 & \quad \text{for } h_n \leq 15m \\
0.015 & \quad \text{for } h_n \geq 30m \\
0.020 - \frac{0.005 h_n - 15}{15} & \quad \text{for } 15m < h_n < 30m
\end{align*}
\]

where

\[h_n\text{: Total height of the structure}\]

These inter-storey drifts are calculated from the elastic response to the design lateral forces and are multiplied by a factor equal to the structural ductility factor $\mu$. 
ASSUMPTIONS ADOPTED IN ALL THE PROTOTYPE STRUCTURES

Basic Assumption

The weight of each floor is the same throughout the height of the structure and the storey height is also the same as shown in Fig.3. With this basic assumption, the lateral force applied to each floor, $F_i$, is represented as follows:

$$F_i = V f_i = CW f_i$$

$$f_i = 0.92 \sum \frac{w_i h_i}{(w_i h_i)} = 0.92 \sum i$$

where

$w_i = \text{constant}$

$W = \eta w$

$n$ : Number of floors

$\Sigma$ : Implies summation from $i=1$ to $n$

Note that for the prototype structures, $f_i$ is only a function of the number of storeys.

The fundamental periods of the prototype structures were calculated by the Rayleigh formula which is represented as follows:

$$T = 2\pi \sqrt{\frac{\sum (w_i u_i^2)}{g \sum (F_i u_i)}}$$

where

$u_i$ : Displacement of $i$th floor

$g$ : Acceleration of gravity

With the assumptions and the relationships obtained above, the Rayleigh formula yields:

$$T = 2\pi \sqrt{\frac{w \sum (u_i^2)}{g V \sum (f_i u_i)}} = 2\pi \sqrt{\frac{1}{g n} \sqrt{\frac{\sum (u_i)^2 \sum (f_i u_i)^2}{\sum (f_i u_i)^2}} \frac{1}{C}}$$

In this equation, it is significant that $\sqrt{\frac{\sum (u_i^2)}{\sum (f_i u_i)}}$ is only determined by the distribution of the displacements of the structure and the maximum deflection. If the displacement distribution and number of storeys are given, the fundamental period of the structure may be calculated from:

$$T = A \sqrt{\frac{1}{C}}$$

where $A$ is the function of the maximum deflection and the number of storeys.
When the deflection limit is checked in NZS4203(1992), the elastic displacements are multiplied by the ductility factor. Therefore, the relation between lateral force coefficient \( C \) and fundamental period is represented as follows:

\[
T = A \sqrt{\frac{1}{\mu C}} \quad \text{or} \quad C = \frac{1}{\mu} \left( \frac{A}{T} \right)^2
\]

**Stiffness Distribution of The Prototype Structure**

As mentioned above, the structure which satisfies the basic assumptions of equal storey height, equal storey weight and maximum allowable drift is only controlled by the displacement distribution and the number of storeys. This means that the structure is also controlled by the distribution of inter-storey drift and number of storeys. When the structure is regular, the distribution of inter-storey drift tends to have the typical distribution shown in Fig.4. where the inter-storey drift is represented as the inter-storey drift angle.

The first storey usually has a small inter-storey drift because of the fixity of the column bases. The upper part of the structure also has small drifts due to the smaller lateral forces in the upper storeys. It should be noted that, under a lateral forces, most framed structures deform in a shear type deformation in each storey and the effects of changes in the column lengths are usually small in comparison. In this example, the maximum inter-storey drift is matched to the deflection limit of 0.015 at the 3rd floor. With this inter-storey drift distribution, the relationship between the fundamental period and the lateral force coefficient is obtained for the structure which also satisfies the maximum deflection limit.

It should be noted that this assumption is not without limitations. The equal displacement principle does not assure equal inter-storey drifts in elastic and inelastic structures. The distribution of inter-storey drifts under large lateral loads and inelastic behaviour could be very different from that assumed in an elastic analysis. The inter-storey drifts would be larger than these assumed from the elastic analysis in the lower storeys of the structure. However, the differences in the distribution of inter-storey drift between elastic analysis and inelastic analysis should not be large, provided the structure ductility factor is limited.

**Ideal Base Shear Coefficient**

If the displacement distribution is selected to satisfy the deflection limit as shown in Fig.4, the relationship between the lateral force coefficient \( C \) and the fundamental period \( T \) is represented as follows:

\[
C = \frac{1}{\mu} \left( \frac{A}{T} \right)^2
\]
As seen in Fig.5, this equation gives a relationship which satisfies the deflection limit and ductility factor $\mu$. The example is for a 12 storey frame with a structure ductility factor of 6. The figure also shows the design spectra from NZS4203(1992) for intermediate soil and ductility factor 6.

There is a point in Fig.5 where both lines intersect where the structure, which is designed to this point, would satisfy the lateral force capacity and deflection limit simultaneously. To the left of this point, the designed structure usually has smaller deflections than allowed and has larger member sizes than required for the stiffness. To the right, the structure does not satisfy the deflection limit requirements.

Therefore, it could be said that this point is an ideal point which satisfies the code required lateral strength capacity with the minimum stiffness of the structure. In all cases, the structure must be designed at or to the left hand side of this point or it cannot satisfy the deflection limit of the code.

**PROTOTYPE STRUCTURE DESIGN**

The prototype structures in this report have the floor plan shown in Fig.3. The prototype structures have an 8m span length and 3 bays in the X-direction. The structural design is considered only for this direction as torsional effects are not taken into account in this report. There are two structural frames which are designed for the lateral forces and gravity load. The frames in between the two structural frames carry only gravity load and do not resist any lateral forces. There are three structures with this plan, 4, 8 and 12 storey structures respectively. The inter-storey height is 4m for all the prototype structures. The member sizes for these structures are shown in Table 2.

As shown in Table 2, beams and columns in a structure all have the same section size. For these members, the effective member moment of inertias ($I_e$) were calculated with the following assumptions[3]:

\[
\begin{align*}
&\text{Beams:} & I_e &= 0.35f I_g \\
&\text{Exterior Columns:} & I_e &= 0.6 I_g \\
&\text{Interior Columns:} & I_e &= 0.8 I_g
\end{align*}
\]

where

$I_g$: Moment of inertia for gross section

$f$: Factor for slab effect ($f = 1.3$)

The load conditions assumed are as follows,

<p>| | | |</p>
<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td><strong>Dead Load</strong></td>
<td><strong>Floor:</strong></td>
<td>5.1 kPa</td>
</tr>
<tr>
<td></td>
<td><strong>Beams, Columns, etc:</strong></td>
<td>3.1 kPa</td>
</tr>
<tr>
<td><strong>Live Load</strong></td>
<td><strong>Floor:</strong></td>
<td>2.5 kPa</td>
</tr>
<tr>
<td><strong>For Earthquake Load</strong></td>
<td><strong>Dead Load + 1/3Live Load</strong></td>
<td>9.03 kPa</td>
</tr>
</tbody>
</table>
The width of the structure, B, was chosen to make the structure fit the ideal point design as shown in Table 3. The first storey column bases were considered as fixed ended. The effects of rigid end-blocks and shear deformations were not taken into account in the elastic analyses.

For the 12 storey structure, the lateral force coefficient is below the code lower limit of 0.03. However, to keep the consistency of the ideal point design concept, 0.022 is used for the design lateral force coefficient.

The control deflections which were obtained at the ideal points are summarised in Table 4. The inter-storey drifts were calculated with the lateral forces which were multiplied by the ductility factor $\mu$.

The beams and columns are designed according to the concept of capacity design[3] and all the hinge mechanisms assumed in design are beam sway mechanisms. In general, the member sizes required to meet the deflection limits were much larger than those required from strength considerations. For almost all the beams and the column bases, the required longitudinal reinforcement was controlled by the minimum reinforcement requirements in NZS3101[4]. Therefore, the members have much greater overstrength factors than expected from the capacity design. Table 5 shows the system overstrength factor obtained for each storey and the bases of the columns in the first storey.

The ideal system overstrength factors are 1.39 for the beams and 1.22 for the column bases. If the system overstrength factors are much different from these ideal values, the assumed moment distribution associated with the static analysis could be very different from the capacity design distribution. This may result in a serious deviation in the behaviour of actions during the earthquake excitation from those assumed in the capacity design approach.

**EFFECT OF IRREGULAR SYSTEM OVERSTRENGTH FACTOR**

The effects of an irregular system overstrength factor is examined in three different ways, a simple method, a pushover analysis and an inelastic dynamic analysis. Note that the model structure is slightly different from which is described above in that the design lateral forces were those of NZS4203(1984)[5]. However this model structure also has the same trends as those presented above in their system overstrength factors.

**Simple Method**

The lateral capacity of the structure is calculated by the summation of the beam hinge moments in one level and the average contraflexure point in the columns above and below obtained from the static analysis used to calculate the design moments. Fig.6 shows this concept graphically.

Fig.7 shows the comparison of design lateral force and lateral force capacity of the model structure. In the upper part of the structure, the increase of capacity which was caused by the excessive system overstrength was seen, and also the excessive column base capacity was obtained as shown in the Figure. The structure had a system overstrength factor 3.06 at the column base, and the ratio of tension reinforcement area to the column gross area was 0.01. The
lateral capacity was calculated by the ideal strength of the member and it should be multiplied by $1/0.9$ to compare with the dependable design capacity.

**Pushover Analysis**

A pushover analysis was carried out for the same structure using RUAUMOKO[6]. The vertical distribution of the lateral force was that of the design lateral forces. All members assumed elasto-plastic moment-curvature relationships and the ideal strength was used for the strength of the members.

Beam hinges appeared first in the lower part of the structure, then beam sway mechanisms spread to the upper storeys. Column hinges also appeared at approximately 80% of the maximum load. The failure step in the analysis was in the formation of a column sway mechanism at the 10th storey resulting from the excessive beam strength in the upper storeys. The lateral force capacity obtained from the pushover analysis was shown in Fig.7.

Because of the vertical distribution of the horizontal forces applied in the analysis, the capacity distribution is the same as that of the design lateral force. However the intensity of the base shear capacity was about 45% larger than the design base shear. Even if the effects of using the ideal strength in the analysis are taken into account, this increase of base shear capacity is not explained. On the other hand, this increase of base shear capacity in the pushover analysis is very small compared with that from the simple method mentioned above.

**Inelastic Dynamic Analysis**

The inelastic dynamic analysis was carried out using RUAUMOKO for the selected excitations of Bucharest(1977 NS), El Centro(1940 NS), Pacoima Dam(1971 S14W). All members have the same elasto-plastic moment-curvature hysteresis rule, and the hinge length was assumed as 0.7 times of the member depth. Fig.8 shows the relationship between the maximum storey shear force and the maximum inter-storey drift for the first storey for the three excitations. It can be seen from this figure that there are two lines which show the behaviour before and after yielding of the storey. Although this relationship becomes confused in the upper storeys of the structure because of the of higher mode effects, it is still possible to define a yield point, though the yield points which were defined in this way are not very precise. However, the capacities which were calculated with this approximate method directly reflect the behaviour of the structure in the real earthquake excitation.

With this approximate method explained above, the capacity distribution obtained from the inelastic dynamic analyses is shown in Fig.7. The structure has excessive capacity in the lower storeys, and the capacity distribution of the original structure became different from the distribution of the design lateral force.
Comparison of the Three Methods

In this section, the effect of the irregular system overstrength factor is discussed with the three methods of analysis, the simple method, the pushover analysis and the inelastic dynamic analysis. Of these three methods, the inelastic dynamic analysis gave the most realistic behaviour in the earthquake excitations. Therefore, the results of the other two methods are discussed on the basis of the results obtained from the inelastic dynamic analysis.

As the simple method is based on the design moment distribution, the capacity distribution of the structure could be very different from that of the design lateral forces when the distribution of the system overstrength factor was irregular. In this case, the excessive capacity at the column bases in the original structure causes a great difference in the first storey capacity. The reason for this is that the assumed contraflexure point of the first storey column was not applicable for this structure. As shown in Fig.9, the contraflexure point of the real action was at a much higher elevation than that assumed. The column cantilever action dominates these column actions. It would be important to note that this problem would not be detected by the elastic analysis itself. The member stiffnesses were assumed as cracked sections, and the contraflexure point of the first storey columns were located within the first storey in the elastic analysis. The excessive capacity of the column base moved the contraflexure point to a higher level when satisfying equilibrium after yielding of the beams.

The pushover analysis gave much better results than simple method. Even if there is a large irregularity of the overstrength factor, the shape of the strength capacity is equal to that assumed in the design lateral force. However, this shape of the strength capacity is also very different from the that obtained in the inelastic dynamic analysis as shown in Fig.7. It can be easily seen from the distribution of the system overstrength factor that the strength capacities in upper storeys obtained in the inelastic dynamic analysis were much larger than those obtained in the pushover analysis. As the distribution of the applied lateral force was maintained in the pushover analysis, it could not have the real strength capacities which would be affected by the large system overstrength in upper storeys.

THE EFFECTS OF OTHER CONDITIONS

The prototype structures discussed above cover only a portion of the many structures which are designed with the code. The basic prototype structures were of 8m span, 3 bays, 4m storey height, and the number of storeys were 4, 8 and 12. It is important to investigate how variations on these parameters would affect the result found above.

Fundamental Period

The prototype structures in this section were designed at the ideal design point in that they have the minimum stiffness which can satisfy the deflection
Therefore, they have long fundamental periods when compared with more traditional structures. The longer fundamental period makes the design lateral load smaller, and as a result, the overstrength factor becomes larger. However, to reduce the fundamental period, the weight of structure must be reduced or the stiffness of the structure must be increased, and this approach also results in the larger overstrength factors. It is important to show how the overstrength factors behave to the left or right of the ideal design point. To investigate these effects, prototype structures which have 0.5 and 1.5 of the weight of the original prototype structures were designed.

It could be said that the structures to the left of ideal point have larger system overstrength factors and that those to the right of ideal point have smaller system overstrength factors than the original prototype structures. Even if the fundamental period of the structure decreases and the lateral force coefficient become larger than that of ideal design point, the weight of the structure also decreases and as a result the design lateral force become much smaller and the system overstrength factors become much larger than that of ideal design point. As mentioned before, the structure to the right of the ideal point does not satisfy the deflection limit. Therefore, the structure designed at the ideal design point has the smallest system overstrength factors.

**Number of Bays**

The capacities of almost all members in the prototype structures were determined by the minimum reinforcement requirements. It may be appropriate to increase the number of bays and reduce all member sizes. The axial forces in the columns would be reduced and the capacity of all members would be reduced. However, at the same time, the design action for each member would be reduced. To investigate this approach, three prototype structures which have 4m spans and 6 bays were analysed.

There is only about a 3% difference between the design lateral forces for these structures and those of the original prototypes. The displacement distributions were also similar to those obtained earlier. The ideal design point where the design lateral force and the deflection limit were satisfied simultaneously were almost the same as to the ideal design point for the original prototype structures. The system overstrength factors obtained from these structures were lower than those obtained in the original prototype structures but were still significantly larger than required.

**Storey Height**

All the prototype structures had a 4m storey height which may be considered as an upper limit as most buildings have a storey height of between 3m and 4m. A smaller storey height would change the relationship between the fundamental period and the lateral force coefficient. To investigate the effects of varying the storey height, the structures were redesigned using a 3m storey height.

The effect of storey height was very large, but still, all the members of these structures were again constrained by the minimum longitudinal
reinforcement requirements. The structures with 3m storey height had a better result in terms of the system overstrength factors.

**Stiffness Distribution**

The prototype structures above maintained their member sizes through all storeys. These member sizes were mainly determined by the deflection limits in the lower storeys of the structures. In a real design, a member size reduction is usually used in the upper storeys of the structure. This would reduce the beam overstrength factors and increase the flexibility of the upper storeys. To investigate this effect, the prototype structure was modified to use three different beam sizes.

Although there were significant differences in the distribution of drift angle between modified and original structure, there was only about a 4.5% difference between the relationship between the fundamental period and the lateral force coefficient. The system overstrength factors in the upper storeys are reduced, but, the system overstrength factors in the lower storeys and column bases are reduced by only 4.5%. The design of members in the modified structure were still determined by the minimum longitudinal reinforcement requirements.

**CONCLUSION**

All three prototype structures discussed in this paper were designed at the ideal point at which the design lateral force would give the deflection limits automatically. This means that the given member sizes in the design were all minimum member sizes. At the same time, the structures designed at the ideal design point should have the minimum system overstrength factors as shown above. Even though the design conditions were ideal, almost all member reinforcement were determined by the minimum longitudinal reinforcement requirement. To make matters worse, the members which were defined by such minimum requirements would change the strength capacity distributions of the structure that were to be provided in accordance with the code requirements. It would be worth noting that the beams in the upper storeys were determined by the minimum reinforcement requirement and not determined by the effects of long term load conditions in these prototype structures.

It is essential that the minimum reinforcement strength of the beam provide the capacity design requirements of the columns if predictable hinge mechanisms are to occur. The member sizes which were dominated by the minimum requirement for longitudinal reinforcement were not only unusual but also uneconomical. The problem comes from the excessively small design lateral force together with the large allowable drift limits. The balance between deflection limit and design lateral force does not seem to be appropriate and it is strongly recommended that the design lateral forces should be increased or the deflection limits decreased.
REFERENCES


### TABLE 1 — STRUCTURAL DUCTILITY FACTOR, $\mu$

<table>
<thead>
<tr>
<th></th>
<th>Structural steel</th>
<th>Reinforced concrete</th>
<th>Prestressed concrete</th>
<th>Reinforced masonry</th>
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<tbody>
<tr>
<td>1. Elastically responding structures</td>
<td>1.25</td>
<td>1.25</td>
<td>1.0</td>
<td>1.25</td>
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<tr>
<td>2. Structures of limited ductility</td>
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<td></td>
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<tr>
<td>(a) Braced frames:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(i) Tension &amp; compression yielding</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(ii) Tension yield only</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>(Two storeys maximum)</td>
<td></td>
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</tr>
<tr>
<td>(b) Moment resisting frame</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>(c) Walls</td>
<td>3</td>
<td>3</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>(d) Cantilevered face loaded walls (single storey only)</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>3. Ductile structures</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Braced frames (tension &amp; compression yielding)</td>
<td>6 $\leq \xi \leq$ 10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) Moment resisting frames</td>
<td>6 $\leq \xi \leq$ 10</td>
<td>6 $\leq \xi \leq$ 10</td>
<td>5 $\leq \xi \leq$ 8</td>
<td>4 $\leq \xi \leq$ 6</td>
</tr>
<tr>
<td>(c) Walls</td>
<td>5 $\leq \xi \leq$ 8</td>
<td>4 $\leq \xi \leq$ 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(d) Eccentrically braced frames</td>
<td>6 $\leq \xi \leq$ 10</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

where $\xi = 20(1 - T_1)$ and $T_1$ fundamental period

### TABLE 2 — MEMBER SIZES OF PROTOTYPE STRUCTURE

<table>
<thead>
<tr>
<th></th>
<th>Beam</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 storey</td>
<td>300 x 600</td>
<td>500 x 500</td>
</tr>
<tr>
<td>8 storey</td>
<td>350 x 700</td>
<td>600 x 600</td>
</tr>
<tr>
<td>12 storey</td>
<td>400 x 800</td>
<td>700 x 700</td>
</tr>
</tbody>
</table>

### TABLE 3 — BASIC PARAMETERS OF PROTOTYPE STRUCTURES

<table>
<thead>
<tr>
<th></th>
<th>Width(B) (m)</th>
<th>Height(h) (m)</th>
<th>Coefficient A</th>
<th>Period(T) (sec)</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 storey</td>
<td>6.00</td>
<td>16.0</td>
<td>0.7832</td>
<td>1.50</td>
<td>0.0448</td>
</tr>
<tr>
<td>8 storey</td>
<td>5.20</td>
<td>32.0</td>
<td>0.9172</td>
<td>2.10</td>
<td>0.0320</td>
</tr>
<tr>
<td>12 storey</td>
<td>6.80</td>
<td>48.0</td>
<td>1.108</td>
<td>3.05</td>
<td>0.0220</td>
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</table>
### Table 4 — Assumed Inter-Story Drifts

<table>
<thead>
<tr>
<th>Storey</th>
<th>4 storey Drift (cm)</th>
<th>8 storey Drift (cm)</th>
<th>12 storey Drift (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>1.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>2.41</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>3.14</td>
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<td>9</td>
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<td>3.80</td>
</tr>
<tr>
<td>8</td>
<td>2.00</td>
<td>4.40</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>3.11</td>
<td>4.91</td>
<td></td>
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<tr>
<td>6</td>
<td>4.11</td>
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<tr>
<td>5</td>
<td>4.94</td>
<td>5.69</td>
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<tr>
<td>4</td>
<td>4.50</td>
<td>5.90</td>
<td>5.92</td>
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<tr>
<td>3</td>
<td>6.85</td>
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<tr>
<td>2</td>
<td>7.87</td>
<td>5.71</td>
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</tr>
<tr>
<td>1</td>
<td>5.00</td>
<td>3.29</td>
<td>3.09</td>
</tr>
</tbody>
</table>

4 storey (Drift Limit: 0.0787m)  
8 storey (Drift Limit: 0.0600m)  
12 storey (Drift Limit: 0.0600m)

### Table 5 — System Overstrength Factor

<table>
<thead>
<tr>
<th>Storey</th>
<th>System overstrength factor ($\psi_o$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4 storey</td>
</tr>
<tr>
<td>12</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
</tr>
<tr>
<td>10</td>
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<td>6</td>
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<td></td>
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<td>4</td>
<td>3.38</td>
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<td>2</td>
<td>1.49</td>
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<tr>
<td>1</td>
<td>1.54</td>
</tr>
<tr>
<td>Base</td>
<td>1.81</td>
</tr>
</tbody>
</table>
Fig. 1—Basic seismic hazard acceleration coefficient for intermediate soil sites

Fig. 2—Zone factor $Z$
Fig. 3—Assumed structural layout

Fig. 4—Distribution of inter-story drift angles

Fig. 5—Relationship between fundamental period and lateral force coefficient
Fig. 6—Concept of simple method

Fig. 7—Comparison of strength capacity
Fig. 8—Relationship between maximum story shear force and maximum inter-story drift

Fig. 9—Effects of cantilever action
Seismic Design of Frame Buildings: A European Perspective
by P. E. Pinto and G. M. Calvi

Synopsis: This presentation consists essentially of a review of the aspects considered to be major limitations, from a conceptual point of view if not always from a practical one, in the view of a codified approach to design and, more generally, in our ability of describing the seismic response of structures. The state of progress and the current research efforts on three interrelated subjects are first discussed: capacity design criteria and procedures, definition of the state of collapse and damage of a structure and viable techniques for a probabilistic calibration of safety factors. It should be noted that a fuller rationalization of these aspects is more acutely needed in Europe due to the larger variety of structural typologies, which calls for more refined differentiations and, in turn, for more rational and visible justifications.

In the second part of the paper the specific problem of shear-bending interaction is considered, whose behaviour is qualitatively known, but is neither quantitatively well defined nor commonly implemented in computer programs, and a simple proposal for an analytical model is presented and discussed. The results of some preliminary numerical simulation show interesting results, their main merit consisting in an indication of the relevance of the problems. Similar elements could be easily integrated into non-linear dynamic programs and used for probabilistic calibrations.

The paper is concluded by a brief presentation of a broad experimental and analytical research program just started in Europe to support the final preparation of a unified seismic code.

Keywords: bending; capacity; earthquake-resistant structures; frames; models; reinforced concrete; shear properties; standards; structural design
Paolo E. Pinto has been Professor of Earthquake Engineering at the University of Rome since 1974, where he graduated in 1968. His contributions are in the analysis and behaviour of reinforced concrete structures, and in reliability studies for large systems. He is currently chairman of Eurocode 8 and of CEB Commission III "Design".

G. Michele Calvi is Associate Professor of Structural Engineering at the University of Pavia. He got his M.S. at the UC Berkeley and his Ph. D. at the Politecnico di Milano. His contributions are in seismic testing, analysis and design of reinforced concrete structures. He is presently member of CEB TG III/2 and III/6, and coordinator of the European pre-normative research program for EC8.

TOM PAULAY IN EUROPE

In pursuance of the objective of a harmonized approach to seismic design, in 1979 the Comité Euro-International du Béton assembled an international panel of renowned experts with the task of producing a Model Code for Seismic Design of Concrete Structures. The least renowned among these panelists, who is now writing these lines, was asked to take the chair, with an assigned time limit of three years and a recommendation to pay particular attention to the then recently issued seismic code in New Zealand.

Since twelve out of twenty of the members were from Europe, and the rest from everywhere in the world, the preparation of the drafts was essentially a European affair, while the others were asked to comment and criticize.

New Zealand responded in the person of Tom. Within a short delay after each sending, an annotated copy of the draft was back in Rome, accompanied by a few lines of encouragement. The corrections, at times substantial, were introduced directly into the text, with explanations kept to a minimum, and frequent references to the sources.

I was greatly impressed by this type of approach, at the same time direct, generous and authoritative: every time I rushed to learn from the references, to revise the text, to discuss it with the European colleagues, and then to send it back to Tom, thanking him for the consideration he was giving us and almost incredulous that he would be willing to persist. Which he naturally did, until the conclusion.

This is the story of one of the channels, not a secondary one, through which the New Zealand thought and expertise started and steadily continued to flow to Europe, to such a pervasive extent that we observe today.

To Tom our appreciation for the rare combination of wealth and knowledge with a liberal attitude in communicating them to others.
PROLOGUE: TOWARDS A EUROPEAN SEISMIC CODE

As common for most of the important facts of life, the decision of setting up Eurocodes for structural design of structures, whose great consequences originated the title of this presentation, was not a conscious act of political vision and foresightedness: it is probably closer to the truth to affirm that their deficiency was likely to be at the origin of the whole process.

Actually, only after having pursued the objective of eliminating trade barriers between EC countries by defining strict standards for "EC products" for many years, it became apparent that barriers for construction products could be introduced de facto through apparently "neutral" provisions in national codes. Their harmonization was therefore seen as an inevitable step, of a purely technical nature, along the way of the ultimate objective of the free market, rather than as a desirable goal per se.

No thought was initially (1979) spent for the examination of implications of any sort. And again a collateral aspect, not anticipated nor necessarily desired, ended in becoming an important issue of its own. In the folds of a national technical code is hidden much more than impersonal technical knowledge: pragmatical and scientific attitudes towards the concept of building as developed in the course of local history are rather reflected and expressed. And European countries are not short of historical background nor of differences in social, economic and climatic conditions, nor - there is no advantage in pretending to ignore it - of more fundamental aspects related to the philosophy of science.

Yet, about ten years after the decision, the first Eurocodes started to appear. Eurocode 8, on Seismic Design, initiated in 1984, will be delivered for experimental use within 1993.

The fruits borne by such an intense effort cannot be appreciated in full while still ongoing. The need for objective principles and solid knowledge to base the development of rational procedures has been recognized from the beginning in the case of EC8, because of the disparate situations existing over Europe in relation to seismicity and seismic design philosophy.

Traditional practices have been called into question, and existing solutions have been borrowed only after deep scrutiny of their suitability in the new context. In the process national peculiarities have tended gradually to smoothen and to harmonize, while a new sense of purpose developed among the people involved: the development of original solutions in the numerous areas where the scientific base was felt as inadequate. This situation resulted in the fact that all major research projects in Europe in the field of earthquake engineering are now being funded by the Community and carried out by several scientific institutions from different countries of the EC. Under these circumstances, it is reasonable to talk of a "European perspective" to seismic design as a reality.

This presentation consists essentially of a review of the aspects considered to be major limitations, from a conceptual point of view if not always from a practical
one, in the view of a codified approach to design and, more generally, in our ability of describing the seismic response of structures. The state of progress and the current research efforts on three interrelated subjects are first discussed: capacity design criteria and procedures, definition of the state of collapse and damage of a structure and viable techniques for a probabilistic calibration of safety factors. It should be noted that a fuller rationalization of these aspects is more acutely needed in Europe due to the larger variety of structural typologies, which calls for more refined differentiations and, in turn, for more rational and visible justifications.

In the second part of the paper the specific problem of shear-bending interaction is considered, whose behaviour is qualitatively known, but neither quantitative well defined nor commonly implemented in computer programs, and a simple proposal for an analytical model is presented and discussed. The results of some preliminary numerical simulation show interesting results, their main merit consisting in an indication of the relevance of the problems. Similar elements could be easily integrated into non linear dynamic programs and used for probabilistic calibrations.

The paper is concluded by a brief presentation of a broad experimental and analytical research program just started in Europe to support the final preparation of a unified seismic code.

CAPACITY DESIGN PHILOSOPHIES

Effects of Capacity Design Principles

Capacity design principles have now reached a widespread use in seismic design, as a rational approach to avoid brittle or otherwise undesired failure modes.

The conceptual approach is straightforward, consisting essentially of the following steps (1):

- definition of potential plastic hinge regions;
- design of these regions to ensure strength and ductility required by the assumed seismic action;
- definition of potential undesirable modes of inelastic deformation;
- design of the strength associated with such modes, that must exceed the capacity associated with plastic hinge regions: these modes are therefore forced to behave elastically, irrespective of the intensity of the ground shaking.

On the contrary the implementation in a practical design procedure is far from being a simple exercise, due to the difficulties in establishing a relation between the safety of a structure and the partial factors to be applied in order to obtain adequate capacities, to the potential interaction and compounding of CD
principles, to their relation with the ductility assumed for designing, to the inadequacy of the linear elastic methods of analysis generally accepted in codes.

Typical undesired modes of failure are associated with shear and anchorage failure, soil-foundation failure, and formation of soft-storey or similar flexural mechanisms associated with large local ductility demand and small overall energy dissipation.

When combined with appropriate detailing to ensure a ductile behaviour of the selected plastic mechanisms, capacity design principles should guarantee a stable response and a large energy dissipation capacity.

Interaction and Compounding of Capacity Design Principles

The well known soft-storey mechanism can form when plastic hinges develop in all the columns of a storey rather than in beams, the consequence is the recourse to a "strong-column weak-beam" method of design, which in turn is translated into codes by imposing that in each single beam-column node the column strength is designed according to the capacity of the beams. In this way the mechanism in fig. 1(b) is forced to happen with obvious advantages. It has to be recognized, however, that it is not necessary to avoid the formation of hinges in all columns to avoid a soft-storey mechanism: for example the mechanism in fig. 1(c) has the same ratio between local and global ductility has that in fig. 1(b), with 21 plastic hinges instead of 19, having only one "strong column" out of three.

This example has obviously been presented to make the point, and has to be understood in relation with the problem of the quantification of the partial safety (or overstrength) factors. Actually since it is the probability of the formation of plastic hinges at all the columns of a storey that has to be checked, each ratio of column to beam flexural strength can be kept much lower or close to unity, relying on material strength variability. Along this line Priestley and Calvi (2) have proposed the definition of a storey-sway index as follows:

\[
S_p = \frac{\sum_{i=1}^{n} (\sum M_{B_i,j}) + \sum_{i=1}^{n} (\sum M_{B_i+1,j})}{\sum_{i=1}^{n} (\sum M_{C_i,j}) + \sum_{i=1}^{n} (\sum M_{C_i+1,j})}
\]

[1]

where \( n \) indicates the storey, \( j \) the joint, \( B \) stands for beam and \( C \) for column.

It has also been noted (Dolce and Evangelista, (3)) that the amplification of column stresses, suggested by some codes to simplify the design process, is never able to avoid any yielding in columns, due to the moment redistribution which follows the plastic hinge formation at the beam ends. Again the subject assumes a different light if yielding in all columns of a storey is regarded as the catastrophic event, rather than yielding in one or more column.
Another example of a capacity design principle which could be questioned under some respect can be found observing that in order to avoid foundation failures, difficult to be assessed and repaired, a factor should be applied to the capacity of the column, which has been in turn dimensioned factorizing the beam capacity, with some obvious compounding. This fact is particularly curious if it is considered that some plastic deformation in the soil could, under certain circumstances, provide significant energy dissipation under stable conditions.

Calibration of Safety Factors for Capacity Design

The discussion above clearly points towards the necessity of some investigation of the relation between CD factors and overall safety against collapse of a building. A reliability differentiation among possible inelastic mechanisms can be achieved, as it is well known, through the use of partial safety factors, often simplistically referred to as "overstrength factors". Their quantification requires some consideration of two aspects: the amount of the inelastic deformation that it is intended to exploit (deterministic component) and the variability of the mechanical properties of the materials (random component).

The major obstacles probably reside in our limited ability in the mechanical modeling of the response of a structure and in the knowledge of the conditions leading to both local and global collapse. This limitation of knowledge does not reduce the significance of a probabilistic treatment of the problem, this approach being the most rational to deal with uncertainties.

There is therefore an urgent need for models endowed with the ability to continue the simulation of the response up to what is accepted as collapse, which cannot obviously be identified with some first yielding or first attainment of the ultimate moment in a section.

Secondly, the most appropriate probabilistic simulation techniques should be applied to the problem, in order to determine some consistent values for sets of design parameters (i.e. with equal probability of failure), taking into account structural model error, material variability, uncertainty on the resistance models and randomness of the input motion.

NEEDS FOR A UNIFORM SAFETY AGAINST COLLAPSE

Damage, collapse and limit states

In recent years many studies appeared on possible damage indices, to be used in post-earthquake evaluation and, with more difficulties, in seismic assessment and design. These indices can be based on peak (e.g. some maximum displacement or
ductility) or cumulative (e.g. some absorbed or dissipated energy) parameters, and constitute, in the long run, the most promising approach to a rational definition of different limit states.

Clearly, when properly defined such indices could be used as basic parameters to quantify strength and stiffness degradation, or, in other words, they could constitute the base for rational member constitutive equations.

The basic problem in this area of research lies in the lack of experimental data, in the difficulties of correlating data from tests performed by different researchers and particularly in the complexity of the phenomena to be represented in the model.

A definition of collapse, on the contrary, has a more conventional and philosophic flavour: with a possible spectrum going from the attainment of first yielding in a member to the inability to sustain gravity load. The authors are in favour of this last definition, which is less conventional, allows a full use of damage indices and intermediate limit states, but also requires very sophisticated models to perform a complete simulation.

Features of analytical models up to collapse

A recent review of the state of the art in non-linear cyclic modeling of reinforced concrete structures is presented in a recent CEB Bulletin (4), where it is clearly stated that only for the case of uniaxial flexure, without axial load and shear, our capacity of modeling the inelastic cyclic behaviour can be considered quite satisfactory. This situation is not essentially due to difficulties in modeling, but rather to the lack of reliable experimental data: "for uniaxial or biaxial bending with varying axial load, for interaction of shear and axial load effects, the addition of one or more control and output variables increases the difficulty of the empirical curve-fitting problem and the volume of experimental data required for the fitting by at least one order of magnitude". On the contrary, tests results for these more complicated cases are sparse and models are more based on theoretical speculation than on experimental evidence.

The case of uniaxial or biaxial flexure with axial load is on the way of being solved through the use of fiber models, where each fiber is characterized by a non-linear uniaxial stress-strain law, with load reversal, appropriate for concrete or steel. The research in this field is more concentrated on the definition of efficient integration procedures to reduce the enormous computational effort presently needed, without impairing the accuracy of the solution.

It has to be underlined that all available computer programs based on fiber models disregard important phenomena, such as interaction with shear, bond slippage, buckling of bars, joint failure.
Shear-bending interaction

In 1964 Kani published the well known paper "The Riddle of Shear Failure and Its Solution" (5), but twenty nine years later Priestley (6) still stated "shear design of reinforced concrete is full of myths, fallacies and contradictions".

The importance of shear-bending interaction is apparent when considering that in most cases the real collapse of a structure is triggered by shear damage.

Recent experimental studies (Priestley at al., (7)) suggest to compute the nominal shear strength by an additive equation of the form:

\[ V_n = V_c + V_s + V_p \]  \[2\]

where \( V_c \), \( V_s \) and \( V_p \) are components of shear strength from concrete mechanism, steel truss mechanism and axial force mechanism respectively. The concrete contribution is assumed to be dependent on the rotational ductility according to figure 2, the steel contribution is constant and the axial force contribution, also constant, increases when the column aspect ratio \( (M/V_d) \) decreases. The separation of the axial force term from the concrete term constitutes a difference from that proposed in most code equations, and is needed to consider some possible internal arch action, with formation of an inclined strut.

Shear-flexure interaction is obviously more important for low aspect ratio values, in which case also the strength and stiffness degradation due to shear can be very significant. A review of available experimental data and numerical models is presented in the already mentioned CEB Bulletin (4).

The numerical application presented in the second part of this paper has been based on the implementation of a model based on both shear and flexural degradation, inspired, for what concerns shear, by a model recently proposed by Fardis (8). Shear bending interaction is also present, consistently with what has been previously discussed.

Techniques for probabilistic calibrations

Given the state of uncertainty that permeates most of the facets of the problem, there are in principle no rational alternatives, when attempting a quantitative assessment of the safety against collapse, to a probabilistic approach.

An argument, however, that today could not be easily discarded against this statement, relies in the poorness of our knowledge: so many arbitrary assumptions should be accepted that the probabilistic results could become scarcely relevant.

The only defense against this argument is that for code calibrating the differences on the design parameters necessary to keep the probability of failure \( (P_f) \) constant over the design space are essentially required (the global scaling factor can be derived from experience) and these differences can be presumed to be less sensitive to alternative choices made in calculating \( P_f \).
A second argument, and not a weaker one, is that at present the tools for performing complete probabilistic safety analyses are simply not available: the existing proposals (see for example (9), (10), (11)) invariably disregard or simplify one or more of the aspects of the problem. Exhaustivity would in fact require consideration of: randomness of the input, material variability, structural model error and uncertainty on the collapse limit state, in presence of a highly non linear path dependent behaviour. Additionally, code calibration is much more than calculating $P_f$ for a number of structures: it is a problem of optimization.

If we denote with $q$ the set of unknown design parameters (i.e. force reduction factors, overstrength factors, and capacity design procedures), and with $P_f^*$ the target value of the collapse probability, then the solution for $q$ is the one minimizing the squared error:

$$Q = \sum (P_f^* - P_f)^2$$

where the summation extends conceptually to the entire population of structures intended to be covered by the particular set of $q$ (a sub-set of the design space). Considering that calibration of partial safety factors made thus far could only deal with isolated problems of load combination factors and elementary sectional strength models, the purpose of calibrating a much larger number of factors in a full system reliability context appears to be simply out of reality.

If the problem cannot be attacked on its whole front, however, a feasible and useful alternative is to consider isolated sub-problems in sequence. Examples of priority cases include:

a) to find the overstrength factors necessary to protect the columns from yielding (protection formulated in probabilistic terms) given:
   - a structural typology
   - the value of the global $q$ factor used in design (possibly a variable parameter)
   - the admissible level of ductility demand,
   and taking into account: randomness of the input, model uncertainty and material variability;

b) to find in probabilistic terms the maximum forces that can be transmitted to the foundations system (as a by-product of point a));

c) starting from a preliminary mechanical model for shear strength under cyclic bending, to use available test data to make a Bayesian updating of the model, and to derive iso-reliable overstrength factors for shear design in order to allow for not hindered flexural dissipation.

Even in these reduced forms the tasks are not simple: two techniques having shown some potentiality for yielding useful results - the Monte Carlo (MC) and the Response Surface (RS) approaches - are briefly introduced in what follows.

Given the large number of variables involved a direct application of the MC method would require an excessive amount of samples. In many cases, and particularly when low values of $P_f$ are of interest, the efficiency of the MC method is enhanced by the used of the techniques of directional sampling (DS). Further, if the critical part of the failure surface is at least approximately known, an estimate
of \( P_f \) in this region can be made much more accurate by the use of the importance sampling (IS) technique. The effectiveness of combining DS with IS with specific reference to problems of non linear dynamic response under seismic excitation has been explored in (11), with mixed success. The response surface approach is also computationally not affordable if the space of the events includes all the variables involved. To make the RS usable for the present purpose, it has been proposed by Veneziano (10) to partition the full vector \( \mathbf{X} \) of all the uncertain variables into a small one, \( \mathbf{X}_1 \), containing the most influential variables that have to be treated individually, and its complement \( \mathbf{X}_2 \). The components of this latter are viewed as random effects parameters and their influence is lumped into a few additive random variables (r.v.).

The RS takes the following form:

\[
S_R = g(\mathbf{X}_1, \Theta) + \Sigma \epsilon_i
\]  \hspace{1cm} [4]

where \( \mathbf{X}_1 \) is the reduced vector of influential r.v.'s, \( \Theta \) contains the unknown parameters of the surface and each \( \epsilon_i \) is a r.v. which accounts for the effects of a sub-vector of \( \mathbf{X}_2 \).

Techniques of experiment planning and of analysis of variance are used to determine \( \Theta \) (as random variables themselves) and the parameters of the r.v.'s \( \epsilon_i \). After having determined the (random) RS as outlined above, \( P_f \) can be evaluated by means of standard reliability procedures. This approach has been applied in (10) to the determination of fragility curves for a 4 storeys, 3 bays reinforced concrete frame.

A more recent and conceptually appealing proposal based on RS is discussed in (9): the RS is not constructed in the whole domain of definition of the variables, but in the proximity of the design point (i.e. the most likely failure point) only. This is achieved by means of an adaptive procedure, which starts by defining the RS in the points representing the mean values \( x_i \) of the r.v.'s, and in the points: \( x_i + f_i \sigma_i \). In this way, second moment information is introduced in the construction of the RS from the beginning. Simple quadratic interpolation functions without mixed terms are used for representing the RS in the neighborhood of the chosen points. The initial RS is then used to obtain an estimate of the design point, which is subsequently taken as the new center point for the construction of a second iteration RS. After convergence the variability analysis can proceed by using either first or second order procedures or MC simulations with importance sampling.
AN INTERACTION MODEL FOR MEMBERS UNDER FLEXURE AND SHEAR

Analytical Description of the Model

It has been discussed how the interaction between flexure and shear may become critical not only for the deformability of the element, but for its survival as well, particularly in the case of low shear span ratios. The degradation of shear strength due to cyclic action can precipitate failure of an otherwise steadily degrading element under flexure alone.

Composite models describing this interaction are not available, mainly due to lack of sufficient experimental data on the failure surface in the M-N-V space under cyclic loading. Few existing behavioural models which account for the contribution of both flexure and shear to the total deformation treat separately the two components, which are then superimposed (series models). The simple interaction model shown in figure 3 has been developed along these lines, but also considering a mutual influence of shear and flexural behaviour.

The element shown is in a state of constant shear and linearly varying bending moment. It is assumed that initially the diagonal cracking due to shear does not interfere with flexural cracking, which is concentrated near to the base. Repeated and/or increased cyclic displacements induce a state of progressive disintegration, eventually leading to failure. It is emphasized that the model should be regarded as conceptual, since there is not adequate evidence to support a more realistic quantitative implementation.

For the two-springs stick model in fig. 3(c) the top displacement is expressed as:

\[ x = x_1 + x_2 = F l (1/k_\gamma + l/k_\phi) \]  \[ \text{[5]} \]

where \( k_\gamma \) and \( k_\phi \) represent the non linear hysteretic coupled stiffnesses for shear distortion and flexural rotation, respectively, modeled using the expressions proposed by Bouc (12) and extended by Wen (13), because of their ability of representing various forms of non-linearities, from ideal elastic-plastic to locking behaviour.

The generic force-displacement relation takes the following form:

\[ F = \alpha k_0 x + (1-\alpha) k_0 z \]  \[ \text{[6]} \]

where the first right-hand term is linearly elastic, while the second one is hysteretic and controlled by the auxiliary variable \( z \), which obeys the first order non-linear differential equation:

\[ \dot{z} = (A\dot{x} + \sqrt{B\dot{x}}|\dot{z}|^{\gamma-1} z + \gamma\dot{x}|z|^{\gamma-1})\eta \]  \[ \text{[7]} \]
A detailed analysis of the role of the eight parameters in equations [6] and [7] is complicated by the fact that they mutually influence the different mechanical properties of the model. Basically, the product $\alpha k_0$ defines the asymptotic slope of the F-x curve, while the stiffness for $z = 0$ (hence, in particular the initial tangent stiffness) is equal to:

$$\left. \frac{dF}{dx} \right|_{z=0} = \alpha k_0 + (1 - \alpha) k_0 A / \eta \quad [8]$$

The hysteretic term has the asymptotic value:

$$(1 - \alpha) k_0 (A / v_1 (\beta + \gamma))^{1/n} \quad [9]$$

The constants $\beta$ and $\gamma$ govern the general shape of the hysteresis loop and the exponent $n$ the sharpness of yield, while the two parameters $v$ and $\eta$ regulate the degradation process. If these latter are allowed to vary as a function of the response, then an increase of $v$ affects (reduces, eq. [8]) the strength only, while an increase of $\eta$ operates similarly on the stiffness (eq. [9]).

The intersection between the initial and the asymptotic value tangents to the F-x curve defines a conventional yield point $(F_y', x_y')$, which can be used to normalize the curve. In what follows shear and flexural mechanisms will be referred with numbers 1 and 2 respectively, the normalized displacements being defined as:

$$\bar{x}_1 = x_1 / x_{1y}$$
$$\bar{x}_2 = x_2 / x_{2y}$$
$$\bar{x} = (x_1 + x_2) / (x_{1y} + x_{2y}) \quad [10]$$

which can obviously be regarded as ductilities.

Any quantity related to the response can in principle be used as a control variable to describe the degradation of strength and stiffness. Common choices use a measure of the dissipated energy and/or an extreme variable, e.g. the peak ductility. This last has been the choice adopted in this study, essentially for simplicity, given the lack of alternative quantitative indications and the paradigmatic nature of the study.

The specific degradation criteria adopted are as follows.

**Shear.** The strength reduces linearly with the total ductility $\bar{x}$, starting at a lower threshold value $\bar{x} = \bar{x}_{\text{min}}$ and decreasing to a given percentage of the initial strength for $\bar{x} = \bar{x}_{\text{max}}$.

The stiffness also reduces linearly with $\bar{x}$, starting at $\bar{x}_{\text{min}}$ down to a value related to the residual stiffness of the flexural component for $\bar{x}_{\text{max}}$.

**Flexure.** Both strength and stiffness decrease linearly with $\bar{x}$, with a different rate of speed, for example depending on the axial force level. The flexural degradation does not depend on the shear response.
Typical shapes adopted for shear and flexural components are shown in figure 4: the shear component is intentionally idealized, and based on intuitive reasoning more than on experimental evidence. In particular, the energy loss per cycle has been intentionally kept at an almost negligible level up to large magnitude cycles characterized by a significant degradation. Emphasis has been placed on a reasonable description of the stiffness and strength degradation and of their evolution. The initial cycles are practically linear elastic, with a high stiffness value; gradually the stiffness decreases and the response departs earlier from a linear behaviour, showing a softening trend. Upon unloading, almost the same path is followed, implying a small variation of force for the first part of the displacement reversal, followed by a hardening which continues upon load reversal until the new softening in the opposite direction takes place.

For what concerns the flexural behaviour figure 4 (b) shows the expected features for a well behaving element: a limited amount of strength degradation can be traced on the peak force levels at equal deformation in different cycles, similarly some light stiffness degradation is evident both in loading and unloading branches.

**Model Behaviour**

The model behaviour is governed by a set of four equations:

\[
\begin{align*}
\mathbf{x} &= \mathbf{x}_1 + \mathbf{x}_2 \\
\mathbf{F} &= \mathbf{F}_1 = \mathbf{F}_2 \\
\dot{\mathbf{z}}_1 &= \mathbf{f}_1(\mathbf{x}_1 + \mathbf{z}_1) \\
\dot{\mathbf{z}}_2 &= \mathbf{f}_2(\mathbf{x}_2 + \mathbf{z}_2)
\end{align*}
\]

in the four unknowns \( \mathbf{x}_1, \mathbf{x}_2, \mathbf{z}_1, \mathbf{z}_2 \), that can be solved numerically by imposing a discrete history of displacement.

In incremental terms, the distribution of the total displacement \( \mathbf{x} \) between the two mechanisms (flexure and shear) depends on their relative stiffness, which changes during the time history, together with the relative strength. Since the force must be the same in both components, its value must result to be the one offered by the instantaneous weaker element, and therefore the force level previously sustained by the stronger one can be reduced. This continuous interaction can be anticipated in a qualitative way, considering the strength degradation functions.

For a well designed element the initial response will be governed by the flexural strength (\( \mathbf{V}_{\text{Mmax}} \)), in principle always lower than the shear strength (\( \mathbf{V}_{\text{Vmax}} \)); the initial shear stiffness is also generally much higher than the flexural stiffness, resulting in an almost unnoticeable contribution of shear to the global \( \mathbf{F}-\mathbf{x} \) diagram. As shown in figure 5, \( \mathbf{V}_V \) starts degrading after a given threshold value of \( \mathbf{x} \), together with the associated stiffness, while \( \mathbf{V}_M \) may have started degrading in both characteristics earlier, but with a slower rate. For \( \mathbf{x} \geq \mathbf{x}_{\text{min}} \) the contribution of the shear deformation is therefore increasing even if the flexure still dictates the strength value. Then, for \( \mathbf{x} = \mathbf{x}^* \), it may occur that shear starts dominating the force level: the flexural cycles have then to shrink, not being able to reach the cycle
amplitudes previously attained, and the shear deformability becomes of the same order of magnitude, possibly greater, than the flexural deformability. The overall shape of the $F-x$ curve becomes more similar to the shape of the shear mechanism.

After both mechanisms have reached their final degraded state ($x \geq x_{2,\text{max}}$, figure 5), the mathematical model reaches a sort of steady state: this is due to the dependence of strength and stiffness degradation on displacement only, and could be easily modified using some energy based parameter obtained by proper experimental tests.

Results from pilot applications

The number of free variables characterizing the even elementary model just described is so high that intelligible results can only be obtained assigning reasonable constant values to some of them, selecting the most relevant as variable parameters.

In the first applications, discussed in what follows, three parameters have been assumed as variable:

- aspect ratio of the element (height, $l$, over width, $h$), as a governing parameter of the relative importance of flexure and shear;
- ratio of the initial shear and flexural strength, which depends essentially on the design factor adopted to assure a flexural behaviour;
- ratio of final and initial shear strength, which depends on the relative importance of concrete, steel and axial action contributions to the total shear strength.

The major parameters kept constant, for the time being, are:

- a 20\% degradation of the flexural strength for $x$ varying in the interval 2 - 6;
- a 50\% degradation of the flexural stiffness in the same interval;
- a degradation of the shear stiffness from its initial value to the same value of the flexural stiffness for $x = 6$: a choice totally arbitrary but probably scarcely influential;
- a loading history, with displacement cycles of increasing amplitude up to $x = 6$.

A total of twenty four numerical experiments have been run, i.e. all the possible combinations resulting from table 1. The results have confirmed the negligible influence of the aspect ratio, as expected because of the normalization adopted for the displacement ($\bar{x} = x / x_{1y} + x_{2y}$).

Out of the six remaining cases the three described in the following points a), b) and c), and in table 2, have been selected for illustration.

a) This is a common design situation, for what concerns either the margin of safety against shear failure and the amount of residual shear strength, consistent with a rather low aspect ratio.

The results are plotted in figure 6(a), showing a response essentially flexural up to a ductility of 2, with cycles reaching a maximum amplitude close to unity.

For $\bar{x}$ varying between 2 and 4 the shear strength decreases to half of the initial
value (i.e. $1.25 / 2 = 0.625$). At this point shear already dominates the deformability, contributing with 2.5 out of a total ductility of 4. The flexural cycles shrink in amplitude from 1 to 0.625. For ductilities larger than 4 the response reaches a steady state situation with shear characterizing the shape of the F-$\bar{x}$ diagram: physically this stage might be considered as corresponding to the combined contribution of dowel action and friction.

b) This case is similar to a), but with a significantly lower residual shear strength, it was therefore expected that the behaviour would have been strongly influenced by this weakness. In fact, the response (fig. 6(b)) is in this case identical to case a) up to $\bar{x} = 2$, followed by a precipitous degradation of the shear strength to the imposed minimum value ($1.25 \times 0.25 = 0.3$) for a ductility value of $\bar{x} = 4$. The shape of the global F-$\bar{x}$, is in this phase similar to that of the shear alone, which actually contributes to three quarters of the total deformation for $\bar{x} = 4$. At the maximum displacement ($\bar{x} = 6$) the flexural component is practically no more active, and the behaviour could be physically considered a shear sliding.

c) This case represents the opposite extreme, with a large margin of strength between shear and flexure and a mild strength degradation in shear: the behaviour is expected to be essentially flexural. The response, plotted in figure 6(c), shows how the shear strength never degrades below the initial value of the flexural strength, its minimum value being $2.0 \times 0.75 = 1.5$, well above 1. The behaviour is therefore influenced by the different velocity of degradation of the stiffness of the two components. The faster rate of the shear stiffness degradation can be observed comparing the two diagrams in figure 6(c). At maximum displacement the shear component contributes with a ductility of 2, the flexural component with a ductility of 4. The observed behaviour is obviously satisfactory.

**CONCLUSIONS: A PRE-NORMATIVE RESEARCH PROGRAM**

The conceptual model developed to study the interaction between bending and shear allowed discussion of two fundamental aspects of the problem: shear-bending interaction is of fundamental importance for modeling frame structures up to collapse, and, it will be possible to introduce this phenomenon into deterministic and probabilistic programs of analysis without impossible complications.

The problem is then moved to the availability of experimental data and to their interpretation towards simple phenomenological models.

A very ambitious research program has just been started in Europe, with the objective of investigating weak aspects involved in the present draft of EC8. This program deals also with infilled frames, bridges and soil-foundation interaction, but its main body is oriented to reinforced concrete frames. Four shake tables (in Athens, Bristol, Lisbon and Bergamo), the big reaction wall of the Joint Research Center in Ispra, and fourteen other Institutions (in Bagneux, Basilicata, Darmstadt, Liege, London, Madrid, Milan, Paris, Patras, Pavia, Rome, Saclay, St. Martin d'Hôres) are involved.
For what concerns reinforced concrete frames a number of topics are on the table, such as:
- evaluation of the effects of new kinds of steel produced in Europe;
- numerical studies on available ductility in single members;
- conception, design and analysis of typical frame buildings;
- identification of parameters of regularity;
- evaluation of the effects of capacity design provisions;
- experimental tests on members;
- study of vulnerability functions and damage indices by non-linear dynamic analysis;
- full scale tests on frame buildings (a four storeys, two bays by two bays building has already been constructed to be tested in Ispra);
- shaking table tests on scaled specimens and partial sub-assemblages.

The main expected outcome should deal with some revision of EC8 clauses, dealing in particular with:
- requirements for reinforcing steel;
- definition of structural regularity and its implication on models and analysis;
- evaluation of behaviour factors and capacity design procedures and factors;
- provisions for member and joint design.

In the occasion of this project the wish of the authors of this paper is to have some other "Tom Paulay in Europe".

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The authors are indebted to their colleagues in TG III/2 and III/6 of the CEB for long and profitable discussions on the subject, as well as with all the researchers of nineteen Institutions participating in the pre-normative research in support of EC8. The contribution of Dr. T. Pagnoni has been fundamental in the implementation of the shear-bending interaction model.
TABLE 1 — MAIN VARIABLE PARAMETERS CONSIDERED

<table>
<thead>
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<th>ASPECT RATIO</th>
<th>STRENGTH RATIO</th>
<th>SHEAR DEGRADATION</th>
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<td>1 / h</td>
<td>$K_M / K_V$</td>
<td>$V_Y / V_M$</td>
</tr>
<tr>
<td>1</td>
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</tr>
<tr>
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TABLE 2 — CHARACTERISTICS OF THREE CASES PRESENTED IN FIG. 6

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<th>(b)</th>
<th>(c)</th>
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<td>shear strength ratio</td>
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Fig. 1 — Plastic collapse mechanisms
Lateral Force Transfer in Buildings

Fig. 2—Degradation of concrete shear strength with ductility [Priestley et al., (5)]

Fig. 3—Interaction model for flexure and shear

Fig. 4—Typical shapes adopted for shear and flexural components

Fig. 5—Typical strength degradation of shear and flexure as a function of total ductility
Fig. 6—Response of three exemplificative cases (see Table 2)
Multi-Story Precast Concrete Framed Buildings

by A. J. O'Leary

Synopsis:

In this paper design and construction aspects of precast concrete moment resisting frames for the lateral load resistance of multi-storey buildings are described. Discussion will concentrate on the particular aspects of the framing system of a 13 storey building constructed in Wellington, New Zealand. The building is octagonal in plan with a perimeter lateral load resisting frame consisting of two storey high precast reinforced concrete elements. Each element includes a column plus two levels of beam stubs. In-situ concrete mid-span beam splices and grouted steel sleeve column rebar splices form the joints between individual units.

The paper will briefly present other similar precast systems used for multi-storey buildings. A review of laboratory testing recently completed will be given which confirms the good structural performance of the framing systems described.

Keywords: concretes; earthquake-resistant structures; frames; loads (forces); precast concrete; reinforced concrete; resistance
APPRECIATION

My first memory of Tom Paulay goes back to my final undergraduate year at the University of Canterbury School of Engineering. My class, and numerous classes before and since, had the great fortune to have Tom as mentor and lecturer for 'drawing and design III'. I guess this was our first introduction to the real world of structural design. It is no coincidence that many of us, and indeed a large proportion of Canterbury civil engineering students specialised in structural engineering because of the enormous influence of Tom. His lectures were an inspiration for all those vaguely interested in structures. Tom’s enthusiasm as a lecturer in the basics of structural design and detailing was every bit as "catching" as his enthusiasm in presenting the latest research information on beam/column joints, shear walls, capacity design and many other aspects of reinforced concrete and general structural engineering. Our knowledge of these has made enormous advances because of the influence of Tom’s research.

I was so impressed with Tom’s attitude to structures that after completing my undergraduate studies I enrolled for a Ph.D. at Canterbury with Tom as my supervisor. At times he must have despaired of me ever making the grade as I was a mere mortal when it came to advancing the detailed knowledge of ‘shear, flexure and axial tension in reinforced concrete members’. However, his cajoling, the extremely rewarding discussions, and the directing of where I should proceed to next, achieved the goal in a little over three years. I understand that I was the second student that Tom supervised to graduate Ph.D. from Canterbury.

To the lighter side of Tom’s personality. For the last 24 years every time we see him he has been enquiring of my wife Marie whether the family is increasing in numbers. When she is present Tom’s native Hungarian gallantry allows him to forget all about aggregate interlock, and confinement and concentrate on what he probably always did prefer, charming the ladies.

The development of the precast reinforced concrete framing system that I am about to describe in this paper was in no small way influenced by a morning’s discussion with Tom several years ago. From the discussion evolved the overlapping hook concept for the in-situ ‘stitching’ for the precast beam connections. It was the mark of the man that after all the years as an academic he was still able to very quickly assimilate the problems being faced by the designer and help me to come to a very practical and buildable detail which retained all the features required for structural integrity.

For all your help and inspiration my heart felt thanks Tom.

A J O’Leary
Arthur J. O'Leary, B.E. (Hons), Ph.D., M.I.P.E.N.Z. Chief Structural Engineer and Principal of Kingston Morrison Limited, Consulting Architects, Engineers and Planners. Completed his doctoral thesis on "Shear Flexure and Axial Tension in Reinforced Concrete Members" at the University of Canterbury New Zealand. Specialist field is the design of seismic load resisting structures.

INTRODUCTION

Recent trends in reinforced concrete building construction in New Zealand have included substantial precasting of column and beam elements of lateral load resisting and gravity loaded frames. The major reason for this is the speed with which such systems can be erected, thus enabling the structures of buildings to be erected more quickly than where conventional cast-in-situ concrete construction is used. There are other beneficial aspects of precasting, such as reduced site labour, relief of congestion created by formwork on restricted sites, and the ability to fabricate reinforcement cages in "shop" conditions.

The basic analysis and design of such framing systems are very similar to those for cast-in-situ systems. On the other hand detailing of the reinforcement can be somewhat different. The most critical aspects of detailing are the understanding of the tolerances required for successful precasting along with the special requirements of joint details between precast elements.

UNISYS HOUSE STRUCTURE

Lateral Load Resisting Frame

The lateral load resisting frame that will be discussed in detail in this paper is the 13 storey frame used in the construction of Unisys House, an office building in Wellington, New Zealand. The completed building structure is shown in Fig. 1. The building is an octagonal tower with each floor approximately 860m² in plan. The facets of the tower are of equal length but the octagon has different major and minor axes. The included angles of the corners of the octagon are 120° and 150°. Each facet includes three equally spaced columns. The columns are offset from the intersection points of the facets but the beams are continuous around the corners between facets. The typical floor structural layout is shown in Fig. 2.

Lateral load resistance is provided by the perimeter frame, which is a closed ring. There are two internal frames along the long axis of the octagon but these are designed to resist only gravity loads. The suspended floor slabs
consist of precast hollow core units spanning between the two internal frames and the perimeter frame. An in-situ 65mm topping completes the diaphragm for each floor.

Because the internal frames have long relatively slender beams the lateral load resistance contribution from these frames is minimal and thus for analysis all lateral load resistance was assigned to the perimeter frame. However, because significant post-elastic deflection would occur in the internal frames under severe earthquake attack the internal frames were detailed for ductility.

The unusual feature of Unisys House is that the perimeter lateral load resisting frame is almost entirely precast reinforced concrete. The design of this building also utilised other precast components to the maximum possible extent and included the gravity frames, the stairs, the service core slab, and the flooring system. In-situ concrete, where used, was the minimum necessary to tie the precast elements together and comprised the precast frame beam joints, the structural slab toppings, and the gravity frame beam/column joints.

The precast system described in detail in this paper is the perimeter lateral load resisting frame. This frame comprises precast "cruciform" units consisting of a two storey high column with two levels of beam stubs (Figs. 3 and 4). The connections between cruciforms comprise an in-situ beam joint and mechanically spliced column rebar.

**ANALYSIS**

A response spectrum analysis for the building was carried out with ETABS 84, a micro-computer version of ETABS. Only the perimeter frame was included in the model, the internal frames were assumed not to contribute to the lateral load resistance of the building. The response spectrum used was the tri-linear spectrum from NZS 4203:1984 [1]. The first three modes in each of the directions of earthquake attack considered were combined by the square root of the sum of the squares method. In accordance with the requirements of NZS 4203 the combined modal base shear was scaled to 90% of the code base shear. The differences between the scaled cumulative storey shears were then used to carry out a static elastic analysis of the frame. Accidental torsional eccentricities were allowed for by applying the inertia forces at a distance of one-tenth of the building width from the theoretical centre of mass of each floor. The fundamental periods of the frame along the major and minor principle axes of the building were 1.68 and 2.08 seconds respectively.

Earthquake attack parallel to the oblique faces of the frame was found to govern the design of members of those oblique faces, so analyses were also carried out with earthquake attack parallel and perpendicular to the oblique sides of the building.
Preliminary analyses were performed assuming points of contraflexure at the corners of the frame. Subsequent analyses with full continuity of the beams at the corners and including allowance for beam torsional stiffness indicated this assumption was correct. It is apparent that the torsional stiffness of the beams provided little bending restraint at the beam change of direction.

**DESIGN**

The design was carried out in accordance with NZS 3101:1982, the New Zealand Code of Practice for Design of Concrete Structures [2]. Beam shear design, column design, and beam/column joint design were carried out to the capacity design requirements of NZS 4203 using the procedures recommended in the Commentary to NZS 3101 [3]. This procedure aims to ensure that flexural yielding will generally occur in only the beams. Other components (except the columns at ground floor) were designed to be protected from yielding by applying suitable overstrength factors to the beam actions obtained from the static analysis. Beams were designed to yield at the column faces.

The frame configuration of a closed ring results in a stiff and efficient structure. Because in this frame beams frame into columns in only one plane concurrency effects did not need to be considered. Also, column axial loads induced by lateral forces remain relatively low because of the presence of beams on both sides of all columns. Even the corner beams carry significant shear forces thus balancing the column axial loads induced by the actions of the beams at the opposite face of the column.

The in-situ beam joints needed to be as short as possible to ensure that the rebar splices remained outside the potential plastic hinge region. NZS 3101 requires beam rebar laps do not encroach within twice the beam effective depth from the column face. Thus straight lap splices were too long so the 180° hooked splice shown in Fig. 5 was used, the bar tension being transferred through compression of the concrete enclosed by the overlapping hooks. Two transverse bars were placed in contact with the inner hooks to prevent local crushing of the concrete. Testing at the University of Canterbury [6] subsequent to this building being completed has shown that the two effective beam depths is unduly conservative and an amendment to NZS 3101 has relaxed this restriction to one effective beam depth.

The column rebar reinforcement was connected with epoxy filled metal sleeve splices, commercially known as NMB splices. The system enables deformed rebar to be butt spliced by inserting the bars into a double frustrum shaped steel sleeve, and filling the space between the bar and the sleeve with high strength, non-shrink epoxy grout [4]. The system can develop at least 125% of the yield force of the rebar which is the minimum mechanical splice strength allowed by NZS 3101 in areas of load reversal. A sketch of an NMB splice is shown in Fig 10.
DETAILING

Introduction

By appropriate structural planning within the architectural constraints imposed, the number of dimensionally different sets of formwork for the cruciforms was limited to three. Any other dimensional differences in beam lengths were taken up in the length of the in-situ beam joints. All variable elements of the cruciforms such as rebar size, spacing and shape were assigned parameters, and this information was then presented for the precaster in tabular form.

Column Splices

The column splice comprised up to 18 bars. Because of the diameter of the NMB splice sleeves four of these bars had to be cranked inward to maintain adequate clearance between splices (Fig. 6). This requirement, together with the larger diameter of the splices and their tapered shape, necessitated special stirrups at the splices.

Fundamental to the performance of NMB splices is the adequacy of the embedment of the bar inside the steel sleeve. Both the embedment in the sleeve at the bottom of the cruciform and the projection of the bar at the top were closely scrutinised by the designer during fabrication. Projection lengths were set by a stop on the formwork and tolerance control on bar and formwork lengths ensured that the minimum embedment in the sleeves was achieved.

Grout fill and bleed tubes were fitted to finish outside the finished concrete face. These fill and bleed tubes were arranged, as far as practicable, to exit on the inside face of the column to facilitate the subsequent grouting operations from the floor of the building thus eliminating any need for exterior scaffolding.

In-Situ Beam Joint

Three different beam joint splice configurations occurred being, straight, 120° corner, and 150° corner. The straight joint consisted of overlapping hook bars from adjacent cruciforms, offset as necessary to facilitate placement. Joint construction required the sliding on of three purpose made stirrups and tying in the transverse bars. The purpose made stirrups consisted of two U-bars shop welded together because the congestion in the joint did not permit standard hooks. Ref. Fig. 7.

In the corner joints the beam rebar on either side terminated in hooks as above, but these did not overlap. A prefabricated reinforcing cage was then dropped into position and tied in a similar manner to the straight joint (Fig. 8).
Precast Beam to Slab Connection

Two different details for the connection of the precast beam to the floor slab were used depending on the location in the structure.

The precast hollow core slab units were seated in continuous rebates cast into the perimeter precast beams. The rebate can be seen in Fig 8. Where the hollow core units spanned parallel to the perimeter frame beams no rebate was required, the hole core unit being installed against the vertical side of the beam. Slab topping starter bars were cast into the sides of the precast beams. These are shown (temporarily bent out of the way) in Figs 7 and 8. The perimeter beams were precast to the finished floor level so that when the topping concrete was cast on the hollow core units the perimeter beams set the slab finished level, and formed the external edge of the slab. A construction joint was formed in the topping at the line of the back of the hollow core seating rebate or precast beam face as appropriate.

The internal beams were precast up to the soffit level of the precast hollow core floor units as can be seen in Fig 4. Topping starters were incorporated into the exposed top main rebar and stirrup tops and the in-situ topping was later cast over the internal beams thus forming both the top of the internal beams and slab topping as a continuous whole.

Tolerances

The subject of tolerances discussed here is limited to the column splices. Other areas on the project posed no significant problems.

The overall objectives of the detailing were to ensure that minimum specified embedments in the NMB splice sleeves were achieved and that the gap between cruciforms could be readily grouted. The minimum workable gap was 10mm, so the nominal gap was set at 15 ±5mm.

At the outset of the project the question of achievable tolerances was discussed and agreed with the contractor. The following tolerances were accordingly specified:

- Longitudinal Rebar Length
  + 2mm (all bars were square cut, not cropped).
- Formwork length
  + 3mm
- Longitudinal Rebar Placement
  +0, -3mm

This last tolerance is in one direction only as the rebar was placed up to a stop end hence could not extend past, but could only be short of the stop.
Because the longitudinal rebar projection from the concrete of the precast element was positioned at the upper end of the cruciform the embedment of that bar in the NMB sleeve at the lower end of the next cruciform above was subject to the sum of the errors of the rebar and formwork length, i.e. \( \pm 5\text{mm} \). Accordingly, embedment lengths were specified as 5mm more than the required minimum.

Table 1 shows the embedment and projection lengths for the various sizes of longitudinal rebar used. In addition the resulting gap between rebar in the NMB sleeve is shown for the extremes of allowable gap between cruciforms (i.e. 10mm minimum and 20mm maximum). This gap has been shown for the situation that occurs when the steel has been placed hard up against the stop.

CONSTRUCTION

Placing Cruciforms

Cruciforms were transported lying flat by truck and trailer from the precast yard. On arrival at the site they were lifted from four lifting points located in the beam sides and placed in the horizontal position on the working slab. The bottom of the column was positioned in a specially designed pivoting heel that was secured to the slab to prevent it moving during the subsequent lift (Fig. 3).

While on the working slab the braces were attached and the lifting chains removed and connected to two lifting points in the top face of the upper beam stubs. The cruciform was then lifted into the vertical position, craned to its appropriate location and lowered into position. To facilitate placement a purpose made guide aligned the cruciform over the projecting longitudinal rebar of the cruciform below. The cruciform came to rest on four steel shims which had been pre-placed to the correct level.

The straight in-situ beam joint required the steel hooks from adjacent cruciforms to overlap. Accordingly as the cruciform was lowered it had to be guided carefully to ensure that the hooks meshed together correctly. The only difficulties encountered here occurred when hooks had been bent or cast out of square and this was not detected prior to lifting. These hooks however in all cases could be easily straightened with the minimum of delay while the cruciform was still on the crane.

With the crane still attached the braces were secured back to the working slab and props were placed under the lower beam stubs. Final plumbing was performed by adjusting the braces and the props as required. The crane was then released to repeat the procedure for the next cruciform. The entire procedure took about 30 minutes.
Prior to placing any cruciform the projecting length of the column rebar on the lower cruciform was checked to ensure that the minimum embedments in the NMB sleeves would result, in conjunction with an acceptable gap between cruciforms. This was achieved by accurately placing to the correct level a template over the column bars and measuring the required distances. Any problems this highlighted were then remedied as appropriate prior to placing the subsequent cruciform. For instance, if the bars were too long they were ground down to the required length.

**In-Situ Beam Joints**

Once the cruciforms were positioned the rebar to the in-situ beam joints was tied from cradles clipped to the beam studs. When complete, prefabricated steel shutters were positioned and the in-situ joints cast. Generally the in-situ joints were cast as soon as possible after the cruciforms were placed as this helped brace the whole assembly.

**Column Splices**

Once the cruciforms were secured in position grouting of the column splices began. The grout was hand mixed and then transferred to an electric pump capable of grouting to a pressure of 200kPa. Single bag lots were mixed at one time and this was sufficient for approximately one and a half column joints. To ensure continuity of the work grouting was delayed until at least half of the cruciforms on a given floor had been positioned.

The grouting of the column splice was achieved by pumping the grout into the gap between cruciforms and the NMB sleeves comprising the column joint. It was found that the entire splice could be filled from one point. With this method the gap was sealed and grout was introduced via the fill tube of a conveniently located NMB sleeve. The grout progressively filled the gap (the displaced air being 'bled' through the unfilled sleeves) and then the NMB sleeves. As the grout rose inside the sleeves the individual fill and bleed tubes were sealed by the insertion of tight fitting bungs. Any sleeves that were unable to be filled could then be individually grouted as required.

The success of the system hinged on three factors:

1. **Gap between cruciforms (15mm ± 5).** The lower limit of 10mm proved to be a practical working minimum. Gaps any less than this would have been difficult to pump through and hence would have increased the risk of blocking. On site any gaps which were smaller than this minimum were increased to 15mm by scabbling back the top of the lower cruciform.
2. Workability of the grout. Grout was mixed in small amounts to ensure maximum workability.

3. Seal to gap between cruciforms. The majority of problems with the grouting operation resulted from inadequate sealing of this gap. Time delays inremedying the seal meant the subsequent grouting was performed at less than maximum workability.

**SPEED OF ERECTION**

Although the construction period was essentially free of industrial problems the following factors slowed overall progress:

1. Delays in delivery of cruciforms.

2. Delays in delivery of the precast flooring system. The precasting yards at that time were over committed and could not keep up with the building construction programme.

3. Limitations on transporting cruciforms. Delivery times were limited to the off-peak traffic periods. This meant that at most six cruciforms per day could be delivered. Although the work was programmed around the cruciform deliveries it was generally felt that floor turnaround could have been significantly improved upon had it not been for this constraint.

4. Lack of space on site meant that storage of cruciforms was not possible.

Despite these factors the construction time for the building was short by New Zealand standards at that time. The first cruciform was placed on site on 25 August, and the roof slab was cast on 14 February the following year, i.e. thirteen floors were erected in a five and a half month period which included a three week Christmas/New Year holiday. Turn-arounds of seven days for the floors related to the first level of the cruciform and five days for the second level were consistently achieved.

**ALTERNATIVE PRECAST FRAMING SYSTEMS**

During the mid and late 1980s the author’s practice designed and detailed several multi-storey buildings incorporating precast reinforced concrete frames. The majority of the framing configurations incorporated a perimeter lateral load resisting frame or combinations of frames, plus internal beam/post construction which was detailed for ductility but was not designed to participate in the lateral load resistance of the building.
One such alternative system consisted of cast in-situ reinforced concrete columns cast up to beam soffit level. Precast beams were then placed over projecting column bars. These precast beams incorporated sleeves through which the column bars projected. Beam/column joint stirrups were wrapped around the sleeve formers rather than directly around the column bars. In this way the difficult beam column junctions were precast under ‘shop’ conditions, and thus were more easily assembled. In-situ beam splices were incorporated at mid span of the beams much as in the cruciform type construction described earlier in this paper. The column bars were then grouted into the preformed sleeves along with the horizontal joint between the beam soffit and the column.

The advantage of this system was that the cranage capacity required to lift the precast beams was not as great as that for the cruciforms. It also allowed simple precasting of beams at corner columns. Threading of the precast beam units over the column bars proved difficult on occasions because of column bars being out of plumb, or out of location. Placement of the column rebar in in-situ construction required tolerance control that could easily be achieved in the precasting yard, but was difficult to achieve in a site fabrication situation.

Single storey height cruciforms were also used on two multi-storey buildings. Both of these buildings incorporated columns at corners with a change of direction of 45° in one case and 60° in another case. These single level cruciform columns had the same attendant tolerance requirements as the double height cruciforms. However they were successfully incorporated into the building structure. The advantage of precasting these cruciforms rather than using an in-situ column precast beam solution was that the whole of the rather difficult beam/column joint reinforcing was built up in the precasting yard. Refer to Fig. 9 which illustrates one of these units.

Some of the precast systems described here are developed in more detail in reference [5].

**EXPERIMENTAL VERIFICATION**

There is a tendency in New Zealand for such systems as have been described in this paper to be developed and incorporated in actual buildings before any significant laboratory verification is carried out. The structural engineering profession is usually able to advance sufficiently convincing arguments to the local permit issuing authorities to satisfy them that the novel systems that have been designed comply with governing standards. The permit issuing authorities are in general sympathetic to new developments provided that they do not depart too far from accepted practice.
However it is always consoling when testing is carried out on newly developed systems to verify their performance in practice. Research has been undertaken at the University of Canterbury into the performance of the beam splice arrangements used in the two storey cruciforms (the overlapping 180° hooks). Restrepo et al [6] has shown that the performance of the in-situ beam splices used in the two level cruciforms is very good. He has also shown that the NZS 3101:1982 code restriction which does not allow splices within twice the effective depth of the beam from the column face is conservative and that this criteria can be relaxed to one times the beam effective depth from the column face.

It is always nice to know that the systems evolved and used in real buildings will work as has been envisaged in the design phase.

CONCLUSIONS

The use of precast elements in a 13 storey building showed that large precast concrete components can be successfully connected to form a lateral load resisting frame. This method proved to be a viable alternative to more conventional means of construction in reinforced concrete.

The use of the NMB rebar splice system necessitated stringent tolerance control, both in the precasting and the site erection. The final detailing was based on achievable but very tight tolerances. Although the tolerances were tight, the supervision by the designer and the quality control measures of the contractor ensured they were met. The NMB system proved to be successful and would be used again in similar circumstances.

The in-situ beam splices incorporating 180° hooked overlapping top and bottom longitudinal bars was simple, quick to site assemble and effective. The performance of the system has been shown in laboratory testing to perform well.

ACKNOWLEDGEMENTS

The author wishes to thank Professor Paulay for his contribution to the development of the beam splices. The support and encouragement from the shareholders of Morrison Cooper Limited for the development of the framing system is much appreciated. Finally thanks must go to the contractor for his input into the buildability of this project.
REFERENCES


| Sleeve Type | Bar Size | Sleeve Length | Embedment | Bar Projection | Min. Distance between bars in sleeve when cruciform gap is
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Fig. 1—Unisys House completed building structure
Lateral Force Transfer in Buildings

Fig. 2—Floor plan showing building structure

Fig. 3—Precast cruciform being lifted into position
Fig. 4—Precast cruciform units erected
Fig. 5—Beam rebar splice details

Fig. 6—16-bar column rebar splice arrangement
Fig. 7—Rebar detail at straight in-situ beam splice

Fig. 8—Rebar for typical corner in-situ beam splice
Fig. 9—Single-story precast unit incorporating a 45 deg corner

Fig. 10—NMB splice
Myths and Fallacies in Earthquake Engineering — Conflicts between Design and Reality

by M. J. N. Priestley

Synopsis: Current practice in seismic analysis and design is examined, with particular reference to reinforced concrete structures. The attitude of the paper is deliberately iconoclastic, tilting at targets it is hoped will not be seen as windmills. It is suggested that our current emphasis on strength-based design and ductility leads us in directions that are not always rational. A pure displacement-based design approach is advanced as a viable alternative. Improvements resulting from increased sophistication of analyses are seen to be largely illusory. Energy absorption is shown to be a mixed blessing. Finally, accepted practices for flexural design, shear design, development of reinforcement, and the philosophic basis of capacity design are questioned.

Keywords: capacity; earthquake-resistant structures; elastic analysis; flexural strength; shear properties; strength; structural design
Appreciation

My association with Tom Paulay goes back more than 30 years, to the 'Engineering Design II' class of 1962 at the University of Canterbury, where I learned the fundamentals of structural design from Tom. This qualifies me as one of Tom's earliest 'victims,' as he still insists in terming those of us who have been fortunate enough to study under him. I have continued to learn from him ever since, as a graduate student first, but later during collaboration on papers and recently on a co-authored book. The discussions we have had about some of the finer points of seismic design have undoubtedly been amongst the highlights of my career, and have been invaluable in keeping me on (or perhaps directing me towards, would be more accurate) the structural straight and narrow.

As well as Tom's eagle eye for a violation of equilibrium, or a woolly train of logic, the attributes that characterize Tom for me are his enormous generosity with his time, his sense of humor, and his sense of the theatrical. I have never met anyone more willing to spend hours uncomplainingly reading and commenting on substandard research drafts. This must almost be considered a failing of Tom's — not to be less open with his time. His sense of humor is legendary, and Hungarian. As he explains, Hungarians can live without food, but not without jokes — they have to have one for each day of the year: it's a form of calendar, as well as nourishment, I suspect. His sense of the theatrical is that essential attribute that separates a great teacher from merely an excellent teacher. His design classes were three-hour long experiences in performance art, where Tom would use all his wiles and energy to involve the class in the lectures. This included liberal overuse of his hardness of hearing, as a weapon. In response to a question, spoken a little softly perhaps, he would storm up the tiers of the lecture room, white hair and elbows flapping, would cup his ear in his hand a few inches from the 'victim' and enjoin him or her to 'speak up, I'm deaf, you know!' We loved it! His high-energy sorties into the class during lectures had something of the flavor of the academic equivalent of a Hungarian cavalry charge.

Tom has always been larger than life — long may he remain so!
Nigel Priestley is a Professor of Structural Engineering at the University of California, San Diego. He was educated in New Zealand, and spent 10 years on the faculty of the Department of Civil Engineering, University of Canterbury, before moving to the U.S.A. in 1986. Research interests include seismic response of bridges and buildings.

INTRODUCTION

It has long been recognized that our codified approaches for seismic design bear a comparatively tenuous relationship to expected performance. Design is based on a static 'snapshot' simulation of the dynamic event, using methods extrapolated from approaches felt to be adequate and conservative for gravity load design. A major difference between gravity load effects and seismic response is that ultimate strength should never be developed under gravity load, while it is almost certain to be developed under seismic response, typically at a level of excitation which may be a fraction of the design level of seismic attack. Further, although ductile seismic response implies greater dependency on displacements than forces, we still, as a matter of convenience and tradition, design for specified force levels, and treat displacements in a comparatively cursory way.

When the design approach is based on carefully considered philosophy, as in the capacity design approach pioneered in New Zealand and gradually becoming accepted in many other seismic regions, excellent results are to be expected because the design structure is comparatively insensitive to the various assumptions made.

It appears, however, that the enormous approximations involved in seismic design are perhaps becoming less appreciated, rather than more, as sophisticated analytical techniques become specified by codes and accepted into common design practice, as a matter of routine. In the United States, and I suspect elsewhere, this has resulted in a tendency for the functions of analysis and design to be separated, and performed by different specialists. The analyst is responsible for modeling the structure and running the lateral force analysis – typically a 3-D modal analysis process. Results from the analysis are presented to the designer who determines member sizes, reinforcement quantities (if reinforced concrete construction) and detailing aspects. The analyst is typically more involved in the analytical process than the correct simulation of member characteristics, with potential dangers. The result of the separation of design and analysis tends to be that analysis drives the design process, rather than the reverse, which might seem to be more appropriate.

There is also room to examine current design and detailing practice, much of which is also extrapolated from gravity load considerations. Occasionally, this process can lead us in directions which are inappropriate for seismic behavior. Even when tenets of structural performance are based on purely dynamic characteristics, such as energy dissipation under cyclic response, the directions we are accustomed to taking are not necessarily the best for survival and damage control.

In this paper, some of the accepted seismic design and analysis procedures are identified as 'myths' or 'fallacies' – an overstatement perhaps, to make a rather dry topic seem more interesting. Nevertheless, a critical examination of the bases
of our design processes is always appropriate, since the origin of these are often obscure, and lost in the history of design practice, or worse, in code committee minutes. Some of the points to be made are well known, others perhaps less so.

THE ELASTIC SPECTRAL ANALYSIS FALLACY

The fundamental basis of seismic design is still the assumption that an elastic (or modified elastic) acceleration response spectrum provides the best means for establishing required performance of a structure. The limitations of the approach are well known, and accepted because of the design convenience, and because of the lack of a viable design alternative. A case can be made that this is a fallacy, and that viable design alternatives exist, or could be developed with comparative ease.

To summarize the limitations:

1. Response is based on a ‘snapshot’ of structural response: that is, response at the moment of peak base shear for an equivalent elastically responding structure. Duration effects, which tend to be period-dependent, with short-period structures suffering a greater number of response cycles than long-period structures are not considered. The merits of using modal combination rules to provide some insight into higher mode effects seems hardly worth while when these will have to be considered by largely empirical rules later in the capacity design process.

2. The relationship between peak displacement response of elastic and inelastic systems is complex, and more variable than commonly accepted. Various rules, such as the ‘equal energy’ and ‘equal displacement’ rules are commonly employed, but without much consistency or logic. If we consider a typical elastic acceleration spectrum, as in Fig. 1, four distinct zones can be identified. At zero period, displacements of elastic and ductile systems cannot, by definition, be related. The structure will be subjected to peak ground acceleration (PGA), regardless of ductility capacity, and will fail if a lesser strength than that corresponding to PGA is provided. In the rising portion of the acceleration spectrum, displacements of inelastic systems are greater than those of elastic systems with equivalent initial stiffness, and the ‘equal-energy’ relationship has some application. In the initial stages of the falling portion of the acceleration spectrum, elastic and inelastic displacement responses are often similar, leading to the ‘equal-displacement’ rule. As the structural flexibility increases still further, the ‘equal-displacement’ rule tends to become increasingly conservative. At very long periods, there is essentially no structural response to the ground motion, and the concept of an absolute displacement (independent of period or ductility) could be advanced, where the relative displacement of the center of mass of the structure is equal to the absolute peak ground displacement.

Different codes rely on startlingly different relationships between elastic and inelastic displacements. In the U.S.A., the relationships between force reduction factors ($R_w$), seismic load factors (1.4) and design displacements ($\Delta_y \times \frac{3}{8} R_w$) included in the Uniform Building Code approach\[1\] can be interpreted as implying that inelastic displacements are expected to be about 50% of equivalent elastic displacements. At the other end of the spectrum, many Central and South American codes rely on the equal energy approach, with a base ductility of about 4. This implies inelastic displacement about 150% of equivalent elastic displacements.
Although we accept displacement capacity to be more fundamental to seismic response than strength, it appears that different groups of experts cannot agree within a factor of 3 as to what these should be, from a given elastic acceleration spectrum.

Although these points have been recognized and partially considered in codes which define inelastic spectra with variable ratios between elastic and ductile coordinates, such as NZS4203\[^2\], confusion is still almost universal.

(3) The elastic acceleration approach places excessive emphasis on elastic stiffness characteristics of the structure and its elements. As discussed subsequently, we are less careful than we should be in determining these characteristics. The question remains as to whether better alternatives might be considered. I believe that they can, and that a more consistent approach may be achieved by complete inversion of the design process.

It should be noted that although many researchers have discussed displacement-based design, the processes described are in fact still strength-based. Moehle\[^3\], for example, discusses the relative merits of ductility-based and displacement-based design, but in his comparison, the starting point is still a given strength and stiffness (and hence period) with the difference being whether displacements or ductilities are checked. As acknowledged by Moehle, when properly carried out, the two approaches are directly equivalent. In the approach described below, strength and stiffness are the end product of the design process, rather than the starting point.

The procedure is initially illustrated by reference to the simple multi-column bridge pier shown in Fig. 2, for which displacement based design is comparatively straightforward. A set of elastic displacement response spectra for different levels of equivalent viscous damping are required, as shown in Fig. 3(a). These can be generated in much the same way as elastic acceleration response spectra. The shape, with a resonant region as shown in Fig. 3(a) with reducing displacement response at large periods is characteristic of displacement spectra, as can be seen from the example in Fig. 3(b).

The sequence of operations involves the following steps:

1. An initial estimate for the structural yield displacement $\Delta_Y$ is made. Since final results are not particularly sensitive to the value assumed, $\Delta_Y$ could be based on a typical drift angle of about $\theta_Y = 0.005$. For a building design, a lower value would generally be appropriate.

2. The limit to acceptable plastic rotation of critical hinges is determined. This will be a function of the importance of the structure, and also the section geometry, and acceptable level of transverse reinforcement. For the bridge bent of Fig. 2, hinges are assumed to develop at top and bottom of the columns.

3. The maximum acceptable structural plastic displacement $\Delta_P$ at the center of seismic force, corresponding to the plastic rotation limit of the most critical hinge is found from considerations of mechanisms deformation.

4. A first estimate of total acceptable structural displacement is thus $\Delta_m = \Delta_Y + \Delta_P$. 


5. An estimate of effective structural damping is made, based on the implied displacement ductility level \( \mu_\Delta = \Delta_m / \Delta_y \) from Fig. 4, where curves are given based on typical hysteresis characteristics for structures with beam hinges or column hinges, respectively.

6. With reference to elastic response spectra for the site (e.g. Fig. 3a), the effective response period can now be estimated. The effective stiffness of the substitute structure at maximum response can thus be found from

\[
T = 2\pi \sqrt{\frac{M}{K}}
\]

as

\[
K = \frac{4\pi^2}{T^2} M
\]  

(1)

and the required structure yield strength, or base shear capacity is

\[
F_y = K\Delta_m
\]  

(2)

7. With a knowledge of the required shear capacity, the member sizes can now be proportioned, and an initial estimate of reinforcement made. The elastic stiffness can thus be calculated, and a refined estimate of the yield displacement obtained.

8. The total displacement, structure ductility, and hence effective structural damping are thus revised, and steps 4 – 7 repeated until a stable and satisfactory solution is obtained. Individual flexural strength requirements for potential plastic hinges are finalized, based on statics.

The approach outlined above has considerable flexibility, since plastic hinge rotational capacity can be related to transverse detailing (or vice versa), and the design is not dictated by somewhat arbitrary decisions about force-reduction factors.

It would appear that this method of displacement-based design could also be applied to multi-story frame or shear wall buildings, provided some additional assumptions are made. The two critical pieces of information required are (1) the relationship between maximum interstory drift and structural displacement at the height of the center of seismic force; and (2) the shape of the lateral force vector to be applied. These aspects are illustrated in Fig. 5 for an idealized frame of \( n \) stories each of equal height \( h \). The center of seismic force is approximately at 2/3 of the building height, and the maximum displacement at this height can thus be expressed as

\[
\Delta_u = \Delta_y + \frac{2}{3} nh \theta_p K
\]  

(3)

where \( K \leq 1 \) defines the non-uniformity of drift up the building height and \( \theta_p \) is the maximum acceptable rotation of the plastic hinges and hence the maximum story drift angle. On the basis of inelastic analyses of frames, Paulay and
Priestley\cite{4} recommend that the distribution of drift should be assumed to be that shown in Fig. 5(c), where the drift in the lower half of the stories is equal to twice the average drift at the roof level. Assuming further that this distribution can also be applied to the plastic component of drift, Eqn. (3) can be simplified to

\[ \Delta_u = \Delta_y + 0.5 n \theta_p \]  

(4)

That is, \( K = 0.75 \).

It is suggested that improved estimates of plastic drift would be obtained by elastic analysis of a substitute structure\cite{5}, where stiffness of members containing hinges is reduced in proportion to their expected ductility. Hence, if beam hinges are expected to have rotational ductilities of \( \mu_0 = 7 \) (which might correspond to a structure displacement ductility of \( \mu_A = 4 \)) then the appropriate stiffness for the beams in the elastic analysis would be \( K_u = \frac{K_e}{\mu_0} = 0.14 K_c \). The adequacy of the design can thus be checked by a lateral elastic analysis of the substitute structure.

If the inelastic displaced shape can be approximated by Fig. 5(c), it follows that the vector of lateral inertial forces to be applied to the structure should also take the same shape.

The displacement-based design approach outlined above appears attractive in principle, but will need to be checked by specific examples covering a wide range of structural types and periods.

THE REFINED ANALYSIS MYTH

In the Introduction to this paper, it was noted that structural analyses for design purposes have become more sophisticated in recent years, with the consequences that the analysis and design functions are frequently separated and carried out by different people. The reason for the increased sophistication in the analysis is principally related to the availability of powerful computers rather than a perceived inadequacy of earlier, and simpler analysis techniques.

Although 3-D modal analysis is undoubtedly useful in structures with unusual or irregular geometry, it is doubtful if it produces better results than those obtained from simpler methods – say simple lateral analysis based on an assumed lateral force distribution. The myth here, then, is that refinement of the analysis produces more 'accurate' results. It is appropriate to consider the refinement of the analytical process in light of the approximations still remaining.

Elastic modal analysis essentially relies on the equal-displacement approximation, since it is not feasible to use different force-reduction factors associated with different modes of elastic response. As noted above with reference to Fig. 1, this is appropriate for a comparatively narrow band of periods.

Deflection profiles from elastic modal analyses tend to underestimate drift levels in the lower stories of a building. As noted earlier, it is felt that agreement could be improved by use of a substitute structure approach.
Elastic analyses are generally based on approximations of member stiffness that should be considered gross, even in the elastic range. As an example of this, let us consider that the columns of the lowest story of the frame shown in Fig. 5(a) are reinforced concrete $600 \times 600$ columns, reinforced with $8 - D28$ bars of yield strength $455$ MPa. Concrete strength is $f_c = 31$ MPa. It will be normal in the analysis to assume that all three columns at the lowest level have the same stiffness, though it is possible the central column might be allocated slightly higher stiffness because of increased axial load. It would seem to be impossible to allocate different stiffnesses to the two outer columns when multi-modal response is considered.

For the sake of argument, it is assumed that the outer columns carry gravity loads of $P_{(D+L)} = 0.2 \, f_c \, A_g$, and that seismic axial forces of $P_E = \pm 0.2 \, f_c \, A_g$ can be expected. Figure 6 shows moment-curvature relationships for the compression and tension columns, including the effects of varying axial force as the lateral base shear increases, and the relationship assuming a constant axial force of $P = 0.2 \, f_c \, A_g$. Onset of first yield of tension reinforcement, and attainment of a compression strain of $\varepsilon_c = 0.003$ are also noted.

Taking the yield condition to correspond to a curvature of about $0.0084/m$ (which is the yield point for a bilinear approximation to the $P = 0.2 \, f_c \, A_g$ curve) we find that the effective stiffness of the compression column is more than twice that of the tension column. As a consequence, the distribution of elastic forces in the lower stories is likely to be substantially different from that predicted by the 'refined' elastic analysis. Note, however, that it would be comparatively straightforward to consider these effects in an equivalent lateral force approach.

In some design codes (e.g., [2]), it is permissible to redistribute up to $30\%$ of moment from a tension column to a compression column. It is perhaps of interest to note that using the $30\%$ redistribution rule, force levels predicted by constant-stiffness analysis for the tension column could be reduced by $30\%$ of the average of the tension and compression column capacities, resulting in a minimum permissible strength of $600$ kNm for the tension column, or about $15\%$ higher than the capacity. Design based on variable stiffness would, however, not require any redistribution beyond that naturally resulting from the stiffness difference. It should also be noted that the redistribution limit of $30\%$ is set to avoid excessive ductility demand. Although it will be seen that the tension column does in fact reach yield at a curvature $16\%$ lower than the $P = 0.2 \, f_c \, A_g$ case, the onset of crushing (very conservatively estimated at $\varepsilon_c = 0.003$) occurs at a curvature more than twice that for the $P = 0.2 \, f_c \, A_g$ column, and more than three times that for the compression column. Ultimate curvatures are similarly affected. Thus, at least so far as the tension column is concerned, the $30\%$ limit to redistribution would seem quite unreasonable and the critical condition is likely to be the compression column, whose ductility demand we imagine to be reduced by the redistribution process (herein appears another fallacy).
THE STRENGTH/DUCTILITY TRADE-OFF FALLACY

The current design emphasis on force-based design, together with the general adoption of the equal displacement approximation leads us to the natural conclusion that required strength, \( S \), and displacement ductility demand, \( \mu_\Delta \), are related by the expression:

\[
S \cdot \mu_\Delta = \text{constant}
\]

for a given structure or critical element, with the usual caveat that for short period structures, the approximation may be inappropriate.

The fallacy of this observation becomes obvious when we invert the logic. Consider that we are designing a structural element, say a bridge column, and we decide that, as designed, the ductility capacity (and hence in a more basic sense the displacement capacity) is inadequate. As a consequence, we decide to increase the strength to reduce the ductility demand. We do this by increasing the longitudinal steel ratio, and keeping the section size constant. Have we really improved anything? Probably not – the equal-displacement approximation still says we require the same ultimate displacement capacity, even though the ductility demand has apparently reduced, and it is certainly not clear that increasing the longitudinal reinforcement ratio has increased the ultimate displacement.

In fact, quite the opposite is more likely. Figure 7 plots the results of varying longitudinal reinforcement ratio for circular columns with axial load ratio \( 0.1 f_c A_g \). Results for moment capacity, stiffness, ductility capacity and ultimate displacement are expressed in dimensionless form by reference to the value pertaining to a 'standard' longitudinal ratio of \( \rho_L = 0.0015 \). It will be seen that as \( \rho_L \) is increased, the ultimate moment capacity (\( M_u \)) increases almost proportionately, but the ultimate displacement capacity reduces somewhat (by about 10% at \( \rho_L = 0.03 \)), and the ductility capacity reduces even more. Of course, the argument also contains a fallacy, since the stiffness (\( K_e \)) has increased almost as much as the strength has, and thus the period will have changed. Nevertheless, it is unlikely that this stiffness variation will have been included in the original calculations, and the required increase in ultimate displacement has clearly not been achieved. It can even be reasonably argued that if we were worried about the ductility or displacement capacity, we would have been better off reducing the reinforcement ratio, and hence the strength.

THE ENERGY MYTH

One of the more pervasive myths in earthquake engineering is that energy dissipation should be maximized to obtain optimum seismic response. In this myth, it is supposed that we should strive towards obtaining hysteretic characteristics as closely approximating elastic/perfectly-plastic response as possible. Although there are situations, particularly those involving very short period structures where this is, in fact, desirable, there are many cases where better response can be obtained with apparently less desirable loop shapes.
Figure 8 compares three idealized hysteresis loop shapes — elastic/perfectly-plastic, degrading stiffness model typical of a reinforced concrete column hinge and the bilinear elastic characteristic theoretically appropriate for a plastic hinge with unbonded prestressing tendons. Response is shown both with and without P-Δ effects.

Consider first response without P-Δ effects. If response is in the 'equal-displacement' domain of Fig. 1, the peak response displacements of the three systems are likely to be very similar if each has the same initial stiffness. In actual fact, the displacements of the EPP system will on average be a little smaller than the other two, but the difference will not be great. Let us assume that a maximum drift of 2%, corresponding to a displacement ductility factor of $\mu_\Delta = 5$ is obtained. After the earthquake, the residual drift of the EPP system could be as high as 1.6%, that of the column hinge about 0.9%, and the bilinear system of Fig. 8(c) will return to its initial position. Which system has exhibited better response? It is at least arguable that residual displacements are ultimately more important than maximum displacements, given the difficulty of straightening a bent building after an earthquake.

Consider now the influence of hysteresis loop shape on response as affected by P-Δ effects. MacRae has rather convincingly shown that the tendency for instability under P-Δ is strongly related to the shape of the loop. In Fig. 8, the influence of P-Δ moments on the inelastic response shape is shown by dashed lines. With the EPP loop of Fig. 8(a), response at a given instant of the seismic response has resulted in a residual deformation corresponding to point B. The structure will oscillate with the elastic stiffness about this point until response acceleration sufficient to develop the yield strength develop. As will be seen in Fig. 8(a), the acceleration required to make the system reach the upper yield line is much less than that for the lower yield line. It is thus probable that plasticity will develop in the direction of increasing, rather than reducing residual displacement. With a long duration and an EPP loop shape, the system is inherently unstable under P-Δ effects.

With the degrading stiffness model of Fig. 8(b), and a residual deformation corresponding to point B, the lower yield line is closer to the zero acceleration line, and is thus more likely to be attained than the upper yield line. The system is thus inherently stable, since the probabilities of inelastic deformation favor decreased residual set. MacRae has demonstrated the validity of this argument with a very large number of dynamic inelastic time-history analyses.

Since the system in Fig. 8(c) is elastic non-linear, there are no residual displacements to be considered, and the system is stable for P-Δ effects.

It should also be noted that for longer period structures, where P-Δ effects are likely to be significant, the equal displacement rule would indicate that P-Δ effects are unlikely to significantly increase the maximum displacement of the stable systems significantly. This is also supported by time-history analyses. For the EPP system, which as noted is inherently unstable, neither the equal displacement nor equal energy rules can be applied, since the increase in maximum and residual displacement is strongly influenced by the duration of the earthquake record.
One should not, of course, dismiss the value of hysteretic energy absorption. However, it is clear that current emphasis on the loop shape is overstated. Steel structures, with deformation characteristics approximating EPP loops, have a greater tendency for undesirable deformation response than the equivalent reinforced concrete structures.

THE DISTRIBUTION OF FLEXURAL REINFORCEMENT FALLACY

The discussions above have largely related to analysis issues. However, it is clear that many aspects of design and detailing could also bear critical review. The remainder of this paper will examine a few issues specifically related to reinforced concrete design, though more could be identified, both with reinforced concrete, and with other materials.

One of the most pervasive fallacies relates to the way we distributed reinforcement in beams of ductile moment-resisting frames. By use of moment redistribution, we frequently end up with positive and negative moment demands that are equal, or nearly so. We then place reinforcement in two bands, as close to top and bottom respectively of the beam, as shown in Fig. 9(a), in the mistaken view that this provides the most efficient distribution. Wong et al. [9] have shown that essentially the same moment capacity can be achieved by distributing the total amount of reinforcement down the sides of the beam, as shown in Fig. 9(b). Figure 10 compares the flexural strength of the alternative distributions of Fig. 9, as a function of the mechanical reinforcement ratio $\rho = \frac{f_y}{f_c}$, where $\rho = \frac{A_s}{bh}$ is the total reinforcement ratio. The strength differences between the distributions are insignificant.

There are, however, good reasons for adopting the distribution of Fig. 9(b). Congestion at beam-column joints of two-way frame is considerably eased, a greater proportion of the joint shear force can be associated with the diagonal concrete strut, thus reducing the demand for joint shear reinforcement, and flexural overstrength resulting from strain-hardening of reinforcement is reduced. This latter point could be taken advantage of by reducing the overstrength ratio used to develop member forces in the capacity design process, thus resulting in design efficiencies.

Design efficiency could also be improved by more appropriate use of strength reduction factors in the basic capacity design equations, which can be generally stated as

$$\phi_s S_n \geq \omega_s \phi_o S_r$$

where $S_n$ is the nominal strength of a particular action (flexure, shear, etc.), $S_r$ is the strength required for that action resulting from the basic analysis assumptions, $\phi_s$ is a strength reduction factor applied to $S_n$ to provide a dependable strength, $\omega_s$ and $\phi_o$ are dynamic amplification factors and overstrength factors, which again relate to the analysis assumptions, design efficiency and action considered.

Currently in seismic design, we associate a strength reduction factor to the basic flexural strength of plastic hinges, but not to members or actions protected by...
capacity design principles, on the basis of perceived conservatism in the values of \( \omega_s \) and \( \phi_0 \) currently specified.

It would appear that the logic may have thus been inverted. It is clear that we do not need a flexural strength reduction factor for plastic hinges, since small variations in strength from the specified value will only result in small variations in ductility demand. As noted in relation to Fig. 7, increasing the reinforcement content, which is the end result of application of a flexural strength reduction factor, may not improve overall safety.

However, if we wish to totally proscribe non-ductile inelastic deformation (e.g. shear) we need a high degree of assurity that the dependable strength of that mode cannot be exceeded. If we believe strength reduction factors need to be associated with that action (e.g. shear) as a result of possible non conservatism of design equations, or possible material under strength, then they should be utilized in the capacity design process. If the product \( \omega_s \phi_0 \) is felt to be such that no strength reduction factor is needed, this implies that \( \omega_s \phi_0 \) is too high, and should be reduced.

This may seem to be a matter of semantics, since the end result would probably be little different in current design practice. However, if design is to be permitted in accordance with more advanced analytical processes (e.g. time-history analysis to determine expected dynamic influences once strength of plastic hinges has been determined) then values for \( \omega_s \) and \( \phi_0 \) might become determined by the results of the analysis process. In this case, it would be better to have the variability of design strength properly associated with the correct parameters.

Of course, the process could be greatly simplified if we did away with strength reduction factors completely, as is the case in Japanese design. The value of \( \phi \) would thus be inherent in the equations for strength. It is hard to see that we would lose much in the process.

THE SHEAR MYTH(S)

Shear design of reinforced concrete is so full of myths, fallacies and contradictions that it is hard to know where to begin in an examination of current design. Perhaps the basic myth, and that central to our inconsistencies in shear design is that of shear itself. It has been argued that we tie ourselves into intellectual knots by separating flexure and shear, and considering them essentially independent entities. Compression field theory as developed by Collins et al.\[10\] is an attempt to integrate the actions. Similar attempts have been made elsewhere\[11\]. The fact remains, however, that it is very convenient to separate the flexural and shear actions, and also that the more fundamental approaches are not only inconvenient from a design viewpoint, but also do not produce notably better agreement with experimental results, particularly when shear strength of ductile linear members characteristic of framed structures is considered.

Our understanding of the mechanisms of shear transfer in plastic hinge regions seems particularly weak. If we consider a beam hinge adjacent to a
column, as depicted in Fig. 11, the design assumption is that concrete shear mechanisms such as aggregate interlock, dowel action and compression shear transfer are undependable, because of the potential presence of a full depth flexural crack (see Fig. 11(a)) and hence shear must be entirely transferred by a 45° truss mechanism, involving transverse reinforcement, as shown in Fig. 11(b).

A little reflection reveals that the two halves of this assumption (wide full depth crack; truss mechanism) are mutually incompatible. The truss mechanism of Fig. 11(b), whether based on 45° or some other angle, relies on the development of diagonal compression struts, stabilized by vertical tension in the stirrups or ties, and changes in the longitudinal beam tension and compression resultants at the 'nodes' formed by intersection of the tie and diagonal forces. If a vertical section is taken at any position, the vertical component of the diagonal compression forces continuous across the section must equal the shear force transferred by the truss mechanism. At the critical section at the column face, there is a full depth flexural crack formed by inelastic action in the two opposite direction of response. As a consequence, there can be no diagonal compression forces crossing this section, and the shear carried by the truss mechanism must also be zero.

This apparent, and obvious, dilemma has been 'rationalized' by the assertion that the intersection of diagonal cracks, shown in Fig. 11(a) allows the cracks to dilate and close in the middle section of the beam, thus permitting diagonal compression struts to develop. There are two concerns to adopting this solution: first, if diagonal compression can develop across this crack, then perhaps the assumption that $V_c = 0$ should not be made since at least part of the rationale for discarding $V_c$ has been eliminated. Second, the assumption of diagonal compression implies a reduction to apparent flexural strength of the section. Consider equilibrium of the stress resultants in Fig. 11(b). The resultant of the diagonal compression forces may be assumed to act at midheight. If a 45° truss is assumed its vertical and horizontal components must both be equal to $V$ as shown. The flexural compression force acts at or near the center of compression reinforcement. Without the diagonal compression, the moment capacity is

$$M_i = T(d - d')$$  \hspace{1cm} (7)

If the diagonal compression force is included, the moment capacity reduces to

$$M_{iv} = T(d - d') - V\frac{(d - d')}{2}$$

$$= \left[ T - \frac{V}{2} \right](d - d')$$  \hspace{1cm} (8)

Neglecting the dead load shear, assuming equal moment capacity at opposite ends of the beam, and a beam length of $x(d - d')$,

$$V = \frac{2M_{iv}}{x(d - d')},$$  \hspace{1cm} (9)
and hence
\[ M_{iv} = T(d - d') - \frac{M_{iv}}{x} \]

or
\[ M_{iv} = \left( \frac{1}{1 + 1/x} \right) M_i \] (10)

In a deep beam with (say) \( x = 4 \), this implies a 20\% reduction in moment capacity, which is not supported by experimental results. Even with more slender beams (say \( x = 10 \)) the moment reduction should be evident. Note that when dead-load shear is added, negative moment capacity should be further eroded, if the truss mechanism were correct.

Since this behavior is not apparent in experimental results, it would appear that alternative mechanisms must be relied upon. If a full depth crack can develop, it is difficult to escape the conclusion that all shear must be carried by dowel action. If this is the case, the primary shear transfer function of transverse reinforcement in the plastic hinge region must be to reduce the unsupported length over which dowel action of the longitudinal reinforcement occurs, hence increasing the shear that can be transferred. Rational models for the amount of shear reinforcement required can be developed, which are very different from those resulting from the truss model. Critical aspects include diameter of longitudinal reinforcement (the bigger the better), and location of the first stirrup from the column face – which should be as small as possible.

As mentioned, there are many other inconsistencies in shear design. A particularly troublesome one is the way in which we treat the enhancement of shear strength by axial compression. Our codes indicate that the degree of enhancement depends on whether the structural element is a beam or a wall, and whether the axial compression comes from applied gravity forces, or from prestress. Differences in influence of more than 100\% are possible. Since the differences appear to result from semantic definitions, a degree of skepticism is appropriate.

DEVELOPMENT OF REINFORCEMENT FALLOACY

The final example considered in this paper is the way in which we consider development of reinforcement. Development is a catch-all term used to describe embedment or anchorage, splicing, and flexural bond. The same equations, with modifiers in some cases, are used to describe the three situations. Although this is convenient for design, it is unlikely to be realistic in practice, considering the wide number of possible situations occurring, as illustrated for a bridge column in Fig. 12. A number of rather different conditions exist depending on location, whether or not confinement is present, and whether or not the splitting cracks assumed in the basic development length equation can actually develop.

The fundamental basis of the basic development length equation can also be questioned. In the ACI code, this length is given by

\[ \ell_{db} = 0.019 A_b f_y / \sqrt{f_c} \] (11)
Where $A_b$ is in mm$^2$ and $f_y$ and $f_c$ are in MPa. Modifiers are included to represent the influence of cover, bar spacing, location in the concrete pour, confinement etc. Figure 13 shows the ratio of basic development length to $d_b$ diameter for U.S. bar sizes, for $f_y = 414$ MPa and $f_c = 22.5$ MPa. In Fig. 13, the equation definitions are in American Standard units. It will be seen that the basic development length increases in terms of number of bar diameters as the diameter increases. In fact, considering the range of bar sizes used in the USA ($#3 \pm 10\text{mm}$ to $#18 \pm 56\text{ mm dia.}$) Eqn. (11) implies the dimensionless development length increases by a factor of six, or the actual development length increases by a factor of 36. Tests on scale models do not support a scale dependency of this nature.

To some extent, of course, the modification factors applied to $l_{db}$ reduce the apparent scale dependency, but it would appear from Fig. 13, that the fundamental basis of our development calculations may be flawed. Returning to more fundamental approaches results in equations that are more intellectually satisfying$^{[12]}$, and fit the data better.

CONCLUSIONS

In this paper, a somewhat irreverent examination of aspects of seismic analysis and design has been presented. In order to establish inconsistencies and inaccuracies commonly accepted by the design profession, a number of the examples have been deliberately overstated. A critical review of this paper will show that some of the arguments are at best simplistic, if not flawed.

It should be emphasized that it is not contended that current seismic design practice, particularly the version adopted in New Zealand is unsafe. If this paper has a message or conclusion, it is simply the following cautionary note related to the tendency for increased complexity in analysis: given the wide range, and occasional gross nature of the assumptions and approximations inherent in seismic design, we might be better keeping the design and analysis processes simple enough so that we still understand what we are doing.

A second, and more serious (perhaps!) point brought out at the start of this paper related to displacement-based design: if we accept that displacements are more important than forces, it is time we started basing our designs on displacement, rather than acceleration spectra.

REFERENCES


Fig. 1—Design acceleration response spectrum

(a) Structure (b) Plastic Displacement

Fig. 2—Bridge bent example
a) Design displacement spectra

b) Actual displacement spectra (Brawley, 1979, 315 deg)

Fig. 3—Displacement response spectra for design
Fig. 4—Equivalent viscous damping versus displacement ductility level

Fig. 5—Maximum response of frame building
Fig. 6—Influence of axial force on moment curvature relationship for rectangular column (1 kNm = 8.85 kip in., 1 m = 39.37 in.)
Fig. 7—Dimensionless change in structural response of cantilever bridge pier as function of longitudinal reinforcement ratio.

Fig. 8—Response for different hysteretic models including P-Δ effect.
(a) Conventional Reinforcement  (b) Vertically Distributed Reinforcement

Fig. 9—Arrangements of longitudinal reinforcement in beams

Fig. 10—Dimensionless flexural strength for beams with conventional and vertically distributed reinforcement
Lateral Force Transfer in Buildings

a) Conditions at high ductility

\[ T = A_f f_y \]

b) 45 deg truss mechanism

\[ C_p = V \]

\[ C = T - C_p \]

Fig. 11—Shear transfer in beam plastic hinges
Fig. 12—Anchorage and splicing of reinforcement


c) Column top  
d) Column midheight lap splice

Fig. 13—Basic development length for deformed bars in tension \( f'_{c} = 3.25 \text{ ksi}, f_{y} = 60 \text{ ksi}, \\
1 \text{ ksi} = 6.90 \text{ MPa, 1 in. = 25.4 mm} \)
Formulation of a Conceptual Seismic Code

by V. V. Bertero and R. D. Bertero

Synopsis: A new conceptual code format has been developed for earthquake-resistant design (EQ-RD) of buildings. It consists of: (1) guidelines for conceptual overall design of entire building systems and (2) a conceptual methodology for numerical EQ-RD of building systems in compliance with the worldwide-accepted EQ-RD philosophy and based on energy concepts, fundamental principles of structural dynamics, mechanical behavior of entire building facilities, and comprehensive design. The numerical EQ-RD methodology considers the desired seismic performance of the entire building system explicitly from the beginning of the EQ-RD process, and concludes by evaluating whether such performance would be achieved. A discussion of the main aspects and problems involved in the preliminary numerical EQ-RD procedure is presented. Main results from its application to a 30-story RC space-frame building are discussed and compared to results from analysis of the performance of the same building designed according to 1991 UBC, showing the weakness of present UBC seismic regulations when applied to tall buildings, particularly regarding performance under service-level EQ ground motions. The main advantage of the proposed conceptual methodology is that uncertain quantifications of its concepts can evolve without changing the format of the codified methodology as new and more reliable data are acquired.

Keywords: ductility; earthquake-resistant structures; hinges (structural); loads (forces); standards; stiffness; structural design
APPRECIATION

It is an immense pleasure for the senior author to be able to participate in this international symposium to honor the 70th birthday of Prof. Tom Paulay. He met Prof. Paulay for the first time in 1969 at the Fourth World Conference on Earthquake Engineering in Santiago, Chile, where Prof. Paulay presented a paper on "the Coupling of Reinforced Concrete Shear Walls." Since that opportunity, the senior author has been following Prof. Paulay's publications very closely and has had not only the pleasure of listening to many of his inspiring presentations of papers and lectures on earthquake-resistant design of RC structural walls and RC ductile frames, but also the privilege of having valuable personal discussions in these important areas of earthquake-resistant construction. Professor Paulay has been the leading researcher in these areas and his work has opened the way to improving the design procedure, construction details and building codes around the world. In congratulations, we wish him the very best in his future activities.

Vitelmo V. Bertero
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INTRODUCTION

A recent statistical study [1] shows that seismic risks in our urban areas are increasing with the years. Most human and economic losses due to moderate and severe EQ Ground Motions (EQGMs) are caused by the failures of human-made facilities, which presumably are designed and constructed to protect their occupants against expected natural hazards. In practice, the design and construction of any facility located in a region of seismic risk follows code design procedures and regulations. Analysis shows that current seismic code design methodologies fail to realize the goals and objectives of the worldwide-accepted philosophy of Earthquake-Resistant Design (EQ-RD). Also, present seismic codes are not transparent, i.e., their regulations do not present in a visible way the basic concepts that govern EQ performance of structures. Thus, there is a need to improve EQ-RD by identifying the basic problems created by EQs and finding out how these problems can be solved and how the solutions can be codified in a transparent way. From analysis of the various approaches and methodologies suggested for such an improvement, it has been concluded that in the end the best solution is a new transparent format for seismic codes, covering in logical sequence all aspects that a seismic code should regulate, whose regulations will invoke basic concepts rather than empirical rules. For reliable application of these regulations in practice, the code regulations must remain simple and in accordance with the education in EQ Engineering of the practitioners. Therefore a three-step approach is proposed for the final formulation of the simple seismic code regulations.

First Step -- Based on the state of the art in EQ Engineering, a conceptual seismic code should be developed, covering all aspects that a seismic code should regulate. Given the different groups of aspects and problems involved in EQ-RD and EQ-RC [1, 2], the conceptual seismic code will consist of regulations that can be grouped as follows.

Group 1. Guidelines for assessing seismic activity and sources of potential seismic hazards (damage); restrictions for land use and guidelines for the selection of building sites and corresponding siting restrictions; and procedures for site suitability analysis.
Group 2. For a selected site and function of a building, conceptual establishment of the EQ-RD criteria, design EQGMs, and design methodology.

Group 3. Conceptual overall design of the entire building system, covering restrictions and/or guidelines regarding: selection of building configuration or form (size and shape), foundation, structural layout, structural system, structural materials, and nonstructural components (potential unintentional structural components) and their materials.

Group 4. Conceptual preliminary numerical design of the whole facility system, which requires prediction of the mechanical behavior of such a system and involves: proper modelling of the entire system; estimation of the demand on the structure and its contents (structural and stress analysis) at the different levels of design EQGMs; preliminary sizing and detailing through estimation of the capacities to be supplied to the structure.

Group 5. Reliable analysis of the performance of the preliminarily designed facility when subjected to the expected critical EQGMs at each of the limit states contemplated in the design criteria.

Group 6. Final design (detailing).

Group 7. Monitoring of field construction, function (use) and maintenance (alterations, repair, and/or upgrading) of the constructed structure.

Group 8. Conceptual methodology for the upgrading of hazardous facilities.

Second Step -- The conceptual code regulations will be applied to the design of building facilities with different regular and irregular configurations, structural layouts and structural systems, and to the upgrading of different types of existing hazardous facilities. In order to judge current or modern code procedures, the designed buildings will preferably be selected from among those that have been designed and constructed according to current or modern seismic codes and have available records or predictions of the response to EQGMs, and the existing hazardous facilities preferably will be selected from among those that have been recently upgraded.

Third Step -- From analysis of the results obtained in the second step, a simplified conceptual code that can be applied properly by the practitioners should be developed. It should state clearly all restrictions in siting and in selection of configuration (or form), foundation systems and structural systems for which such simplified code regulations could be used. For complex buildings, a peer review process should be required in which the conceptual code to be developed in the first step could be used.

Importance and Advantages of Formulating a Conceptual Seismic Code.

The importance and advantages of developing conceptual seismic code provisions
are discussed in Refs. 1-3. The importance of a conceptual "overall design" (i.e., an overall conception of the configuration or form of a building and its foundation; and selection of the structural layout, structural systems and material and nonstructural components that could become unintentional structural components) is discussed in detail in Refs. 4-8. The importance of formulating a conceptual methodology for numerical preliminary EQ-RD of structures based on well-established fundamental principles of structural dynamics, mechanical behavior of the entire facility system and comprehensive design, and in compliance with the worldwide-accepted EQ-RC philosophy, is discussed in Refs. 1-3. Its main advantages are: (1) it leads to a transparent numerical design procedure that considers and checks the selected or desired performance objectives; and (2) in spite of the great uncertainties in the quantification of some of the concepts involved in its codification, such quantifications can be improved as new or more reliable data become available without changing the philosophy and particularly the format of this codified methodology. Another important advantage is that such a formulation can be used as a basis for improving the education of architects and engineers, as well as for the establishment of the much-needed prioritization and program of the focused research needed to improve EQ-RC.

Objectives and Scope of Paper.

The main objective of this paper is to summarize the development of a "conceptual seismic code," with particular emphasis on a framework for what can be called a conceptual methodology for numerical EQ-RD. The basic ideas of the proposed conceptual methodology and some of the main observations and results obtained in its application to a 30-story RC frame building are presented. These results are compared to results obtained in the analysis of the performance of the same building designed according to the 1991 UBC. Main conclusions and recommendations for research needed to improve the quantification of the developed conceptual methodology are offered.

FORMULATION OF A CONCEPTUAL METHODOLOGY FOR THE EQ-RD OF BUILDING STRUCTURES

Among the different groups of aspects or problems that the conceptual seismic code should regulate, as listed in the introduction, the following groups must be considered in order to formulate a conceptual methodology and the corresponding code provisions for the EQ-RD of a building facility.

Group 2. (a) Conceptual establishment of the design criteria according to the desired function (occupancy) or performance of the building. (b) According to the selected site, conceptual establishment of the design EQGMs, as well as any other
source of potential hazard that needs to be considered in the design. (c) Selection of the design method.

**Group 3.** Conceptual overall design (conception) of the entire building system.

**Group 4.** Conceptual preliminary numerical design of the whole facility system.

**Group 5.** Reliable analysis of the performance of the preliminarily designed building when subjected to the established critical EQGMs.

**Group 6.** Final design (detailing).

All of the above groups, with the exception of Group 3, can be considered under the umbrella of **conceptual methodology for numerical EQ-RD**. Therefore, the conceptual methodology for EQ-RD can be considered to consist of two main parts: the conceptual overall design (conception) of the facility system; and the conceptual methodology for numerical EQ-RD. Although these two main parts are discussed separately below, they are actually intimately interrelated.

**Conceptual Overall Design.**

Conceptual overall design is the avoidance or minimization of problems created by effects of seismic excitations, using understanding of behavior rather than numerical computations [7].

Conceptual overall design of the facility system involves not only the choice of overall shape and size of the building, but also the selection of the structural layout, the structural system, the structural material, type of nonstructural components (particularly those that could become unintentional structural components), and the foundation system. Both the architect and the engineer have to understand how design decisions regarding building layout may have serious seismic effects on the structure. The inertial forces depend on the mass (amount and distribution), the damping, and the structural characteristics (stiffness, yielding strength, maximum strength and energy absorption and energy dissipation capacities). In 1979, Arnold [6] reported that 65% to 80% of buildings designed within the last 15 years were of irregular form. This percentage does not agree with the conceptual basis upon which the UBC specifications were then based. Although there is no universal ideal building configuration, certain basic principles of EQ-RD can be used as guidelines to select adequate building and structural configuration [4-8].

**CONCEPTUAL METHODOLOGY FOR NUMERICAL EQ-RD**

The conceptual methodology for numerical EQ-RD of new structures covers problem groups 2 and 4-6. A detailed discussion of the conceptual methodology, which is based on energy concepts, and its solution of the problems in group 2 and some of the problems in groups 4-6, is offered in Refs. 3 and 9. In the following
formulation of the problem of EQ-RD of building structures, it is assumed that the conceptual overall design part of the overall EQ-RD (group 3) has already been solved [9].

**GIVEN:** • Function of building; site of building; and general configuration of building (structural layout, structural system, structural material and nonstructural components and their materials).

**REQUIRED:** • An efficient (optimum) EQ-RD of the building.

**SOLUTION:** • A technically efficient and economical solution requires an iterative procedure, starting with an efficient preliminary EQ-RD and ending with a final design.

The procedure for achieving efficient EQ-RD of a structure should be rational, transparent and reliable. It is convenient to divide the preliminary EQ-RD procedures into two main phases: the establishment of design EQs (design EQGMs) and the design procedure for the building against them (Fig. 1).

**First Phase: Establishment of Design EQGMs.**

This phase covers acquisition and processing of data for establishment of design EQGMs (Figs. 1 and 2).

**Acquisition of Data** -- The needed data and the problems involved in acquiring them can be summarized as follows.

**GIVEN:** • The site of the building (soil profile and topography).

**REQUIRED:** • Return periods of different levels of possible EQGMs at the site and their damage potential to the entire building system for at least its service and safety limit states.

**SOLUTION:** • Conduct a reliable analysis of the site; identify all of the sources of EQGMs that could affect the building; define the seismic activity at the site due to all possible EQ sources in the form of time histories and recurrence periods \( T_r \) of EQGMs; select \( T_r \) for at least two limit states of EQGMs considered in the general philosophy of EQ-RD (the service or functional level and the safety level).

Ideally, acquisition of the needed data should be based on EQGM records from the site. If there are not enough records, the data can be obtained either from EQGMs recorded at sites with similar soil profile and topography, or by using numerical synthesis [10] to generate several probable EQGM time histories.

**Processing of Data** -- In this key step, the available data about probable future EQGMs at the site are processed to facilitate reliable selection of the design EQs. Conceptually, a design EQ should be the critical EQGM for the limit state under consideration, i.e., the EQGM that drives the structure to its critical (maximum)
response for the failure stage under study. However, the application of this simple concept in practice meets with serious difficulties, and it has been shown that the reliability of the design EQs recommended by current seismic codes is highly questionable. According to recent studies [1-3, 9, 11, 12], the problems involved in this step, and their solutions, can be summarized as follows.

**GIVEN:**
- Time histories of probable EQGMs for at least service and safety limit states.

**REQUIRED:**
- For serviceability limit state: Smoothed Linear Elastic Design Response Spectra (SLEDRS) for strength ($C_s$) and displacement ($S_d$) for different damping ($\xi$).
- For safety limit state: the SLEDRS and Smoothed Inelastic Design Response Spectra (SIDRS) (for different values of the displacement ductility ratio, $\mu$, and $\xi$) for $C_s$ and $S_d$, and for the parameters needed for evaluation of the cumulative damage caused by cyclic load reversals [input energy ($E_I$), damping energy ($E_\xi$), plastic hysteretic energy ($E_{H\mu}$), cumulative ductility ratio ($\mu_d$), Number of Yielding Reversals (NYR), and Number of Equivalent Yielding Cycles at $\mu_{\text{max}}$ ($\text{NEYC}_{\mu_{\text{max}}}$)].

**SOLUTION:**
- Computation of the Linear Elastic Response Spectra (LERS) and the Inelastic Response Spectra (IRS) (for different values of $\mu$ and $\xi$) for $C_s$ and $S_d$ for each possible EQGM that can be generated at the site from the EQ sources. From statistical studies of LERS and IRS find SLEDRS and SIDRS. In smoothing the LERS and IRS, close consideration should be given to the standard deviation, $\sigma$, as well as to uncertainties in the estimation of the dynamic characteristics of future EQGMs and of the entire building system. To obtain the critical EQGMs to be considered for safety level, where some damage is tolerated (i.e., $\mu > 1$), it is necessary to compute for each EQGM the following spectra: $E_I$, $E_\xi$, $E_{H\mu}$, $\mu_d$, NYR, and $\text{NEYC}_{\mu_{\text{max}}}$, and hysteretic behavior history. Selection of critical EQGMs can be simplified using recently proposed damage indices and by introduction of the $\gamma$ factor, as discussed in Refs. 3, 9 and 13.

In current practice, design for the safety limit state is done on the basis of only strength, $C_s$, Smoothed Inelastic Design Spectra (SIDS) and in some cases a displacement, $S_d$, SIDS derived directly from the $C_s$ SIDS. As these spectra do not reflect the effect of the duration of the strong motions on inelastic demands ($E_I$, $E_\xi$, and particularly $E_{H\mu}$, $\mu_d$, NYR and $\text{NEYC}_{\mu_{\text{max}}}$), which can have significant effects on the damage potential of an EQGM, it is necessary to compute the spectra of these new parameters or to use the simplifications discussed in Refs. 2, 3 and 9.

**Second Phase: Design Procedure.**

This second phase of the proposed conceptual methodology for numerical EQ-RD is devoted to the design (sizing and detailing of the members and their connections and supports) of the entire building system against the critical combinations of the established design EQs with other excitations that can act simultaneously on the
Preliminary Design Procedure -- The main objective of this phase is a design which is as close as possible to the desired final design. As illustrated in Fig. 1, the preliminary design phase consists of three main steps: (i) preliminary analysis, (ii) preliminary design, and (iii) analysis of preliminary design.

1. Preliminary Analysis. The objective of this first group of steps is to establish the design criteria and estimate the acceptable maximum fundamental period (T) and the design forces (critical combinations among all of the forces that can be induced). In the conceptual methodology, preliminary analysis of the design problem can be formulated as follows.

GIVEN: • Function of building; • general configuration of the building, structural layout, structural system, structural materials and nonstructural components and contents; • gravity, wind, snow and other possible loads or excitations; and • SLEDRS and SIDRS for expected service and safety EQGMs. The SIDRS should be based on the selected acceptable value of the damage index (alternatively, the spectra of the γ factor must be given [3, 9, 13]).

REQUIRED: • Establishment of the design criteria, the minimum stiffness (or maximum T) capable of controlling the damage (maximum deformation and deformation rates of the building), the design seismic forces, and the critical load combinations.

SOLUTION: • Based on a transparent approach that takes into account from the beginning that: the structure is a Multi-Degree-of-Freedom System (MDOFS); there can be important torsional effects even under service EQGMs (i.e., in the linear elastic response) and that for safety EQGMs these effects can be different; and it is necessary to consider the desired value of the damage index (control of damage) for selecting the appropriate μ that can be used, as well as the expected overstrength.

Figure 3 shows a flow chart of the steps involved in the preliminary analysis for estimating the design seismic forces. Note that although the main purpose of this step is analysis of the problem (i.e., what is given, what is known, and what is needed), it is clear that preliminary design of member sizes is in fact necessary in order to control the maximum deformation and deformation rates and to obtain the T₁ to be used for estimating the design forces.

2. Preliminary Design. This step (assuming that preliminary sizing for stiffness was done in the preliminary analysis) can be stated as follows for a RC building:

GIVEN: • Gravity, wind, snow and seismic design loads for service and safety limit states; critical load combinations; and mechanical characteristics of the structural and nonstructural materials.

REQUIRED: • Preliminary sizing and detailing of both the structural elements
[beam and columns sizes and their flexural reinforcement (in the case of moment-resisting space frames)], and the unintentional structural (sometimes called nonstructural) components, which can affect the seismic response of the building.

**SOLUTION:** Based on an application of linear optimization theory, the design of beams and columns in each story minimizes the volume of flexural reinforcement (in the case of RC), using practical requirements and service forces and moments as constraints so that the preliminary design simultaneously considers the demands for serviceability and safety.

### 3. Analysis of Preliminary Design.

**GIVEN:**
- General configuration of building, structural layout, structural system, structural material and its mechanical characteristics, and nonstructural components and their materials and mechanical characteristics;
- Sizes and reinforcement of intentional and unintentional structural components; and
- Design EQs, critical load combinations and possible critical EQGMs for service and safety limit states.

**REQUIRED:** Determine the acceptability of the preliminary design, i.e., check if it satisfies the desired performance according to the established design criteria.

**SOLUTION:** Check interstory drift index (IDI); floor velocity and acceleration; stress-ratios; axial, flexural, and shear stresses in members and joints; adequacy of foundation; and local damage index (DMI) under critical EQGMs at each limit state using static and dynamic load analyses.

Because of the importance of an efficient preliminary EQ-RD that is as close as possible to the desired final design, a discussion of the main aspects and problems involved in the preliminary design procedure is presented below.

**Discussion of 1. Preliminary Analysis.** One of the most important data for attaining a reliable numerical design is the reliable quantification of the excitations against which the structure is to be designed. Thus, proper selection of the design EQs is an important and very difficult task in efficient preliminary EQ-RD of a structure. As summarized above under "Establishment of Design EQs" and discussed in detail in Refs. 3 and 9, it is necessary to compute a series of spectra. At present most of these spectra are computed just for SDOFS, but because a real building generally is a MDOFS, it is necessary to modify the obtained SDOFS spectra.

** Modifications of SDOFS Spectra to Account for MDOFS** -- In selecting the design global displacement ductility ratio, $\mu_g = \mu_{SDOF}$, to be used to find the SIDRS for the design of the equivalent SDOFS, it must be considered that for an MDOFS the demanded story ductility ratio, $\mu_s$, will not usually be uniform along the height of the structure, i.e., there will always be a story whose $\mu_s$ will be larger than the global $\mu_g$. Therefore, the $\mu_g$ selected for the design of the SDOFS equivalent to the MDOFS should be somewhat smaller than that selected for the design of a real SDOFS. The taller the building and the larger the structural irregularities along its height, the smaller this equivalent $\mu_g$ should be [14]. As indicated in Fig. 3, the SIDRS developed for the SDOFS considering a $\mu_g$ modified to consider that the real structure is a MDOFS still needs some additional modifications, depending on the
type of design to be conducted and the possible effects of torsion. These modifications, which are discussed below, are needed to obtain a preliminary design as close as possible to the desired final design.

**Modifications of the SIDRS to Account for Design Method** -- Because in EQ-RD the critical regions of the members, and therefore the members themselves, are usually provided with a larger strength than is required, and particularly because critical regions are usually provided with a large local \( \mu_p \), elastic design methods usually result in buildings with strength higher than their design strength, resulting in designed and usually constructed structures with significant lateral overstrength over code-required strength. This overstrength varies with the fundamental period, \( T_1 \), of the structure. The taller the building (the larger the \( T_1 \)) and the fewer structural bays, the smaller the overstrength will be. It should be noted that if the designer tailors the main reinforcement so that all of the critical regions of each of the members reach their demanded strength simultaneously, the resulting structure’s overstrength will be reduced. Thus it is not only difficult, but even dangerous, to attempt to codify just one constant value for such overstrength. Similarly, if the designer uses the ACI code [15] strength method with the redistribution due to plastic deformation allowed by this code, the overstrength will also be reduced. Use of an inelastic design method that accounts for plastic redistribution of the internal forces that are demanded elastically from the structure will result in a decrease in overstrength. However, the degree of decrease depends on how the design is conducted. For a design based on an optimization, such as that suggested in Ref. 9, the overstrength will be reduced to a minimum, but can still be significant. The actual dynamic overstrength during the dynamic response to recorded EQGMs is even higher than that estimated under equivalent static load (pushover test) [16].

In specifying the possible reduction in the ordinates of the SIDRS for \( C_s \) due to overstrength, it is necessary to consider both the possible overstrength and its effect on maximum IDI. An increase in IDI beyond the acceptable limit can control the reduction. Thus, the final design will generally have a maximum yielding strength larger than that required by the adopted yielding strength spectra, \( C_s \), and therefore the response ordinates of such SIDRS can be reduced by a factor \( R_{OVS} \). The problem is, how much can the value of \( R_{OVS} \) be? Because this value depends on many variables (design method, dynamic effects, \( T_1 \) and \( T_1/T_\Theta \), etc.), at present its selection requires seasoned judgement and should be done conservatively, until needed research produces the calibration data for its proper selection. Detailed discussion of potential overstrength sources is given in Refs. 9, 14 and 17.

**Modifications due to Torsional Effects** -- Because of torsion, the demanded strength and IDI at certain parts of the structure increase over those required by just translational deformation. The larger the eccentricity between the center of rigidity and the center of mass, the larger the torsional effects. These effects differ under service and safety EQGMs. At the safety level involving inelastic behavior, they can increase significantly, depending on the initial location of the center of torsional resistance and how this center and the resulting yielding resistance eccentricity are
changing in the yielding of the structure \[18\]. Torsional effects are currently considered in the estimation of seismic design forces either by modifying the distribution of computed total design base shear and average IDI obtained from the design spectra, or even later, in the analysis of the preliminary design. In the proposed conceptual methodology, torsional effects are considered from the beginning by direct modification of the value obtained from the spectra (Fig. 3). The authors have developed a series of practical equations for doing this \[9\].

Discussion of 2. Preliminary Design. As stated in the summary of the preliminary design steps, it is proposed to obtain the required sizing of the structural elements and the amount of their reinforcement story by story, starting from the roof. Plastic design and optimization theory will be employed, using as an objective function the minimization of the flexural reinforcement (in the case of a RC structure), and introducing as constraints all practical requirements as well as the axial-flexural strength required by the service design EQ, so that the preliminary design will satisfy simultaneously the demands imposed by the service and the safety design EQs. For a detailed discussion of this preliminary design step, the reader is referred to Ref. 19. For solving the numerical problems involved in the preliminary design, it is suggested that an electronic spreadsheet, rather than a Fortran computer program, be used.

Discussion of 3. Analysis of Preliminary Design. As indicated in the flow chart of Fig. 1 and in the summary presented previously, the main objective of this step is to determine the acceptability of the preliminary design according to the desired performance in the adopted design criteria. This requires a series of reliable analyses of the demanded values for the main response parameters used in the establishment of the design criteria, considering the main limit states through which the whole facility system could pass during its service life. The main parameters are: total weight (\(W_T\)) and weight of the reactive mass (\(W\)); IDI; floor velocities and accelerations; stress ratios; shear stresses in the members and their joints; global, story and local ductility; and local damage index (DMI). These should be done for the critical EQGMs for each different limit state considered in the design criteria. To carry out the above checks it is necessary to conduct linear and nonlinear analyses. The ideal would be to conduct a 3-D dynamic analysis which would consider all of the components of the critical EQGMs (or at least their two translational horizontal components) acting simultaneously. While at present this is easily done for the serviceability limit state because it requires the use of just dynamic linear analysis, checking the safety limit state is more difficult, requiring the use of reliable 3-D dynamic nonlinear analysis computer programs for multi-story buildings. Available general 3-D dynamic nonlinear computer programs require large computers and significant computer time. Therefore, at present, attempts should be made to conduct the 3-D analysis using the static lateral load (pushover) method with a proper load pattern, or, even better, analyses considering the probable bounds of such a pattern. If 3-D analysis programs are not available or can not be used because of the powerful computers required, attempts should be made to use pseudo-3-D programs. The disadvantage of pseudo-3-D programs such
as DRAIN-2DX is that they cannot estimate the effects of torsion and multidirectional input.

While the static pushover method can give an idea of the first significant local yielding strength, it can significantly underestimate the actual dynamic global yielding strength. Furthermore, the results of such pushover analyses can significantly underestimate the local cumulative plastic rotation, and will not reveal the possibility of a shakedown problem, particularly in very slender, tall RC buildings [9]. Thus, efforts should be devoted to developing practical and reliable 3-D dynamic nonlinear analysis computer programs that can conduct 3-D time-history analyses of the response of buildings when subjected to the time-history components of the critical EQGMs.

APPLICATION OF CONCEPTUAL NUMERICAL METHODOLOGY TO THE PRELIMINARY DESIGN OF A 30-STORY BUILDING

The proposed conceptual methodology for numerical design of tall buildings has been applied to the preliminary design of a building similar to an existing 30-story RC building designed and built in Japan (Fig. 4) [20]. A summary of the design follows.

Problem Statement.

2. Site location: east shore of San Francisco Bay.
3. Site condition: soft soil, 10-14 m bay mud (1 sec<T_g<1.5 sec, T_g = soil period).
4. General configuration: structural layout and structural system (Fig. 4).

REQUIRED: An efficient EQ-RD of the building.

SOLUTION: The steps for the solution of this problem follow.

Establishment of Design EQs (Design EQGMs) -- Given the site of the building and using the procedure described previously for acquisition and processing of the data, the following spectra needed for preliminary design were obtained.
- For serviceability limit state: the mean + σ SLEDRS for C_s and S_d.
- For safety limit state: the mean + σ SLEDRS and SIDRS for different values of μ for C_s, S_d and parameter γ for evaluation of the cumulative damage caused for cyclic load reversals.

Preliminary Design Procedure --

Preliminary Analysis. In the following sections, the equations developed to introduce the effects of MDOFS, torsion, damage index and overstrength shown in Fig. 3 are presented. These equations allow the designer to control in a rational and
simple way the main parameters that influence the seismic response of a structure from the beginning of the design. The theory behind these formulas is presented in Ref. 9.

Selection of Design Criteria. Two levels of design ground motions are used: serviceability and survivability (safety). Table 1 summarizes the limit state design criteria for the two levels of earthquake hazard for a particular building.

Critical Load Combinations.

a) Service limit state. For serviceability, instead of considering the loads at the allowable stress level, it was decided to consider the load combination that in a period of ten years has a probability of 20% of producing the first yielding in the structure, e.g., the following load combinations:

\[ D_n + L_n \quad D_n + 0.4 L_n \pm W_n \quad D_n + 0.4 L_n \pm EQ_{ser} \]

where \( D_n \) = nominal dead load, \( L_n \) = nominal live load (reduced considering the tributary area), \( W_n \) = nominal wind load, and \( EQ_{ser} \) = seismic demand for service.

b) Safety limit state. For safety [i.e., the load combination that in a period of 450 years has a 20% probability of producing a level of damage, represented by a damage index \( DMI = DM = 0.8 \) (pre-collapse) of the structure] the following load combinations were selected.

\[ 1.4 D_n \quad 1.2 D_n + 1.6 L_n \quad 0.9 D_n - 1.3 W_n \quad 1.2 D_n + 0.5 L_n + 1.3 W_n \]
\[ 0.9 D_n - 1.0 EQ_{saf} \quad 1.2 D_n + 0.5 L_n + 1.0 EQ_{saf} \]

where \( EQ_{saf} \) = seismic demand for safety.

Estimation of Mode Shapes. For preliminary design, the torsional effects will be introduced by amplifying the assumed translational response of the building. Then, considering only translational modes, a good estimation of the response of tall buildings can be obtained using the first three modes. For preliminary design, the mode shapes and the ratio between periods can be selected from the following three types of behavior: uniform shear beam, linear first mode, or flexural beam. Once the type of behavior is assumed, the following information is easily obtained: the mode shapes, \( \varphi_1(x) \), \( \varphi_2(x) \) and \( \varphi_3(x) \), and the ratio between periods \( T_2/T_1 \) and \( T_3/T_1 \), as well as the global parameters, \( E \) and \( M_i^* \). For the 30-story building the mode shapes given in Table 2 were selected.

Required Stiffness for Serviceability Limit State. Assuming that the LEDRS for displacement, \( S_d(T) \), was obtained (following the guidelines discussed previously and in detail in Ref. 9), the following equation was developed to estimate the design period, \( T_1 \), for the first mode of the structure assuming a first mode shape \( \varphi_1(x) \) and a distribution of mass \( m(x) \) along the height of the building, \( H \).
In this equation, the factor $\beta_1$ takes into account the increase in displacement due to the dynamic effects of torsion for each planar frame (i.e., each plane of stiffness). This factor was introduced in order to carry out preliminary design of buildings. It was found to be mainly a function of the value of the ratio of eccentricity to the radius of gyration, $e/r$, of a typical story, and the ratio between the pure torsion and the pure translational periods, $\alpha_\Theta = T_T/T_\Theta$. The output of this step is the maximum period of the building, $T_{1ser}$ that can satisfy the required maximum IDI for the service limit state (see Fig. 5).

**Required Stiffness for Safety Limit State.** Assuming that the IDRS for displacement, $S_d(T, \mu_{SDOF})$, was obtained, in order to find the required stiffness for the safety limit state it is necessary: (a) to estimate the story ductility, $\mu$, that can be allowed in order not to surpass the target Park and Ang damage index $(DMI)_{PA} = 0.8$; (b) to estimate an equivalent SDOFS ductility, $\mu_{SDOF}$, taking into account the concentration of damage due to MDOFS effects; (c) to estimate the relationship between the maximum displacement of the equivalent SDOFS $S_d(T, \mu_{SDOF})$ and the IDI of the building considering inelastic torsion and the possible concentration of IDI in a particular story.

**Story Ductility for the Target Damage Index, $(DMI)_{PA}$.** In view of the limitations of using constant displacement ductility or constant hysteretic energy dissipation as damage criteria [3, 9], a damage index, $(DMI)_{PA}$, that combines these two factors was used. Using this criterion, the maximum story ductility ratio, $\mu_{max}$, that takes into consideration reductions due to $E_H$, can be computed as a function of the ultimate monotonic ductility, $\mu_{u,mon}$, from the following equation.

$$\mu_{max} = \frac{\sqrt{1 + 4(DMI)_{PA} \beta \gamma^2 \mu_{u,mon}} - 1}{2\beta \gamma^2}$$  \hspace{1cm} (2)

where $\mu_{u,mon} =$ ultimate monotonic ductility story capacity, $\beta =$ model parameter, and $\gamma =$ damage parameter obtained for the expected EQGMs at the site.

**Equivalent SDOF Ductility Demand.** The possibility of a concentration of ductility demand in one story in the case of MDOFS is considered using a factor $\beta_2$, which depends mainly on the structural period and system. The maximum displacement ductility [such that the target damage index, $(DMI)_{PA}$, for the real MDOFS building is not surpassed] of an equivalent SDOF system is computed using this factor as $\mu_{SDOF} = \mu_{max}/\beta_2$.

**Maximum Interstory Drift Index, IDI.** The following equation was developed to
obtain the maximum period, \( T_1 \), so that the maximum IDI_{saf} for the safety limit state is not surpassed.

\[
IDI = \frac{\partial \varphi}{\partial x} \beta_2 \beta_3 S_d(T, \mu_{SDOF}) \leq IDI_{saf} \Rightarrow T_{1safr} \tag{3}
\]

In this equation the factor \( \beta_3 \) takes into account the increase in displacement due to torsion at the ultimate stage for each plane of strength. This factor depends on the position of the center of rotation when the structure yields in a torsion-translation mode. The output of this step is the maximum period of the building, \( T_{1safr} \) that can satisfy the required maximum IDI_{saf} for safety limit state (Fig. 6).

**Preliminary Design of Member Sizes.** At this stage, the maximum allowable period of the building, \( T_{1\text{max}} = \text{minimum of} \ T_{1\text{ser}} \text{ and } T_{1\text{safr}} \) has been obtained. An iterative design of member sizes until the fundamental period of the designed structure, \( T_1 \), is computed to be smaller than \( T_{1\text{max}} \) is required next.

**Service Design Forces.** With the period, \( T \), selected from the above requirement of IDI and the assumed linear shape for the first mode and the computed modal parameters summarized in Table 2, the other periods, \( T_2 \) and \( T_3 \), are computed. The total base shear can be estimated from the elastic response spectra and the assumed modal parameters as

\[
V = W \sqrt{\sum_{i=1}^{3} \left[ \frac{\partial \varphi_i}{\partial x} \beta_1 \beta_2 \beta_3 S_d(T, \mu_{SDOF}) \right]^2} \tag{4}
\]

The base shear in each structural plane \( j \) can be computed, taking torsion into account, as \( V_j = (k_j / K) \beta_1 \beta_j V \), where \( k_j / K \) is the relative stiffness of the frame (in the plane) \( j \) with respect to the total stiffness, \( K \), in the considered direction, and \( \beta_1 \beta_j \) is the factor that takes into account the torsion for the \( j \) plane of stiffness.

The shear, \( S_k \), in the \( k \) story can be computed as

\[
S_k = \sqrt{\sum_{i=1}^{3} \sum_{l=1}^{a} M_i \varphi_i(z_l) \frac{\partial \varphi_i}{\partial z_l} \frac{S_d(T, \mu_{SDOF})}{M_i}} \tag{5}
\]

where \( M_l \) = reactive mass lumped in floor \( l \), and \( z_l \) = coordinate of floor \( l \).

The shear, \( S_{jk} \), in the \( k \) story of the frame (structural plane) \( j \) can be computed, taking torsion into account, as \( S_{jk} = (k_j / K) \beta_1 \beta_j S_k \).
Finally, the service design forces, $F_{jk}$, in each story $k$ of the frame (structural plane) $j$ can be computed using $F_{jk} = S_{jk} - S_j(k+1)$.

**Safety Design Forces.** The total base shear can be estimated from the SIDRS and the assumed modal parameters as

$$V_T = W \sqrt{\frac{3}{2} \sum_{i=1}^{3} \left[ \frac{L_i^2}{M_i} \cdot \frac{1}{M} \cdot C_s(T_1, \mu_{SDOF}) \right]^2}$$

(6)

However, because usually the final designed and constructed structure has an overstrength, OVS, over its design strength, the demanded design shear base can be reduced as follows: $V_D = V_T / (1 + OVS) = V_T / R_{OVS}$. Overstrength may come from a variety of sources [17]. The selected design method is a main factor: because its selection is based on plastic limit design (formation of mechanism), the OVS was assumed to be $\leq 0.30$, i.e., a $R_{OVS}=1.3$ was selected.

Finally, the torsion effects are included through the use of a coefficient $\beta_4 = 1 + 2 \left| \frac{X_R}{j_R} \right|$, which was developed using rigid-plastic analysis [9]: $X_R$ is the distance between the center of mass and the center of resistance, and $j_R$ is an index of torsional strength of the structure.

The base shear, $V_j$, in each frame (structural plane) $j$ can be computed so that $\Sigma V_j = \beta_4 V_D$. Note: because service constraints regarding strength are satisfied independently, much freedom exists in selecting $V_j$ for each plane of strength.

The shear, $S_k$, in the $k$ story can be computed as

$$S_k = \sqrt{\sum_{i=1}^{3} \sum_{i=k}^{n} M_i \varphi_i(z_i) \frac{g}{M_i} g \cdot C_s(T_p, \mu_{SDOF})}$$

(7)

The shear, $S_{jk}$, in the $k$ story of structural plane $j$ can be computed, taking torsion into account, as $S_{jk} = (V_j / V) \beta_4 S_k$.

Finally, the safety design forces, $F_{jk}$, in each floor $k$ of structural plane $j$ can be computed using $F_{jk} = S_{jk} - S_j(k+1)$.

**Preliminary Design.** The design shear and forces and the simultaneous design for service and safety limit states were obtained using an electronic spreadsheet. The advantages of using a spreadsheet, and the process and equations used, are described in Ref. 9.
Analysis of Preliminary Design. In this last step of the preliminary design procedure, the preliminary design of the 30-story RC building on soft soil is evaluated at the limit states considered in the established design criteria. Linear elastic modal and time-history analysis of a 3-D model of the building, nonlinear "pushover" analysis using a lateral force pattern obtained from modal spectral analysis, and nonlinear time-history dynamic analysis on a 2-D model of the building were conducted. Only the main results of the analysis of the conceptual preliminary design are presented here and compared to the results of analysis of the same building designed following the UBC code. A complete description of the results and comparison to other design methods can be found in Refs. 9 and 21.

Weight (Table 3). The Conceptual Design (CD) results in a building 23% heavier than the UBC design. The weight of the columns increases 77% over UBC, and the weight of the beams 50%.

Period (Table 4). CD period is 33% smaller than UBC, mainly because of the larger $I_{eff}$ of the beams. $T = 1.7$ sec is the estimated maximum period to obtain $I_{max} < 0.003$.

Strength (Table 5). The strength, $V_{mechanism}$, computed using a "pushover" test with P-Δ effects, is 2.72 times larger for CD than for UBC. The reasons for this difference are discussed below where the comparisons of the "pushover" tests are presented.

LERS Displacements for Design EQGMs with Return Period, $T_R = 10$ Years. Figure 7 shows the displacements for the center of mass and the exterior frame for CD and UBC. The UBC displacements are about twice the CD displacements, due to the larger period. The torsion effects increase the displacement 13% and 8% for the exterior frames of the CD and UBC designs, respectively.

LERS IDI for Design EQGMs with $T_R = 10$ Years. As shown in Fig. 8, the maximum IDI for UBC exterior frame is about 0.0072 at story 9. The IDI have a shape nearly ideally uniform for CD, because the stiffness of the beams is reduced progressively from the bottom to the top, following the shape of story shear. A more irregular shape is obtained for UBC design, because although the stiffness of the columns is reduced from the bottom to the top, the effective stiffness of the beams have small changes with the height.

Maximum Stress Ratio for LERS for Design EQGMs with $T_R = 10$ Years (Fig. 9). Because CD is developed to obtain first yielding under EQGMs with $T_R = 10$ years, the whole structure remains elastic, with a maximum stress ratio = 1. For UBC design, extensive yielding is expected in the beams along the building for EQGMs with $T_R = 10$ years. First yielding is expected for EQGMs with Peak Ground Acceleration (PGA) as low as $PGA = (0.07/2.6)g = 0.027g$. 


Maximum Displacements and IDI for Time-History Analysis (Mexico SCT PGA = 0.07g) (Figs. 10 and 11). Note that the CD results correspond to a 3D linear elastic analysis, while, because of the expected inelastic behavior, a 2D nonlinear response is plotted for the UBC design. Therefore, torsion effects and bi-axial EQGMs are not considered for UBC. Comparison of Figs. 8 and 11 shows that for CD the IDI for SLERS and linear elastic time history are similar. However, a concentration of IDI appears for UBC design around story 7, with a maximum IDI = 0.0076. It can be concluded that nonstructural damage can be expected each 10 years for UBC design.

Pushover Test (Fig. 12). For CD design, the structure was designed for a \( V_{\text{1st yield}} = 0.16W \) and for ultimate \( V_{\text{Du}} = 0.19W \), expecting an overstrength \( R_{\text{OV,S}} = 1.3 \), so that \( V_{\text{mechanism}} = 0.25 \) was expected. Because torsional effects were considered in the design and the pushover test is only translational, the first yielding was obtained for \( V = 0.18W \), which is a result of amplifying the design \( V_{\text{1st yield}} \) by a factor accounting for elastic torsion. Also, because the service limit state controlled the design of exterior frame, a larger overstrength than was expected was obtained at ultimate. Because of the smaller period required to satisfy IDI under service and the larger required design capacity to avoid yield under service, the capacity of CD is about three times that of UBC.

Because UBC design has smaller strength, and owing to its displacement shape has a larger P-\( \Delta \) effect (Fig. 13), P-\( \Delta \) effect is much more important for it than for CD.

Pushover Test: Displacement and IDI. Figures 13 and 14 show the displacement and IDI for UBC and CD without P-\( \Delta \) effects. The values are plotted at \( \Delta_{\text{roof}} = 0.26 \text{ m} \) (first yielding), 1.0 m and 2.05 m for CD; and \( \Delta_{\text{roof}} = 0.30 \text{ m} \), 1.0 m and 2.0 m for UBC. The displaced shape obtained for CD is nearly a straight line (i.e., nearly uniform IDI). However, a large concentration of IDI is obtained around story 8 for UBC design.

Time History Mexico SCT PGA = 0.30 g: Displacements and IDI. Figures 15 and 16 show the envelope of displacements and IDI for CD and UBC design. Again, a uniform pattern for CD and a concentration of IDI around story 7 for UBC design are obtained. Note that serious damage to the nonstructural elements for stories 4 to 10 (IDI>0.02) can be expected for UBC design.

Time History Mexico SCT PGA = 0.30 g: Beams and Columns Plastic Hinge Rotation. As shown in Fig. 17, the maximum and accumulative plastic hinge rotation (\( \theta_{\text{max}} \) and \( \theta_{\text{acc}} \), respectively) in beams for UBC and CD are comparable. Note the amplification around story 7 in the maximum and accumulative plastic hinge rotation for UBC design. Because of the larger strength of CD beams, the seismic axial loads in the CD column are larger and net tension occurs in the external columns. As is discussed below, the result of this is that in the beams \( \theta_{\text{max}} \) is equal to \( \theta_{\text{acc}} \) for stories 1 through 4 (Fig. 17), and large accumulative rotations under tension are obtained for columns of the CD (Fig. 18).
Generalized SDOFS Time History Mexico SCT PGA = 0.30 g. The maximum displacement, $\delta$, of the generalized SDOFS representing the UBC and CD designs subjected to the SCT EQGM are close to those obtained for the analysis of the actual MDOFS structure. Table 6 gives the values obtained from such analysis of MDOFS ($\mu_{u,\text{mon}}$ = ultimate monotonic ductility ratio = 6 was considered to compute the damage indices).

Discussion of Beam and Column Plastic Hinge Rotations. Figures 17 and 18 show that some unexpected results have been obtained from the time-history analysis. From Fig. 17, it can be seen that the largest plastic rotation ($\theta_{\text{max}}$) in the CD beams is about 0.046 (at the 4th floor), significantly higher than the $\theta_{\text{max}}$ in the beams of the UBC, which is 0.026 and occurs at the 8th floor. Figure 17 also shows that at the 1st to the 4th floors the $\theta_{\text{max}}$ of the CD beams is practically equal to the accumulative plastic hinge rotation, $\theta_{\text{acc}}$. From Fig. 18 it is obvious that the $\theta_{\text{max}}$ and $\theta_{\text{acc}}$ in the columns of the CD design are the same and are significantly larger than those occurring in the columns of the UBC design. Why is this so? These results, at first sight surprising, can be explained on the basis of the significant effect of the axial force on the column yielding resistance and using the simple example of Fig. 19, which analyzes what can happen to a simple one-story frame with two columns having the Axial Force (P)/Moment (M) interaction shown in the diagram in Fig. 19(m), and a beam with bending moment capacity $M_b$. Consider an elasto-perfectly plastic behavior for the members. Assume that when a positive lateral force $V^+$ is applied, we have tension $P^+$ on the left column and compression $P^-$ on the right column [Fig. 19(a)]. When the sign of the lateral load changes, the sign of the axial load also changes, as shown in Fig. 19(g). Assume that while the bending moment capacity of the column under tension load $P^+$ is $M_t$, under compression load $P^-$ it is $M_c$, and that $M_t < M_b < M_c$. If the structure is loaded with $V^+$, at point (1) in Fig. 19(n) the first yielding is reached in the left column under tension and the plastic hinge $\theta_{CL}$ is formed. At point (2) the beam yields, the plastic hinge $\theta_{BR}$ is formed and then the structure becomes a mechanism [Fig. 19(c)]. If the lateral displacement, $\Delta$, is increased, and then the structure is unloaded (point 1), at this point residual moments, $M_t$ [Fig. 19(e)], frozen plastic deformations [Fig. 19(f)], and a lateral deformation remains in the structure. If the structure is now loaded in the other direction, the axial loads reverse and the first plastic hinge, $\theta_{CR}$, is formed in the right column (point 3). If the load is increased more, a mechanism is formed when the plastic hinge $\theta_{BL}$ is formed at the left end of the beam [Fig. 19(i)]. Note that new cycles of load-displacements always produce plastic hinge rotations of the same sign in all of the plastic hinges. In fact, joint mechanisms have been formed. From this example, it is clear that for exterior columns and beams of the lower stories of a tall and slender building where the effects of the tension in the columns are important, the accumulative and maximum hinge rotation tends to be equal. Obviously, this accumulation of plastic hinge rotation results in a shakedown problem undetectable by pushover analysis, as illustrated in Figs. 20 and 21, where the maximum plastic hinge rotations obtained for time-history analysis are compared to those from the pushover test for the same roof displacement ($\delta=0.82$ m).
Regarding the above observations, it should be noted that the determination of the required resistance at the critical regions of the columns was based on a capacity approach in which the evaluation of the magnitude of the axial forces was done according to the procedure presented by Prof. Paulay in 1977 [22].

CONCLUSIONS AND RECOMMENDATIONS

Because of space limitations, only some concluding remarks and main recommendations for research needs are offered below. For a detailed formulation of conclusions and recommendations, the reader is referred to Refs. 9 and 21.

Concluding Remarks.

Modern seismic code regulations, which try to reflect great advances in knowledge and understanding in a very simple way that practitioners can apply in the EQ-RD of buildings, are not transparent about the expected level of performance of the entire building system (soil-foundation-superstructure and nonstructural components). The true nature of the EQ-RD and EQ-RC problem is to obtain satisfactory building performance, but this is implicit rather than explicit in the code regulations because it is obscured by a series of empirical factors.

The new conceptual code format summarized herein aims mainly at attaining regulations that cover in a rational and transparent way all of the aspects that a seismic code should regulate to ensure the desired building performance when significant EQGMs occur.

The proposed conceptual methodology for numerical EQ-RD complies with the worldwide-accepted EQ-RD philosophy. It is based on the use of energy concepts and on fundamental principles of structural dynamics and comprehensive design, considering realistic mechanical behavior of the entire building system. It takes into account from the very beginning of the EQ-RD procedure (i.e., from the preliminary design) the simultaneous demands for strength, deformation and their combined effects on the demanded and supplied energy capacities of the entire facility system.

The main advantage of the proposed conceptual methodology for numerical EQ-RD is that, notwithstanding great uncertainties in some of the concepts involved in its codification, the numerical quantification of these concepts can be improved without changing the format of this codified methodology as new and more reliable data are acquired.

The proposed conceptual methodology was applied successfully to the EQ-RD of a 30-story RC space-frame building, which was also designed using the 1991 edition
of UBC. Comparison of the performance of the two designs reveals the weaknesses of present UBC seismic code regulations when applied to tall buildings, especially the lack of control of the general demands imposed by the service EQGMs (Figs. 7-11), which results in an unacceptable performance for EQGMs with a $T_r = 10$ years. This weakness was anticipated in the studies reported in Ref. 23.

It should be clearly noted that it is not the intention of the authors to propose that such a detailed methodology be implemented in present building codes for all kinds of structures. What is proposed is that such a conceptual methodology be used in the second step of the proposed three-step approach. The proposed conceptual code regulations can be used to carry out case studies whose results can be used to formulate practical seismic codes for the design of standard regular buildings that are simple, but more transparent and reliable than present ones. These practical codes should clearly state all restrictions in siting and in selection of configuration, foundation, structural layout, structural system and nonstructural components for which such simplified codes should be used.

**Recommendations for Needed Research**

Extensive probabilistic studies of the factors $\beta_1, \beta_2, \beta_3, \beta_4, \gamma$ and $R_{OVS}$, which are used to introduce elastic and inelastic torsion, concentration of IDI, MDOF effects, damage and overstrength during the preliminary analysis, are needed. These studies must build a database to improve the quantification of the concepts involved in the proposed design methodology.

There is a need to improve the evaluation of the magnitudes of the axial forces, shear and moment to be used in the preliminary design of the columns to avoid the shakedown problem observed for the CD 30-story building. This shakedown problem was created by the development, in the columns of the lower stories, of axial tensions higher than those considered in the preliminary design.

A practical and reliable 3-D dynamic nonlinear analysis computer program should be developed that will enable 3-D time-history analysis of realistic 3-D models of the entire building system to be conducted when such a model is subjected to the time-history of the critical EQGM components acting simultaneously.

**ACKNOWLEDGEMENTS**

Most of the studies reported herein were supported by grants from CUREe-Kajima and the National Science Foundation. The authors gratefully acknowledge this support. Thanks are also extended to Amador Terán-Gilmore, Research Assistant, for his valuable collaboration, and to Brad Young for editing and typing this paper.
REFERENCES


### TABLE 1 — LIMIT STATE DESIGN CRITERIA

<table>
<thead>
<tr>
<th>Return Period</th>
<th>SERVICEABILITY</th>
<th>SURVIVABILITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Response Condition: Structural Damage</td>
<td>$T_A = 10$ years</td>
<td>$T_B = 450$ years</td>
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<tr>
<td></td>
<td>$DM = 0$ (Undamaged Elastic)</td>
<td>$DM &lt; 0.8$ Pre-Collapse</td>
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<tr>
<td>Response Condition: Nonstructural Damage</td>
<td>$IDI &lt; 0.003$</td>
<td>$IDI &lt; 0.0125$</td>
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<tr>
<td></td>
<td>$PF &lt; 0.20$</td>
<td>$PF &lt; 0.20$</td>
</tr>
<tr>
<td>Damping Coefficient</td>
<td>0.02</td>
<td>0.05</td>
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### TABLE 2 — ASSUMED AND COMPUTED MODAL PARAMETERS

<table>
<thead>
<tr>
<th>$n$</th>
<th>$\phi_n(z)$</th>
<th>$T_n$</th>
<th>$\phi_n^2/M_n$</th>
<th>$\phi_n^2/M_n$</th>
<th>$k_n'(0)$</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>$z/H$</td>
<td>$T_1$</td>
<td>1.5</td>
<td>0.75</td>
<td>$1/H$</td>
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<tr>
<td>2</td>
<td>$-3/2\left(z/H\right)+5/2\left(z/H\right)^3$</td>
<td>$0.41T_1$</td>
<td>0.875</td>
<td>0.109</td>
<td>$1.5/H$</td>
</tr>
<tr>
<td>3</td>
<td>$15/8\left(z/H\right)-70/8\left(z/H\right)^3+63/8\left(z/H\right)^5$</td>
<td>$0.26T_1$</td>
<td>0.687</td>
<td>0.043</td>
<td>$1.87/H$</td>
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</table>

### TABLE 3 — SUMMARY OF WEIGHTS, IN TONS, OF 30-STORY BUILDINGS

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>Japanese Design</th>
<th>Conceptual Design</th>
<th>UBC Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>slab</td>
<td>10170 ton</td>
<td>8130 ton</td>
<td>8130 ton</td>
</tr>
<tr>
<td>beams</td>
<td>7970</td>
<td>7820</td>
<td>5200</td>
</tr>
<tr>
<td>columns</td>
<td>5910</td>
<td>6450</td>
<td>3625</td>
</tr>
<tr>
<td>nonstructural and others</td>
<td>4790</td>
<td>3750</td>
<td>4240</td>
</tr>
<tr>
<td>reactive live load</td>
<td>1770</td>
<td>0</td>
<td>1440</td>
</tr>
<tr>
<td>TOTAL</td>
<td>30610 ton</td>
<td>26150 ton</td>
<td>22635</td>
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### TABLE 4 — PERIODS, IN SECONDS, OF FIRST THREE TRANSLATION MODES OF 30-STORY BUILDING

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>$T_1$</th>
<th>$T_2$</th>
<th>$T_3$</th>
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<tbody>
<tr>
<td>Japanese</td>
<td>1.67</td>
<td>0.55</td>
<td>0.30</td>
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<tr>
<td>Conceptual</td>
<td>1.70</td>
<td>0.59</td>
<td>0.34</td>
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<tr>
<td>UBC</td>
<td>2.53</td>
<td>0.84</td>
<td>0.46</td>
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TABLE 5 — ULTIMATE AND DESIGN BASE SHEARS OF 3-STORY BUILDINGS

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Ultimate Base Shear</th>
<th>Design Base Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Japanese</td>
<td>7000 ton (0.24 W)</td>
<td>0.18 W at ultimate (0.12 at allowable stress)</td>
</tr>
<tr>
<td>Conceptual</td>
<td>7850 (0.30 W)</td>
<td>0.25 W at ultimate</td>
</tr>
<tr>
<td>UBC</td>
<td>2451 (0.11 W)</td>
<td>0.063 W at first significant yielding (0.045 at allowable stress)</td>
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TABLE 6 — MAIN RESULTS FROM TIME-HISTORY ANALYSIS OF GENERALIZED SDOF SYSTEM SUBJECTED TO MEXICO SCT RECORD NORMALIZED TO PGA = 0.30g

<table>
<thead>
<tr>
<th>Conceptual Design</th>
<th>UBC Design</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( \delta_u ) = 87.6 cm</td>
<td>( \delta_u ) = 144 cm</td>
<td>Maximum Displacement</td>
</tr>
<tr>
<td>( \mu = 2.03 )</td>
<td>( \mu = 4.12 )</td>
<td>Maximum Ductility Ratio</td>
</tr>
<tr>
<td>( \mu_a = 11.0 )</td>
<td>( \mu_a = 24.8 )</td>
<td>Cumulative Ductility Ratio</td>
</tr>
<tr>
<td>DM = 0.86</td>
<td>DM = 1.30</td>
<td>Damage Index</td>
</tr>
</tbody>
</table>

Fig. 1 — Flow-chart for EQ-RD
Fig. 2—Steps needed for establishment of design EQs

Fig. 3—Flow chart for preliminary analysis
Fig. 4—30-story building: Structural system.

(a) Elevations of exterior frame; interior frame.

(b) Typical Floor Plans

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Fig. 21—Maximum plastic hinge rotation in beams from: time-history analysis (Mexico SCT, PGA = 0.30 g); and pushover test at same roof maximum displacement.
Japanese PRESSS Design Guidelines for Reinforced Concrete Buildings

by S. Otani

Synopsis: The paper briefly introduces an ultimate strength design method for reinforced concrete buildings on the basis of the capacity design concept. A design guideline was developed in Japan as a part of the U.S.-Japan PRESSS (Precast Seismic Structural System) project. The design for earthquake loading is specified for the serviceability limit state and ultimate limit state. This paper introduces the concept of earthquake resistant design for the ultimate limit state using a nonlinear static analysis under monotonically increasing force.

Keywords: capacity; deformation; dynamic characteristics; earthquake-resistant structures; limit state design; loads (forces); nonlinear analysis; precast seismic structural system (PRESSS); reinforced concrete; serviceability; strength; structural design
Appreciation

Professor Tom Paulay flanked me many times, too many times that I cannot remember each occasion. My first personal encounter with him probably was right after my presentation of a paper at the Fifth World Conference on Earthquake Engineering in Rome in 1974. My presentation was the last in a session. When the chairman declared the end of the session, a silver-haired giant rushed me with an excited expression of a naughty boy, and held me on the shoulders with huge hands. Who could protect himself against such a surprise attack while he was in a relaxed mood after the presentation? I was almost choked to death. Then the giant said "You did a good job!" and shook me several times before he ran away from the room. I observed Tom in many conferences before, but I, as a person raised in a country of rigid hierarchy, thought he belonged to a flock of fame and did not expect such warm words from him. A few minutes later when I went down to a restaurant and waited in a long line to grab pieces of sandwich, I saw Tom at a distance already chatting with ladies with a glass of wine and foods.

I have maintained my respect to his technical perspectives and affection to his personal warmth and encouragement.

Tom and I do not necessarily agree on every technical matters, but if we do not, we understand the difference in our standpoint which leads to varied interpretation of the same phenomenon. I feel it lucky to have him as a precursor who studies deeply on various matters and develops new concepts for us rather than to have him as a hot chaser on the track. Since I returned to Japan, I studied his work on the resistance of reinforced concrete beam-column joints and on the concept of capacity design. There has been so much to learn from his efforts.

We sincerely hope that he continues to watch the work of youngsters and gives encouragement and guidance to them.

Shun Otani
Shunsuke Otani, consulting member of ACI committee 442 and liaison member of ACI committee 318, is professor in Department of Architecture, Faculty of Engineering, University of Tokyo. He obtained M.Sc. and Ph.D. degrees in civil engineering at University of Illinois at Urbana-Champaign.

INTRODUCTION

The Building Standard Law of Japan (1) was proclaimed in 1950, accompanied by the Law Enforcement Order (Cabinet Order), in which two levels of allowable stresses were introduced; one for the long-term (gravity) loading to ensure serviceability and the other for the short-term extraordinary (earthquake, strong wind and snow) loading to ensure safety. Since then the Law Enforcement Order has maintained the allowable stress design format. The Law Enforcement Order was revised in 1981 to add the requirement of an ultimate lateral force resistance. The Architectural Institute of Japan (AIJ) published "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings based on Ultimate Strength Concept (2)" in 1990 on the basis of capacity design philosophy.

The current PRESSS design guidelines, developed in Japan as a part of U.S.-Japan PRESSS (Precast Seismic Structural system) project, extended the concept of the AIJ guidelines, and introduced serviceability and ultimate limit state design requirements. Two separate procedures are prepared for earthquake resistant design: (a) a procedure using static nonlinear analysis of a building under monotonically increasing lateral forces, and (b) a procedure using a linearly elastic analysis and a simple limit analysis for a building less than 31 m in height. This paper introduces the earthquake resistant design procedure using an incremental static nonlinear analysis.

SCOPE OF PRESSS GUIDELINES

The guidelines are intended to provide minimum requirements for the structural design of cast-in-situ reinforced concrete (RC) buildings with/without precast reinforced concrete (PCa) structural elements. PCa elements and their connections must be properly designed and constructed to exhibit behavior as good as, or superior to, that of their corresponding monolithic reinforced concrete assemblages.

The application is limited to ductile moment resisting frame buildings of regular structural configuration, with/without continuous structural walls, and that are not taller than 60 m. The Law (1) requires the special approval of the Minister of Construction for the design of a building taller than 60 m.

The structural regularity (1) is judged by a stiffness ratio Rsi and an eccentricity ratio Rei of story i, defined by Eqs. (1) and (2);
\[ Rsi = \frac{rsi}{rs} \quad (1) \]
\[ Rei = \frac{ei}{rei} \quad (2) \]

where, \( rsi \): reciprocal of inter-story drift angle at story \( i \) under the design earthquake force; \( rs \): arithmetic mean of all \( rsi \)'s; \( ei \): distance of eccentricity from the center of stiffness to the center of vertical gravity loads at story \( i \); and \( rei \): elastic radius of gyration at story \( i \), defined as the square root of the torsional stiffness divided by the lateral stiffness. The stiffness ratio should be equal to or greater than 6/10 and the eccentricity ratio should be equal to or less than 15/100.

Design concrete strength is limited to 21 to 36 MPa for normal concrete, and 21 to 27 MPa for light-weight aggregate concrete. The grades of deformed bars are SD295A/B, SD345 and SD390, in which the number indicates a specified yield stress in MPa. The bar sizes are limited to 10 to 41 mm in nominal diameter. High-strength prestressing bars (yield stress ranging from 700 to 1,300 MPa) may be used as shear reinforcement in beams and columns if the use of specific products with their design specifications is approved by the Minister of Construction.

**PERFORMANCE CRITERIA**

The performance criteria of the super-structure, basement and foundation structure of a building, as designed, are specified for serviceability and ultimate limit states.

**Serviceability Limit State** -- A structure must be serviceable without major repair work immediately after a medium intensity earthquake motion, which may be expected to occur several times during the use of the building (e.g., a return period of approximately 50 years). A structural member must not yield, nor should non-structural elements and mechanical facilities be damaged.

**Ultimate Limit State** -- A structure must be usable, even though this may involve an extensive repair work, after a high intensity earthquake motion, which may occur once in the lifetime of the building (e.g., a return period of approximately 400 years). A moment-resisting frame structure with continuous structural walls is allowed to develop flexural yielding at beam ends, and at the base of first-story columns and structural walls (weak-beam strong-column mechanism). The structure must develop lateral resistance, which is greater than the specified value, at the design limit deformation (Limit Deformation), which is the maximum possible deformation under a design earthquake motion for the ultimate limit state. The region where yielding is expected must maintain its resisting capacity to the design proof deformation (Proof Deformation), which is the upper bound deformation (two times the design limit deformation) taking into consideration the uncertainty in characteristics and intensity of design earthquake motions and the reliability of structural analysis and member strength evaluation. Shear and bond failure must be prevented in any member to Proof Deformation.

The foundation and basement must transfer the actions by gravity loads and lateral earthquake forces from the super-structure to the ground. Yielding, as a general rule, must not develop in the foundation structure, including foundation girders, foundation slabs, and piles. Non-structural elements and attachments must follow the deformation of the structure.
STRUCTURAL ANALYSIS METHOD

A complete history of member and structural resistance with deformation must be obtained for the structure, as designed, under monotonically increasing lateral forces. The analysis may be terminated when the maximum drift at a story reaches the Proof Deformation. As the analytical results are significantly affected by modeling of a structure and its members, the method of modeling and the resistance-deformation relation of members are outlined in the commentary.

Structural Modeling -- A structure must, as a general rule, be analyzed including the foundation structure, idealized as a series of plane frames interconnected by rigid truss members at floor levels. The three-dimensional effect of a structural wall (the contribution of girders orthogonal to the wall to the vertical movement at the wall boundary columns) must be included in the analysis. The structure may be analyzed separately in the two principal directions if the effect of torsion and transverse frames can be neglected.

Modeling of Structural Members -- Inelastic deflection of a beam and column may be assumed to be concentrated at the member ends, and represented by the rotation of rigid-plastic rotational springs. The stiffness characteristics of a column and beam under monotonically increasing force may be assumed to be of the trilinear type with stiffness changes occurring at flexural cracking and yielding. In modeling a structural wall, the shift of neutral axis in the section by flexural cracking, resulting in sizable vertical deformation in the tensile boundary column, must be included. A beam-column connection may be assumed to be rigid.

Loads -- A structure must be analyzed under dead and live loads specified for gravity loading, and under monotonically increasing lateral forces distributed in the same pattern as used with the design earthquake force for the serviceability limit state.

SERVICEABILITY LIMIT STATE DESIGN

Design Earthquake Force -- It is desirable to define the intensity and characteristics of earthquake motions expected at each construction site, and to perform realistic nonlinear earthquake response analyses of a structure to examine its safety. However, due to the lack of reliable earthquake information at present, the PRESSS guidelines adopt the design earthquake forces defined in the Law Enforcement Order (1).

The design earthquake story shear $Q_i$ is calculated by multiplying the total of the dead and live loads (reduced for earthquake loading) $W_i$ at and above story $i$, by a seismic story shear coefficient $C_i$ at the story:

$$Q_i = C_i W_i$$

$$C_i = Z R_t A_i C_B$$

where, $Z$: a seismic zone factor (= 0.7 to 1.0); $R_t$: a vibration characteristic factor of the building taking into account the type of soil; $A_i$: a factor representing the vertical distribution of the seismic story shear coefficient; $C_B$: standard base shear coefficient. Coefficient $R_t$ is given as follows:
where, $T_c$: critical period of subsoil (0.4 sec for stiff sand or gravel, 0.6 sec for other soil, and 0.8 sec for alluvium mainly consisting of organic or other soft soil); $T$: primary period of the building, calculated by

$$T = 0.02 \ h \quad (6)$$

where, $h$: total height of the building in m. The coefficient $A_i$ is given by the following expression:

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)[2 \ T / (1 + 3 \ T)] \quad (7)$$

where,

$$\alpha_i = W_i / W_1 \quad (8)$$

The standard base shear coefficient $C_B$ is 0.2 or more for the serviceability limit state.

**Performance Criteria** -- At the design earthquake load, a building must not develop flexural yielding at any member end, and the drift angle (inter-story drift divided by story height) must be less than $1/200$ rad, in order to ensure that the non-structural elements are protected. The drift limit requirement, when based on a realistic stiffness, including stiffness degradation by cracking, often governs the design.

**ULTIMATE LIMIT STATE DESIGN**

**Required Lateral Force Resisting Capacity** -- A structure must develop a lateral force resistance larger than 90 percent of the required lateral force resisting capacity, defined by Eqs. (3) and (4) with a standard base shear coefficient $C_B$ specified in Table 1, when a story drift angle at any story reaches Limit Deformation (drift angle $R_u1$). The lateral force resistance at Proof Deformation (drift angle $R_u2$) must be greater than the required lateral force resisting capacity. The required lateral force resisting capacity is consistent with that required for a ductile structure by the Law Enforcement Order (1).

The values of $R_u1$, $R_u2$ and $C_B$ are listed in Table 1, in which the values are varied with the ratio of base overturning moment resisted by the structural walls to the total overturning moment of the structure at Limit Deformation $R_u1$. The required base shear coefficient $C_B$ varies from 0.3 to 0.4, and Limit Deformation $R_u1$ in terms of drift angle varies from $1/100$ to $1/150$ rad (Fig. 1). These values were selected recognizing that structural walls enhance the stiffness and lateral load resistance of a structure, but that the structural walls reduce the deformation at the maximum resistance. The value $R_u1$ of Limit Deformation does not have any rational ground, but the drift limitation of $1/100$ rad is commonly used in the design of high-rise moment-resisting frame buildings.
Where yielding is permitted in design, the region must be provided with sufficient ductility to develop a deformation amplitude calculated by the nonlinear static analysis at Proof Deformation $\delta_{u2}$.

**Member Design Actions** -- A region other than the permitted region of yield hinges must be provided with resistance sufficient to prevent flexural yielding, shear failure and bond failure. Basic member design actions are those calculated by the nonlinear static analysis at Proof Deformation $\delta_{u2}$; these actions are multiplied by corresponding amplification factors to yield the member design actions.

The amplification factors must take into consideration, (a) increase in material strength above the specified value, (b) lateral force distribution during an earthquake excitation different from the one assumed in the nonlinear static analysis, (c) bi-directional earthquake loading, (d) reliability of structural analysis method, (e) reliability of member strength estimate, and (f) reliability of workmanship. The first three items are considered in the guidelines.

**Amplification by Material Strength** -- The upper bound (statistical average plus twice the standard deviation) of yield strength of reinforcing bars, as tested, is 1.30, 1.25 and 1.25 times the specified yield strength for grade SD295AIB, SD345 and SD390 steel, respectively (2), where steel grades SD345 and SD390 are commonly used as the longitudinal reinforcement. The specified yield strength may be multiplied by 1.1 times in the calculation of flexural resistance in design (1). Therefore, flexural resistance at a yield hinge may be increased by approximately 1.1 ($=1.25/1.1$) by the steel strength. For example, the basic shear for a girder, normally calculated for yielding at both ends, must be increased by an amplification factor of 1.1 to prevent shear failure.

**Amplification by Bi-directional Earthquake Motion** -- An earthquake motion is not limited to one direction although the structural design of a building is routinely performed for the longitudinal and transverse directions, separately; uni-axial design earthquake force is assumed to act in any direction. For simplicity and discussion purpose, it is assume that (a) the collapse mechanism is formed by simultaneous yielding at all girder ends and at the base of first-story columns in a moment-resisting frame structure under monotonically increasing lateral force, (b) the lateral force resisting capacity is identical in the two orthogonal directions, (c) the maximum response deformation during a design earthquake motion is comparable in any direction, and (d) the yield surface of a column under bi-directional loading is approximated by a circle.

The lateral resistance in a principal direction is governed by the flexural resistance of girders, which is not influenced by the loading in the orthogonal direction (Fig. 2). Under loading in the diagonal direction, yielding resistance and deformation become $\sqrt{2}$ times larger than the corresponding resistance and deformation in a principal direction.

If yielding does not take place in a structure (Zone 1 in Fig. 2) during a principal loading, then, by assumption (c) above, the diagonal loading will not develop column moments and shears larger than those at the yielding in the principal direction; i.e., the design moment and shear of columns need not be amplified for diagonal loading. On the other hand, if a large plastic deformation takes place in a structure (Zone 3 in Fig. 2) during a principal loading, the diagonal loading is likely to develop simultaneous yielding in both principal
directions; i.e., the design moment and shear of columns must be amplified by \( \sqrt{2} \) from those under principal loading to prevent column flexural yielding or shear failure.

Therefore, the basic moments and shears of a column must be amplified by a factor between 1.0 to \( \sqrt{2} \), depending on the expected plastic deformation under earthquake excitation in a principal direction.

**Amplification by Dynamic Effect** -- The lateral load distribution during an earthquake is different from the distribution assumed in a static analysis; this phenomenon is due to dynamic effects of excitation. The dynamic effect increases with the degree of plastic deformation, and larger actions tend to be attracted to stiffer vertical members (2). The value of the dynamic amplification factors must be determined on the basis of nonlinear response analyses of typical buildings to design ground motions.

A series of simple structures of 5-story (15 m tall) to 20-story (60 m tall) buildings were analyzed under design earthquake motions, and maximum column actions were calculated. Dynamic amplification factors were determined as the ratios of maximum response column actions to the corresponding column actions (basic member design actions) calculated at the Proof Deformation \( R_{d2} \), which is much greater than the response expected during a design earthquake motion. For a column moment, the larger of the column top and bottom moments was used at a floor. The amplification factors are generally larger for column bending moments than those for column shears, due to the shift of the inflection point. External columns tend to attract larger earthquake forces than interior columns.

Following the general practice in the design of a building taller than 60 m, the intensity of earthquake motions for a response analysis was selected to be \( v_{\text{omax}} = 500 \text{ mm/sec} \) in maximum ground velocity (the intensity corresponding to an expected return period of approximately 400 years in Tokyo). The El Centro (NS) 1940 record \( (a_{\text{omax}} = 5.0 \text{ m/sec}^2) \), Taft (EW) 1952 record \( (a_{\text{omax}} = 5.2 \text{ m/sec}^2) \), Tohoku University (NS) 1978 record \( (a_{\text{omax}} = 3.7 \text{ m/sec}^2) \) and Hachinohe Harbor (EW) 1968 record \( (a_{\text{omax}} = 2.4 \text{ m/sec}^2) \), were used in nonlinear earthquake response analyses.

According to the results of the nonlinear earthquake response analyses of simple buildings, designed in conformance with the Law (1), the response story drift and ductility demands decrease with the structural height (Fig. 3); i.e., the lateral load resisting capacity specified in the Law includes a larger safety margin for a taller structure. By the same token, a structure, provided with a lateral load resistance larger than that required in the Law develops a maximum story drift smaller than the Limit Deformation \( R_{u1} \). The larger safety margin for a taller building was thought to be desirable; i.e., the required lateral force resisting capacity and the Limit Deformation were not reduced although small drift and ductility demand were calculated for a tall building in the earthquake response analyses.

**Amplification Factors for Design Actions** -- The expected response zones in Fig. 2 were selected for buildings of three ranges of height and lateral force resistance (expressed in terms of base shear coefficient \( C \) of the building calculated at Limit Deformation \( R_{u1} \)) as listed in Table 2. Expected response zone increases for buildings of low lateral force resistance and low height.
Amplification factors for design actions were determined, as listed in Table 3, for the three expected response zones on the basis of the response analyses, taking into consideration the effect of dynamic response, the effect of bi-directional earthquake response, and the increase in material strength. Note that the effects of bi-directional response and dynamic response increase with the ductility demand (higher response zone). Therefore, the amplification factor was reduced for a structure with a larger safety margin (a tall or strong structure). The amplification factor for column moment was deliberately kept low at the level of that for the column shear because the column yielding for a short duration by oscillation in higher modes would not lead to a large plastic deformation in a story mechanism as long as some ductility is provided in the columns for flexural yielding.

**Axial Load Limitation** -- Axial force in columns must be less than $2/3 \, N_u$ and $3/4 \, N_t$, where $N_u$: ultimate compressive strength of the column, and $N_t$: ultimate tensile strength.

**FOUNDATION STRUCTURES**

The foundation and basement must be provided with sufficient rigidity and strength against gravity loading during strong wind and snow loading and medium intensity earthquake motions, to prevent excessive settlement, inclination and sliding for serviceability, and to prevent excessive cracking for durability.

Design stresses for basement structures include the effect of soil pressure and hydraulic pressure in addition to the effect of long-term gravity load and short-term snow, wind and earthquake loads transmitted from the super-structure. Stresses caused by uneven settlement and lifting or deflection of the soil and piles must be considered if appropriate.

At an earthquake load level for the serviceability limit state of the super-structure, the stresses must not exceed the allowable stresses for the short-term loading; furthermore, the foundation must not lift. At an earthquake load level for the ultimate limit state of the super-structure, forces in the foundation structure must not exceed the ultimate strength of the members and the soil.

The foundation structure, such as foundation girders, slab and piles, must not form a part of the specified yield mechanism. The properties of sub-structures are not clearly understood in relation to the behavior of super-structures. The sub-structure must be protected because it is often difficult to investigate the damage in the foundation after an earthquake and because the cost to repair the foundation damage is enormous.

**ACKNOWLEDGMENT**

The guidelines is drafted and discussed by a working group with the following members; S. Otani, chairman, M. Teshigawara, Secretary, M. Hayashi, Y. Inoue, T. Kaminosono, I. Kawabata, M. Kimizuka, S. Nakata, K. Yagishita, K. Yoshioka. The contribution of the working group members is deeply acknowledged. Nonlinear analyses of idealized structures were carried out by Dr.
Y. Sakai, Dr. D. Saito, Dr. H. Kinugasa and Mr. K. Fu. Benchmark structures were designed by working group (Mr. N. Kani, chairman) of Japan Structural Consultants Association.

REFERENCES


<table>
<thead>
<tr>
<th>Structural Type</th>
<th>$C_B$</th>
<th>$R_u1$ (rad)</th>
<th>$R_u2$ (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta &lt; 0.3$</td>
<td>0.30</td>
<td>1/100</td>
<td>1/50</td>
</tr>
<tr>
<td>$0.3 &lt; \beta &lt; 0.7$</td>
<td>0.35</td>
<td>1/120</td>
<td>1/60</td>
</tr>
<tr>
<td>$0.7 &lt; \beta$</td>
<td>0.40</td>
<td>1/150</td>
<td>1/75</td>
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</tbody>
</table>

$\beta$: ratio of base overturning moment resisted by structural walls to the total overturning moment of the structure at Limit Drift Angle $R_u1$

<table>
<thead>
<tr>
<th>Base Shear Coeff., $C$</th>
<th>H&lt;20 m</th>
<th>20 m&lt;H&lt;45 m</th>
<th>45 m&lt;H&lt;60 m</th>
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</thead>
<tbody>
<tr>
<td>0.27 &lt; C &lt;0.315</td>
<td>Zone 3</td>
<td>Zone 2</td>
<td>Zone 1</td>
</tr>
<tr>
<td>0.315&lt; C &lt;0.36</td>
<td>Zone 3</td>
<td>Zone 2</td>
<td>Zone 1</td>
</tr>
<tr>
<td>0.36 &lt; C</td>
<td>Zone 2</td>
<td>Zone 1</td>
<td>Zone 1</td>
</tr>
</tbody>
</table>

$C$: Base shear coefficient of the building calculated by a nonlinear static analysis at Limit Deformation. Expected Response Zone: See Fig. 2.

<table>
<thead>
<tr>
<th>Members</th>
<th>Actions</th>
<th>Expected Response Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Zone 1</td>
</tr>
<tr>
<td>(a) Girders</td>
<td>Shear</td>
<td>1.1</td>
</tr>
<tr>
<td>(b) Interior Columns</td>
<td>Moment</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>1.2</td>
</tr>
<tr>
<td>(c) Exterior Columns</td>
<td>Moment</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>1.4</td>
</tr>
<tr>
<td>(d) Structural Wall</td>
<td>Moment</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>1.3</td>
</tr>
<tr>
<td>(e) Beam-column Connection</td>
<td>Shear</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Amplification factors should be used for the actions of the building calculated by a nonlinear static analysis at Proof Deformation.
Fig. 1—Performance criteria for frame-wall structures

Fig. 2—Effect of bidirectional loading
Fig. 3—earthquake response and building height

1. Maximum story drift angle
2. Maximum girder ductility factor
Development of Canadian Seismic-Resistant Design Code for Reinforced Concrete Buildings

by S. M. Uzumeri

Synopsis: This paper summarizes the developments and changes to the seismic design provisions of the National Building Code of Canada (NBCC 1990) since its 1977 edition and discusses the changes to the seismic design provisions of the Canadian Standards Association Standard, Design of Concrete Structures for Buildings (CAN3-A23.3).

The paper outlines the philosophy of seismic-resistant design of the Canadian code, discusses the changes to the zoning maps, static design procedures, and the introduction of the force modification factors. The paper also deals with the changes to the Canadian reinforced concrete code and especially with the section on ductile walls, changes to load combination factors, and the explicit links between the concrete code sections containing the detailing requirements and the NBCC sections on determination of the design seismic forces.

Keywords: building codes; ductility; earthquakes; hinges (structural); reinforced concrete; structural design; walls
APPRECIATION

Tom Paulay was an officer in the Hungarian cavalry. He has been a "cavalry officer" all his life. He has been charging into the unknowns of behaviour and design of concrete structures, along well selected paths with deliberation, imagination and perseverance. His charges have been penetrating, sustained, and executed with wisdom and finesse. We have been the beneficiaries of his successes. Thank you Tom!

I had an opportunity to get to know him during his first sabbatical leave at the University of Toronto. His graduate class CIV 1099S - Special Studies in Civil Engineering, was most successful. Lectures were well attended by staff, students and practitioners in Toronto. From that period grew many things: Toronto research on confinement of concrete, our joint paper criticizing part of the Canadian code of the day, which was awarded the Gzowski Medal by the Engineering Institute of Canada. We had many discussions on behaviour and design of beam-column joints of frames, and certainly learned a great deal from him about the behaviour and design of "ductile walls".

The relationship continued during my visit to University of Canterbury for eight months, Tom's subsequent visits to University of Toronto and at many meetings, conferences and visits. I am happy that our association will continue while he is the President of the International Association for Earthquake Engineering.

It has been a most pleasurable and rewarding relationship. Both Jean and I also appreciated Tom the gardener and story teller, and Herta, the most charming hostess. We wish the very best for the future to Tom, Herta and Dorothy, Esther and Gregory and their families.

I cannot help but remind Tom that such a symposium is not an act of thanks alone, but it is also an act of investment in our future. Thank you very much Tom, but now it is time for you to get back to work!!!

S. M. (Mike) Uzumeri
Ankara, July 1993
S. M. Uzumeri, FACI, is a Professor of Civil Engineering at the University of Toronto, Toronto, Canada and currently is a visiting professor at the Middle East Technical University in Ankara, Turkey. He was a member of the joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures and Chairman of the ACI Committee 442, Response of Buildings to Lateral Forces. His research interests include the development of seismic design provisions, design of beam-column joints, and behaviour of confined concrete columns.

Code of Hammurabi, c. 1770 B.C.

*If a builder has built a house for a man and has not made his work sound, and the house which he has built has fallen down and so caused the death of the householder, that builder shall be put to death.*

**INTRODUCTION**

If the Code of Hammurabi were in effect today, many deaths in recent earthquakes would have to be followed by many executions of builders, designers and even code writers. In self-protection, the society has moved from 1770 B.C. to today by spreading and thus diffusing the responsibilities among its many parts. This diffusion naturally results in lengthy and costly legal arguments and court cases about who is responsible. The vehicle used to reduce these arguments is the development and use of building codes in the building process. It is expected that building codes, drafted by "knowledgable" experts, will protect the public by leading to safer buildings and will also reduce endless litigation by providing clear and unambiguous rules that can be of help in a court of law, to respond to the adversarial questions with a simple "Yes" or "No".

Building codes are "living" documents that change with time. They are intended to reflect the "state-of-the-art" at a given time. Therefore, it is crucially important to have documentation and writings explaining and describing the background of the code provisions. It is a folly for any group to look only at a code and decide on changes to some of its provisions without appropriately informing themselves of the reasons for the provision, or the compromises that have already taken place to arrive at the published version. There are many papers in the literature comparing one or two parameters among various building codes in the world as if there was a uniform basis among these codes. It is of doubtful merit to compare a single function or a parameter among various codes without considering other
variables such as load factors, detailing rules, and system selection
requirements also. In the author's opinion, the only reliable comparison of
the requirements of two codes would be the comparison of the final detailed
designs resulting from each of those codes, for two similar buildings located
in similar risk areas. Even then, the comparisons may be of little value in
assessing which building code would result in a safer building and by how
much, if the variation in construction materials and the quality of construction
in the two jurisdictions is not considered.

The purpose of this paper is to provide an overview and some
background on the developments in Canadian code requirements for
earthquake-resistant concrete buildings and to give a brief description of the
evolution of these requirements since 1977 [1,2,3]. A previous study in 1978
[4] dealt with the developments up to the 1977 version of the National
Building Code of Canada (NBCC 1977) [1,2], and the 1973 version of the
Canadian Standards Association (CSA) Standard (A23.3-1973) for the design
of concrete structures for buildings [3]. Since then, the NBCC has undergone
major revisions in 1985 and 1990 and the concrete code has undergone major
changes in 1984.

DEVELOPMENT OF CANADIAN BUILDING CODE

The objective of earthquake-resistant design standards in Canada is similar to
many other jurisdictions and it is to produce structures that will provide an
acceptable level of public safety in an earthquake. This is considered to have
been achieved if, in a moderate earthquake, no significant structural damage
occurs, and if, in a major earthquake, the primary structure does not fail and
evacuation of the occupants is therefore possible. While it may be feasible to
design buildings that will sustain no damage in an earthquake, such
extraordinary measures would be uneconomical. Accordingly, the Canadian
earthquake codes aim to reduce the probability of fatalities while accepting
some structural damage and even total economic loss in a major earthquake.

However, earthquake-resistant design of buildings is yet more Art than
Science. It is based on information obtained from past earthquakes, with the
hope that any future event will bear some resemblance to the past in
frequency, location and intensity. For countries such as Canada, where the
history of earthquake data collection is quite short, there are serious
uncertainties about projecting to the future.

The development of building codes involves the following basic steps:

1. Assembly of information on geology, seismicity, soil conditions and past
   earthquake records and formulation of seismic risk maps.
2. Definition of Acceptable Risk Levels: Society, according to its social and economic priorities, will define various acceptable risk levels. These levels will differ among buildings, also for other types of structures such as dams and nuclear power plants, and will relate to the desired level of performance of structures subjected to various levels of earthquakes.

3. Definition of the Specified Strength and the Accompanying Deformability: In most building codes, specified strengths of structures are arrived at by using the information described in the steps above. For buildings, these specified strengths are lower than the forces to which a building would be subjected if it remained in the elastic range throughout the earthquake. Therefore, each specified strength level is accompanied by an implicit expectation of deformation, damage and inelastic energy dissipation capability, and the necessary design and detailing requirements for buildings must be set out.

The National Building Code of Canada (NBCC) [1,2], provides for the requirements described in item 3 above. In addition, the NBCC specifies the material design standards that should be used in the design of buildings. For reinforced concrete buildings, this approved design standard is the Canadian Standards Association, "Design of Concrete Structures for Buildings" (CSA A23.3)[3]. The NBCC is a model code approved by the National Research Council (NRC) of Canada's Associate Committee on the National Building Code. NRC Committees include in their membership individuals from various sectors: government, universities, industry etc.

NBCC with its approved material standards, have no legal status until they are adopted by a Province for use in that Province. The earthquake provisions were prepared by the Canadian National Committee on Earthquake Engineering (CANCEE). CANCEE was (until 1992) an Associate Committee set up by the National Research Council of Canada. CANCEE also prepared a commentary, Commentary J [2], on the earthquake-resisting design provisions of the NBCC.

The concrete design provisions are prepared by the Canadian Standards Association technical committee established for this purpose. Commentary to the concrete code [5], was published by the Canadian Portland Cement Association (CPCA), with the review and approval of the CSA technical committee chaired by James G. MacGregor of the University of Alberta. A Concrete Design Handbook, published by the CPCA [5] includes the code commentary and examples for the application of the seismic design provisions [5, section 11], authored by Michael P. Collins of the University of Toronto and Denis Mitchell of McGill University, Montreal. At the time of development of NBCC 1985 and 1990 [1,2], and the development of CSA A23.3 - 1984 [3] the author was the Chairman of the Canadian National Committee on Earthquake Engineering (CANCEE), Vice-Chairman
of the concrete code committee and Chairman of the sub-committee responsible for the seismic design provisions of the concrete code. This paper is written to summarize the background of the Canadian design specifications. This review will take two parts, the first part dealing with the specification of design loads and performance requirements, and the second part dealing with materials considerations and the design and detailing requirements for seismic design in reinforced concrete buildings.

PHILOSOPHY OF THE CANADIAN BUILDING CODE

Every building code reflects the philosophy of code writers as to the best tactic for resisting the effects of earthquakes. The design decisions and actions affecting the survival of usual buildings subjected to earthquakes may be summarized, in order of their importance, as follows:

Selection of Structural System

The most important decision is the selection of a good structural system. Sharp changes of stiffness and mass should be avoided and as far as possible symmetry and continuity should be provided. Maximum effort should be directed toward the avoidance of architectural features that condemn a structure from the start. Judicious use of stiff elements (shear walls) with frames (providing an alternate load path), or coupled wall systems could achieve both damage control and structural survival. Selection of a good structural system is the most important action, because if a designer starts with a poor structural system, all that he or she could hope to achieve would be to improve a bad system, which is a self-limiting approach.

Good Detailing

The importance of good detailing cannot be overemphasized. The only language with which the designer can convey his response instructions to the structure is the manner in which he details the structure. Detailing of the structure must be based on the "as-built" strength of its components. Since inelastic action is relied upon, shear force attracted to a region is related to the bending moments corresponding to the yielding of a section and not to the specified or nominal shear forces. Careful attention must be given to the effect of "nonstructural" components so that the behaviour of the structure is not altered in an undesirable manner. The designer does not have reliable means to compute the system ductility of the structure he is designing. What are available to him are the detailing rules arrived at through the study of experimental behaviour as well as the observations of the behaviour of structures during previous earthquakes.
Precision of Structural Analysis

The importance of the analysis of the structure for earthquake forces is obvious. However, this is ranked behind the first two factors in importance. Given the uncertainties involved in determining the characteristics of a future earthquake and the difficulties and uncertainties involved in the mathematical modelling of a structure, especially for inelastic response, efforts for a "precise" analysis can be more usefully directed toward the first two objectives.

Quality of Construction

This is an overriding factor. However good the construction quality may be, it will not compensate for a poor structural system or poor detailing. On the other hand, regardless of the quality of the system chosen, detailing provided and the analysis carried out, if the quality of construction is not acceptable, the structure has very little chance of survival in a major earthquake.

The Canadian code (NBCC) is in keeping with the above. For example, it favours and encourages "good" systems such as ductile walls, even to the extent of giving an advantage on the lateral forces that they must be designed for. In addition, great care is taken in requiring good details. The NBCC also subscribes to the idea of de-emphasis of "precision" in analysis by accepting the equivalent static load analysis as the only specified procedure for determining the base shear.

SPECIFICATION OF DESIGN LOADS AND PERFORMANCE REQUIREMENTS

Specification of Design Earthquake Forces

Design earthquake forces are specified in the National Building Code of Canada and are computed on the basis of an equivalent static load procedure. There is no reference in the code to an accepted dynamic analysis procedure for the determination of the base shear force, $V$, although dynamic analysis procedures are provided for the distribution of the base shear force.

These specifications are not intended to apply to unusual, irregular or special-purpose structures that may require special design procedures or reference to other standards. The NBCC makes the simplification that, for normal buildings, independent design about each principal, horizontal axis as well as for torsional forces will provide adequate resistance in all directions.
Design Base Shear, $V$ -- The NBCC specifies that structures shall be designed for a minimum equivalent static base shear, as given in following equations:

Provisions of 1985 NBCC [1,2]:

$$V = vS\cdot K\cdot I\cdot F\cdot W$$  (1)

where $V$ is the unfactored minimum base shear force.

Provisions of 1990 NBCC [1,2]:

$$V = [V_e/R] \cdot U$$  (2)

where:

$V$ is the minimum lateral seismic force at the base of the structure to be used with a load factor of 1.0;

$V_e$ is the equivalent lateral force at the base of the structure representing the elastic response,

$$V_e = vS\cdot I\cdot F\cdot W$$  (3)

$R$ is the force modification factor, given in the code (Table 1);

$U$ is a factor representing level of protection based on experience, and is equal to 0.6.

The definition and a discussion of the symbols used in Eqs. 1, 2 and 3 follows.

$v$, Zonal Velocity Ratio -- the Canadian seismic zoning map was entirely revised for the 1985 edition of the NBCC [1,2]. The previous map (Fig. 1), which was in effect from 1970 until 1985, was the first probabilistic-based map and was developed by Milne and Davenport [6] using extreme-value statistics applied to known Canadian events. The map contained seismic zones, the boundaries of which were established using peak horizontal acceleration values with an annual probability of exceedance of 0.01.

Canadian seismic zoning maps in effect since 1985 are shown in Fig. 2 and in Fig. 3. These maps are based on a statistical analysis of the earthquakes that have been experienced in Canada and adjacent regions, and have been prepared by the Earth Physics Branch of the Ministry of Energy, Mines and Resources of the Federal Government of Canada [7]. The following are the significant differences of the current maps from previous maps as listed in 1990 NBCC Commentary J [2] and described in references thereof:
(a) the data were analyzed using a method proposed by Cornell and a seismic risk computer program developed by McGuire, instead of the extreme-value method. ...

(b) new strong seismic ground motion attenuation relations were employed,

(c) both peak horizontal acceleration and peak horizontal velocity have been mapped, and

(d) the probability of exceedance of the seismic ground motion parameters has been changed to 10 percent in 50 years (which is mathematically equivalent to a probability of exceedance of 0.0021 per annum) from the previous value of 0.01 per annum.

The peak values are expressed in terms of zonal ratios, the zonal acceleration ratio, $a$, being the peak horizontal acceleration normalized to 1.0g (taken as 10 m/s$^2$) and the zonal velocity ratio, $v$, being the peak horizontal velocity normalized to 1.0 m/s. Finally, the seismic data from these risk maps, including the acceleration and velocity-related zone numbers, $Z_a$ and $Z_v$, respectively, and the zonal velocity ratio, $v$, have been tabulated for locations across the country.

$S$, Seismic Response Factor -- This factor pertains to the response of structures to ground motion. $S$ is a function of the period of the structure, $T$, and the velocity-related and acceleration-related zone numbers $Z_v$ and $Z_a$ for a given location.

The seismic response factor for NBCC 1985 was the empirical relation shown in Fig. 4. The new seismic zoning maps altered the distribution of risk across Canada. However, the committee was of the opinion that there was no basis for altering the level of overall protection against earthquakes in Canada. Therefore the NBCC 1985 was derived on the basis of maintaining the average degree of protection, as measured by the specified base shear force, unchanged across Canada (taking into account the population distribution) from the level of protection provided in the NBCC 1977 [2,8,9]. In NBCC 1990 a similar derivation was made, taking into account the revised load factors for earthquake action in the NBCC (Fig. 5).

Equations for the estimation of the period of the structure are given in the code. For frames, $T = 0.1N$, where $N$ is the number of storeys. Both NBCC 1985 and 1990, require that if the period of the structure is calculated using more rigorous techniques, the period used in the design may not exceed 1.2 times the period calculated from the empirical equations given in the code. This was essentially to restrict the use of periods calculated on the basis of unrealistic structural characteristics, ignoring many elements.
stiffening the structure, thus obtaining longer computed periods and thus lowering the design base shear force. Although there was a significant interest in providing two different empirical equations to estimate the periods of reinforced concrete and steel frame buildings, consideration of this matter was left to a later revision.

**K, Coefficient (NBCC 1985 only)** -- This coefficient was intended to reflect the capacity of a structure to dissipate energy through damping and inelastic deformation. It was also intended to reflect the performance of various types of structures in past earthquakes. Typical K values would range from 0.7 for a complete ductile, moment-resisting space frame, to 1.0 for ductile flexural wall structures, to 1.3 for nominally ductile structures, to 2.0 for buildings of unreinforced masonry, to 3.0 for essentially elastic structures such as cross-braced towers supporting elevated tanks. This coefficient was replaced in NBCC 1990. One of the reasons for its replacement was that its position in the force equation was thought to obscure from designers the fact that forces used in design were significantly lower than those a structure would sustain if it were to remain elastic (as in a wind loading case).

**R and U, Force Modification Factor and a Factor Representing the Level of Protection Based on Experience (1990)** -- It was indicated above that the code writer's goal was to ensure that, on an average basis across Canada, similar buildings be designed for similar base shear forces using the NBCC 1985 and the NBCC 1990. This meant that the result obtained from Eq. 3 (after being divided by a factor to account for inelastic action etc.) be approximately equal to the force level indicated (after the effects of load factor revisions taken into account) by Eq. 1.

It is here that a force modification factor, reducing the base shear force given by Eq. 3 is utilized. If two moment-resisting ductile frame structures were to be subjected to equivalent base shear forces, the reduction factor needed was in the order of 6.7. In the profession, an undeniable link has been formed between any such "reduction factor", and the "system ductility factor". Furthermore, there have been a number of publications linking "curvature ductility factor" and the "system ductility factor". Indeed, in an earlier version of the Canadian code, a misunderstanding had crept into the commentary for the concrete code, resulting from an inappropriate interchangeable use of the term "ductility factor" for curvature and for system. This resulted in a publication dealing with that matter [10]. Figure 6 is reproduced from Reference 10. It clearly shows that it would be extremely difficult to attain the curvature ductility levels required to reach the "system ductility factors", if "reduction factors" are thought of as if they were system ductility factors. After extensive debate, it was decided that the maximum value for the "Force Modification Factor, R" be 4.0 [11]. Accordingly, the "Force Modification Factors, R" have been determined and are shown in Table 1.
When $V_e$, as calculated from Eq. 3, is divided by the force modification factor, $R$, shown in Table 1, it is necessary to reduce the resulting $(V_e/R)$ to achieve a base shear force in 1990 approximately equivalent to the base shear force prescribed by the NBCC 1985. It was decided to call this factor $U$ (some thought it to be particularly appropriate to describe the "unknown").

Thus, Eq. 2 is written in the form of $V = [V_e/R] \cdot U$ rather than in the form of $V = [V_e/(R/U)]$, or even in the form of $V = [V_e/R^*]$, where $R^* = (R$ from Table 1)/$U$. It was decided to set the value of $U = 0.6$.

It is regrettable that, despite the efforts to prevent future confusion in matters related to deformability of various structural elements tested, some writers have interpreted the $(1/U)$ as "the overstrength factor" and there has been some discussion in the literature on this interpretation [12]. The term "overstrength" means that some item is made larger than its known required strength. It is confusing to discuss "overstrength", when one is uncertain as to what the required strength is or should be. Had the committee foreseen the possibility of this confusion related to considering the factor $(1/U)$ as an "overstrength factor", I am certain it would have preferred to remove the $U$ factor, and have the $R$ factors appropriately larger. In that event, certainly there would be no room to discuss an "overstrength" factor. I believe there is no basis for such a discussion, and there is no "overstrength factor" in the formulation of the NBCC 1990.

**F. Foundation Factor (1990)** -- This factor is intended to account for the amplification of ground motions through the soil. A wide variety of soil conditions have been reduced to four categories. Foundation factors are assigned on the basis of the soil type and depth. Typical values of $F$ range from 1.0 for rock or very dense, stiff soils of any depth, to 2.0 for soft fine grained soils with depth greater than 15 meters.

**I. Importance Factor (1990)** -- All structures that should remain serviceable after an earthquake, including those housing essential public services are assigned the factor of 1.5; schools are assigned a factor of 1.3, with all other structures having an $I = 1.0$.

**W. Weight of Structure (1990)** -- In addition to the dead load of the structure, $W$ includes portions of the live load due to snow and permanent storage (25% of snow, 60% of storage and 100% of tank contents).

**Additional Considerations**

**Distribution of Lateral Seismic Force (1990)** -- The base shear force, $V$, is distributed along the height of the building according to well known
procedures. The use of dynamic analysis procedures is encouraged for the
determination of the distribution of the base shear along the height of the
building.

Torsional Moments (1990) -- Where the centres of mass and rigidity do not
coincide, this eccentricity must be accounted for in the design. In every
edition of the Canadian code, this item has been subject to revision. I am
certain that it will be revised at the next edition of the code as well. In any
case, a minimum design eccentricity is imposed to account for possible
torsional ground motions and for accidental torsion.

Load Factors (1990) -- The Canadian code is a limit states design code. The
design loads and their combinations (at ultimate state) are arrived at through
the following expressions, which are presented in simplified form. In these
expressions, $Q$ represents earthquake loading, $L$ represents the live load and $D$
represents the dead load.

a) $1.25D + 1.5L$

b) $1.25D + 1.0Q$

c) $1.25D + 0.7(1.5L + 1.0Q)$

d) $0.85D + 1.0Q$

In addition to the ultimate load state, there are serviceability and
fatigue requirements to be satisfied.

Additional Specifications (1990)

Interstorey deflections computed by elastic analysis are to be multiplied by
the $R$ factor, in anticipation of plastic deformation during an earthquake. In
these cases, maximum interstorey drift is limited to 1.0% for post-disaster
service buildings and to 2.0% for all other buildings. Adjacent buildings
should either be connected or else separated by the sum of their individual
deflections calculated as above. Foundations must be designed so as not to
yield before the superstructure.

Large differences in the stiffness or ductility in the orthogonal
directions of a framing system are to be avoided.

Floor systems that act as diaphragms are to be checked to ensure the
proper distribution of loads.

The use of unreinforced masonry is not recommended because of the
likelihood of brittle failure. It is prohibited in velocity-related zones of $Z_v$ of
2 and higher.
SPECIAL PROVISIONS FOR SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS

The National Building Code of Canada stipulates that buildings of plain, reinforced, or prestressed concrete must conform to the Canadian Standards Association "Design of Concrete Structures for Buildings" (CSA A23.3) [3]. Moreover, for seismic design, the NBCC allows the designer to use a relatively lower statically equivalent base shear in design, on the basis that the structural system will be capable of dissipating substantial energy through inelastic deformation.

The special provisions of Clause 21 (CSA A23.3) are in addition to the general requirements of the code. Clause 21 includes articles dealing with ductile moment-resisting space frames (NBCC \( R = 4.0 \)), ductile flexural walls (NBCC \( R = 3.5 \)) and building members with nominal ductility (NBCC \( R = 2.0 \)). In addition, there are Articles dealing with shear strength requirements and with frame members not considered as part of the lateral load-resisting system. The following are the highlights of these various requirements.

General Requirements

Clause 21 requires that all of the general provisions relevant to reinforced concrete design must be satisfied in addition to Clause 21, except as modified by Clause 21. Also, while Clause 21 does not contain provisions specifically for precast, composite or prestressed concrete structures, under the general requirements, these structural systems may be used, provided it can be demonstrated that the proposed system will have strength and toughness equal to a comparable monolithic reinforced concrete structure satisfying Clause 21.

In the design, all material strengths must be reduced by appropriate material resistance factors (\( \phi_c = 0.60 \) for concrete and \( \phi_s = 0.85 \) for reinforcing steel), which are meant to take account of variability in material properties and deviations in the dimensioning and placement of member elements. This approach is similar to that adopted by the CEB-FIP Model Code (CEB-FIP 1978) [13]. Member resistance factors, similar to ACI \( \phi \) factors (ACI 318 1983) [14], need only be applied where the possibility of pre-emptive brittle shear failure exists.

Three different types of internal resisting moments or forces are used in Clause 21. They are "factored resistance", calculated using \( \phi_s = 0.85 \) and \( \phi_c = 0.60 \); "nominal resistance" calculated using \( \phi_s \) and \( \phi_c = 1.0 \); and "probable resistance" calculated using \( \phi_s \) and \( \phi_c = 1.0 \) and using \( f'_c \) and \( 1.25f_y \) as the strengths of the concrete and the steel respectively (Table 2).
The reinforcement to be used in ductile frames and ductile walls is to be steel of controlled composition with yield strengths of 300 or 400 MPa. The actual yield strength is not to exceed the specified yield strength by more than 125 MPa and the actual ultimate strength is not to be greater than 1.25 times the actual yield strength.

Members of Ductile Moment-Resisting Frames (NBCC $R = 4.0$)

These provisions are very similar to the provisions of Appendix A of the American Concrete Institute Code (ACI 318 1983) [14]. There are separate Articles dealing with ductile frame members subjected to flexure, and flexure plus axial load and with joints in frames.

The energy dissipation necessary for a multi-storey frame to survive a severe earthquake should, in general, occur by the formation of ductile plastic hinges in beams. Plastic hinges in beams are capable of tolerating larger rotations than hinges in columns. Further, mechanisms involving beam hinges cause energy to be dissipated at many locations throughout the frame. An additional consideration is that extensive hinging in columns may critically reduce the gravity load carrying capacity of the structure. Accordingly, the flexural resistances of the columns and the beams shall satisfy

$$\Sigma M_{rc} \geq 1.1 \Sigma M_{nb}$$  \hspace{1cm} (4)

where:

$\Sigma M_{rc} =$ the sum of moments at the centre of the joint, corresponding to the factored resistance of the columns framing into the joint. The factored resistance of the columns shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, which results in the lowest flexural resistance and

$\Sigma M_{nb} =$ the sum of moments at the centre of the joint, corresponding to the nominal resistance of the beams and girders framing into that joint, consistent with the direction of the lateral forces considered. The nominal resistance shall include the effects of slab reinforcement within a distance 3 times the slab thickness measured from each side of the beam.

Flexural resistances shall be summed such that the column moments oppose beam moments. Equation 4 shall be satisfied for beam moments acting in either direction. In addition, to ensure the distribution of the longitudinal steel around the perimeter of the column, the clear distance between adjacent longitudinal reinforcing bars in compression members is not
to exceed 300 mm. Where cross-ties are used for transverse reinforcement, they must be anchored around longitudinal bars, not around other transverse reinforcement. The requirements for the design of joints in ductile, moment­ resisting frames are very similar to those of the ACI, Appendix A (ACI 318 1983). Horizontal shear stress limits are used as proportioning indices. As a result of the requirement for the distribution of column longitudinal steel around the perimeter of the column, it is felt that an additional calculation for vertical shear at the joint, similar to the New Zealand Code (NZS 3101 1982) [15], is not necessary.

**Ductile Flexural Walls (NBCC \( R = 3.5 \))**

The provisions for ductile flexural walls with an \( R \) value of 3.5 are unique to the NBCC. Encouragement of their use through an improved competitive position clearly reflects the Canadian code writers’ preference for ductile walls and coupled ductile wall systems as an efficient and reliable lateral load-resisting system.

The requirements as outlined below, are similar to those of the New Zealand building code (NZS 3101 1982) and are significantly different from the ACI Code, Appendix A (ACI 318 1983).

**General Requirements** -- Unless a rational analysis shows that hinging will not occur at a particular region, all sections of the wall must be detailed to accommodate plastic hinging. However, in the absence of any discontinuities of strength or stiffness of the lateral load-resisting system, only the bottom half of the wall need be so detailed.

In all cases, the shear resistance design provisions must also be satisfied.

**Dimensional Limitations** -- Effective flange widths for I, L, C, or T-shaped sections may not exceed half the distance to an adjacent wall web or 10% of the total wall height (Fig. 7).

In order to avoid potential stability problems associated with relatively thin walls under axial and lateral loading, the wall thickness within plastic hinge regions must not be less than \( l_u /10 \). This slenderness restriction can be waived in the three cases illustrated in Fig. 7, where lateral support is provided by the addition of boundary elements, by the wall itself outside the compression zone, and by the presence of flanges.

**Reinforcement** -- All anchorage, splicing and embedment must conform to the specifications for reinforcement in tension.
Welded splices and mechanical connections must be able to develop 1.5 times the yield strength of the bar and only alternate bars may be so spliced at any level. The longitudinal separation between such splices in adjacent bars must be at least 40d_b.

To avoid congestion, the reinforcement ratio in regions of concentrated reinforcement may not exceed 0.06.

The diameter of the bars at any section may not exceed one-tenth the wall thickness.

Distributed and Concentrated Reinforcement -- The requirements for distributed and concentrated reinforcement are summarized in Table 3.

In addition to the requirements of Table 3, the total wall must be proportioned to resist all factored loads including earthquake, and in this calculation, the distributed reinforcement should be accounted for. Also, minimum reinforcement must be provided to ensure that the cracked flexural resistance exceeds the uncracked flexural resistance. In computing strengths in this calculation, the axial dead load is to be included with a load factor of 0.85 but the axial live load is to be neglected.

Ductility -- The depth of the compression zone, c_c of a wall is taken as an indicator of its ductility and is used in the code as a dimensional index. In order to ensure the ductility of walls in which the depth of the compression zone exceeds 0.1γ_u l_w, the compression zone must be tied as a column and have a minimum vertical reinforcement ratio of 0.005. Walls in which c_c exceeds 0.4l_w are considered to be too brittle to be permitted.

A simple equation is provided for the calculation of c_c

\[ c_c = \frac{(P_s + P_q + \phi_c f'_c A_f)}{\phi_c f'_c b_w} \]

Coupled Walls -- A coupled wall is a system proportioned so that a significant amount of the overturning moment is resisted by axial loads resulting from vertical shear in the coupling members connecting the various wall elements. Wall systems not satisfying this requirement shall be designed with ductility limits on c_c calculated using the length of the individual wall elements.

Where the shear stress from factored loads exceeds 0.1(l_u/d)\sqrt{f'_c}, coupled walls must be connected by ductile coupling beams able to resist the in-plane factored shear and flexure by means of confined diagonal reinforcement as illustrated in Fig. 8.
In a severe earthquake, the primary energy dissipating elements should be the connecting beams so as to minimize damage to the walls. To this end, the factored resistance of the walls must be greater than the nominal resistance of the coupling beams.

**Building Members Requiring Nominal Ductility (NBCC \( R = 2.0 \))** -- These buildings are very popular in parts of Canada. Prior to the 1984 code, it was possible to design buildings using reduced seismic forces while omitting all special provisions for possible earthquakes. This article details a set of minimum requirements to ensure that the structure that has been built according to the non-seismic portions of the code will have some ductility to survive a potential earthquake.

The following is a brief listing of these provisions.

**Beams:** Continuous reinforcement and some positive and negative moment capacity are required throughout the beam. Also, confinement in the form of closely spaced stirrups is required at each end of the beam where inelastic action is most likely.

**Columns:** Closely spaced ties are required in regions of potential inelastic action.

**Walls:** The depth of the compression zone, \( c_c \), must not exceed \( 0.25y_{w}J_{w} \) unless additional concentrated vertical reinforcement is provided over the outer half of the compression zone. The ratio of distributed reinforcement along vertical and horizontal axes may not be less than 0.0025. Concentrated longitudinal reinforcement must be tied. To ensure unity of action at corners and junctions of intersecting walls, longitudinal reinforcement must be tied and all horizontal bars must extend to the far face of the joining wall and be anchored with a standard 90° hook around a vertical bar.

**Frame Members Not Part of the Lateral Force-Resisting System:** -- These provisions are intended to impart a minimum level of ductility to members designed for gravity loads only, so that they will continue to function under the excessive lateral deformations that may be associated with a severe earthquake.

Unless it is shown that the factored resistance of the member is not exceeded when the structure deflects laterally through twice the deformation due to factored lateral loads, it must be assumed that the member will form a plastic hinge mechanism. Where plastic hinges occur in beams, adequate rotational capacity must be provided and where they occur in columns, hoop reinforcement must be provided over the entire length of the column.
Shear Strength Requirements -- These requirements apply to all beams, columns, ductile flexural walls, and joints. Ductile flexural walls must resist a shear force greater than that which corresponds to the formation of a plastic hinge mechanism.

Traditional methods may be used to calculate the amount of shear reinforcement required except that the shear resistance of the concrete must be assumed to be zero in beams and at the plastic hinge locations of ductile flexural walls where reversed cyclic loading into the inelastic range is expected. Use of Compression Field Theory [16,17] is also permitted for calculating the required shear reinforcement, in which case $\theta$ must be chosen between $30^\circ$ and $60^\circ$, $f_{2,\text{max}}$ must be reduced by 20% and the assumed value of $\varepsilon_x$ must be increased from 0.002 to 0.004. Because for the potential for spalling, all shear reinforcement must be in the form of closed hoops.

CONCLUSIONS

This paper summarizes the development and features of the National Building Code of Canada for 1990 for the earthquake-resistant design and construction of reinforced concrete buildings. Over time, construction of ductile moment-resisting space frames has become increasingly difficult and costly. A recent discussion, by a practising engineer, of a paper on design of ductile frames [18] focuses on this issue.

In the review paper in 1978, Uzumeri, Otani and Collins [4], listed ten problem areas and hence research needs. The following is a brief list of those ten items as well as an indication of what action has been taken in dealing with them in the interval.

1. Use of 100-Year Peak Ground Acceleration as the basis for seismic zoning and design equations: In the 1985 version of the NBCC, all seismic and zoning information has been based on 10% probability of exceedance in a 50-year period.

2. Structures with Low Ductility in High Seismic Risk Zones: This issue has been addressed in NBCC 1985.

3. Protection of Post-disaster Service Buildings: The Importance Factor has been increased.

4. Comparison of Static and Dynamic Design Procedures: This matter was addressed by removing the optional dynamic analysis procedure from the code for 1985 and later editions.
5. Period Estimation and Design Seismic Force: Little improvement in period estimation, but the procedures for the determination of design seismic forces have been improved.


7. Use of Lightweight Concrete in Ductile Structures: Nothing has been done.

8. Yield Strength of Reinforcing Steel: Nothing has been done.

9. Limit States Design: NBCC has become a fully limit states design code.

10. Compatibility of Canadian and U.S. Practices in the Design of Earthquake-Resistant Buildings: Much less of a problem since moving to 10% probability in 50 years as the basis for maps. Although there is little difference in the design of ductile moment-resisting frames, there are significant differences in the provisions for design of ductile walls. Since 1941, the NBCC has been under continuous development. No doubt, as we learn more from earthquakes and research, the code will be further improved.

ACKNOWLEDGMENTS

I would like to express my thanks to Mr. Peter Leesti, Research Associate for his contributions to the preparation of this paper. I gratefully acknowledge the help of my colleagues who contributed to the appreciation of these complex issues, through their writing and spending many hours of their time in discussing these issues.

REFERENCES


14. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, 1983, 111 pp.


### TABLE 1 — FORCE MODIFICATION FACTORS — NBCC 1990 (1)

<table>
<thead>
<tr>
<th>Case</th>
<th>Type of Lateral Load Resisting System</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel Structures Designed and Detailed According to CAN/CSA-S16.1-M</td>
<td>4.0</td>
</tr>
<tr>
<td>2</td>
<td>ductile moment-resisting space frame</td>
<td>3.5</td>
</tr>
<tr>
<td>3</td>
<td>ductile eccentrically braced frame</td>
<td>3.0</td>
</tr>
<tr>
<td>4</td>
<td>moment-resisting space frame with nominal ductility</td>
<td>3.0</td>
</tr>
<tr>
<td>5</td>
<td>braced frame with nominal ductility</td>
<td>2.0</td>
</tr>
<tr>
<td>6</td>
<td>other lateral-force-resisting systems not defined in Cases 1 to 5</td>
<td>1.5</td>
</tr>
<tr>
<td>7</td>
<td>Reinforced Concrete Structures Designed and Detailed According to CAN3-A23.3-M</td>
<td>4.0</td>
</tr>
<tr>
<td>8</td>
<td>ductile moment-resisting space frame</td>
<td>3.5</td>
</tr>
<tr>
<td>9</td>
<td>ductile flexural wall</td>
<td>2.0</td>
</tr>
<tr>
<td>10</td>
<td>moment-resisting space frame with nominal ductility</td>
<td>2.0</td>
</tr>
<tr>
<td>11</td>
<td>wall with nominal ductility</td>
<td>2.0</td>
</tr>
<tr>
<td>12</td>
<td>other lateral-force-resisting systems not defined in Cases 7 to 10</td>
<td>1.5</td>
</tr>
<tr>
<td>13</td>
<td>Timber Structures Designed and Detailed According to CAN/CSA-O86.1-M</td>
<td>3.0</td>
</tr>
<tr>
<td>14</td>
<td>nailed shear panel with plywood, waferboard or strandboard</td>
<td>3.0</td>
</tr>
<tr>
<td>15</td>
<td>concentrically braced heavy timber space frame with ductile connections</td>
<td>2.0</td>
</tr>
<tr>
<td>16</td>
<td>moment-resisting wood space frame with ductile connections</td>
<td>2.0</td>
</tr>
<tr>
<td>17</td>
<td>other systems not included in 12 to 15</td>
<td>1.5</td>
</tr>
<tr>
<td>18</td>
<td>Masonry Structures Designed and Detailed According to CAN3-S304-M</td>
<td>1.5</td>
</tr>
<tr>
<td>19</td>
<td>reinforced masonry</td>
<td>1.0</td>
</tr>
<tr>
<td>20</td>
<td>unreinforced masonry</td>
<td>1.0</td>
</tr>
<tr>
<td>21</td>
<td>Other Lateral Load-resisting Systems not Defined in Cases 1 to 17</td>
<td>1.0</td>
</tr>
<tr>
<td>Col. 1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 2 — FACTORED, NOMINAL, AND PROBABLE FLEXURAL RESISTANCES (5)

<table>
<thead>
<tr>
<th>Type of Flexural Resistance</th>
<th>Definition</th>
<th>Where Used</th>
<th>Flexure</th>
<th>Flexure and Axial Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_f$ = factored flexural resistance</td>
<td>calculated using $\phi_c = 0.60$ and $\phi_a = 0.85$</td>
<td>all members must satisfy $M_f \geq M_i$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_n$ = nominal flexural resistance</td>
<td>calculated using $\phi_c = 1.00$ and $\phi_a = 1.00$</td>
<td>to ensure columns stronger than beams at a joint $\sum M_{nc} \geq 1.1 \sum M_{nb}$</td>
<td>$M_n = 1.20 M_i$</td>
<td></td>
</tr>
<tr>
<td>$M_p$ = probable flexural resistance</td>
<td>calculated using $\phi_c = 1.00$ and $\phi_a = 1.25$</td>
<td>to ensure shear capacities exceed shear corresponding to flexural hinging</td>
<td>$M_p = 1.47 M_i$</td>
<td>$M_p = 1.57 M_i$</td>
</tr>
</tbody>
</table>
TABLE 3 — REINFORCEMENT REQUIREMENTS FOR DUCTILE FLEXURAL WALLS (5)

<table>
<thead>
<tr>
<th>Region</th>
<th>Regions of Plastic Hinging</th>
<th>Other Regions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distributed Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Amount</td>
<td>$\rho \geq 0.0025$</td>
<td>$\rho \geq 0.0025$</td>
</tr>
<tr>
<td>Spacing</td>
<td>$\leq 300$ mm</td>
<td>$\leq 450$ mm</td>
</tr>
<tr>
<td>Horizontal Reinforcement</td>
<td>develop $f_y$ within region of concentrated reinforcement</td>
<td>extend into region of concentrated reinforcement</td>
</tr>
<tr>
<td>Anchorage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Where required</td>
<td>at ends, corners, and junctions of walls</td>
<td>at ends of walls</td>
</tr>
<tr>
<td>Amount* (at least 4 bars)</td>
<td>$\rho \geq 0.002 b_w t_w$</td>
<td>$\rho \geq 0.001 b_w t_w$</td>
</tr>
<tr>
<td></td>
<td>$\rho \leq 0.06 \times$ area of concentrated reinforcement region</td>
<td>$\rho \leq 0.06 \times$ area of concentrated reinforcement region</td>
</tr>
<tr>
<td>Concentrated Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hoop requirements</td>
<td>must satisfy general specifications for transverse reinforcement and more stringent limits on hoop spacing, imposed to ensure column action and ductile behaviour</td>
<td>hoop spacing governed by general specifications for transverse reinforcement</td>
</tr>
<tr>
<td>Splice requirements</td>
<td>not more that 50 percent spliced at same location</td>
<td>100 percent may be lap spliced</td>
</tr>
</tbody>
</table>

* Note: Amount of reinforcement must be adequate to ensure post-cracking capacity and acceptable ductility. Wall (taking into account the distributed reinforcement) must be proportioned to resist all factored load effects.

Note: Bar diameters must be less than or equal to $\frac{1}{10}$ of wall thickness.
Fig. 1—Seismic zoning map of Canada — NBCC 1970 (1)

Fig. 2—Contours of peak horizontal ground accelerations in units of g, having a probability of exceedance of 10 percent in 50 years — NBCC 1985, 1990 (1, 2)
Fig. 3—Contours of peak horizontal ground velocities in m/s having a probability of exceedance of 10 percent in 50 years — NBCC 1985, (1, 2)

Fig. 4—Seismic response factor $S$, NBCC 1985 (1, 2)
Fig. 5—Seismic response factor $S$, NBCC 1990 (1, 2)

Fig. 6—Curvature ductilities required to develop various system ductilities in cantilever shear walls with different height-to-length ratios and different plastic hinge length assumptions (10)
Lateral Force Transfer in Buildings

Fig. 7—Minimum wall thicknesses in plastic hinge regions (5)

Fig. 8—Diagonal reinforcement in coupling beams (5)
NOMENCLATURE

\( a \) = zonal acceleration ratio: the specified zonal horizontal ground acceleration expressed as a ratio to \( g \) (10 m/s\(^2\))

\( A_f \) = area of flange, mm\(^2\)

\( A_s \) = area of nonprestressed tension reinforcement, mm\(^2\)

\( b_w \) = beam web width, or diameter of circular section, or wall thickness, mm

\( c_c \) = depth of the compression zone measured from the compression edge of a wall section, mm

\( D \) = specified dead loads

\( d \) = distance from extreme compression fibre to centroid of tension reinforcement, mm

\( d_b \) = bar diameter, mm

\( F \) = foundation factor (1985 and 1990)

\( f'_c \) = specified compressive strength of concrete, MPa

\( f_y \) = specified yield strength of reinforcement, MPa

\( f_{2\text{max}} \) = diagonal compressive strength of concrete, MPa

\( I \) = importance factor of the building (1985 and 1990)

\( K \) = numerical coefficient that reflects the material and type of construction, damping, ductility, and/or energy-absorptive capacity of the structure (1985)

\( L \) = specified live loads due to intended use and occupancy

\( l_u \) = clear distance between floors or the effective horizontal lines of lateral support, or clear span, mm

\( l_w \) = horizontal length of wall, mm

\( M_{nb} \) = the nominal resistance of a beam, N\( \cdot \)mm
\begin{align*}
M_{rc} &= \text{the factored resistance of a column, } N \cdot \text{mm} \\
N &= \text{the total number of storeys above exterior grade} \\
P_{q} &= \text{the earthquake induced transfer force resulting from interaction between elements of a coupled system and shall be taken as the sum of the end shears corresponding to the nominal flexural resistance in the coupling beams above the section, } N \\
P_{s} &= \text{axial force at section resulting from specified dead load plus specified live load, } N \\
Q &= \text{live load due to wind or earthquake, whichever produces the more unfavourable effect} \\
R &= \text{force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour (1990)} \\
R &= \text{zonal factor in NBCC 1970 (see Fig. 1). Not related to 1990 factor.} \\
S &= \text{seismic response factor for the structure (1985 and 1990)} \\
T &= \text{fundamental period of vibration of the building, in seconds, in the direction under consideration} \\
U &= \text{factor representing level of protection based on experience, and is equal to 0.6 (1990)} \\
v &= \text{zonal velocity ratio: the specified zonal horizontal ground velocity expressed as a ratio to 1 m/s (1985 and 1990)} \\
V &= \text{minimum lateral seismic force at the base of the structure (1985 and 1990)} \\
V_{e} &= \text{equivalent lateral force at the base of the structure representing elastic response (1990)} \\
V_{f} &= \text{factored shear force at section, } N \\
W &= \text{dead load of the building plus the following:} \\
&\quad 25\% \text{ of the design snow load} \\
&\quad 60\% \text{ of the storage load} \\
&\quad 100\% \text{ of the contents of any tanks} \\
Z_{a} &= \text{acceleration-related seismic zone}
\end{align*}
\[ Z_v = \text{velocity-related seismic zone} \]

\[ \gamma_w = \text{wall overstrength factor equal to the ratio of the load corresponding to nominal moment resistance of the building to the factored load of the building} \]

\[ \varepsilon_x = \text{longitudinal strain at mid-depth of the member when the section is subjected to the factored moment at the section, the factored axial load at the section occurring simultaneously with the factored shear at the section, and the equivalent factored axial load caused by shear and torsion} \]

\[ \theta = \text{angle of inclination of the diagonal compression stresses to the longitudinal axis of the member, } ^\circ \]

\[ \rho = \text{ratio of nonprestressed longitudinal reinforcement} \]

\[ \phi_c = \text{resistance factor for concrete (0.60)} \]

\[ \phi_s = \text{resistance factor for reinforcing bars (0.85)} \]

**UNITS**

The Canadian code is based on the S.I. system of units.

1 MPa = 145 psi = 10.2 kg/cm²
Ductility of Columns, Wall, and Beams — How Much is Enough?

by W. G. Corley

Synopsis:

Two hypothetical reinforced concrete buildings (one with special moment resisting frames and the other with structural walls) were designed. Using a time-history inelastic behavior approach, both buildings were analyzed. Drifts were determined for these structures when subjected to severe earthquakes similar to those expected in North America. In addition, drifts associated with an analysis based on ground motions measured for the 1985 Mexico City earthquake were also determined. Measured drifts from components detailed under 1990's North American code requirements are compared with calculated building drifts. These comparisons indicate that 1990's code requirements provide significantly more capacity than calculated to be needed for the structures and components considered. Finally, minimum drift requirements for components to be used in ductile frame buildings and in shear wall buildings are suggested.

Keywords: beams (supports); columns (supports); ductility; earthquake-resistant structures; loads (forces); moments; reinforced concrete; structural forms; walls
Appreciation

The influence of Tom Paulay extends far beyond the University where he did his outstanding research and teaching, far beyond his many successful students — yes, even far beyond his New Zealand homeland. Tom Paulay is recognized as one of only a very few world leaders in seismic resistant design.

During my early days of doing research on seismic resistance of reinforced concrete, I began to read laboratory reports generated in this place I could hardly find on the globe, New Zealand. Long before I met him, the name and work of Tom Paulay became well known to me. I had already developed a high regard for the genius of this outstanding researcher.

Soon I began to meet some of Tom Paulay’s students—Professor Mike Collins, Professor Nigel Priestley, Professor Bob Park, to name but a few. The quality of these researchers demonstrated that Tom was an outstanding teacher who had the unique ability to inspire others to achieve success and to serve their profession.

Based on these contacts, I developed a mental image of Tom as a brilliant, articulate researcher who was also an inspirational teacher. When we finally met, I found all of these perceptions were well founded but not complete. Tom Paulay is also an enthusiastic yet humble man. As an example, during one conversation, he was noting that it was difficult for him to understand how New Zealand (a country that [according to him] has more sheep than people) could have any significant effect on seismic design elsewhere in the world. He was unable to come up with a credible answer.

After many years of consideration, I believe that I have the answer that Tom was too humble to give. New Zealand has had great influence on seismic resistance design in other countries because of one unique individual — Tom Paulay.

W. Gene Corley
W. Gene Corley is Vice President of Construction Technology Laboratories. A registered structural engineer, Dr. Corley has authored numerous technical publications related to design and behavior of reinforced and prestressed concrete structures. He is Chairman of ACI Committee 318, Standard Building Code; a member of ACI Committee 341, Earthquake-Resistant Concrete Bridges; and of joint ACI-ASCE Committee 343, Concrete Bridge Design. He has received many awards, including the Wason Medal for Research and the Turner Award from ACI.

INTRODUCTION

Performance of a structural concrete building subjected to earthquake forces is a function of stiffness, strength, deformation demands, and deformation capacity. Selection of strength provided for a given design earthquake has a significant effect on deformation or drift demands. As illustrated in Fig. 1, increasing strength for a given event reduces deformation demands. A key item affecting the balance between strength and drift capacity is reinforcement details. Details must be provided to insure that strength and drift capacities can be attained for the design earthquake.

Fig. 1—Recommended limits for drift capacities of test components
With the exception of essential facilities, economic considerations demand that most buildings located in regions of high seismicity be provided with a resistance considerably smaller than that required to remain elastic under seismic forces resulting from a major earthquake. In such cases, reliance is placed on the available inelastic capacity of the building members to dissipate energy and avoid collapse.

Seismic-resistant design of reinforced ductile frame buildings (1-6) in regions of high seismicity provides a relative condition of a strong column and weak beam at any junction. The intent is to encourage hinging in the beams rather than in columns. However, building performance during major earthquakes (7-10) and results of analysis (11-14) indicate that hinging can form in columns. Hinging can be attributed to several factors, including the following:

1. Design generally based on first mode of vibration, whereas influence of higher modes of vibrations could significantly alter the moment distributions among frame members.

2. Strain hardening of the flexural reinforcement in beams.

3. Bi-directionality of seismic forces.

4. Differences between actual behavior of structures during seismic events and mathematical modeling of the building in design.

Therefore, the possibility of plastic hinge formation at column ends demands that building columns in seismic areas have significant curvature ductility.

Previous research has shown that ductility of columns can be improved with suitable confinement. (15-17) Testing has been directed toward understanding the functions of lateral ties as confinement for the column core, restraint against buckling of longitudinal reinforcement, and shear reinforcement.

Sakai and Sheikh (18) have summarized major research conducted on the subject of confinement in concrete columns. A general finding of most reported experimental studies is that both concrete stress and strain capacity are increased by an increase in the total amount of transverse reinforcement, and increase in the number of longitudinal bars located around the column perimeter and tied in the corner of a hoop bar.

In general, research conducted to evaluate the effects of the transverse reinforcement arrangement and detail on column performance has been limited. Research related to beam and joint hinging is more extensive. (19)

On September 19, 1985 and again on September 20, Mexico City was subjected to major ground shaking resulting in thousands of fatalities and the destruction of numerous structures. In the days following the earthquake, the author along with a team of engineers visited the quake site and later reported findings. This paper deals primarily with performance of concrete structures, the comparison of field observations to predicted performance, comparison of field observations to laboratory data, and recommendations for determining how much testing is "enough" for laboratory components.
The format includes a brief overview of the general observations made during the quake site visit, followed by more specific observations related to concrete construction. A comparison is made between calculated behavior of a fictitious but critically tuned building subjected to the ground motion measured in Mexico City and behavior of full-size laboratory test specimens. Finally, conclusions applicable to performance of structures under 1990's seismic codes are summarized.

BACKGROUND

Reports concerning the faulting and general geology of the Mexico City area have documented the historical seismicity and is not repeated in this paper. The 1985 earthquake epicenter was approximately 240 miles (400 km) west of Mexico City. The Richter magnitude was reported at 8.1, with a strong aftershock of 7.5. Accelerations on the order of 20% g were recorded in the vicinity of the epicenter. At the outskirts of the old lake bed in Mexico City, accelerations were measured to have diminished to 4% g. Within the lake bed, they were again amplified to approximately 20% g and showed a pronounced 2-second period with a duration of 60 seconds as shown in Fig. 2. (20) The elastic response spectrum, shown in Fig. 3, developed from the ground motion records using 2% damping, indicate extremely large peaks in the 2-second period range. (21)

The shape of the spectrum is significant. It is substantially different than that expected in California and on which North American codes are based. It is generally accepted that buildings respond to ground motions with an initial predominant period and that this period may become longer as the earthquake "loosens" the structure. Based on the El Centro type spectrum, shown in Fig. 3, lengthening the period would diminish the response of the structure, thus, generally, moving the structure away from resonant period problems.

In looking at a similar line of thinking for the Mexico City quake, it is apparent that buildings with periods less than two seconds for initial response would tend to move toward more critical response periods as they were "loosened" by the initial ground motions. This is clearly not the type of response anticipated by North American codes and has direct implications with regard to the performance of non-structural infilled walls.

As reported elsewhere, (22) a remarkable number of buildings, such as the one in Fig. 4, were apparently undamaged. Foundation problems were significant, including settlement due to loss of bearing; differential settlement reportedly due to loss of skin friction on piles; pile failures both compression and pull out; and loss of veneer from the skin of buildings.

Structural damage to concrete buildings varied from no damage to complete collapse. Of the collapsed buildings, the weakness appeared to be predominantly in the columns. As shown in Fig. 5, lack of adequate ties and hoops was observed. The search for well reinforced concrete construction was difficult, but perhaps encouraging in that none of the collapsed concrete construction was of the level required by codes of the 1990's. (23-25)
Fig. 2—S60E component of ground accelerations determined from N and E components measured at SCT in Mexico City.

Fig. 3—Comparison of velocity responses, El Centro, 1940, N-S and SCT, Mexico City, 1985, S60E component.
Fig. 4—Undamaged building

Fig. 5—Columns damaged in September 1985 Mexico City earthquake
ANALYSIS OF HYPOTHETICAL BUILDING

To permit comparison of response of a building designed to meet the current codes, (23,24) a hypothetical building shown in Fig. 6 was designed and analyzed. Details of the structure are given in Reference 25 and its analysis is described in Reference 26. The building analyzed has seven levels and has a Special Moment-Resisting Frame. A similar building with shear walls was also analyzed.

Spans and dimensions of the structure analyzed were chosen to make the building extremely flexible so that it would be in resonance with the predominant 2-second period of the Mexico City Earthquake. A computer program, DRAIN II, developed at the University of California, Berkeley, (27) was used to do inelastic time history analyses of the structure. One set of analyses assumed 5% damping and another set assumed 10% damping. Although it can be argued that the 10% damping may be too high, 5% damping should be attainable.

As can be seen in Fig. 7, maximum drift for the building with a 2-second period was calculated to be 2% for the lower amount of damping and 1.8% for the higher amount of damping. These calculated displacements are significantly greater than the 1.5% drift generally assumed acceptable for design. The large displacements are a direct result of the resonance of the hypothetical building with the measured ground motions.

COMPARISON WITH LABORATORY TEST RESULTS

In an attempt to gage anticipated performance of a building designed by 1990's codes, (23-24) calculated drift demands are compared with drift capacities measured on large laboratory specimens. The codes intend that hinging will occur in beams at or near joints. In Mexico City, it was observed that many of the collapsed buildings had hinging in the columns above or below the joints. Consequently, separate comparisons are made with test results for hinging occurring in joints, in beams, in columns, and in walls.

Beam Hinging

Figure 8 shows test results from a large specimen where hinging occurred in the beam. (28) Hoop reinforcement and stirrup reinforcement in this specimen met requirements of current codes. (23, 24)

The two vertical dashed lines in Fig. 8 show the calculated drift demand calculated for the 1985 Mexico City Earthquake. Drift capacity of the test specimen was found to be on the order 6%. This is more than that calculated as being needed. Consequently, if hinging occurs in the beams, it appears likely that the required drift capacity would be available.
Fig. 6—(a) Plan; (b) E-W frame member sizes; (c) N-S frame member sizes
Joint Integrity

It is well-known that loss of capacity may occur if beam-column joints are not properly reinforced. Figure 9 shows results of a test where no joint reinforcement was provided. (29) Vertical dashed lines are again used to show calculated drift demand. As can be seen, this joint lost capacity at drifts less than those calculated for the Mexico City Earthquake.
Fig. 8—Load versus drift capacity measured with test specimen beam hinging

Fig. 9—Load versus drift capacity measured for joints without hoops
Figure 10 shows results of tests where joint reinforcement meeting the requirements of current codes (23,24) was provided. The capacity measured for this test specimen was very similar to that shown in Fig. 8 for beam hinging. A measured capacity of about 6% was obtained giving a margin of about three times that needed. This suggests that joint reinforcement required by the Blue Book could be expected to provide drift capacity necessary to survive the earthquake considered in the analysis.

![Figure 10: Load versus drift capacity measured for joints with hoops](image)

**Calculated Drift For 1985 Mexico City Earthquake**

1.0

Calculated Drift

For 1985 Mexico City Earthquake

Fig. 10—Load versus drift capacity measured for joints with hoops

**Column Hinging**

In the 1985 Mexico City Earthquake, column hinging was observed to have occurred more often than beam hinging. Remains of damaged columns show that the amount of ties provided was much less than that required by the current codes. (23,24)

Figure 11 shows results of a test where hinging was purposely caused in the column. (30) This column had only one-half the confinement reinforcement required by current codes. However, the amount of reinforcement in the test component is substantially greater than that observed in any of the damaged columns seen in Mexico City.

As indicated in Fig. 11, drift capacity of this column was only slightly over 2%. This is very close to the calculated drift demand for the earthquake. It appears likely that buildings with light reinforcement in the columns such as
Fig. 11—Measured moment versus drift capacity for column hinging with one-half confinement required by blue book

that observed in Mexico City, would have a significant chance of failure in this earthquake. However, they should be able to survive an earthquake of a magnitude anticipated by 1990's codes. Figure 12 shows results of a test where column hinging was purposely caused and where full confinement reinforcement was provided.

Reinforcement in this test specimen was detailed to meet requirements of the current codes (23,24) including the six-diameter extension for tails of hoops.

As can be seen, drift capacities of about 4% were obtained. This drift is about double the calculated demand for the hypothetical building. It appears then that even if hinging occurs in the columns, a building detailed by the current codes would be likely to survive the 1985 Mexico City Earthquake.

Structural Walls

Although buildings with structural walls (shear walls) generally performed well in the 1985 Mexico City Earthquake, reinforcing details observed would not satisfy current code requirements. In general, boundary elements contained widely spaced ties rather than confinement reinforcement.
Figure 13 shows load versus drift measured for shear wall governed by flexural capacity. (31) This wall met reinforcing requirements of current codes. (23,24) Boundary elements contained full confinement reinforcement.

As indicated, drift capacity of the wall was greater than 4%, significantly higher than that estimated for the 1985 Mexico City Earthquake (25) and shown by the vertical dashed lines. As in the case of beams, columns, and joints, the test results suggest that shear wall buildings designed by current codes should survive ground motion similar to those of the 1985 Mexico City Earthquake.

**HOW MUCH IS ENOUGH?**

For more than 4 decades, researchers have been testing buildings and components to determine potential resistance to earthquakes. Most of these tests have been carried out to determine the deformational capacity of the test specimens. This is essential information to have for the purposes of design.

The most important information needed by the designer includes the amount of deformation that will occur during the design earthquake and whether or not details provided will result in a structure that can reach this deformation without collapse. In recent years, researchers and designers have come to the realization that unlimited amounts of deformation capacity are neither possible
nor needed. Consequently, a finite amount of deformation capacity should be sufficient. The problem has been in defining what the finite amount is.

In general, North American Codes anticipate a total drift of about 1-1/2% under the design earthquake. With this drift, damage will occur to the building but the possibility of collapse is expected to be small. In this paper, calculations show that even when the severe ground motions of the 1985 Mexico City earthquake applied to the building whose predominant period is essentially the same as that of the Mexico City earthquake, drift is still in the range of just over 2%. For shear wall buildings, this drift may be significantly less.

Several researchers including Sozen at the University of Illinois and Park in New Zealand, have referred to a drift limit of about 1-1/2%. Although these drift limits were not related to a specific building type, the implication is that 1.5% should be the overall limit for drift capacity. This limit would be applicable to a special moment frame building. In this report, comparisons between calculated drifts and test results of components designed by current North American codes suggest that drift requirements with earthquakes exceeding design values should still be in the range of 2% for ductile frame buildings. For combination shear wall and ductile frame buildings, drift limits may be in the range of half this amount. Consequently, it is the author's opinion that components with capacities that allow at least 3% drift when used in the building, should be considered to have adequate inelastic capacity. For walls, it is suggested that components that have test values of 2% drift should be considered adequate. Suggested limits are shown in Fig. 1. In both cases, the drift capacities are almost double those calculated for the design earthquake. Consequently, these limits should give ample reserve for unexpectedly large ground motions.
CONCLUDING REMARKS

This paper discusses two hypothetical structures (one with special moment resisting frame and the other with structural walls) designed to meet current North American code requirements and analyzed using an inelastic time-history computer program as described elsewhere (25) are discussed. Calculated drifts for selected hypothetical buildings are shown in comparison with results of tests on components reinforced to satisfy current code requirements. (23, 24)

Calculated drifts under severe earthquake loading are in good agreement with the approximately 1-1/2% drift limits implied by the design procedures of the codes. Even when more severe earthquakes are considered, calculated drifts do not significantly exceed 2%.

Comparison of results of component tests indicate that drift capacities of beams, joints, columns, and walls designed to meet current codes, significantly exceed those calculated to be required. Based on these observations and observations of buildings in the 1985 Mexico City earthquake, it is suggested that the following minimum drift requirements are adequate for components:

1. Ductile frame buildings.............................. 3%
2. Buildings containing shear wall ..................... 2%

If details required by 1990's North American codes are provided, buildings should be expected to survive earthquakes even more severe than those considered in design.
REFERENCES


The Use of Rational Design Methods for Shear

by M. P. Collins

Synopsis: The essential features of the "modified compression field theory" are described. A group of behavioral models, based on these assumptions is presented. The use of these models is illustrated and reference is made to experimental data and to existing design codes. A simple, unified design method for shear, that is able to approach both routine and "unusual design problems," is presented. The method is applicable to both prestressed and non-prestressed concrete members; it treats members subjected to either axial tension or axial compression, and it treats both members with web reinforcement and members without web reinforcement.

Keywords: aggregate interlock; axial loads; building codes; crack width and spacing; models; reinforced concrete; shear strength; structural design
AN APPRECIATION

It is now more than thirty years since I first met the extraordinary man who was to have such an influence on so many young New Zealand engineers. He was a new lecturer who had recently joined the School of Engineering on the new campus at Ilam, and for us new civil engineering students, he was an irresistible role model.

He first taught us in a course called "Drawing and Design", which was scheduled for 2 to 5 pm, Tuesdays and Thursdays. Sitting at our draughting tables in that large room, flooded with daylight, we began to learn what it is to be an engineer. "Design a two storey timber frame house by next week!" At first the task seemed impossible, but with the endless help of this tall Hungarian-New Zealander, it turned out that all that was required was clear thinking, common sense, a little ingenuity, and a very large amount of hard work. In the "optional" Saturday morning tutorials, we all somehow received one-on-one coaching and came to know the fascinating person we addressed as Mr. Paulay, but spoke of as "Tom".

In our later courses he introduced us to the design of reinforced concrete structures and his delight in this subject was infectious. From simple beams to continuous, post-tensioned bridges with variable cross-sections, the approach was the same. Work from first principles. Draw free body diagrams. Sketch deflected shapes. Visualize the flow of forces. Carefully draw all the details.

He told us about fascinating research that was being conducted around the world on the behaviour of reinforced concrete, and in the structures laboratory he showed us the experiments he was conducting to try to understand how cracked reinforced concrete transmits shear. The problems were challenging, but he had already shown us how deeply satisfying it is to achieve a very difficult task, and had given us confidence in the power of the engineering principles he had taught us.

Like many other New Zealand engineers, I started as Tom Paulay's student, and was privileged to become his friend.

I owe you so much Tom - thank you.

Michael Patrick Collins
Oakville, August 1993
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**INTRODUCTION**

It is now twenty years since the ACI-ASCE Shear Committee [1] concluded the introduction to its state-of-the-art report with the words:

> During the next decade it is hoped that the design regulations for shear strength can be integrated, simplified and given a physical significance so that designers can approach unusual design problems in a rational manner.

In the two decades since the above words were first published a considerable amount of research has been conducted with the aim of developing behavioral models for reinforced concrete in shear, comparable in rationality and generality to the plane sections theory for flexure. Such models should make it possible to predict not only the shear strength but also the complete load-deformation response of reinforced concrete elements subjected to shear. One group of such models is based on a collection of assumptions about the behaviour of reinforced concrete that is known as the "modified compression field theory". This paper will summarize the key aspects of this theory and will illustrate the use of models based on this theory.

**MODIFIED COMPRESSION FIELD THEORY**

Perhaps one of the reasons why the "shear problem" was so difficult to understand was that the traditional type of shear test, while simple to perform, was difficult to analyze. In such a test (see Fig. 1) the behaviour 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 modified compression field theory, experiments were conducted on elements subjected to uniform stresses (see Fig. 1). While these tests were more difficult to perform, they were easier to analyze.
Even for reinforced concrete elements subjected to simple, uniform loading such as pure shear, the behaviour is rather complex. As the load is increased new cracks open and pre-existing cracks close. The cracks have rough surfaces capable of transmitting considerable shear stresses. The local stresses in both the concrete and the reinforcement vary from point to point, with high reinforcement stresses but low concrete tensile stresses occurring at crack locations.

The modified compression field theory attempts to capture the essential features of the behaviour of cracked reinforced concrete without considering all of the details. The equilibrium conditions which relate the concrete stresses and the reinforcement stresses to the applied loads (see Fig. 2) are expressed in terms of average stresses. That is, stresses averaged over a length greater than the crack spacing. In a similar fashion, the compatibility conditions relating the strains in the cracked concrete to the strains in the reinforcement are expressed in terms of average strains, where the strains are measured over base lengths that are greater than the crack spacing (see Fig. 3).

One of the key simplifying assumptions of the modified compression field theory 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" [2]. That is, the principal stress direction in the cracked concrete ($\theta_c$ in Fig. 2) is assumed to coincide with the principal strain direction ($\theta$ in Fig. 3).

It was found [3] that the principal compressive stress in the cracked concrete, $f_2$, is a function, not only of the principal compressive strain, $\varepsilon_2$, but also of the coexisting principal tensile strain, $\varepsilon_1$ [see Fig. 4(a)]. A suitable, simple relationship is

$$f_2 = f_{2,\text{max}} \left[ 2 \frac{\varepsilon_2}{\varepsilon_c} - \left( \frac{\varepsilon_2}{\varepsilon_c} \right)^2 \right]$$

where

$$f_{2,\text{max}} = \frac{f_c'}{0.8 + 170\varepsilon_1} \leq f_c'$$

Figure 5 compares the observed maximum compressive stresses, $f_{2,\text{max}}$, for 68 biaxially loaded specimens, with their associated principal tensile strains, $\varepsilon_1$. Specimens plotted in this figure all exhibited compressive failures of the concrete, with $\varepsilon_2/f_c'$ at failure exceeding 0.5. Also shown in Fig. 5 are the predictions of Eq. 2, which is the expression recommended by the Canadian concrete code, CSA A23.3 1984 [4]. The Norwegian concrete code
NS 3473 1989 [5] gives a similar expression except that the factor 170 in Eq. 2 is changed to 100, which makes the Norwegian expression less conservative. The third line shown in the figure represents an earlier, more conservative expression recommended by the author in 1978 [2]. The message of Fig. 5 is clear: as the principal tensile strain in the cracked concrete increases, the maximum compressive stress that the cracked concrete can resist is reduced.

Tests of reinforced concrete elements subjected to pure shear [3] demonstrated that even after extensive cracking, tensile stresses still existed in the cracked concrete and these stresses significantly increased the ability of the cracked concrete to resist shear stresses. The average principal tensile stress in the concrete, \( f_1 \), was related to the principal tensile strain, \( \varepsilon_1 \), and to the crack width, \( w \) [see Fig. 4(b)]. Prior to cracking (i.e., \( \varepsilon_1 < \varepsilon_{cr} \))

\[
f_1 = E_c \varepsilon_1
\]  

(3)

After cracking

\[
f_1 = \frac{f_{cr}}{1 + \sqrt{500\varepsilon_1}}
\]  

(4)

where

\[
f_{cr} = 0.33\sqrt{f'_c} \text{ MPa}
\]  

(5)

but

\[
f_1 \leq \frac{0.18\sqrt{f'_c} \tan \theta}{0.3 + \frac{24w}{a + 16}} \text{ MPa and mm}
\]  

(6)

For large values of \( \varepsilon_1 \), the cracks will become wide and the magnitude of \( f_1 \) will be controlled by the yielding of the reinforcement at the crack and by the ability to transmit shear stresses across the crack, which is a function of the crack width, \( w \), and the maximum aggregate size, \( a \).

In checking the conditions at a crack, the actual complex crack pattern is idealized as a series of parallel cracks, all occurring at angle \( \theta \) to the longitudinal reinforcement and spaced a distance \( s_\theta \) apart. The crack width can then be estimated as

\[
w = s_\theta \varepsilon_1
\]  

(7)
PREDICTING THE SHEAR RESPONSE OF BEAMS

The modified compression field theory is a procedure for predicting the stress-strain response of elements of reinforced concrete. In using this procedure to predict the response of a beam such as that shown in Fig. 1, a number of additional assumptions will usually be required.

Engineering beam theory assumes that plane sections remain plane and hence, can study the response of a beam section-by-section, without being concerned with the details of how the forces are introduced into the member. The modified compression field theory has been used as the basis for a number of such sectional models, three of which will be briefly discussed below.

The most powerful sectional model developed so far is called the "dual-section" model [6] and forms the basis for program SMAL. In this analysis, the biaxial stresses and strains and the manner in which they vary over the height of the beam are considered. It is found that the inclination, $\theta$, of the principal compressive stress changes continuously over the height of the beam, becoming steeper near the flexural tension face and shallower near the flexural compression face. By considering 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 a considerable redistribution of shear stresses resulting in higher shear stresses near the stiffer flexural compression side and lower shear stresses near the more flexible flexural tension side.

The detailed, dual-section analysis requires considerable computation and hence, a simpler model, which forms the basis for program RESPONSE, was developed [7]. This approach makes use of the following simplifications:

1. The redistribution of shear stresses that occurs prior to failure is ignored. The shear stress in the web is assumed to be equal to the shear force divided by the effective shear area, $b_d v_f$ (see Fig. 6).

2. The biaxial stresses and strains are considered at just one level of the web. The longitudinal strain at this location, $\varepsilon_y$, is used to calculate $\theta$, which is then assumed to remain constant over the depth of the web (see Fig. 6).

While programs SMAL and RESPONSE are 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", sectional model, referred to as the beta method, has been developed. In this method the shear strength of a section, such as that shown in Fig. 6, is expressed as
\[ V_u = V_c + V_s + V_p \]
\[ = \beta \sqrt{f' c \cdot b v \cdot d v} + \frac{A_v f_y}{s} \cdot d v \cdot \cot \theta + V_p \]  

(8)

where \( V_c \) is the shear strength provided by the residual tensile stresses in the cracked concrete, \( V_s \) is the shear strength provided by tensile stresses in the stirrups, and \( V_p \) is the vertical component of the force in the prestressing tendons.

Values of \( \beta \) and \( \theta \), determined from the modified compression field theory [7], are given in Fig. 7 for members with web reinforcement, and in Fig. 8 for members without web reinforcement. The residual tensile stress factor, \( \beta \), which indicates the ability of the cracked concrete to transmit tensile stresses, has been calculated from Eqs. 4, 5 and 6. In using these equations the highest longitudinal strain, \( \epsilon_x \), occurring within the web is used to calculate the principal tensile strain \( \epsilon_1 \). As the straining increases, \( \beta \) decreases. For simple design calculations \( \epsilon_x \) can be approximated as the strain in the "bottom chord" of an equivalent truss. Hence:

\[ \epsilon_x = \frac{(M_u/d_v) + 0.5N_u + 0.5V_u \cot \theta - A_{ps} f_{po}}{E_s A_s + E_p A_{ps}} \]  

(9)

where \( A_s \) and \( A_{ps} \) are the areas of non-prestressed and prestressed longitudinal reinforcement on the flexural tension side of the member, and \( f_{po} \) is the stress in the prestressed reinforcement when the surrounding concrete is at zero stress. The terms \( M_u \) and \( V_u \) are taken as positive, while \( N_u \) is positive for tension and negative for axial compression.

In determining the \( \beta \) values for members with web reinforcement (Fig. 7) it was assumed that the stirrups would limit the spacing of the diagonal cracks to about 300 mm. For members without stirrups the diagonal cracks will typically be more widely spaced, with the cracks becoming more widely spaced as \( \theta \) approaches zero. The crack spacing when \( \theta \) equals 90° is called \( s_x \), and this spacing is primarily a function of the maximum distance between the longitudinal reinforcing bars, or between the reinforcing bars and the flexural compression zone (Fig. 8). As \( s_x \) increases, \( \beta \) decreases and hence, the shear strength decreases.

Shear causes tensile stresses in the longitudinal reinforcement as well as in the stirrups. 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 must satisfy the following requirement:
It should be emphasized that the three sectional models described above, SMAL, RESPONSE, and beta, all rely on engineering beam theory and hence, cannot predict the behaviour in "disturbed regions", close to abrupt changes in cross-sectional forces or cross-sectional dimensions. Such regions require the use of procedures that more closely approximate the actual flow of forces. These procedures include computer based non-linear finite element formulations [8, 9, 10] and more simple strut-and-tie models [7, 11].

Figure 9 compares the predictions of a strut-and-tie model [7], and the beta sectional model for a series of simply supported reinforced concrete beams loaded with two point loads. For this series of beams, which were tested by Kani [12], the prime variable was the length of the shear span. It can be seen that a beam can resist a very high shear force if the shear is caused by a load that is close to the support. Further, it can be seen that the sectional model gives very conservative predictions when the support and the load are less than about 2d apart. This observation justifies the approach of not considering the shear response of sections that are closer than dv to a support or to a major concentrated load.

**COMPARISON OF BEAM ANALYSIS MODELS**

As explained above, there are a number of analysis models, all based on the modified compression field theory, which can be used to investigate the shear response of a reinforced concrete beam. These range from detailed, complex, non-linear finite element formulations (e.g., program FIERCM [10]) to simple section strength models (e.g., the beta method).

Figure 10, which is from Reference 10, demonstrates the excellent results that can be obtained from program FIERCM (pronounced "fearsome"!). As a basis for comparison, the beta method will be used to estimate the shear strength of the beam described in Fig. 10.

From Eq. 8 we have

\[ V_u = \beta \sqrt{f_c} b_v d_v + \frac{A_v f_y}{s} d_v \cot \theta \]

\[ = \beta \sqrt{24.1 \times 307 \times 0.9 \times 466} + \frac{2 \times 32.3 \times 325}{210} \times 0.9 \times 466 \cot \theta \]

\[ = 632 \beta + 41.9 \cot \theta \text{ kN} \]

As a first estimate assume

\[ \beta = 0.2 \text{ and } \theta = 36^\circ \]
Hence,

\[ V_u = 126 + 58 = 184 \text{ kN} \]

and

\[ \frac{v}{f'_c} = \frac{184000}{307 \times 419 \times 24.1} = 0.059 \]

The critical section, with the highest moment, will be the section \( d_v \) (i.e., 419 mm) from the face of the loading plate. That is, the section 1829 - 0.5\( \times \)305 - 419 = 1257 mm from the centre of the support. At this section

\[ M_u = 1257V_u = 231 \text{ kNm} \]

Hence, from Eq. 9

\[ \varepsilon_x = \frac{231000000/419 + 0.5 \times 184000 \cot 36}{200000 \times 4 \times 658} \]

\[ = 1.29 \times 10^{-3} \]

But from Fig. 7, if \( v/f'_c \) equals 0.059 and \( \varepsilon_x \) equals \( 1.29 \times 10^{-3} \) then

\[ \beta = 0.18 \quad \text{and} \quad \theta = 39^o \]

As the calculated values of \( \beta \) and \( \theta \) differ somewhat from the estimated values, the calculations should be repeated until closer convergence is obtained. With \( \beta \) taken as 0.18 and \( \theta \) as 38°, \( V_u \) is 167 kN, \( v/f'_c \) is 0.054, \( \varepsilon_x \) is \( 1.15 \times 10^{-3} \) and convergence is close enough.

The above estimate of the failure shear and the resulting failure load (2 \( \times \) 167 = 334 kN) are considerably more conservative than the estimate obtained from FIERCM (see Fig. 10). The finite element model could better account for the redistribution of shear stress that occurred prior to failure and could more accurately estimate the web strains. The \( \varepsilon_x \) values were lower, resulting in lower values of \( \theta \) (see Fig. 10), and higher values of \( \beta \).

The shear strength of the beam described in Fig. 10 could also have been estimated from the traditional ACI expression [13] which in metric units becomes

\[ V_u = 0.17\sqrt{f'_c b_w d} + \frac{A_v f_y d}{s} \]

\[ = 0.17\sqrt{24.1 \times 307 \times 466} + \frac{2 \times 32.3 \times 325}{210} \times 466 \]

\[ = 164 \text{ kN} \]
Note that this simple estimate of the shear strength, which ignores the influence of moment and of longitudinal reinforcement, is nearly identical, in this case, to that obtained from the beta method. In other situations the two methods can give substantially different results. One such situation is described below.

In the beta method presented in this paper, the shear strength of members not containing stirrups is a function of the distance between the longitudinal reinforcing bars. The increase in shear strength that results from having more closely spaced longitudinal reinforcement is illustrated in Fig. 11, which gives the failure shears of four large, lightly reinforced, high-strength concrete beams. The ACI shear design procedures do not account for the lower failure shear stress of large, lightly reinforced beams and hence, these procedures can be seriously unconservative for such types of members. The unconservative nature of Eq. 11 for these situations is even more notable if it is recalled that, for beams without stirrups, this equation is supposed to predict the load at which significant diagonal cracks form, rather than the final failure load. For the stronger pair of beams ($f'_c = 86$ MPa) this estimate of diagonal cracking shear would be

$$V_u = 0.17 \sqrt{86 \times 295 \times 920}$$

$$= 428 \text{ kN}$$

Figure 12 shows that for the beams without intermediate longitudinal bars, diagonal cracks 0.7 mm wide had formed when the applied shear was only 184 kN. With the more closely spaced longitudinal bars, a shear force of 343 kN was applied before diagonal cracks 0.6 mm wide formed (see Fig. 13). It is believed that these two photographs provide strong evidence for the link between reinforcing bar spacing and diagonal cracking shear.

SHEAR DESIGN OF CONCRETE OFFSHORE STRUCTURES

One significant advantage of a rational design model is that it can be used to "approach unusual design problems". Deciding whether the heavily loaded walls of concrete offshore structures need to contain stirrups is an example of such a problem.

Whereas, for land based structures, the sectional forces are usually determined by analysing an equivalent frame, for offshore structures it has become the practice to find these forces using an elastic finite element analysis (see Fig. 14). The loading demand placed on a particular location of the structure can be expressed in terms of eight stress resultants, that is, three membrane forces ($N_x, N_y$ and $V_{xy}$), three moments ($M_x, M_y$ and $T_{xy}$), and two transverse shear forces ($V_{xz}$ and $V_{yz}$). Note that these stress resultants are obtained by integrating the stresses obtained from the finite element analysis over the thickness of the element and hence, they are expressed in terms of
force or moment per unit width (e.g., \( N_x = 5000 \text{ kN/m} \)). Even with the aid of modern super-computers, finding these stress resultants for several thousand locations in the structure, for several hundred different loading cases, is a major engineering challenge. Recent finite element models typically involve more than 500,000 degrees of freedom and formulating and checking such large models takes a substantial team of engineers many months.

After the sectional forces are calculated the required reinforcement at each location in the structure must be determined. It must be demonstrated that the section has adequate strength and also satisfactory service load performance (e.g., the crack widths must remain suitably small). Again, this is a major task with considerable difficulties being encountered in extending traditional simple beam procedures to the more complex situations encountered in shell structures. For example, in designing the wall sections to resist shear it is not a simple matter to extend the empirical shear design rules for beams to the loading situation shown in Fig. 14. It does not help that in this problem we need to consider the triaxial stress-strain response of cracked concrete.

In the last few years a shell section analysis program called SHELL474, which can be regarded as a generalization of program RESPONSE, has been developed. This program was first formulated as part of a verification study [14] of the new Canadian code for concrete offshore structures, CSA S474-M89 [15]. It is hoped that procedures such as those incorporated in SHELL474 will replace the existing "equivalent beam" shear design procedures, which are believed to be unsatisfactory.

The shear failure of the Sleipner A platform in August 1991, as it was being ballasted down prior to deckmating, dramatically demonstrated some of the deficiencies of existing design practice. A wall in one of the water-filled interior tricells (see Fig. 15) failed, allowing water to rush into a drillshaft (D3 in Fig. 15). The emergency de-ballasting procedure could not keep up with the water flow and hence, the structure sank. As it went deep into the fjord, the buoyancy cells imploded, totally destroying the three hundred million dollar structure.

The wall that failed was 550 mm thick, spanned about 4.4 m and was subjected to water pressure corresponding to about 67 m of water (i.e., 670 \( \text{kN/m}^2 \)). At the elevation where it failed the wall did not contain any stirrups (see Fig. 16). In addition to the high shears and high moments caused by the wall acting as a beam spanning across the opening, the wall was also subjected to high axial compression resulting from the inward push of the water pressure on the exterior walls. The wall did not contain stirrups because the finite element analysis seriously underestimated the magnitude of the applied shear, while the sectional design procedure overestimated the shear strength of the wall.
When a section is subjected to a loading in which both the shear and the axial compression are increased at the same time, the predicted failure load is very sensitive to the assumed interaction between axial load and shear force. This is illustrated in Fig. 17, which compares the predictions of two sectional models, one based on the ACI Code (ACI 318-1989) and the other calculated by program SHELL474, which is based on the CSA offshore code. For the loading ratio shown, the ACI procedure results in a predicted failure load about three times higher than the CSA procedure.

In the weeks following the Sleipner accident, use was made of a number of the modified compression field theory models to study the cause of failure. For example, Fig. 18 shows the results of a number of studies using the three-dimensional, non-linear finite element program SPARCS [16]. These studies were aimed at determining how the failure pressure and failure mode changed as the length of the T-headed bar across the throat of the tricell changed. The studies indicated that failure would have been avoided if either the T-headed bar had been longer or if stirrups had been provided.

CONCLUDING REMARKS

This paper has reviewed the use of shear models based on the modified compression field theory. In particular, a simple, unified shear design procedure has been presented. In this procedure, the angle of inclination of the principal compressive stresses, $\theta$, and the ability of the cracked concrete to resist tension, $\beta$, are both related to a longitudinal strain parameter, $\varepsilon_x$, which accounts for the influence of moment, axial load, prestressing and longitudinal reinforcement ratios. In addition, for members without stirrups, $\beta$ and $\theta$ are strongly dependent on the maximum distance between the longitudinal reinforcing bars, for it is assumed that this distance will govern the crack spacing.

The August 1955 airforce warehouse failures [17] demonstrated the deficiencies of the shear design procedures of that time, which neglected the influence of both axial load and member size. It was perhaps unfortunate that the investigations that followed concentrated on the detrimental influence of axial tension and continued to neglect the detrimental influence of large size. As a result, the shear provisions developed in the 1960's gave a large penalty to members with axial tension and a large bonus to members with axial compression. The failure of the Sleipner platform in August 1991 demonstrated that members designed by such provisions could be unsafe.
REFERENCES


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Lateral Force Transfer in Buildings

Section

Longitudinal Stresses

Shear Stresses

TRADITIONAL TESTS

Section

Longitudinal Strains

TRADITIONAL TESTS

Loading

Longitudinal Strains

Shear Stresses

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Fig. 1—Testing reinforced concrete in shear

Fig. 2—Equilibrium conditions for cracked reinforced concrete

Fig. 3—Compatibility conditions for cracked reinforced concrete
Fig. 4—Stress-strain relationships for cracked concrete

(a) Softening of compressive stress-strain curve due to transverse tensile strain

(b) Average tensile stresses in cracked concrete as a function of $\varepsilon_1$

Fig. 5—Maximum concrete compressive stress as a function of principal tensile strain
Fig. 6—Beam subjected to shear, moment, and axial load

Fig. 7—Values of $\theta$ and $\beta$ for members with stirrups
Fig. 8—Values of $\theta$ and $\beta$ for members without stirrups

Fig. 9—Comparison of observed and predicted shear capacity for strut-and-tie versus sectional model
Beam tested by Bresler and Scordelis

Finite element mesh

30 Nodes: 180 D.o.F.
23 Reinforced concrete elements with a total of 480 integration points
12 Steel elements with a total of 72 integration points
Selfweight included

Fig. 10—Comparison of calculated and observed response of reinforced concrete beam
Fig. 11—Comparison of predicted and observed failure shears of four large high-strength concrete beams

Fig. 12—Diagonal cracking of large high-strength concrete beam without intermediate longitudinal bars ($V = 184 \text{ kN}$)
Fig. 13—Diagonal cracking of large high-strength concrete beam with intermediate longitudinal bars ($V = 343$ kN)

Fig. 14—Determining sectional forces in offshore concrete platform using linear elastic finite element analysis
Fig. 15—Section through the buoyancy cells of Sleipner A platform

Fig. 16—Cross section of drill shaft D3 of Sleipner A platform
Compression failure of concrete under combined moment and axial load

\[ f_y = 500 \text{ MPa} \]
\[ f_c' = 60 \text{ MPa} \]
\[ M/V = 0.6 \text{ m} \]

Fig. 17—Shear force-axial load interaction diagrams for wall section

Deflections x30

\[ f_c' = 60 \text{ MPa} \]
\[ f_y = 550 \text{ MPa} \]

Fig. 18—Influence of stirrups and length of T-headed bar on failure pressure and failure mode of Sleipner A platform
Non-Hooped Reinforced Concrete Columns and Beam-to-Column Connections Laterally Confined in Bellows Square Steel Tube

by M. Tomii

Synopsis: The method of transversely reinforcing columns and beam-to-column connections with bellows square steel tubes was devised to develop a construction method necessary to realize reinforced concrete frame high-rise buildings which are easy to design and execute in zones where high earthquake resisting performance is required.

To secure a ductile seismic behavior for columns subjected to heavy load, strong shear reinforcement and transverse reinforcement are necessary so as to prevent brittle failure such as shear failure, bond split failure along the longitudinal bars, and failure of the compressed extreme fiber of concrete, or to change it into ductile failure.

It was manifested by concentric compression tests of 1/4 scale columns, combined compression, bending and shear tests of 1/3 scale columns, seismic load tests of 1/3 scale and 1/4 scale beam-column subassemblages and bond tests of main bars embedded in 1/4 scale columns that no dangerous collapse of the building is likely to occur even if shear forces of some of the columns and/or beam-to-column connections in the same story reach the loading capacity, because the mechanical behavior of the columns and beam-to-column connections is very ductile even when the webs of their tube yield in shear.

Field execution tests of this structure have been conducted (Fig.1).

Keywords: beams (supports); columns (supports); earthquake-resistant structures; reinforced concrete; shear properties; steel tubes
First of all I would like to pay tribute to Professor Paulay, who has contributed to seismic design of reinforced concrete through obtaining various experimental results and established the foundation of modern seismic design.

I have been acquainted with Professor Paulay for nearly twenty years and throughout the years I have learned a great deal from him: a lot of useful information and ideas of research and education.

In 1977 when the Sixth World Conference on Earthquake Engineering was held in New Delhi, I visited him in his room in the hotel. I clearly remember the hot discussion we had for over an hour in his room.

When I argued that a framed shear wall of which the frame members did not fail in shear but the concrete of the wall panel failed in slip was a ductile shear wall, Professor Paulay tried very hard to persuade me that ductility of reinforced concrete cannot be obtained unless the reinforcement yields in tension. During the discussion, I said "but" so many times that he nicknamed me "But-Man." 'But' afterwards I realized that, though the wall panel is confined in the plane by the frame, it is not confined out of the plane, and that it is not ductile, as professor Paulay said.

I have successively researched and developed truly ductile structural members laterally confined by steel plates in two directions, such as columns laterally confined by undulated steel tubes or bellows square steel tubes, and wall panels laterally confined by corrugated steel plates on both sides and by vertical steel partitions. In these studies I owe a great deal to the discussion with Professor Paulay.

I am hoping Professor Paulay will continue to be ductile both in his mind and body, and not become brittle, even after he is seventy years old.

Masahide Tomii
Masahide Tomii is Professor Emeritus of Kyushu University, Japan, and a consulting advisor of Aoki Corporation. He is member of ACI, ASCE, IABSE, JCI and AIJ, and is recipient of AIJ's 1971 Research Award. He has mainly studied on shear walls, C.F.T. columns.

INTENTION OF USING BELLOWS SQUARE STEEL TUBES FOR LATERAL CONFINING AND SHEAR REINFORCING OF COLUMNS AND BEAM-TO-COLUMN CONNECTIONS

During an earthquake, even if one column cannot sustain the vertical load and fails or one beam-to-column connection cannot transfer the stresses at ends of the joining beams and columns, the building collapses partially. This is dangerous.

When a reinforced concrete frame structure of high-rise buildings (30 stories considered) is to be built in a strong earthquake zone (a strong earthquake zone in Japan considered), the lower columns and beam-to-column connections are required to sustain a heavy load and at the same time have strong, stiff and sufficiently ductile earthquake resisting performance. To satisfy these requirements, the sectional shape of the column must be small in depth and width, large in sectional area and geometrical moment of inertia, and suited for arranging a large number of main bars to go through the mesh of the main bars of the joined beams at the beam-to-column connection: a square section satisfies all those conditions.

Columns and beam-to-column connections can display strong, stiff, and ductile earthquake resisting performance, if the high-strength concrete (specified design strength = 52.9MPa) in the square section and the high-strength big steel bars (thread steel bars of which the yield point is 390 <= 510MPa, and the largest diameter is 41mm) densely arranged therein (the largest main reinforcement ratio about 10%) resist monolithically to the ultimate state.

Taking these into consideration, the author devised a high-strength bellows square steel tube which can produce a far higher performance of lateral confinement and shear reinforcement than hoops or an ordinary square steel tube without undulation (Figs.2,3) and has advantages in structural design and execution of work. The bellows square steel tube is to be used in place of hoops on the columns and beam-to-column connections and as permanent forms of concrete (Fig.4).
SHAPE AND MANUFACTURING METHOD OF UNIQUE BELLOWS SQUARE STEEL TUBES

To decide the shape of the bellows and the sectional shape of the steel tube, four types of bellows square steel tubes of which the scale of the section was about 1/4 of the actual width and the tube length was twice the average tube width were made by 'built-up welding method' described later on and filled with concrete. With the columns thus made, compression tests were conducted. The four types were equal in wall thickness, length of the bellows amplitude, and wave length which was twice the amplitude, but two of them had semicircular wave shape and the other two 45-degree-folded wave shape. In one of either wave shape the phases of undulation of web and flange were half cycle shifted from each other and in the other the phases of undulation were the same. The ratio r/D of the radius of the semicircle of the bellows r (corresponding to 1/2 of the amplitude) to the average width of the tube D was decided as 0.065 so that r in the steel tube in actual size would be almost equal to the thickness of the cover concrete of the main bars in an ordinary reinforced concrete column with hoops. The test results showed that there was scarcely any difference of the effect of laterally confining the concrete in the four types of steel tubes.

Based on these results the author has decided to adopt the semicircular wave shape for the bellows so that high-strength thin steel plates (JIS SM570, yielding point: over 460MPa, tensile strength: 570 - 720MPa) which are 4.5, 6.0, or 9.0mm thick can be easily bent to form the shape. As for the phase of undulation of web and flange, he has adopted the half cycle shifted one so that the reduction of strength and rigidity caused by partial loss of sectional area will be the least. Bellows square steel tubes of this shape are almost constant in the inside sectional area in the axial direction and in the perimeter. So the bellows tubes can be formed simply by bending steel plates without expanding and contracting them (Fig.4).

One method of manufacturing bellows square steel tubes is to build up four corrugated steel plates into a tube and weld the four corners: 'built-up welding method.' In this method, however, since the welding line is a curve, it is necessary to gradually change the welding styles from fringe welding to edge welding and vice versa, which makes groove cutting and welding work very difficult. Besides, the total welding length of the four corners of the bellows is \(1.91 \times 4 = 7.64\) times the length of the bellows tube.
So quality control will not be easy and manufacturing cost will be very high (Fig.3).

A second method is to manufacture a steel tube the outline of whose section is in a shape of a petal and whose length and perimeter are the same as the undulated wall length and perimeter of the intended bellows square steel tube. This is the first forming. Then the petal shaped tube is squarely surrounded by semicircular metal molds to form concave parts at an interval of the length of the tube wall in a unit wave. With hydraulic pressure put inside the tube, the length of the tube and the interval of the molds are contracted so as to form the tube wall in a square bellows: 'hydraulic pressure forming method.' The author's team has been developing this method and succeeded in manufacturing trial products.

It has become clear that though the plant cost of 'hydraulic pressure forming method' is higher than that of 'built-up welding method,' manufacturing cost is lower, because there is no welding joint on the corners of the tube and quality control is easier (Fig 3).

For the above reasons, it is planned to mass produce standard size bellows square steel tubes by the hydraulic pressure forming method and mass production techniques are being developed. Nonstandardized tubes are to be made by the built-up welding method. To release residual strain of forming and welding, in the hydraulic pressure forming method the tubes are annealed after the first forming and the final forming, and in the built-up welding method after the welding is finished.

WHY BELLows SQUARE STEEL TUBES CAN REMARKABLY IMPROVE THE EARTHQuAKE RESISTING PERFORMANCE OF THE COLUMNS AND BEAm-TO-COLUMN CONNECTIONS

When an ordinary square steel tube without undulation is filled with concrete and compressed, since the apparent Poisson's ratio of the elastic steel tube 0.3 is larger than the Poisson's ratio of the infilled concrete 1/6, it is not until the infilled concrete becomes plastic and its Poisson's ratio approaches 0.5, the value at the time of compressive failure, that the infilled concrete sticks to the steel tube and lateral confinement effect is produced. However, the strain of concrete 0.003 - 0.005 is larger than the yield strain of the steel tube 0.0015 - 0.003. Therefore, when Navier hypothesis can be assumed with regard to the infilled concrete and steel tube, the increment of the compressive load capacity of the infilled concrete caused by the confining effect of the tube is as small as to be neglected in structural design.
In the case of a bellows square steel tube, however, the diameter hardly changes even when it is contracted in the axial direction and the apparent Poisson's ratio is considered nearly zero. Therefore, when it is filled with concrete and compressed, lateral confinement effect is produced at the first stage of loading when the infilled concrete is behaving elastically (Fig. 5). Besides, since the axial rigidity of a bellows square steel tube is extremely small, the in-plane stress of the tube wall is mostly the tensile stress in the ridge direction which contributes to lateral confinement. As a result, like high-strength hoops whose yield strength is below 500MPa, a bellows square steel tube is expected to produce a strong lateral confinement stress corresponding to the yield strength of the steel tube. Moreover, since, compared with hoops or the square steel tube without undulation, a bellows square steel tube has extremely large flexural rigidity with regard to the bulge, it has an excellent lateral confinement effect (Fig. 2). To increase the lateral confinement effect and shear reinforcing effect it is necessary to increase the thickness of the tube wall and/or the amplitude of the bellows, but it does not spoil the concrete filling performance or decrease the reinforcing effect. This is one of the reasons why a bellows square steel tube can improve earthquake resisting performance more remarkably than hoops.

With its strong lateral reinforcement, the bellows square steel tube can increase the compressive loading capacity of the infilled concrete by more than 50% and prevent the buckling of the large amount of main reinforcement whose main reinforcement ratio is about 10%, thus increasing the compressive strength remarkably (Fig. 5). As a result, a ductile flexural and shear behavior, like that in yielding, can be expected even under a heavy compressive load.

Given a behavior of an earthquake, a reinforced concrete column laterally confined, not by hoops, but in a bellows square steel tube, in which the tube and the infilled concrete mesh with each other, can transfer at the flange such a strong vertical thrust as to cause a slip failure to the meshed part of the concrete by punching shear, and unless the slip failure occurs, the tube and the infilled concrete behave monolithically (Figs. 6, 7). It is proved by bending-shear tests that for this reason, when the top and bottom of the column are reinforced so as not to yield in flexure, the shear strain of the tube wall at the web surpasses the yield strain and the column displays ductile behavior like yielding in shear (Fig. 8). It was observed by cutting columns
along the main reinforcement after the tests that the excellent lateral confinement of the bellows square steel tube prevented the bond splitting cracks of the concrete along the main reinforcement from developing (Fig.6). The same effect was observed in the shear tests of beam-column subassemblages, about the connections reinforced with bellows square steel tubes instead of hoops (Figs.9,10,11).

The results of these tests show that bellows square steel tubes not only increase the ultimate compressive strength of concrete in the reinforced concrete columns and beam-to-column connections by confining effect but produce excellent shear reinforcing effect as well.

For these reasons, it is considered that even if flexural yield or shear yield of some of the columns of a story is allowed as failure mechanism of a frame structure at the time of an earthquake, local collapse of the structure will not be induced.

When the story deformation angle of a frame structure subjected to lateral load is below the allowable value $1/100 - 1/80$ rad, bellows square steel tube reinforced concrete columns that yield in flexure or in shear are, according to a trial design, limited to those columns to which spandrel walls and/or hanging walls are attached and which behave as stub columns. However, it is possible that the edge columns of an earthquake resisting framed wall may yield in shear or in compression and the intermediate columns of a continuous earthquake resisting framed wall may yield in shear. As a wall panel of which the elongation, contraction and shear behaviors can follow those of the reinforced concrete columns laterally confined by bellows square steel tubes, the author has devised to put two corrugated steel plates facing with each other with the ridges in the horizontal direction at an interval of the wall thickness, insert steel partitions into the vertical slits made through the inside corrugation at a given interval, fix the corrugated plates and the partitions from the outside by fillet weld so as to make them into a monolithic wall reinforcement as well as a wall form, and fill it with concrete (Fig.12). The vertical steel partition serves as a spacer to keep the wall thickness constant and a stiffener to increase the confining effect on the infilled concrete wall panel, as well as a vertical reinforcing member, since the corrugated steel plates have small elongation and contraction rigidity in the direction orthogonal to the ridge line. Since the corrugated steel plates are welded together with the vertical steel partitions, they prevent the outward collapse of the concrete, carry part of the shear force of the
wall panel and restrain the enlargement of the diagonally cracked infilled concrete wall. As a result, wall bars which impede concrete filling become utterly unnecessary. It is made clear by estimation tests of structural performance of 1/4 scale wall panels that the infilled concrete wall panel thus made is prevented by the lateral confining effect of the corrugated steel panels from bulging outside and behaves as required (Fig.12).

In apartment houses and hotels there are many spandrel walls, hanging walls, and partitions. If the story deformation angle due to lateral load can be reduced by utilizing those walls and partitions as structural members, and, depending on the conditions, by adopting seismic isolation system and vibration control system, it is possible to design slender columns that do not fail in bending-compression when sustaining a heavy vertical load.

Bellows square steel tubes are strong lateral confining and shear reinforcing members in which axial rigidity can be neglected. Taking this fact into consideration, it is possible to manufacture precast members with extremely strong and ductile bending-shear behavior, by using prestressed bars instead of main bars and giving prestress to infilled concrete.

BEAM-TO-COLUMN CONNECTIONS LATERALLY CONFINED AND REINFORCED IN SHEAR BY BELLOWS SQUARE STEEL TUBES NOT UNDULATED AT THE TOP AND BOTTOM PARTS

The top and bottom parts of a beam-to-column connection where the main beam bars of the joining beams cross orthogonally have flat tube walls which are suited to make holes for the bars, and by confining the bulging out of the cover-plated tube walls with nuts fixed to the threaded steel bars and using a bellows steel tube in between, the connection is laterally confined and shear reinforced (Fig.11).

This square steel tube with flat parts on both sides of the bellows is made of high-strength thin steel plate of the same material as that of the bellows square steel tube of the column joined to the connection. The flat parts and undulated part of the tube wall of each face of the tube were formed monolithically by pressing, and the tube was manufactured by the built-up welding method.

The transfer of shear force between the column and the beam-to-column connection relies on cotter action of the end of the column buried 5cm deep into the concrete of the connection, shearing resistance of the concrete placing joint between the column and the connection, and dowell action of the main bars of the column which go through the placing joint.
The transfer of shear force between the beam and the connection relies on cotter action of the end of the beam meshing with the undulated tube wall of the connection, frictional resistance of the flat tube wall and the concrete of the beam, and dowell action of the main bars of the beam crossing the joint.

Slip of the main bars of the beam crossing the connection is prevented by hook action of the cover-plated tube walls fixed to the end of the crossing main bars with nuts put to the thread steel bars (Fig.11). However, prevention of slip of the main bars of the column crossing the connection relies only on the splitting resistance between the concrete whose splitting is controlled by the lateral confinement of the steel tube of the connection and the main bars of the column. So if the steel tube of the connection yields by hoop tension and shear, lateral confining stress does not increase and shear strength at the connection may be dominated not by the shear failure of the concrete but by the slip of the main bars of the column (Fig.10).

At the connection, as a rule, concrete of the same strength as that of the column joined there is to be placed.

The width of the flat tube that confines the top and bottom part of the connection is 6cm larger than that of the bellows square steel tube of the column so as to fill the hollow space between the two tubes with 3-cm-wide concrete and secure the transfer of compressive stress from the concrete of the beam to the connection as well as to get rid of the play of the cotter action of the end of the column which contributes to the transfer of shear force from the column to the connections.

OTHER ADVANTAGES OF BELLOWS SQUARE STEEL TUBES THAN LATERAL CONFINEMENT AND SHEAR REINFORCEMENT

Bellows square steel tubes at beam-to-column connections work as forms of concrete at the connections. Therefore it becomes possible to place lower-strength concrete into the beam, which can generally be designed to use lower-strength concrete, than that placed to the connection, and cost reduction can be expected.

Bellows square steel tubes as forms have several advantages. Since they are manufactured in the factory, there is little deviation in quality, shape and dimension. They are water tight. Since they have high thermal conductivity and large surface area, if temperature rise by the radiant heat from the sun is suppressed by shading the sunlight and water is sprinkled to evaporate and taking off the
heat caused by the curing of high-strength concrete, it is possible to prevent excessive temperature rise.

Since the bellows square steel tube of the column do not carry the axial force and bending moment, the joint of the steel tube is to be placed at the end of the column as a lap joint which is buried about 5cm deep to reach the main bars of the beam in the center of the section at the top or bottom of the beam-to-column connection laterally confined by a square steel tube which is 6cm wider than the end of the tube of the column (Fig.11). This method needs neither welding nor high-tension bolt joint and the execution of the work is very easy. It is proved by seismic behavior tests of beam-column subassemblages and bending-shear tests of columns that the lap joint can sufficiently transfer the shear force of the column to the connection.

Since there are no hoops that impede the placing of concrete when bars are densely arranged, it is easy to place dense concrete even when a large number of main bars are arranged in the column.

Moreover, bellows square steel tubes used in place of hoops prevent the cover concrete from segregating from the bars and causing an earthquake disaster, and immediately play the role of reinforcement when the infilled concrete is damaged by the earthquake.

Since bellows square steel tubes do not carry vertical load or bending moment, design stress of the reinforced concrete column inside the tube due to permanent load does not change even when the tube suffers fire damage. Therefore, if it is not necessary to assume that the frame structure be subjected to large lateral force during a fire, fire resistive covering is considered unnecessary. However, when the tensile yield strength of the steel tube damaged by fire has become smaller than the specified tensile yield strength and the horizontal load-carrying capacity has become less than the required horizontal load-carrying capacity, it is necessary to make the reinforcement strong enough for the building to be used.

CONCLUSIONS

The bellows square steel tube has been developed as a strong lateral reinforcing member to prevent a dangerous brittle failure in reinforced concrete frame structure buildings, even if the shear force of some of the columns and beam-to-column connections of the same story reach the shear loading capacity of the members. Through compression tests and compression-bending-shear tests of columns, bond tests of main bars, and lateral loading tests of beam-column
subassemblages, the bellows square steel tube is proved to serve the purpose perfectly.

The story shear of frame structures with the strong and ductile columns and beam-to-column connections, where the compressive force of the columns is restricted within two thirds of the concentric load capacity, does not reach the shear capacity unless the seismic story deformation angle exceeds 0.02 rad. When the angle exceeds 0.01 rad, the effect of columns (Fig. 7) cannot be neglected in the seismic response analysis of the frame, and the specifications of nonstructural members and equipments must have the ductility to follow the large story deformation angle in earthquakes. If using spandrel walls, hanging walls, and partition walls as structural members, and/or adopting structural control and intelligent systems enables the maximum story deformation angles of the frame structure to remain less than 0.01 rad during any strong earthquake, not only will the designing become simple, but also the columns can safely sustain heavier load and more slender columns or higher frame structures can be designed. Then the merit of the large concentric load capacity of the columns can play a more important role in design.

The future problems are to design a structure with a smaller story deformation angle at the time of an earthquake and develop a mass production technique of bellows square steel tubes.

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This paper is based on the achievements of the technological development of reinforced concrete 30-story high-rise apartment houses in which the Structural Engineering Laboratory of the Technical Research Center of Aoki Corporation has been engaged. The contribution of all the members of the Laboratory is deeply appreciated.

REFERENCE

Fig. 1—Field execution tests of 2nd and 25th stories of 30-story reinforced concrete frame structure built by the method in this paper.

Fig. 2—Comparison of lateral reinforcing effects of circular, square, and bellows square steel tubes.
Fig. 3—Steel tube with full size section in 2-cycle wave length and formed with hydraulic pressure, and the one in 1/4 full length and built up with welding.

Fig. 4—Hanging down a steel tube to be set around arranged main bars of nonhooped column.
339 mm

\[ N = c \sigma_a \cdot D_{min}^2 + 3.4 \sigma_a \cdot D_{min}^2 \]

\[ \bar{\sigma}_r = 0.81 \pi (t/D) \sigma_y \]

\[ \bar{\sigma}_a = \text{average confining stress} \]

Fig. 5—Compression test results of columns
Fig. 6—Punching shear failure of meshed parts and bond splitting cracks along main bars of concrete in vertical cut section of column

\[ M = \rho_c M_1 + \rho_c M_2 + \rho_c M_3 + \rho_c M_4 \]

\[ V = \rho_c V_1 + \rho_c V_2 + s V \]

confinement effect

shear reinforcement effect

Fig. 7—Transfer mechanism of seismic stresses between reinforced concrete column and jacket of steel tube
Thickness of tube wall = 3.2mm

Diameter of main bars = 16mm

All dimensions in mm

Fig. 8—Bending-shear test results of compressed columns
Mechanism line when pins are assumed at a $D_{\text{min}}$ beyond top and bottom of the column, where $D_{\text{min}}$ is minimum depth of the column.

The point where both webs of the square steel tube yield by shear.

Fig. 8 cont.—Bending-shear test results of compressed columns.
 webs of the square steel tube of the connection yield by shear

- Mechanism line dominated by flexural strength of both joints of the beams
- The point where both webs of the square steel tube of the connection yield by shear

Fig. 9—Estimation test results of earthquake resisting performance of beam-column
Lateral Force Transfer in Buildings

\[ V_a = 1.84 V_c \]

\[ V_c \]

\[ N_c = -43.7 \sim 151.0 \text{tonf} \]

\[ -428.6 \sim 1480.8 \text{kN} \]

All dimensions in mm

Fig. 9 cont.—Estimation test results of earthquake resisting performance of beam-column subassemblages
Fig. 10—Bond splitting cracks in concrete at vertical cut section of beam-to-column connection along vertically penetrating main bars

Fig. 11—Steel tube laterally confining beam-to-column connection
Fig. 12—Shear test result of concrete wall panel laterally confined in pair of corrugated steel plates.
Seismic Retrofit of Beam-to-Column Joints with Grouted Steel Tubes

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Synopsis:
This paper presents the findings of an experimental study to evaluate a method of retrofit which addresses a particular weakness that is often found in reinforced concrete structures, especially older structures, namely the lack of sufficient reinforcement in and around beam-to-column joints. Many of these structures lack the required confining reinforcement within the joints and in adjoining beams and columns. The result is a reinforced concrete frame that is weak in the joint areas and lacks sufficient ductility during a seismic event.

The proposed retrofit method consists of encasing the reinforced concrete joint with a grouted steel jacket that provides confinement to the joint area, and imparts ductility to the frame. In this study, two styles of retrofit jacket were tested: a circular steel tube and a rectangular casing. The circular steel jacket provided direct confinement as well as a ductile force transfer mechanism through the jacket itself, but it was more difficult and expensive to fabricate than the rectangular casing. Although the rectangular jacket did not provide the same amount of concrete core confinement, it seemed to be sufficient to prevent damage in the joint area. The load transfer mechanism of the rectangular jacket was found to be adequate in withstanding the loads and deflections typical for seismic events. In the paper, the two jacket styles are evaluated for strength, stiffness and ductility, and their relative merits are discussed.

Keywords: beam-column joints; composite materials; earthquake-resistant structures; frames; grout; hysteresis; joints (junctions); reinforced concrete; shear properties; steel tubes
APPRECIATION

I cannot remember the circumstances and time when I first met Tom Paulay, but no one forgets the infectious enthusiasm, energy, warmth, sense of humor and dedication that are a part of whatever he does, and that help to define the superb lecturer that he is. The impact of his beautiful, multi-colored presentations, the leading-edge research that they describe, the clarity with which they are delivered, and even the unique "Hungrokiwian" accent in which they are offered, are all stamps of an authority that is grounded in a distinguished academic career.

Tom's important and meaningful work on the earthquake resistant design of reinforced concrete structures has earned him a solid international reputation, and a place in the history of the subject - many of his books and papers have become "classics" in their field. His ability to define and solve important problems facing earthquake engineers, and his concern with the transfer of technology into practice, have had a profound influence on improving the safety and economy of engineering works in the seismically active regions of the world.

Committed to public and professional service, Tom Paulay has been called on to fill many prestigious appointments at both the national and international level, an indication of the high esteem in which he is held by the world community of earthquake engineering scholars and practitioners. He has been instrumental in establishing a first-rate research laboratory at the University of Canterbury where, as a devoted member of the faculty, he has trained students and welcomed distinguished scholars from many parts of the world. Although he has been honoured in many different ways and countries for his contributions and ability, and his influence on the next generation of earthquake engineers is already confirmed, I suspect that he is touched most deeply by the respect, admiration and affection in which he is held by former students and colleagues-at-large.

I am happy and privileged to be counted as one of Tom's friends and admirers and to be a part of this event recognizing his person and achievements and celebrating his youth at seventy. And I rejoice also in the delightful memories of a friendship, initially nurtured in Vancouver, and maintained undiminished over distant miles and many years.

As a young man, Tom Paulay was an officer in the Hungarian cavalry. He has been galloping with undiminished enthusiasm and vigor ever since. I expect his ride into the next decade and century will continue to be characterized by the same involvement, service and dedication that have marked his entire career. May the journey be a pleasant one.

She/Cherry
INTRODUCTION

The 1971 San Fernando, California, earthquake led to significant changes in the practice of seismic design, particularly in the high risk seismic zones of North America. Many reinforced concrete structures that conformed to design and construction standards at that time behaved poorly, prompting many modifications to the codes of the American Concrete Institute (1) and Canadian Standards Association (3).

Code modifications (2,4) that were implemented after the San Fernando earthquake specifically addressed the ductility requirements of reinforced concrete members and, in particular, the joint region between beams and columns. This region is subject to large shear forces during lateral seismic loading, particularly when beam moments on opposite faces of a column have the same orientation. The longitudinal reinforcement is stressed in the same direction in this situation, and the bond between the reinforcement and the concrete is heavily relied upon to provide the required transfer of forces through the joint (Fig. 1). Under severe loading, plastic hinges are expected to form at the ends of the beams adjacent to the joint, and transverse reinforcement in both the beam and the column are required to provide confinement to the concrete in the core region, thereby safeguarding the ductility of the joint. Pre-1971 design codes were modified to address these concerns. Most of the changes are reflected in the recommendations of ACI-ASCE Committee 352 for the design of connections in reinforced concrete frames with ductile moment-resisting capacity (2).
While updated design codes address the construction of new structures, older structures that were built according to earlier design codes may not meet today’s seismic standards. Many are inadequate, and pose a severe risk to society. What can be done about them? One available option is to retrofit such structures; that is, to modify them to assure compliance with current design provisions.

OBJECT AND SCOPE OF INVESTIGATION

The purpose of this study was to investigate effective methods of retrofit that can be applied to strengthen the beam-to-column joint region in a reinforced concrete frame. Deficient (or damaged) reinforced concrete frame sub-assemblies were retrofitted by encasing the joints with a steel jacket and filling the void with concrete grout (Fig. 2). This is an elegant and simple solution; the steel provides both lateral confinement and shear reinforcement, thereby adding strength and ductility to the joint. This approach can be applied to undamaged deficient structures and, when appropriate, to frames that have been damaged by an earthquake.

The effectiveness of introducing a steel jacket around the beam-to-column connection was examined by performing six cyclic loading tests on a total of four specimens built according to typical 1960’s design specifications (Fig. 3). The testing program included two preliminary tests on unretrofitted specimens, causing some initial damage and intended as a likely scenario after an earthquake. The last four tests were performed on the two damaged and the two undamaged specimens after they were retrofitted with a confining steel jacket. Two jacket types were tested: two specimens were retrofitted with a circular jacket, two with a rectangular jacket. Each jacket type was tested with one damaged and one undamaged specimen, to compare the effect of the initial damage on the behaviour of the retrofit. As the testing programme progressed, it became evident that a failure of the retrofitted region was virtually impossible under standard load conditions. Although the focus of the study therefore changed as testing progressed, the initial objectives are stated here to explain the choice of test specimens.

The study allows conclusions to be drawn concerning the use of circular or rectangular tubes. The relative effectiveness of the jacket type on the behaviour of the joint region is compared with existing studies that focus on the effectiveness of jacket geometry (5)(6) on the behaviour of plastic hinge regions of columns alone.

The specifics of the steel jacketing used in each retrofitting case conformed to the current Canadian Design Code (4). Some of the parameters considered included: the outside dimensions of the steel section, the diameter to
thickness ratio (or width to thickness ratio) of the steel section, the use of grout or concrete, the appropriate length of retrofit (hinge area vs. entire length of section), and the need for steel section continuity through a joint.

Although the tests that were performed reflect only a simple external beam-to-column connection, this research was intended as a preliminary study to determine the viability of incorporating steel jackets at a connection as a retrofit measure.

**PREVIOUS RESEARCH**

The usual method of providing confinement in reinforced concrete columns is to use transverse reinforcing steel in the form of hoops or spirals, depending on the geometry of the column. A number of options can be employed to increase the effectiveness of the confinement, and hence the column strength and ductility levels. Studies have shown the following approaches to be effective in achieving such increases: increasing the transverse steel ratio (7); increasing the yield strength of the steel (8); adjusting the longitudinal reinforcement geometry (9); using steel fibre reinforced concrete (10), welded-wire fabric (11), prestressed bolts (12) and, with limited success, increasing the concrete strength (13).

When assessing existing structures, based on present design rules, it is commonly found that earlier designs of reinforced concrete columns frequently exhibit poor detailing of transverse reinforcement, either in terms of the use of ties, anchorage of ties and hoops, or simply excessive spacing of transverse reinforcement (14). Other problems include inadequate lapping of longitudinal reinforcement in hinge areas (15) or, in the case of short columns, designs in which the flexural strength exceeds the shear strength, often resulting in brittle failure modes (16). In general, an effective retrofit approach involves an improvement in "caging" to confine the concrete and prevent spalling.

Confinement for concrete columns can also be provided by encasing the concrete core within a steel shell. In new construction this is typically achieved by filling hollow steel sections with concrete. When reinforced concrete columns need to be retrofitted, they can be enclosed in a steel jacket, and the space between the two elements filled with grout. In both cases, the external steel completely encloses the entire concrete core, and effectively confines all the concrete, inclusive of the cover concrete, thus assisting in reducing bond failures in columns. It is preferable that the steel jacket provide confinement only, and not carry any of the axial load. If the steel jacket were to contribute directly to the load carrying capacity of the member, the stiffness characteristics of the structure would be altered, possibly resulting in lower ductility ratios and higher moment capacities at critical sections (17)(18). In reality, however, the structural
separation of steel tube and concrete is difficult to achieve since chemical bond and friction are usually sufficient to assure strain compatibility (19).

Reinforced concrete columns are frequently retrofitted by either partially or fully encasing the member in a steel jacket. Short reinforced concrete columns are particularly susceptible to brittle shear failure and the entire column length is typically encased (16). In columns with higher slenderness ratios, which are typically found in building frames, only the plastic hinge region needs to be confined. Columns which have to support large axial loads need jacketing along the entire length to enhance the compressive strength as well. In this case, the steel jacketing is not expected to actually carry any of the axial load, a function that is left to the added concrete alone.

Comparisons between circular and rectangular jackets have shown that columns retrofitted with circular jackets exhibit greater ductility due to superior confinement, a larger overall strength, and greater bond strength between the concrete and the longitudinal reinforcement as well as the steel jacket (5)(6)(7).

During extreme cyclic loading, the moment and shear forces within a beam-to-column connection could conceivably exceed the load-carrying capacity of the joint region. Since the joint area is considered part of the column, it ideally should remain elastic to avoid failure of the shear panel, and subsequent anchorage failure of the reinforcement (20). Under such conditions, the energy absorption capacity of adjacent plastic hinges is maintained without shear or anchorage failure of the joint core (21). To avoid collapse of the structure, plastic hinges are designed to occur in the beams following the rules of standard practice: strong column/weak beam, strong joint/weak element, strong shear/weak moment (22). Modern design codes address these concepts by specifying a ratio of strengths between beam and column. For instance, to avoid the development of a flexural hinge in the column, the ACI-ASCE Committee 352 on the design of reinforced concrete connections specifies that the ratio of flexural strengths between column and beam should be at least 1.4.

Over the last 20 years, a fair amount of research has been done on the behaviour of beam-to-column connections, which is reflected in the recommendations of ACI-ASCE Committee 352. Before these recommendations, the joint was considered to behave much like a deep beam, and designed accordingly (23). Today, we know that this is not entirely the case. During lateral loading, the moment and shear force transfer between the beam and column can be considered as similar to a truss action, consisting of a concrete compressive strut and a tension tie formed by the transverse reinforcement. Force transfer between these components is activated by bond stress and anchorage. Recent experimental results suggest that the concrete strut transfers most of the force, and hence only enough transverse steel to provide confinement is required within the joint (24).
Joint performance appears to be a function of the joint shear stress and confinement level (25). High joint shear stresses were found to reduce the joint energy absorption capacity and cause a rapid loss of load-carrying capacity. The primary role of transverse joint reinforcement is thus one of confinement (23). Joint deterioration due to high joint shear stress can also be prevented by providing sufficient anchorage of the longitudinal steel (24). Other recommendations for the improvement of joint behaviour are the avoidance of large plastic deformations within the joint, the limitation of concentration of damage to prescribed sections, and the avoidance of brittle failures.

Based on previous research, the use of grouted steel casings in the joint area could provide the ideal solution when existing structures need to be retrofitted. Very little research has been done in this area and it was considered worthwhile to investigate this approach.

**EXPERIMENTAL STUDY**

**Test Specimen Design**

As a basis for the experimental work, a hypothetical two-storey frame for an office building, situated in Vancouver, B.C. was designed in accordance with all the requirements of the 1970 National Building Code of Canada, and the CAN3-A23.3-M66 code (3) for reinforced concrete structures. A beam-to-column joint sub-assembly of this structure was subsequently considered for the detailed retrofit study. Its design was modified slightly by omitting all transverse reinforcement from the joint region. This was common practice at the time due to ambiguities in the interpretation of the 1966 code, which did not explicitly state that the joint region should be considered part of the column, with specific transverse reinforcement requirements. Consequently, transverse column reinforcement was not normally specified for the joint, mainly to avoid congestion of reinforcement which frequently caused voids in the concrete. Often, ties were simply omitted in the construction process because of the extreme difficulty they created in the placing of reinforcement.

The dimensions of the test specimens were primarily dictated by the availability of formwork and laboratory testing capacities, resulting in a beam-to-column joint model of approximately half-scale. Four specimens were fabricated, as shown in Fig.4.
Retrofit Design

Steel jackets were used as a substitute for the lateral reinforcement steel that was missing in the original design. They were designed to provide confinement and additional ductility to the joints, without increasing the moment capacity of the specimens. Based on modern design codes, which require a spacing of 35mm for 10mm ties in both the beams and the columns, the necessary thickness of the jackets in the joint region was 2.86 mm (0.110"). The lengths of the retrofit were made equal to the member depth (d) along the columns and twice the member depth (2*d) along the beam, measured from the beam-to-column interface. A gap of 25mm was left halfway along the length of the beam jacket, to create a flexural hinge at that point (Fig. 5). Because confinement of the joint region was a major concern, it was decided to position the gaps a reasonable distance away from the column face. This later proved to be a non-critical issue.

Preliminary tests (26) have shown that the flexural strength of retrofitted beams can be increased approximately eight-fold. A retrofit that consists of an unbroken tube length of 2*d along the beam would thus force the flexural hinge to the end of the retrofit. This would merely increase the overall strength of the beam, without increasing its ductility; furthermore, it would also increase the risk of a shear failure. The gap in the retrofit beam jacket was therefore introduced to assure that a flexural hinge would occur within the retrofitted area. The joint area would still be confined by the steel tube, adding strength and ductility to the joint region. In spite of the gap, an increase in the overall moment resistance of about one-third was expected due to the added concrete. The diameter and length of the steel jackets was kept to a minimum to reduce the amount of disruption to the structure during the retrofit process, and also to limit the increase in strength of the frame members, which could force a failure elsewhere in the structure.

As testing progressed, changes were made to the retrofit schemes because the strength increases of the retrofit sections exceeded expectations and caused failures in the beam and column outside the retrofit regions. These changes are detailed in a later section.

Experimental Apparatus and Procedure

A testing frame in the Civil Engineering Structures Laboratory at the University of British Columbia was used to apply cyclic displacements to the beam while maintaining a constant axial load in the column (Fig.6). A similar loading pattern was used for both the retrofitted and unretrofitted specimens.
Data from load cells, displacement transducers and strain gauges were recorded by an automatic computer controlled data acquisition system. A set of five displacement transducers (LVDT's) were used to separate and identify the various types of movement that make up the total rotation of the joint and beam; this rotation consists of the rotation caused by elastic deformation of the column, the shear deformation of the joint, and the rotation of the beam assuming a fixed joint (cantilever action). The strain gauges were used in the retrofitted tests to determine the behaviour of the steel jackets during cyclic loading. The gauges were concentrated within the joint area to provide an insight into the load path and stress pattern within the steel jacket.

**Testing Procedure**

First, the axial load was applied to the column, and this load was maintained at a constant level throughout the duration of the test. A moment was then applied to the joint by displacing the end of the beam in an upward (positive) direction. The load was steadily increased until the yield moment was reached. Due to the non-linear response of the specimen, which is a result of the particular stress-strain behaviour of the reinforcing steel, the definition of the yield moment depended somewhat on judgment. A ductility factor of $\theta=1$ was associated with the displacement at which a distinct loss of stiffness was observed. This same displacement was then applied in the opposite (negative) direction. For the unretrofitted specimens the full cycle was repeated a second time, followed by two complete cycles at double amplitude. For the unretrofitted specimens, these four cycles of loading caused sufficient distress in the joint to consider it damaged, but repairable. A greater number of load cycles with stepwise increasing amplitude was imposed on the retrofitted specimens.

**TEST RESULTS**

**Material Properties**

All the reinforcing steel used in the testing program originated from the same batch with a nominal specified strength of 400 MPa. Tests on four tensile coupons produced an average yield strength of 566 MPa; the ultimate strength was approximately 800 MPa. Unfortunately no upper limit is specified for reinforcing steel and cases of excessive overstrength are not uncommon, especially in recycled steel with variable and often unknown alloy content. The stress-strain curves did not feature a marked yield plateau and the stress at 0.2% offset strain was used to define the tensile yield stress (Fig.7). This value thus only provided a nominal yield value for use in design; the actual yield point of
the specimen varied, depending on the amount of deformation. The retrofit jackets were fabricated from hot-rolled plate having a yield stress of 267 MPa and ultimate stress of approximately 370 MPa. The compressive strength of the concrete, determined from six uniaxial tests of standard 30 cm. cylinders, was found to be 26.3 MPa at the time of the unretrofitted tests, and 30.3 MPa when the retrofitted specimens were tested. The average of four tests yielded a compressive strength of 31.3 MPa for the grout.

Unretrofitted Specimens

The tests on the unretrofitted reinforced concrete specimens yielded results that were consistent with previous studies, that is, they disclosed lack of shear strength and ductility in the joint area. The hysteresis loops for specimen RCBC1 (Fig.8) exhibit a rapid degradation in stiffness and strength with progressive cycling, virtually identical curves were obtained for specimen RCBC2. After four cycles of loading the damage within the joint was so extensive that only about 40% of the maximum strength and stiffness remained, and failure of the specimen was declared. The hysteresis loops show the characteristic pinching effect that is prevalent in reinforced concrete members without special detailing.

Extensive damage occurred within the joint region with minor flexural cracking in the beam and column. Severe shear cracks within the joint region opened and closed in synchronization with the cyclic load. After the yield displacement was exceeded during negative loading, the shear cracks within the joint no longer closed completely. A large amount of the concrete cover spalled off the column and concrete crushing took place within the hinge area at the interface of the column and the beam.

The failure mechanism within the joint consisted of two distinct parts. Since the longitudinal reinforcement in the beam was made up of 4 bars on top, and two bars in the bottom layer, the response was different for positive (upward) and negative (downward) loading. During positive loading the bottom reinforcement yielded and a plastic hinge formed in the beam near the interface of the beam and column. During the negative part of the cycle, the top layer of steel did not yield and the maximum moment was limited by a bond failure of the longitudinal reinforcement bars in the joint area. This was particularly evident on the rear face of the joint along the column, where the hooked ends of the longitudinal reinforcement moved outward to split the cover. The beam thus effectively pivoted about a point near the lower reinforcement layer.

The experimental strength values for positive bending conformed well with predicted strengths based on standard section analysis of a reinforced concrete specimen. Joint strength during negative loading was reduced because
of bond failure and damage incurred during positive loading. Test results are tabulated in Table 1. The moment $M_\alpha$ is derived from a section analysis at the particular location listed, corresponding to the 2% offset stress in the reinforcement. This moment is then scaled linearly to the centre of the joint ($M_c$). The experimental moment ($M_y$) also corresponds to "first yield" and is calculated with respect to the centre of the joint.

Considering the shortcomings in the design of the older buildings under study, and hence in their representative test specimens, the outcome of the tests was as expected. The lack of detailing for ductility in and around the beam-to-column joint resulted in poor behaviour during cyclic loading, which was characterized by the development of a low rotational ductility of $\theta=2$, accompanied by severe loss of strength and stiffness. This could have serious consequences during a seismic event, particularly in light of the poor ductility of the structure. Ideally, a plastic hinge should form in the beam during both negative and positive loading, thereby avoiding damage to the joint. Although the calculated shear strength ($\tau$) for the beam-to-column joints ($\pm 185$ kN) exceeds the applied shear ($\pm 135$ kN), the lack of transverse reinforcement negates any reasonable comparison.

**Retrofitted Specimens**

A total of four retrofitted specimens was tested. Each had its own peculiarities and will be discussed individually. Although some of the original specimens had been damaged previously, this did not affect the behaviour of the retrofitted specimens. Nevertheless, the designation "undamaged" or "previously damaged" is used to distinguish the different tests. The test results are presented in Table 1, where $M_\alpha$, $M_c$ and $M_y$ are defined as before. For the sake of uniformity, the hysteresis curves are plotted for the moment at the centre of the joint ($M = \text{vertical load} \times 914 \text{ mm}$) against the average rotation of the beam with respect to a horizontal plane (rotation = $1/d \times \text{vertical displacement at the measuring point}$, where $d$ is shown in Fig.5).

**RETRO-SU: Undamaged Specimen with Square Jacket** — This specimen developed two modes of failure, depending on the direction of the loading. With positive (upward) loading, a flexural hinge formed at the gap between the two sections of steel jacket in the beam. During negative (downward) loading, a flexural hinge formed in the reinforced concrete, just outside the retrofit area. With repeated cycling this latter flexural hinge developed into a shear failure resulting in a drastic drop in load-carrying capacity in both directions. The flexural hinge in the gap area thus ceased to undergo further yielding. Under positive loading, yielding occurred in the retrofit gap at an applied moment ($M_y$) of 27.4 kNm (Table 1), which is as predicted. Under negative loading, yielding occurred outside the retrofit area at the end of the
beam, at an applied moment of 42.7 kNm. If a reduced development length is taken into account for the two top bars at the gap, the predicted failure location should have been at the gap. Considering the added confinement due to the steel casing, however, the actual development length would suggest a higher moment resistance and the failure at the end of the retrofit was to be expected.

The hysteresis curves of the joint (Fig.9) show the progressive development of the plastic hinge. Since hinging during negative loading was outside the gap, the top layer of steel in the gap never reached yield stress, and hence little or no plastic action was observed for this portion of the loading cycle. Not much strength loss was recorded during the progression of the experiment, although the stiffness deteriorated significantly. Some pinching occurred during both positive and negative loading; it was much less, however, than that observed in the unretrofitted specimens. The deformations of the joint area consisted mainly of the beam cantilever displacement, while the shear deformation within the joint area was negligible.

During testing, breakage of the bond between the steel tube and the grout was accompanied by a number of "popping" sounds and was also confirmed by strain gauge readings in the vicinity. Some bond must have prevailed, however, otherwise a failure in the joint area would have been inevitable.

**RETRO-CU: Undamaged Specimen with Circular Jacket** — This specimen also developed two modes of failure, depending on the direction of loading. With positive loading, a flexural hinge developed at the gap between the two sections of the steel jacket surrounding the beam. With negative loading, a flexural hinge once again formed outside the retrofitted region, but in this case on the upper portion of the column. To avoid a catastrophic buckling failure of the column, negative loading corresponding to a ductility factor greater than $\theta=1.5$ was avoided, since it was deemed more desirable to observe the development of the plastic hinge within the retrofitted zone. Failure of the specimen was caused by tensile rupture of the lower layer of beam reinforcing steel under positive loading. Based on an approximate assessment of the point when the yield moment was reached, a ductility factor of $\theta=5$ was developed in the fourth cycle. The flexural hinge was concentrated over a very short distance, due to the confinement of the core concrete. Under positive loading, yielding was initiated in the gap region in the beam at an applied moment of 39.6 kNm. This value is somewhat higher than might be expected for this section, because both the top and bottom layers of reinforcing steel were in the tension zone. It was found that the flexural capacity of this specimen under negative loading was governed by the flexural capacity of the column (56.4 kNm) outside the retrofit. As indicated in Table 1, the location of failure and the value of the yield moment can be predicted reasonably accurately by analysis.

During positive loading, when the flexural hinge developed within the retrofit gap, there was no loss of strength of the specimen. However, a reduction
in the strength of the specimen occurred during negative loading due to P-δ effects. Fig.10 indicates that some reduction in the stiffness of the section and some hysteresis loop pinching took place during the test, most noticeably during negative loading, when failure occurred within the unretrofitted section of the column.

There was no audible indication and very little visual evidence of any bond failure between the concrete and the steel jacket. No portion of the jacket was strained beyond yield (1335 microstrain) during the test. Observed tensile transverse strains in the retrofit area were as expected and tended to be higher than those observed in the rectangular retrofit test RETRO-SU. Deformation within the joint area was negligible.

RETRO-SD: Previously Damaged Specimen with Square Jacket —
Based on the results from RETRO-SU, it was decided to extend the steel jacket along the beam by 230 mm. The entire length of the beam was thus effectively wrapped in the steel jacket (Fig.11). The critical section for both positive and negative loading directions was now within the gap region and the specimen developed the desired flexural hinge. Under positive loading, the flexural hinge began to form at the gap in the retrofit at an applied moment of 29.0 kNm. This value agrees with predicted yield moments using section analysis (Table 1). Under negative loading, the flexural hinge began to form in the gap at an applied moment of 44.3 kNm, which is substantially below the value of 57.3 kNm predicted by section analysis, assuming full development of the reinforcing steel. If partial development is taken into account, however, the predicted moment capacity of 42.4 kNm compares well with the test result of 44.3 kNm.

Failure of the specimen was caused by tensile rupture of the bottom layer of steel. The hysteresis curve of the specimen (Fig.12) shows that a maximum ductility factor of $\theta=6$ was reached during the ninth cycle. Only a marginal loss of strength was recorded during cycles four to seven, which can be attributed to the small displacement increments of subsequent cycles. Because of a reduction of stiffness, the displacement excursions might not have been sufficient to reach the full moment capacity. This was alleviated during cycles eight and nine, which had twice the displacement increment of the previous cycles. In this specimen, hysteretic loop pinching only occurred during negative loading, and it was clearly less than the pinching in RETRO-SU and RETRO-CU. This can be explained by the fact that the bending stiffness under positive loading was mainly a factor of the reinforcing steel (tension and compression), whereas the concrete in compression contributed significantly to the stiffness under negative loading. During closing of the flexural cracks under negative loading, the stiffness did not increase until the cracked concrete surfaces came into contact again. The hysteresis loops indicate a stable increase in the ductility ratio. All the plastic deformation occurred within the gap region of the retrofit.
As was the case with specimen RETRO-SU, a number of "pops" could be heard during the initial testing cycles, indicating some loss of bond between the concrete and steel jacket. Plastic deformation of the steel jacket was evident in the area bordering the gap of the retrofit steel on the beam. The deformation was characterized by an outward bulging of the middle of the jacket; the jacket corners remained square.

**RETRO-CD: Previously Damaged Specimen with Circular Jacket**

Based on the behaviour of specimen RETRO-CU a modification of the retrofit was undertaken to avoid development of a flexural hinge in the column. An extra pair of equidistant gaps were cut between the originally designed gap and the column face (Fig.13). This was done to reduce the moment capacity in the beam, and hence move the critical section for negative loading from the column into the beam area. This specimen's failure mode was as expected and designed for - the development of a plastic hinge within the gap region of the retrofit under both positive and negative loading. Under positive loading, the applied moment capacity of this specimen was 29.8 kNm, which was very close to the predicted value of 29.5 kNm from a section analysis. Under negative loading, the applied moment capacity was found to be 44.3 kNm, which was significantly smaller than the theoretical capacity of 51.9 kNm, assuming full development of the reinforcing. Again, when taking into account the reduced development length of half of the top steel, the moment prediction of 43.6 kNm compares well with the applied moment of 44.3 kNm.

Failure of the specimen was caused by flexural yielding and subsequent tensile rupture of the bottom layer of reinforcing steel during positive loading. The extra gaps helped to spread out the yielding length of the reinforcing bars. The remaining steel rings provided sufficient confinement to the concrete to ensure the development of the tensile strength of the bars. The bottom layer of steel failed in tension during the positive loading portion of the 11th cycle, with an overall hinge ductility ratio of $\theta=7$ (Fig.14). A small amount of strength loss was observed during positive loading, but virtually none during negative loading. A small gain in the strength occurred in the two cycles before failure, due to strain hardening of the reinforcement bars. No significant stiffness loss took place, and for positive loading the hysteresis loops were of a rounded shape, without the characteristic pinching effects. For negative loading there was some evidence of pinching, which could be attributed to the loss of bond in the shorter reinforcement bars of the top layer. Most of the bending strain after yielding was concentrated in the first gap (closest to the column), with some deformation in the other two gaps. There was no evidence of loss of bond and the strain gauges indicated confinement stresses of up to 50% of the yield stress.

There had been some concern that the proximity of the gap to the joint area might unduly affect the integrity of the joint. The measured levels and distribution of strain in this area suggest that this concern was unwarranted. The joint area was not confined by the steel jacket on all sides and the bond and
confinement effects could not be relied upon in this complicated region. Except for the greater strains in the joint retrofit steel, and the greater bending capacity of the specimen, RETRO-CD behaved similarly to RETRO-CU.

Summary of Retrofit Testing

The testing program of the four retrofitted specimens highlighted potential mistakes that should be guarded against in the design of retrofit schemes. Flexural failure in the beam or column outside the retrofitted area necessitated changes in the retrofit design of subsequent specimens. This was accomplished by either extending the retrofit along the beam or by cutting the two extra gaps into the jacket of the beam, adjacent to the joint. Some deterioration of the bond between the steel tube and the grout occurred in the two square specimens; this bond failure did not occur in the two circular specimens. All the specimens developed tensile strains in their jackets during testing. However, only the circular jackets were effective in producing significant confinement stress on the core concrete, leading to a generally superior behaviour with regard to pinching.

EVALUATION OF RETROFIT SCHEMES

Before considering the advantages and disadvantages of the different retrofit schemes studied, it is useful to summarize what was actually achieved in the retrofit process under evaluation. First and foremost, the retrofit strengthened the deficient or damaged reinforced concrete joint area to such an extent that hinging or failure was deflected to adjacent areas: to the beam or column outside the retrofit, or to the beam within the intentionally weakened "hinge section". Any comparisons of "before" and "after" are thus somewhat misleading because the "problem" was shifted from the most critical link to the next one in line. It is thus necessary to consider the retrofit process in its entirety, which involves not only assessing the benefits of introducing strengthening measures, but also of evaluating the behaviour of the post retrofit critical failure zone and of assuring that sufficient ductility is provided to maintain favourable structural response.

Reliability of Bond Between Concrete and Retrofit Steel

In the design of the retrofit jacket, the possibility of developing substantial bond between the concrete and the retrofit steel was considered. To break the continuity of the tubes in the longitudinal direction, circumferential
gaps were provided in the casing. Ideally, the steel casing should act only as confinement for the concrete, and should not increase the moment capacity of the specimen. In this ideal situation, no longitudinal bond or interaction would exist between the concrete and the retrofit steel. It is, however, impossible to expect lateral confinement of the concrete, resulting from a biaxial stress state in the jacket, without experiencing some amount of mechanical interaction (friction) in the longitudinal direction. This interaction, of course, would increase the moment capacity, perhaps significantly, particularly where the beam jacket is physically connected to the column jacket.

The chemical bond between the concrete and the steel did not appear to be very consistent, as was evidenced in certain areas of the square jacket retrofits, where bond failure occurred early in the tests. A more likely scenario would be the development of a more reliable mechanism through friction or mechanical bond between the concrete and the retrofit steel.

**Improvement in Ductility**

The overall improvement in ductility of the specimens was substantial, regardless of the jacket shape used in the retrofit. In comparison to the unretrofitted specimens (RCBC1 and RCBC2), even the square retrofits (RETRO-SU and RETRO-SD) provided a substantial improvement, provided that failure occurred within the retrofitted zone. It should be noted that the true extent of the improvement in ductility for RETRO-SU and RETRO-CU cannot be fully assessed, since in both cases failure mechanisms developed outside the retrofit zones, which affected the results of the intended experiments.

**Positioning the Gap for Plastic Hinge Development**

The gap in the beam retrofit was initially incorporated in the design concept in response to the anticipated increase in the moment capacity from chemical and/or mechanical bond between the retrofit steel and the concrete. Its function was to contain failure within the retrofitted zone of the specimen, and also to limit the resulting increase in moment capacity. The gap was placed a reasonable distance away from the joint area, to avoid any risk of a joint failure. Since the joint area was not entirely surrounded by the steel jacket, a gap very close to the beam-to-column interface was thought to reduce the confinement protection provided by the jacket. This was of particular concern with the square jackets which, because of their geometry, were not expected to act as efficiently as the circular tubes. Two major considerations thus governed the positioning of gaps: reduction of the moment capacity, which would be most effective with a gap close to the joint, and preservation of the integrity of the joint zone, which
called for an uninterrupted casing close to the joint. A third consideration entered at a later stage, namely the placement of multiple gaps to provide an extended hinge zone.

**Dimensions of the Retrofit Jacket**

The thickness of the steel jacket was found to be sufficient for the purposes of this particular study. The basis of design was to replace the missing transverse steel in the joint area, on a volume basis, with an equivalent amount of steel in the form of a steel jacket. A major consideration dictating the use of a steel jacket was ease of fabrication. Weldability of the casing required a minimum thickness to avoid excessive distortions and burn-through when joining the component parts of the steel jacket. The weld needed to be able to withstand the high stress concentrations that would occur in the beam-to-column joint area; this could be a severe requirement, especially under repeated yield cycles.

The length of the jacket sleeves and the placement of the gaps were the critical dimensions affecting the behaviour of the retrofit. Two possibilities arose: extend the jackets sufficiently to force a failure in the joint itself, or provide a weakened hinge zone in a non-critical area. In both the square and circular jacket cases, the length of the retrofit was sufficient, provided the gaps were close to the column face. The confinement effect of the steel jacket was mainly to prevent spalling of the concrete, both within the enclosed zones and the narrow gaps in the plastic hinge zones.

**Confinement Effects**

In general it may be stated that the increase in concrete compressive strength due to confinement is only an issue in heavily loaded columns. In beams, the increase in bond and shear strength are of greater importance. The confinement of the concrete was maintained throughout the loading sequence; even in the gap area the arching effect was sufficient to contain the concrete. With the aid of mechanical bond (friction), the failure zone was concentrated in the gap zone. A general bond failure between the beam reinforcement steel and the column concrete in the joint area was thus avoided. There was no evidence of an increase in compressive strength of the concrete due to triaxial confinement. The measured moments at yield were generally consistent with predictions based on the conventional unconfined strength of the concrete.
Rating the Retrofit Schemes

Considering the efficiency of the retrofit schemes, all the specimens behaved satisfactorily, with a slightly better performance being exhibited by the circular retrofits. For the size of the specimens tested it can be said that the square retrofits did behave well enough to merit consideration as a reasonable retrofit scheme. Caution is advised, however, when retrofitting larger sections where square casing might not perform as well. In general it can be said that the increase in the moment capacity provided by the concrete-to-steel bond, and the containment provided by the jacket, regardless of its geometric shape, were the overriding factors in the improvement of ductility. Placing the gap closer to the joint helped the effectiveness of the jacket as designed. RETRO-CD provided the greatest retrofit improvement, with RETRO-SD a close runner-up. Failure was observed to be entirely within the retrofit region of these two specimens. RETRO-SU and RETRO-CU were affected by failure modes outside the retrofit region, and their performances should thus not be compared to RETRO-CD and RETRO-SD. RETRO-CU developed a larger ductility than RETRO-SU, but the increase in ductility was limited by the failure in the column under negative loading. RETRO-SU developed reasonable ductility under positive loading, until flexural and shear failure took place outside the retrofit zone under negative loading.

CONCLUSIONS

An effective retrofit method has been studied for deficient or damaged beam-to-column joints in reinforced concrete frames. Grouted steel tubes provided the necessary strength and confinement to avoid distress of the joint region. Circular tubes, which are more costly to fabricate and install, proved to be more effective than rectangular tubes, although the latter also met the general expectations.

As a side effect, this retrofit method caused an increase in moment capacity of the sections, which should be minimized to avoid undue distress in other regions of the structure that have not been designed to carry additional loads. To limit the increase in moment capacity of the joint area, it is recommended that gaps be placed in the steel casing, close to the beam-to-column joint area. A number of gaps may be advisable to avoid concentrated yielding and reduced ductility of the reinforcement bars within the gap areas. When using square jackets, care should be taken to avoid placing the gaps too close to the joint area. The joint area was shown to be weak, and without the added benefit of the radial confinement provided by a circular retrofit jacket, this joint area, which is effectively within the column, can be significantly affected by a plastic hinge forming too close to it.
A jacket thickness designed to replace the missing transverse steel, on a volume basis, was found to be sufficient. For practical reasons during construction (e.g. welding), a minimum thickness is advisable.

Since none of the retrofitted specimens failed in the same manner as the unretrofitted ones, no conclusions can be drawn pertaining to the effectiveness of the proposed retrofit method in the immediate joint area. Follow-up tests are underway to determine the amount of strengthening and added ductility needed in that region.

ACKNOWLEDGEMENTS

The direct and indirect support of the Natural Sciences and Engineering Research Council of Canada and the Department of Civil Engineering at the University of British Columbia is gratefully acknowledged.

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<table>
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<th>SPECimen</th>
<th>Failure Location</th>
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<th>Negative Loading</th>
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<td>RCBC1</td>
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<td>16.8 18.8 20.0</td>
<td>32.4 36.7 24.8</td>
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<td>16.8 18.8 20.2</td>
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<tr>
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<td>26.5* 42.4* ----</td>
</tr>
<tr>
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* Moment resistance calculated with partial bar development

**TABLE 1 — RETROFIT TEST RESULTS**

---

Reinforced Concrete Frame under Seismic Load  
**Forces in Longitudinal Reinforcement**  
Joint Shear

Fig. 1—Shear forces in joint area
reinforced concrete column
\[\text{grout}\]
\[\text{steel casing}\]
\[\text{gap}\]
reinforced concrete beam

Fig. 2—Beam-to-column joint retrofit

UNDAMAGED REINFORCED CONCRETE

DAMAGED REINFORCED CONCRETE

\[\text{RCBC1 RCBC2}\]

RETRO-CU

RETRO-SU

RETRO-CD

RETRO-SD

CIRCULAR JACKET RETROFIT

SQUARE JACKET RETROFIT

CIRCULAR JACKET RETROFIT

SQUARE JACKET RETROFIT

Fig. 3—Test program
Fig. 4—Reinforced concrete specimen

Fig. 5—Jacket geometry
Fig. 6—Test arrangement
Fig. 7—Steel properties

Fig. 8—Hysteresis curve for unretrofitted specimen RCBC1
Fig. 9—Hysteresis curve for specimen RETRO-SU

Fig. 10—Hysteresis curve for specimen RETRO-CU
Fig. 11—Testing of specimen RETRO-SD

Fig. 12—Hysteresis curve for specimen RETRO-SD
Fig. 13—Testing of specimen RETRO-CD

Fig. 14—Hysteresis curve for specimen RETRO-CD
Reinforced Concrete Buildings in Moderate Seismic Zones: Progress and Problems in Evaluation and Design

by P. Gergely

Synopsis: Earthquake-resistant design of reinforced concrete structures has special problems in moderate seismic zones if the possibility of a very large rare earthquake exists. This is the situation in central and eastern North America. The questions and difficulties associated with introducing a seismic design code for the first time are discussed. The seismic risk to a populated region is not reduced much for many years after the code takes effect; only the rehabilitation of existing structures will reduce the risk significantly in a meaningful time frame. The overall behavior of buildings, especially of existing older r/c buildings, is often nearly elasto-plastic in nature because a mechanism forms soon after the formation of the first hinge and there is little or no overstrength. This may not be an optimum design in most cases. The response of r/c buildings to moderate ground motions designed only for gravity loads is better than expected, with moderate drifts and no premature brittle failures in most building types. That is not the case for the rare catastrophic earthquake.

Keywords: cyclic loads; earthquake-resistant structures; frames; joints (junctions); loads (forces); reinforced concrete; structural design
Appreciation

There are a number of reasons why Tom Paulay has had such a profound and lasting influence on me and on my views on various questions in earthquake engineering. It is unfortunate that I have had so little direct contact with him - our encounters have been brief albeit eventful, intense, and memorable. They were relatively recent (we met the first time in about 1970), but we did meet in many interesting places around the world, ranging from Japan to Spain, from Paris to San Francisco to Ithaca.

I have heard about him in so many different connections. A couple of problems we worked on at Cornell University were also studied by Tom and his associates, almost at the same time. These include cyclic shear transfer across cracks (aggregate interlock), bond, splices, and design approaches based on limited ductility. In some of these questions we looked at problems from nearly opposite extremes: his approach to design in a severe seismic zone has involved elaborate detailing with lots of confinement, whereas our work in a moderate zone tries to get away with as little extra detailing as possible. In that sense we speak a different language, or at least a different dialect.

But my link to Tom Paulay is deeper and stronger in another field - where we do speak the same language. His fame as a Hungarian cavalry officer had reached me before our first encounter. Over the years I have enjoyed and marvelled at his colorful use of our language, even though he has not had much chance to practice his mother tongue in New Zealand.

My appreciation of the receipt of an Honorary Doctorate from the Technical University of Budapest in 1992 was greatly enhanced by the realization that Tom Paulay had also received that honor a few years earlier.

What has impressed me most is the energy, vigor, and intensity of Tom's work, and the breadth and depth of his knowledge linking research to design practice. Our talks would often switch to earthquake engineering and he would always feel very strongly about certain views or innovative ideas. I don't think he is slowing down at all.

Tom, I wish you continued youthfulness, satisfaction in your work, and good health. Kedves Tamés, kívánok jól egészséget, további jól munkát, és Isten áldását! Szeretettel köszöntünk Kingével.

Peter Gergely
Peter Gergely has been on the faculty of Cornell University since 1963. He is active in BSSC, ATC, and NCEER. He served on the Board of Directors of ACI and is a member of ACI Committees 224, 369, 408, and 445. He won ACI’s D. Bloem Distinguished Service Award and a Wason Medal for most meritorious paper.

INTRODUCTION

Earthquake engineering research and practice has progressed much in the past thirty years. Many new concepts have been developed, among them the ductility and capacity-based design, relying on pioneering work by Thomas Paulay and others. Much of this evolution has naturally concentrated on regions of high seismicity where earthquakes occur frequently and the awareness of the risk is relatively high.

Lately rapidly increasing attention has been paid to the seismic risk in moderate seismic zones (MSZ), for example in the eastern and central United States of America, parts of Europe, and in Australia. This paper highlights the problems faced in introducing earthquake-resistant design in MSZ’s and contrasts them with the state of practice in zones of high seismicity. It also examines questions related to the evaluation of existing buildings, since that is by far the most urgent problem, but some discussion is devoted to the design of new buildings, to the introduction of seismic code provisions for the first time, the problems with handling large infrequent earthquakes, and to the retrofit of structures. It is argued that the difficulties in some of these areas are greater because of the knowledge and sophistication already achieved in high seismic zones. A few of the special problems in evaluation are illustrated using the example of gravity-load designed structures.

The author is delighted and honored to have this opportunity to recognize and to show his esteem for his compatriot, Thomas Paulay, who has done so much for the improvement of earthquake engineering, both by advancing fundamental concepts and by developing practical solutions.
MODERATE SEISMIC ZONES

Seismic codes have designations (low, moderate, severe) for regions with various levels of seismic risk. Since nearly all the attention has been paid to high-risk areas, MSZ's have not been defined properly. In most codes the designation is a function of the peak ground motion (acceleration or velocity). Materials-oriented design specifications, such as the ACI Building Code (1), have special reduced detailing and ductility requirements for "moderate" zones. This simplistic approach seemed to be satisfactory until recently when seismologists and structural engineers began to pay closer attention to the risk and to the generation of uniform hazard spectra in MSZ's.

What is a moderate seismic zone? In the simpler case rather frequent earthquakes occur but with limited intensities and the intensities of the largest rare events are not much larger than the frequent ones. This is a model analogous to the situation in high seismic zones in California and is easily understood. Examples for such a zone are central Europe, Scandinavia, the British Isles. The design philosophy of most design codes can be extended to these regions without any conceptual difficulties. However, there is another possibility: a region where there are a few ground motions with limited intensities in any location but there is a probability of very large earthquakes with low recurrence rates. Examples are eastern and central North America, northern India, and parts of China. The handling of risk in these zones is much more complex, as will be discussed below. There are also regions where the tectonics are not understood and it is impossible to classify them into one of the two types of "moderate" zones. In some cases the fault slip rate is so small or the potential of a long active fault developing is remote and the likelihood of a major quake to occur is negligible and the first type of moderate zone exists.

The Catastrophic Earthquake

Most of the following discussion will focus on eastern and central North America (ENA), where the possibility of a
devastating earthquake is a major difficulty in establishing design goals. How do we deal with a low-probability high-consequence event? The thinking developed in other regions may have to be modified to deal with this complex question. In California the design earthquake is not significantly smaller than the rare catastrophic one, but in ENA the motion caused by the rare earthquake (say with a recurrence interval of 2,000 years) is 2 to 4 times greater than the current “design motion” considered in moderate zones (2).

The difficulties in detailing with rare catastrophic events are practical and conceptual (as will be discussed below). Our codes deal with forces below the yield level and extrapolate to and rely on nonlinear behavior. The R-factor approach in current codes is deficient in handling this artificial and over-simplified dualism. In particular, the R factor, among other effects, reflects the overstrength of a building, that is its strength relative to first yield. Often structures, especially many existing buildings (see the next section), possess little overstrength. Yet this property may be a key component of the safety margin in a catastrophic earthquake, especially in MSZ’s. Whether a building collapses during a ground motion greater than the design input depends strongly on the excess capacity (overstrength).

It is relatively easy to deal with essential facilities (hospitals, fire stations, structures containing hazardous materials, and perhaps schools) because society probably wants to have a reasonable assurance that these structures will perform as expected even in the catastrophic event. But what is a suitable performance level for these structures? Little chance for collapse, or continued serviceability? These questions are difficult for new design but they are much more complex when one considers the rehabilitation of existing essential facilities.

The case of ordinary buildings (apartment houses, small office buildings) is even more complicated. Is it up to the owner to assume a certain level of risk? In most cases the owner would not worry about a 2,000-year event, especially for an existing building, but society may demand a reduced risk for life protection.
The problem of the failure of nonstructural elements, such as parapets, partitions, and exterior cladding, is perhaps even more difficult to handle than the collapse of the frames. Yet, the collapse of nonstructural elements is an important potential cause of loss of life and this question is more critical in ENA where many buildings have deteriorated and the drift is often large.

**Introduction of Seismic Code Provisions**

Several states in the USA have recently incorporated seismic design provisions into their building codes for the first time and a number of states are in the process of doing so. These are typically in the "moderate" seismic zone and use force levels from national model codes, without serious or explicit consideration of the catastrophic event. The national model codes (UBC, BOCA, SSBC) are modified to a limited extent. Little information is yet available on the acceptance of the new provisions or problems with their use by the design profession and the construction industry.

As pointed out above, the model codes rely on and believe in the sanctity of ductility. It is the hysteretic energy absorption that saves the structure in a major earthquake. Most experienced engineers in high seismic zones, especially those writing the seismic codes, have "grown up" with the development of codes, starting with the 1950s when there were only a few simple rules to the 1990s when many special details are required to assure adequate ductility. Most designers and constructors have learned the ropes gradually and have a good idea of the appropriate fee structure indicated by the additional work.

However, in MSZ's the introduction of mature seismic provisions is occurring suddenly. Considering the fact that good seismic design is partly an art and the understanding of nonlinear dynamic behavior is essential for good design of all but the simplest structures, it is questionable whether the complex rules associated with inelastic response and detailing should be part of the first code. They could easily be misinterpreted or ignored by some designers or builders.
Perhaps simpler rules, relying more on strength rather than ductility, would suffice and would certainly be much simpler. For essential or other important structures designers would use more sophisticated codes with the help of consultants.

Another difficulty facing code adoption is the fact that seismic codes are continually in the process of changing. For example, the introduction of a two-level design procedure is being considered for several codes; the associated complexity will tax engineers in high seismic zones, let alone their many colleagues new to seismic engineering. A slow and deliberate change of codes, including their seismic zone maps, is advisable.

The introduction of seismic design codes would not significantly reduce the risk to society for many years. (One estimate is that in the East only about 10% of the building stock would be affected after about 50 years.) Also, one should remember that over 90% of all building construction is three stories or less and in many rural communities building codes are not enforced. Thus the risk would not be much greater if a simpler interim code would be introduced (say for about five years) until engineers, architects, inspectors, city officials, and contractors become familiar with the basic concepts.

The extra cost of earthquake-resistant construction is not great for the current moderate zone force levels, but if the codes introduce much higher requirements for the rare event, the costs would increase. As mentioned before, the seismic risk in the East will reduce significantly only if existing buildings are rehabilitated, and that is a much more complex and costly task.

The risk to the ENA is great because (1) large concentrations of population, (2) the possibility of a large-magnitude earthquake, (3) the damaged area extends much farther than in the west for a given magnitude earthquake, (4) higher soil amplification because of the greater impedance difference between hard rock and soil, (5) practically no buildings have been designed for earthquakes and many not even for wind, (6) many buildings are old and deteriorated, (7) the duration of motion may be relatively long.
There are strong arguments in favor of requiring *ductility* in MSZ's to guard against collapse in the catastrophic rare event and of using the current code *force* levels applicable to moderate zones to design for the 10% in 50 years probability earthquake. This appears to be a valid reasoning but the unresolved question remains: how soon should the codes require ductility for the rare event and for what types of structures? These questions are different from those in zones of high seismicity. It is ironic that there has been a trend in the opposite direction in California: to require more strength and stiffness (and perhaps somewhat less ductility) to limit damage. In the East damage is probably not a concern for ordinary buildings (and more flexible structures are acceptable, as discussed in the next section).

The New York State seismic code draft, which is similar to the earlier New York City draft, follows the UBC provisions with a number of notable exceptions. The state is divided into four seismic zones with Z factors ranging from 0.09 to 0.18 (based roughly on 10% probability of exceedance in about 100 years). Ordinary moment resisting frames are allowed only in the lowest zone (a "low" risk zone). The five soil factors range from 0.67 to 2.5, a much large variation than in current codes, reflecting the large impedance contrast between the competent rock and the soil generally found in the East. Rules are included for the probability of liquefaction. Dynamic analysis is recommended but not required for tall or irregular buildings, and for buildings with periods greater than 1 second on the poorest soil type. Building separation is covered by a simple rule.

The proposed European code for seismic design, Eurocode 8 (3), has many rather sophisticated features and strict requirements, such as the need for modal analysis for all buildings with periods of more than about one second. The author believes that this code is too complex for regions having no experience with earthquake resistant design.
Consideration of Risk

Explicit risk analysis has not been an urgent issue in new design because of the relatively low additional cost of seismic construction and the relatively small impact of a catastrophic earthquake in the West. But the high cost of rehabilitation and the threat of a rare event in the East are strong arguments in favor of risk analysis for life safety and for damage reduction. Risk analysis and the determination of acceptable risk are quite different for a specific building than they are for a region. Our handling of risk, especially involuntary risk, is often subjective and fallacious, and is a function of the cost of risk reduction.

Risk analysis is tied to the performance objectives which could range from no collapse to continued operation of the facility. Unfortunately, there is no rational or even irrational but consistent way of estimating acceptable risk to society or to an owner. Comparisons of the probability of loss of life due to earthquakes with that due to everyday events tend to indicate that the former is smaller for a specific project. Since risk can be viewed as the product of the probability of occurrence and the value, for certain buildings or facilities the risk may be higher because of the greater value to the owner. Likewise, the global risk to society (city, state, or country) could be large. The probability of loss of many lives in a single event becomes greater relative to most other hazards. People generally seem to be less tolerant against loss of many lives, even if the probability is low, than they are against loss of very few lives frequently.

It appears that in most cases purely probability-based analysis would not indicate rehabilitation of ordinary structures and perhaps the inclusion of the rare event in design. The decision and the general question of acceptable risk depend on the value or importance of the structure. The threat of litigation may also enter the picture. Society could and probably will insist on better global performance in a region. These questions are becoming acute for the reasons mentioned above.
As described previously, the possibility of a low-probability high-consequence earthquake in most moderate zones creates a dilemma for designers. Is it best to provide the required ductility for this rare event, considering the design and construction complexities in regions where earthquake-resistant construction is a novelty? It appears that the optimal mix of strength and ductility remains an open question in other seismic zones as well.

Several earthquakes have shown that excessive reliance on ductility can result in too much damage to the contents of a building. As a result, most codes have been tightening the stiffness (drift) requirements and put slightly greater emphasis on strength, at least for ground motions with lower return periods; the failure of life-threatening non-structural elements depends on (tangential) interstory drift.

In a design procedure developed by Thomas Paulay (4) and his colleagues, ductile behavior is concentrated in selected areas of the building to guarantee sufficient energy absorption so that other areas can be designed elastically. This is excellent approach in the hands of experts and provided nothing will alter the assumed load path and stiffnesses.

There are advocates of an "optimum" structural behavior involving plastic hinges throughout the structure occurring at about the same load level. This is analogous to plastic design in steel in which the lowest weight corresponds to the simultaneous formation of plastic hinges in the collapse mechanism. However, seismic response also depends on the stiffness and frequencies, and on the characteristics of the input. Such an "optimum" design would result in a nearly elasto-plastic type of load-deflection curve for the building, with little stiffness following the first yielding. (Gravity-load designed buildings often have such characteristics because of their weak columns, as discussed in the next section.) Ground motions having acceleration pulses could cause the collapse of this type of building. Proponents of spreading energy absorption throughout the building might end
up with a similar design (as opposed to the New Zealand design approach mentioned above). In general it appears preferable to have load-deflection characteristics with post-yield stiffnesses to avoid soft-story type of failure, the acceleration pulse effect, and reasonably high overstrength (ratio of peak load to load at first yield). In some MSZ's overstrength may be essential in resisting rare catastrophic earthquakes, as mentioned before. There is a need to develop such design goals and guidelines. In this respect dual systems are advantageous because they lose their stiffness gradually. A two-level design procedure might also avoid this problem (5).

The need to study post-elastic stiffness (which affects strength and deflection) has prompted the development of design procedures involving static nonlinear "pushover" analyses. A lateral load vector (for example from the code) is applied stepwise to the building and the member stiffnesses are reduced at hinge locations. The base shear versus roof displacement, for example, reveals a lot about the participation of the various structural elements in carrying the lateral loads, the damage state at various drift (and load) levels, and the post-elastic stiffness. A comparison of many pushover analyses of actual buildings has shown that a surprisingly large fraction has nearly elasto-plastic shapes, indicating nearly "optimal" design (as defined above). This investigation is continuing.

A number of design procedures are based on the pushover analysis. Probably the earliest is the reserve energy technique proposed by John Blume around 1960. The Capacity Spectrum Method, developed by Sigmund Freeman (6) is rapidly gaining popularity. Certain equivalent nonlinear single-degree systems depend on pushover analyses (those developed by Mete Sozen and his associates). These approaches are attractive though most tend to be conservative. Work is under way at Cornell University and elsewhere to improve these relatively simple but powerful methods.
DAMAGE INDICES

A number of investigators at NCEER and elsewhere are working on the definition and measures of damage in a structure. The goal in most cases is to have a single index which describes the damage state in a building during and at the end of nonlinear analyses. The various damage indices often depend on drift, energy dissipation, or a combination of the two (7). Damage indices are usually developed for members and then a global damage index for the entire structure which gives an idea of the level of damage (slight, repairable, irreparable, or collapse). Ideally, the global damage index should be able to warn against impending collapse. An interesting and relatively simple damage index is a function of the current natural period, averaged over a time window (8). Work is continuing on damage indices for structural systems, mainly because they are valuable in risk analyses. However, much less attention has been paid to damage to nonstructural elements.

GRAVITY LOAD DESIGNED R/C BUILDINGS

An extensive experimental-analytical investigation of the performance of r/c buildings designed only for gravity loads has been under way at several institutions associated with the National Center for Earthquake Engineering Research (NCEER). The goal has been to study the behavior of these frames, representing typical construction for the 1940 to 1970 period and having nonductile details. Only a few selected results are highlighted here.

The chief characteristics of these frames is their high flexibility. The columns are slender and the slipping of the bottom beam bars, which are discontinuous at the joint, provides for additional flexibility (9). At higher loads distress may develop in the joints (which contain no ties or perhaps only a couple); that also represents reduced stiffness but may not indicate global failure. As a result of the high initial period (much higher than the approximate code value) and the increase in period, the seismic input is relatively low.
Shaking table and analytical studies have indicated that in most cases the input energy is small (10,11).

Examination of typical three, six, and ten-story frames has shown that collapse is likely only at maximum ground acceleration levels of about 0.30g or slightly higher, which is certainly satisfactory for design motions in MSZ's but probably not for the rare catastrophic event. Drifts are normally less than 2% as shown in Table 1 (12). It is interesting to note that the gravity load negative moment can be important in counteracting the positive moment causing bar pullout and consequent great loss of stiffness and moment capacity. In taller gravity load designed frames, over about ten stories and with maximum ground accelerations above about 0.20g, the shear in columns may become a problem.

Figs. 1 and 2 show capacity spectra for a three-story and a ten-story frame superposed on response spectra (13). The predicted maximum response indicates damage to the beam ends (bar slip and cracking), moderate drifts, and damage to the joint regions. It should be noted that distress in joint regions does not necessarily signify collapse. Nonlinear time-history analyses showed similar but somewhat better performance, illustrating the conservatism in the capacity spectrum method. Comparison of responses for the Nahanni earthquake, which is thought to be a reasonable representation of an intraplate earthquake (ENA), for two types of soil conditions is illustrated in Fig. 3a and 3b. The flexible frame moves into the amplified region of the spectrum but collapse would still not occur in this case.

Comparison of nonlinear time-history analyses with and without the p-delta effect (12) showed that the axial force had rather small effect on the results for the three buildings considered. The full-scale joint tests indicated, however, that although high column axial load improves joint capacity, the joint deterioration is more rapid.

It has been recognized that the duration of the ground shaking has can have a pronounced effect on the response, yet this factor is not considered explicitly in design. According to the current thinking, major earthquakes in central and eastern North America have much longer durations at
moderate distances and high frequencies than in California. This factor also enters the already complex question of risk assessment in a MSZ.

**Analysis versus Experiments**

The shaking table tests of three-story frame building models at Cornell University and the State University of New York (SUNY) at Buffalo provided data for checking the nonlinear analysis results using the program IDARC. The analyses used trilinear or smooth hysteretic curves, based on frame component tests, which reflected strength and stiffness degradation, and pinching. Although the comparisons were very good, it should be pointed out that the initial period of the structure has a great influence on the analysis results and thus such comparisons hinge on the accurate knowledge of the building periods. The estimation of structural periods in r/c structures is subject to error in the laboratory and even more so in real buildings. It is advisable to perform analyses with a range of periods or use widened spectra (14).

**Rehabilitation (Retrofit)**

The work on the evaluation of gravity-load designed r/c structures has been followed by an investigation of their rehabilitating. The emphasis has been on simple ways of stiffening and strengthening in MSZ's (15, 16). At Cornell University external steel plates and rods have been used. In the case of internal joints the rods served to provide the continuity of the bottom beam bars. Plates attached to the outside faces of external joints delayed the spalling of the cover and along the splices. One problem with structures with brittle regions is that stiffening a weak spot can increase the forces in the structure and shift the failure to another (perhaps brittle) region. In the present case the original structure experienced distress and loss of stiffness in the joints, but little problems with the splices. Partial retrofit of the joints could initiate splitting at the lapped splices.
The retrofit method at SUNY/Buffalo involved column capitals (thickening of the slab) and jacketing the columns with vertical prestressing over the height of the entire building. This approach provided greatly increased stiffness and strength; a strong column - weak beam structure was achieved.

The retrofit investigation is continuing using methods (at Cornell University) which increase the stiffness only modestly so that the foundation forces do not become excessive.

CONCLUSIONS

The consideration of seismic risk is very difficult in some moderate zones where the possibility of a rare earthquake exists which is much larger than the design motion. The question has different implications for an essential building, for an ordinary building, and for a region.

As opposed to most regions of high seismicity where seismic codes have become sophisticated over the years, the sudden introduction of such seismic codes in moderate regions is questionable. Gradual introduction of seismic provisions and making few changes in each editions would greatly assist the design profession and the construction industry. The risk will not be reduced much for many years after the new code takes effect. The rehabilitation of existing buildings does reduce the risk but that is a much more complex and expensive undertaking than the design of new structures.

Optimum design often means minimum weight or smallest member sizes. For such structures a collapse mechanism would form soon after the formation of the first hinge and the structure's overall behavior is nearly elastoplastic. This type of building is susceptible to failure due to an acceleration pulse. It is better to follow other design criteria that assure sufficient stiffness and overstrength after the formation of the first few hinges.
An extensive study of the seismic behavior of r/c frames designed only for gravity loads has shown that this common structural system performs reasonably well for moderate earthquakes. The drifts are limited and brittle failures are not likely to develop. Damage develops in the joints and column shear becomes a problem for taller buildings. Since these structures are very flexible, the demand is relatively low but the damage to the structure and to the contents is high.

A research program is underway to study simple methods of retrofitting these buildings. The primary aim is to provide some increased stiffness and to avoid premature failure of the lapped splices or columns.

ACKNOWLEDGMENTS

Many individuals have cooperated in the research and in the development of some of the concepts discussed in this paper. First and foremost, the author is indebted to his colleague Richard White with whom he has worked closely on the projects at Cornell University. Many others have contributed in various ways, among them Andrei Reinhorn and John Mander at SUNY/Buffalo. Glenn Bell has communicated a wide-range of ideas and suggestions discussed in this paper; his insight and ready help are greatly appreciated. The author is pleased to acknowledge the key role of many graduate research assistants who have worked on the research projects and generated stimulating ideas. Many thanks are due to Klaus Jacob who has often provided intriguing ideas about seismicity and risk, and elucidated some of the notions in this article. The author is indebted to most of the authors in this volume for their friendship, example, and professionalism; foremost among them is Thomas Paulay. Finally, the financial support of NSF and the State of New York to NCEER is acknowledged.
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### TABLE 1 — CALCULATED DRIFTS FOR THREE-STORY FRAME

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Maximum story drift (%)</th>
<th>Max. base shear (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st story</td>
<td>2nd story</td>
</tr>
<tr>
<td>Nahanni hard soil 0.15g</td>
<td>0.09</td>
<td>0.1</td>
</tr>
<tr>
<td>Nahanni soft soil 0.20g</td>
<td>0.17</td>
<td>0.14</td>
</tr>
<tr>
<td>Taft 0.02g</td>
<td>0.09</td>
<td>0.08</td>
</tr>
<tr>
<td>Taft 0.20g</td>
<td>0.84</td>
<td>0.85</td>
</tr>
<tr>
<td>Taft 0.35g</td>
<td>1.48</td>
<td>1.11</td>
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<tr>
<td>El Centro 0.02g</td>
<td>0.08</td>
<td>0.07</td>
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<tr>
<td>El Centro 0.35g</td>
<td>2.15</td>
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<tr>
<td>El Centro 0.52g</td>
<td>2.46</td>
<td>2.32</td>
</tr>
</tbody>
</table>

**Fig. 1—Capacity spectrum analysis of three-story frame**
Fig. 2—Capacity spectrum analysis of ten-story frame
Fig. 3—Capacity spectrum analysis for Nahanni earthquake on soft soil and hard soil
Studies on the Seismic Response of Waffle-Flat Plate Buildings

by M. Rodriguez and R. Meli

Synopsis: Waffle-flat plate buildings are very popular in different countries. Their seismic performance has been very poor. Several research projects on the seismic response of these buildings have been performed at the Instituto de Ingeniería, UNAM; their results and findings are summarized in this paper. First, the main features of the structural system and of its resistance to lateral loads are presented. The most common mechanisms of collapse are described and the observed performance during the 1985 Mexico earthquake is discussed. A case study of building performance during the earthquake is briefly presented. Results of an experimental research on a two-story model of a waffle-slab building are described. The specimen was first tested in a shaking table and later subject to cycles of static lateral loads. The specimen showed a rather poor behavior with very small lateral stiffness and limited energy dissipation capacity. The failure mechanism was mainly governed by the shear and flexural strengths of the columns, and by flexural cracking in waffle slabs. Recommendations for the proper use of the system are given, emphasizing the need to combine it with structural walls, bracings or stiff frames, in order to achieve the necessary lateral stiffness and strength in a building.

Keywords: buildings; earthquake-resistant structures; flexural strength; loads (forces); plates (structural members); reinforced concrete; shear strength; stiffness; waffle slabs
Appreciation

It was when I was on sabbatical leave at the University of Canterbury, New Zealand, that I met Tom Paulay. Since then I have the pleasure of enjoying his friendship and warm personality. His pleasant conversations during several tea breaks at the University of Canterbury, as well as on other occasions, were always stimulating. These conversations were on the possible use of structural wall buildings instead of RC frames, and the pitfalls of walking a dog in a neighbor's yard. Words were also exchanged on slab contribution to building resistance and on his lively memories on horseback riding.

His enthusiastic approach to research in Earthquake Engineering has impacted my own attitude towards research in this field. After Tom's lessons, structures were not inanimate anymore. We can ask them not only to behave in a ductile manner, but also to behave well.

Tom has made an enormous contribution to the field of Earthquake Engineering, a field that challenges us everyday. His work lights our current efforts to continue our research in the field.

Last, but not least, it is said that along with a great man there is a great woman. Again, this is true. Herta, Tom's wife, also deserves the gratitude of earthquake engineers for all the support that she has given to him.

Mario Rodriguez
ACI Member Mario Rodriguez is a Professor in the Instituto de Ingenieria, Universidad Nacional Autonoma de Mexico at Mexico City. He has teaching and research interests in reinforced concrete structures and earthquake engineering. Dr Rodriguez is active on several ACI technical committees.

ACI Member Roberto Meli got his Ph.D. in Structural Engineering from the Universidad Nacional Autonoma de Mexico, where he has been a research professor for more than 25 years. He has been active in research, mainly on behavior of masonry and concrete structures, and on earthquake engineering.

INTRODUCTION

Since the 1960’s and until 1985, waffle-flat plate buildings without structural walls were extensively used in Mexico City because construction procedures were simpler and story heights were lower than in conventional frame structures. Although evidences of poor seismic performance of the system had been observed in experimental research in Mexico and other countries before 1985, as well as in flat-plate buildings in several countries before that year (1), it was the September 19, 1985 Mexico earthquake that clearly showed the significant shortcomings of the system. During this earthquake an important number of waffle-flat plate buildings in Mexico City collapsed or were severely damaged, showing a rate of damage higher than in conventional frame structures.

As a result of the observed building damage in the 1985 Mexico earthquake, several studies have been performed in Mexico aimed at having a better understanding of the seismic behavior of waffle-flat plate buildings. Also, new seismic design provisions for the system have been incorporated in the current 1987 Mexico City Building Code (1987 MCBC) (2).

This paper describes some characteristics of waffle-flat plate buildings in Mexico City and their observed seismic performance during the 1985 Mexico earthquake. Results of analytical and experimental research conducted in Mexico on the seismic behavior of waffle-flat plate buildings are also described, and some of the MCBC seismic design provisions for the system are commented.
CHARACTERISTICS OF WAFFLE-FLAT PLATE BUILDINGS IN MEXICO CITY

Before the 1985 Mexico earthquake, waffle-flat plate systems were commonly used in Mexico City as main lateral load resisting systems for low-rise and medium-rise buildings up to 15 stories. Spans ranged in most cases from 6 to 8 m, with story heights of about 3 m. Typical of the system is a grid of perpendicular ribs and wide shallow beams spanning at column lines, and a solid head around a column.

Building code provisions for Mexico City previous to 1985 typically specified light amount of transverse reinforcement in columns, and code requirements for designing slab shear and flexural reinforcement for transmission of moments to columns were provided only since 1976 when a new code, the 1976 Mexico City Building Code (1976 MCBC) (3) superseded the previous one (1966 MCBC) (4).

The 1976 MCBC provisions were not widely followed in Mexico City before the 1985 earthquake. As a result, a number of waffle-flat plate buildings had small solid heads around columns, and poor detailing for slab shear and flexural reinforcement to transmit moments to columns.

As is discussed later in the paper, code provisions for Mexico City previous to 1985 overestimated the lateral stiffness of waffle-flat plate systems. Hence, very flexible systems resulted when complying with these provisions.

Irregular configurations were often observed in buildings constructed with the system, as well as irregular arrangements of columns, masonry partitions or other so called nonstructural elements.

SEISMIC PERFORMANCE OF WAFFLE-FLAT PLATE BUILDINGS IN MEXICO CITY DURING THE 1985 EARTHQUAKE

A description of observed damage and prevailing modes of failure in waffle-flat plate buildings in Mexico City during the 1985 earthquake is given elsewhere (1). Only some important aspects of the damage pattern are discussed in the following.

Typical characteristics of the seismic behavior of the system during the earthquake were excessive lateral
flexibility, high rate of flexural and shear damage in columns and inadequate shear strength of waffle slabs. Statistics show that column flexural and shear failures were typically observed in most cases of the system's failures. In only a few cases collapse or severe damage were due to punching shear of waffle slabs.

When robust columns or structural walls were provided in waffle-flat plate buildings, flexural behavior of the system was predominant. In these cases, formation of flexural yield lines was frequent, combined with some slab shear cracking in solid heads and mainly in slab ribs in the perimeter of these slab heads. This distribution of damage was not generally associated to building collapse. The behavior is in agreement with the commonly used strong column-weak beam seismic design approach, which is considered desirable for seismic design of frames in high seismic risk areas (5).

MEXICO CITY BUILDING CODE PROVISIONS FOR WAFFLE-FLAT PLATE BUILDINGS

Lateral Stiffness

The 1987 MCBC and previous building codes for Mexico City, have adopted the effective beam width model for the seismic analysis of waffle-flat plate buildings. In this model the slab is replaced by beams spanning in each direction in column lines. The depth and width of these beams are equal to the slab thickness and an assumed effective width, \( b_e \), respectively. The 1966 MCBC and 1976 MCBC specified for \( b_e \) the following expression

\[
b_e = \frac{L_2}{1 + 1.67 \frac{L_2}{L_1}} + 0.6c_2
\]

where \( L_2 \) is the slab span perpendicular to the direction of analysis, \( L_1 \) is the slab span perpendicular to \( L_2 \) and \( c_2 \) is the column dimension in the direction of \( L_2 \). Equation (1) was based on the effective width for flat plates proposed by Khan and Sbarounis (6).

Based on experimental research on waffle slab-column connections conducted in Mexico in the late 1970's (7), the 1987 MCBC defines \( b_e \) as

\[
b_e = c_2 + 3h
\]
where $h$ is the slab depth.

The MCBC definitions for the ratio $b_c/L_2$, along with the Khan and Sbarounis' definition for this ratio, are plotted in Figure 1. The results correspond to the case of square slabs and columns, and for a ratio $h/L_2$ equal to 0.05. According to Figure 1, lateral stiffness of waffle slabs evaluated following MCBC provisions previous to 1985 are considerably larger than those evaluated according to the 1987 MCBC, and give results that agree well with those using Khan and Sbarounis' definition for $L_e$.

Waffle Slab Strength and Other Design Provisions

The design provisions for resisting the unbalanced moment $M_s$ (see Figure 2) between slab and column are similar in both the 1987 MCBC and the 1989 ACI (8). Additionally, the 1987 MCBC requires that the flexural reinforcement of a waffle slab in a $c_2 + 3h$ width must resist the total slab bending moment produced by design seismic forces. This requirement and current provisions of the code for defining the lateral stiffness of waffle flat plate systems are major departures from design provisions for the system that were specified by previous building codes for Mexico City.

Other current code provisions severely limit the use of the system by specifying allowable lateral displacements and response reduction factors that are lower than those specified in previous codes for Mexico City. Hence, new waffle-flat plate systems cannot be constructed in Mexico City for medium to high rise buildings without combining them with other structural elements, as structural walls, bracings or stiff frames, which could resist the major part of seismic forces. Design procedures for reinforced concrete buildings in which earthquake resistance is provided mainly by structural walls are given elsewhere (9).

MEXICAN RESEARCH ON WAFFLE-FLAT PLATE BUILDINGS

The earliest studies on the seismic behavior of waffle-flat plate systems were performed at the Instituto de Ingenieria (II) of the Universidad Nacional Autonoma de Mexico (UNAM) at Mexico City in the late 1970's. Several studies on the subject have been also performed in Mexico after the 1985 earthquake. Some results of
these studies are commented in the following.

**Analysis of the Seismic Performance of Waffle-flat Plate Buildings During the 1985 Mexico Earthquake**

Several waffle-flat plate buildings that resisted the 1985 earthquake in Mexico City have been studied in detail (1,10,11). Both linear and nonlinear analyses have been performed for selected buildings, and comparisons with their observed performance during the earthquake have been made. In some cases, measurements of fundamental periods of damaged buildings were also made.

Results of these studies are generally consistent with the observed damage and high flexibility observed in the system during the earthquake. The results also indicate that, as in the case of conventional frame buildings (12), the lateral strength of the system computed by nonlinear analyses is typically larger than the strength evaluated using building code procedures. These overstrengths had significant incidence in the survival of an important number of buildings during the 1985 earthquake (10). An example that illustrates these results is presented in the following.

Figure 3 shows the plan and elevation of a 15-story waffle-flat plate building in Mexico City. The building was designed according to the 1976 MCBC, and the earthquake resistant system consisted of moment resisting waffle-flat plate frames with robust columns in the E-W direction and waffle-flat plate frame-shear wall in the N-S direction. The building had severe slab flexural and shear damage during the 1985 earthquake, with only minor cracking in columns. A detailed description of the building, results of nonlinear dynamic analysis, and an evaluation of its seismic performance during the earthquake are given elsewhere (11). Results of nonlinear static analysis of the building are commented below.

Figure 4 shows results of nonlinear static analysis of an interior frame in the E-W direction of the building, performed with the nonlinear analysis program DRAIN-2D (13). In this figure, which can be considered representative of the overall behavior of the building, V and W are the base shear and total weight associated to the frame, respectively; thus, V/W represents the base shear coefficient. Dr represents the roof drift ratio of the frame, which is defined as the roof displacement divided by the height of the building. Figure 4
indicates that the calculated lateral strength of the building was about 2.6 times the unfactored design strength, which can be assumed equal to 0.06, considering that the building was designed according to the 1976 MCBC.

Lateral displacements of the building were also computed by linear elastic analysis according to the 1976 MCBC (11). The results showed that lateral displacements were lower than the allowable interstory drift (0.016), although the differences were not significant. When lateral displacements of the building were computed by linear elastic analysis according to the 1987 MCBC, they far exceeded the allowable interstory drift (0.012) (11).

The maximum interstory drift ratio in a frame structure is typically larger than the maximum roof drift ratio. In the analyzed building the former was about 1.6 times the latter. Hence, the results of Figure 4 also highlight the high flexibility of the system, which can be associated to severe building damage.

**Experimental Research on the Seismic Behavior of Waffle-flat Plate Systems**

Lateral load tests on waffle slab-column connections were performed at II, UNAM, in the late 1970's. Results of this research have been published elsewhere (7). Design procedures were developed from this research, and constituted the bases for the changes in the seismic design provisions of waffle-flat buildings of the 1987 MCBC.

Experimental studies of a two-story, one third scale model of a waffle-flat plate building have been also performed at II, UNAM. The overall geometry of the specimen is depicted in Figure 5. A detailed description of the experimental program and evaluation of results are given elsewhere (14,15). Only some important aspects of this research are outlined in the following.

In the first stage of the research, the specimen was subjected to shaking table tests representing small earthquakes. Hence, the specimen responded in the elastic range. Lateral stiffness was determined with an elastic analysis based on gross-section member properties and using the effective width model, with \( b_e \) equal to \( c_2 + 3h \). Results of this analysis and the experimental results showed good agreement.
In a second stage of the research, the specimen was subjected to cyclic lateral load testing. At the end of testing, the specimen showed severe shear and flexural damage in columns and severe flexural cracking in waffle slabs. Figure 6 shows the base shear, $V$, versus roof lateral displacement hysteresis loops measured in the lateral load tests. In this figure, $V_{MC}$ is the resistant base shear calculated according to the 1987 MCBC, and $D_2$ is the roof lateral displacement. The results indicate that the specimen had a low energy dissipation capacity throughout the test, as compared to those found in tests on reinforced concrete elements with good confinement and flexural behavior (16), and a measured overstrength of about 2.5 times the design strength.

If yield displacement is defined as the one corresponding to an equivalent monotonic elastoplastic system with the same monotonic energy absorption as the real system, the maximum displacement ductility factor reached in the test structure would be only 2.5. It was also found that for nominal theoretical base shears lower than about $0.5 V_{MC}$, the measured lateral stiffness of the specimen can be predicted by the current Mexico City code provisions (1987 MCBC) using elastic analysis based on gross-section member properties. For larger base shears the 1987 MCBC provisions overestimate the measured lateral stiffness of the test structure (14).

An experimental research on the seismic behavior of waffle-flat plate systems has also been undertaken at the Universidad Nacional Autonoma del Estado de Mexico at Mexico. A two story, one half scale model of a waffle flat plate building was subjected to cyclic lateral loading. Some results of this research are given elsewhere (17). The results also confirm the high flexibility of the system and the associated severe structural damage.

CONCLUSIONS

1) Waffle-flat plate systems have been extensively used in Mexico City before the 1985 Mexico earthquake, usually as main seismic resisting systems for low-rise and medium-rise buildings. During the earthquake a significant number of waffle-flat plate buildings collapsed or had severe damage. Observed shortcomings in the system were: low lateral stiffness, weakness in shear and flexural capacity of columns, and inadequate shear strength of slabs.
2) Studies for evaluating the seismic response of the system during the earthquake have been performed in Mexico. Several selected waffle-flat plate buildings that resisted the earthquake were subjected to detailed seismic analyses and results were compared with observed seismic performance during the earthquake. The results indicate that code designed waffle-flat plate buildings had important overstrengths that allowed the survival of an important number of these buildings during the earthquake. The results of these analyses also showed that the high lateral flexibility of the system is a typical undesirable characteristic of the system.

3) Seismic load tests on scaled models representing typical waffle-flat plate systems in Mexico City have been also performed in Mexico. Results of these tests confirm the low lateral stiffness and the important overstrength that were found in the analyses of the seismic response of typical waffle-flat plate buildings during the 1985 earthquake. The experimental results also highlight the low energy dissipation capacity and low displacement ductility factor capacity of the system.

4) Design procedures have been developed from the experimental and analytical research conducted in Mexico on the seismic behavior of waffle-flat plate buildings. Most of these procedures have been incorporated in the current Mexico City Building Code, which has now more stringent seismic design provisions than those of previous building codes for the city. For complying with current design requirements, it is expected that waffle-flat plates cannot be constructed in Mexico City without other structural elements, as structural walls, bracings or stiff frames, which could resist the major part of seismic forces.

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Fig. 1—Comparison of several definitions for effective width in flat slabs

Fig. 2—Shear and moment transfer at interior column-slab connection at ultimate load
Fig. 3—Details of waffle-flat plate building

(Dimensions are in meters.)

a) First floor plan
b) Elevation of frame D
Fig. 4—Base shear coefficient versus roof drift ratio in waffle-flat plate building
Fig. 5a—Dimensions of a two-story waffle-flat plate test structure—Typical floor plan

a) Typical Floor Plan

b) Elevation, Frame 1
Fig. 5b—Dimensions of a two-story waffle-flat plate test structure—Elevation, frame 1

Fig. 6—Measured base shear versus roof lateral displacement hysteresis loops for test structure
The Philippines Earthquake of July 1990
— Lessons for us all from the Destruction and Reconstruction

by D. C. Hopkins

Synopsis:

The M7.8 earthquake which hit the Philippines in July 1990 caused extensive and varied damage to a wide range of structures, most of which were of reinforced concrete. Because US Codes are adopted in the Philippines the event provides a unique opportunity for earthquake engineers worldwide to review their approaches to seismic design.

The paper results from the author's involvement in a visit immediately after the event and his subsequent role, in 1991 and 1992, advising the Philippine Government on reconstruction of damaged public buildings and infrastructure. Valuable insights into the real issues were gained through contract with local consultants, government engineers and government agencies such as the Departments of Health and Education.

The Government's Earthquake Reconstruction Project is outlined and the effects of the earthquake briefly described as an introduction to the main issues - structural concepts, ductile detailing, construction practice and supervision, influence of "non structural" elements, and the value of site investigations. Examples are given to illustrate these issues in the Philippine context. The author concludes that proper attention to the basics is sufficient to significantly reduce earthquake risk, not only in the Philippines, but in many developing and other countries. In this International Decade for Natural Disaster Reduction, this has special relevance.

Keywords: confined concrete; damage; earthquake-resistant structures; earthquakes; education; quality assurance; reinforced concrete; standards; structural design
Appreciation

We all knew that Tom Paulay had special qualities when he held us in rapt attention for two hours in drawing and design classes. These lectures were unsurpassed for the quality and quantity of information and their deep insight into structural behaviour. Force diagrams were done on the blackboard with awesome precision and at breakneck speed. They were clear, concise, in colour, absolutely to the point - and fun. This large, enthusiastic, energetic and friendly Hungarian could change coloured chalk in his hands with a magician’s sleight of hand, and always to good effect. There was no question of changing career paths after exposure to such charming competence.

Thankfully Tom’s influence extended beyond the University, geographically speaking, and through the years. Through Conferences and meetings of the New Zealand National Society for Earthquake Engineering we kept plugged in to this dynamo who generated useful information and improved our understanding of the subject. His papers were always on topics of relevance to practicing engineers, and he was as ready to give advice as we were to receive it. Shear walls, coupled or cantilever, capacity design procedures, and of course detailing became simple to understand.

When personal access was not possible we always had his papers, and Park and Paulay, that most thumbed of texts. On overseas assignments these were of inestimable worth. In 1981, Bechtel in San Francisco asked me to develop a capacity design procedure for pipe racks. I turned to Tom’s papers and was able to oblige. In Mexico in 1985 Tom’s insight was with us as we witnessed the superior performance of shear wall structures.

More recently, in the Philippines, I found that copies of Tom’s papers on shear walls and detailing of reinforced concrete proved to be worth many times the cost of the excess baggage required to take them there. The text and especially the diagrams always made it easy to get ones points across convincingly. Shear walls once again featured, and the photocopier worked overtime.

In a wider context, the comprehensive grounding in the fundamentals received by many of Tom’s classes at Canterbury has played an important part in developing the reputation of New Zealand engineers throughout the world. Through his own efforts and those of his graduates, Tom has already contributed much to the reduction of earthquake risk in many countries - a campaigner for international disaster reduction for four decades.

I have had the privilege of knowing Tom Paulay since my early schoololdays, when he first came to the University of Canterbury to join my late father’s department. At Engineering School I had the good fortune to be one of his ‘victims’ - an experience which has benefitted me throughout my career. It is a high honour and a great pleasure to be permitted to contribute to this special Symposium for an outstanding New Zealander.

David C Hopkins
Biographical Sketch

David Hopkins is a consulting engineer based in Wellington, New Zealand who has been involved in a wide variety of projects in United Kingdom, New Zealand, South East Asia and the Pacific. He is a former President of the New Zealand National Society for Earthquake Engineering and has contributed to its publications on precast concrete, lifelines, masonry buildings and architectural elements.

INTRODUCTION

The earthquake of July 16, 1990 was notable for its widespread effects and range of types of damage. As such it provides a unique opportunity for engineers to review their approaches to seismic resistant design.

This paper results from the author's involvement in a technical reconnaissance visit immediately after the earthquake, and his subsequent role in the Philippine Government's Earthquake Reconstruction Project (ERP) as Program Technical Specialist (Buildings and Bridges). Damage, and lack of damage, have been viewed first hand. Reconstruction, and strengthening measures have been reviewed and observed on a range of projects. Contact with local consultants and government engineers has provided further insights into the effects of the earthquake and the key issues faced in the design and construction of seismic resistant buildings and infrastructure.

The earthquake revealed some consistent deficiencies in design and construction which have been evident in past earthquakes in other parts of the world. The lessons available from the event and subsequent reconstruction thus have a wider relevance. This is particularly so because the Philippines has adopted US codes of practice for earthquake design.

The July 1990 Earthquake and Its Effects

The Philippines is known to be one of the most seismically active countries in the world. At least five earthquakes are recorded each day, and there have been six major earthquakes of magnitude 7.3 or greater since 1954, ie one every six to eight years.
Added to this is the fact that the country, with its many fault systems, is recognised as one of the most complex regions of tectonic plate interaction in the circum-Pacific belt.

The magnitude 7.8 earthquake of 16 July 1990 involved rupture of the Philippine Fault over a 120km length, with maximum displacements of seven metres horizontally and one metre vertically. Affected areas and the fault break are shown in Figure 1. Strong shaking at approximately code level occurred in Baguio City and elsewhere. In Manila, intensities were significantly less, but shaking lasted for about a minute and a half.

The range of types of damage and its widespread geographic location is indicated by the following:

- The University and mountain resort city of Baguio suffered major losses to buildings, especially hotels.
- The coastal city of Dagupan, sited on a river delta, lost a key bridge. Many buildings subsided due to the spectacular effects of soil liquefaction.
- Major roads in the mountain regions were devastated by landslip which exposed slopes to subsequent rains.
- Roads on the plains were extensively damaged by liquefaction.
- Many bridges on national, provincial and barangay (village) roads were damaged due to landslip and liquefaction.
- Hundreds of schools and hospitals over a wide area were damaged or destroyed due to ground shaking, liquefaction and ground subsidence.
- Fault movement caused damage, particularly to roads.

Damage to natural environment was substantial. Slope failures laid bare soils which eroded in subsequent rains. Siltation closed at least one hydro dam and has modified river beds, making flooding in subsequent rains much worse in some areas.

Several technical publications have recorded the effects of the earthquake in some detail and indicated likely key issues. (1, 2, 3, 4)

**Earthquake Reconstruction Project**

The Earthquake Reconstruction Project (ERP) was initiated by the Government of the Philippines and provides for the reinstatement, and/or strengthening of damaged public facilities. The ERP is funded by the Philippines Government with the backing of loans from the Asian Development Bank ($US100 million) and World Bank ($US125 million). It commenced in November 1990 and is expected to be completed during 1993.
The ERP consists of 12,000 sub-projects, which range from minor plastering of cracks to replacement of major buildings and bridges. Quite apart from the technical challenges, this represents a considerable organisational task. Overall management is headed by the Department of Public Works and Highways (DPWH) Central Office and implemented through their Regional Offices. Seven local consulting firms have each been assigned a geographic area within which they are responsible for all projects.

Other agencies such as the Department of Health (DOH), Department of Education, Culture and Sports (DECS) and National Irrigation Administration (NIA), are responsible for implementing projects involving their facilities. Figure 2 shows the basic organisation of the Earthquake Reconstruction Project.

A summary of sub-projects is shown in Table 1, indicating the agencies responsible for highways, bridges, roads, housing, hospitals, schools, flood control, water supply and irrigation. A Project Implementation Unit (PIU) and a Project Coordination Unit (PCU) were established to oversee implementation; incorporate mitigation and preventive measures to defined standards; make application to the loan agencies for funds; and to coordinate the efforts of the various agencies.

International Consultants assigned to the ERP include both Implementation and Technical Specialists. The focus of the Technical Specialists, of which the author was one, was on incorporation of mitigation measures in the reconstruction.

Particular tasks of Technical Specialists included:

- Preparation of Detailed Guidelines for Design and Construction
- Establishment of mitigation measures
- Review of design criteria and preparation of contract documents
- Review of site investigations
- Review of engineering supervision
- Provision of training in mitigation technology and its application in construction

This required liaison with DPWH and other implementation units and with Local Consultants responsible for design and construction supervision. Projects were required to meet ERP criteria on mitigation measures to qualify for funding from the loan agencies. The majority of projects are of relatively low value and the efforts of the Technical Specialists were focussed on projects involving significant reconstruction/strengthening and technical content.
MAIN LESSONS FROM THE EARTHQUAKE

With such a wide range of effects it is difficult to cover these adequately in a brief paper. Nevertheless, the experience of working with local engineers from both Government and the private sector, showed up a number of recurring issues. The principal lessons to be drawn were that:

- Sound structural concepts must be adopted
- The effect of "non-structural" elements must be properly considered
- Consistently good construction practices and effective supervision can reduce seismic risk substantially.
- For bridges, special care must be taken in site selection, design and construction.
- Data gathering on seismic risk needs to be augmented.

This is a familiar list to earthquake engineers, but the challenge is to identify ways of implementing these lessons effectively. To do this it is necessary to examine the issues in more detail.

Sound Structural Concepts

In the Philippines, almost all buildings for people are made of reinforced concrete, the vast majority being reinforced concrete frames with concrete hollow block (CHB) partitions. The usual design approach is to assume that earthquake forces are taken by the frame acting alone. No account is taken of the stiffening effect of the CHB.

Such partitions are made from low quality blocks faced with 12mm of cement plaster each side. Nominal reinforcement is generally included, with dowel bars to columns and beams. No effort is made to separate the partitions from the structure.

Many of the hotels which collapsed in Baguio had soft storeys, because the lower one or two storeys had fewer partition walls than the upper floors. Examples from Baguio include the Nevada, FRB (Figure 3) and Royal Hotels where dramatic collapses occurred.

If the architects and engineers for these hotels had got together early in the design, the soft storeys could have been eliminated without detriment to the architectural objectives.

More subtle irregularities brought about the demise of the Skyworld building in Baguio. This building, triangular in plan, occupied a prime corner site on the steep main street. It had reasonably regular stiffness except for two floors at the lower end. This produced a partial soft storey
which caused irreparable damage to the lower columns, resulting in the building being condemned. While it is true that better detailing of the columns may have saved them, the change in stiffness at the lower levels was clearly the prime cause of distress.

An even more subtle conceptual deficiency probably contributed to the collapse of the tower block of the Hyatt Regency Hotel in Baguio. In plan it had reinforced concrete shear walls of seemingly adequate size in both directions, as shown in Figure 4. Verbal reports of the building motion in previous earthquakes suggested a marked torsional response, and it is clear from the plan that the torsional stiffness is very much less than the transverse stiffness, and it is possible that the inter-storey drifts due to torsional motion helped initiate collapse of one of the outer columns or walls. Some stiffer elements restraining torsional movement may have prevented this.

It is evident from inspection of the collapsed building that detailing of the reinforcement in the columns and walls was deficient in modern day terms. There was no effective lateral tying of the compression flange of the walls.

In contrast to the spectacular failures of hotels, many buildings in Baguio did not fail.

In Baguio City, most of the buildings of up to five storeys made of reinforced concrete frames with block infill, survived the earthquake. Although the block was no doubt classed as "non-structural" by the engineers, it acted like a shear wall. Fortunately, in those buildings which survived, CHB partition walls happened to be reasonably symmetrically placed and continuous through the height of the buildings. It appears that in buildings of limited height the forces in these "shear walls" performed satisfactorily. Given the prevalence of this type of construction, it would be helpful to learn more about the reasons for their survival and develop some empirical rules for design. Unfortunately, Philippine resources to do this are lacking.

Recognition of the vulnerability of CHB was evident in some school buildings strengthened under the Earthquake Reconstruction Project. The concept adopted for these two-to-four storey schools was to replace selected CHB walls with reinforced concrete (See Figure 5). Thus reliable elements protect the columns and CHB walls from inter-storey drift. The practical difficulties of placing these new wall elements are considerable, but because the concept is sound, the achievement of high quality becomes less critical, and good earthquake performance can be expected provided construction is satisfactory.

A significant advantage of this concept is that it concentrates special seismic detailing in a few elements, making consistent achievement more likely.
Detailing of Structural Elements

It is well known that for reinforced concrete to behave in a ductile manner, it must be properly detailed. Most importantly this means:

a. Close-spaced stirrups and ties in columns and walls to hold the concrete in place and to prevent the buckling of longitudinal bars.

b. Close-spaced stirrups and ties in potential plastic hinge regions of beams to ensure they retain strength during cyclic loading.

c. Detailing of special transverse steel through beam-column joints in ductile frames to maintain the integrity of the joint during plastic deformation of adjacent beam hinges.

d. Careful attention to connections between all elements on the paths of gravity and earthquake load.

Points a), b) and c) have for some time been covered by the Structural Code of the Philippines in the Special Appendix for seismic requirements. Point d) is fundamental to all structural engineering.

The author has reviewed many sets of drawings for schools, hospitals and other public buildings in the Earthquake Reconstruction Project. Very few have included provisions for points a), b) and c) in spite of these being code requirements. Attention to point d) has not been consistent.

The issue is not whether the stirrups ties are spaced according to some complicated code equation but whether they are at reasonably close spacing, well tied and with $135^\circ$ bends at the terminations.

"Non-Structural" Elements

There are two major aspects to this:

1. The ability to "non-structural" elements, notably concrete hollow block (CHB) walls to alter the assumed structural response.

2. The tendency for "non-structural" elements to be damaged as the building sways.

The first aspect results in a real danger of building collapse. The second results in more costly damage, even in moderate earthquakes, and may even result in the unnecessary abandonment of the building.
The general failure of the engineering and architectural professions in the Philippines to address the structural implications of "non-structural" elements represents a major impediment to the improvement of the seismic performance of buildings. This failure resulted in unnecessary damage in the 16 July earthquake, and it is important that widespread and concerted efforts be made to learn from this experience and adopt a more reasoned approach.

The situation can be addressed at relatively low cost, but requires a change in approach. Fortunately there is evidence in some quarters that this change is happening. The recently issued Guide to Earthquake Resistant Design of Structures (5), published by the Association of Structural Engineers for the Philippines is a major step. Assistance from overseas engineers and architects would undoubtedly help.

On the second aspect of premature damage, there are countless examples of CHB walls which cracked in the July 1990 earthquake. These were walls classified as "non-structural" by the designers but they are almost always regarded as structural damage by the occupants and owners, and on occasion this resulted in evacuation and demolition of the building. This should act as a reminder to all engineers to consider the layman's view of what constitutes structural failure.

It is evident that this "non-structural" damage could be reduced or eliminated by designing selected walls as structural elements in reinforced concrete, or perhaps a stronger, higher quality, "structural" CHB. The reinforced concrete structures built in the 50's and 60's fared noticeably better than the more recent buildings of r.c. frame and blockwork infill, even though they had no seismic detailing. Much of the author's time was spent encouraging designers to incorporate reinforced concrete shear walls in selected locations. This would make the performance of the building more predictable and protect the remaining blockwork elements from damage.

Design Construction and Supervision Practices

The main structural code is the National Structural Code of the Philippines (NSCP) the technical requirements of which are identical to the United States Uniform Building Code (UBC) 1988.

The highest seismic coefficients from UBC are used generally throughout the Philippines. Given the tectonic setting and higher frequency of occurrence of large earthquakes, even higher factors could be argued.

Leading consultants and agency engineers have a reasonable knowledge of earthquake engineering and recognise the merits of good concepts and detailing and so on. Unfortunately, these virtues do not consistently find their way into completed designs, and even less in the
finished work. This difference appears to be related to cultural and 'political' factors. Review of another professional's work is regarded as unethical and constructive criticism and the sharing of knowledge is not prevalent. There is no regular local professional engineering journal in any discipline.

Many other pressures exist which make implementation difficult. These are similar in nature to, but occur to a greater degree than, those in Western countries. In spite of a generally low level of educational resources, leading engineers have considerable knowledge but find it difficult to insist on standards which they know to be appropriate.

For example, close attention by designers and owners needs to be paid to ensure that construction matches the drawings. Contractual specifications need to allow for changes to be made to take account of site conditions. Too often there is no effective mechanism to respond properly to adverse ground conditions.

The importance of detailing of reinforced concrete is not widely known in the construction industry. Tighter control together with instruction and training of site foremen would assist in seeing the designer's intentions achieved. The Philippine Chapter of the American Concrete Institute is about to launch a series of seminars aimed at helping site personnel understand these key issues. More such initiatives are needed and support from overseas quarters is especially appreciated.

Bridges

One of the most striking features of the July 1990 earthquake was the number of failed bridges. Those with discontinuous spans stood out. The disruption caused by even minor damage must serve as a reminder to all engineers to pay special attention to all aspects of bridge design.

In many cases on the ERP, account had to be taken of the drastically modified siltation regimes due to earthquake-induced landslides in the mountains. Riverbeds rose several metres and are still rising.

This changing situation highlights the difficulties of repairing infrastructure in such circumstances. The necessity to reinstate urgently must be balanced against the desirability of considered investigation.

Foundation Design -- Experience on the ERP suggests that investigation of the soil properties and the review of their implications are not as thorough or as commonplace as in other countries. An unfortunate drawback of the AASHTO Code used in the Philippines for Bridge Design is its lack of emphasis on the substructure in developing realistic models for analysis. A bridge must be thought of as a single entity, with proper account taken of the interaction of the substructure (usually piles), the
superstructure (piers and deck) and the soils down to bedrock. Code writers in the US and elsewhere, whose codes are adopted in other countries, should take care to see such fundamentals are appropriately covered.

**Deck Continuity** -- The Design Guidelines for the ERP sub-projects emphasised the desirability for continuity and Technical Specialists promoted and encouraged this practice.

**Soil Conditions**

Many public buildings including schools and hospitals in the Philippines, are built to standard designs prepared by DPWH, DOH and other agencies. The July 1990 earthquake brought many examples of failure of foundation soils, not just in Dagupan. This has brought a greater awareness by many engineers and owners of the need to carry out appropriate soils investigations as input to foundation design. Nevertheless some of the local engineers on the ERP have argued that soil investigations are "not necessary for single storey buildings" even though many single storey school buildings failed due to soil liquefaction.

**Seismic Risk Data**

The ERP objectives include provision to improve the available data on both the overall seismicity of the country and the correlation of building performance with known ground motions.

No strong motion records were obtained from the July 1990 earthquake, and preparations are now being made to install a network of instruments throughout the country. The lack of information on the nature of ground motions in the Philippines is in strong contrast to the wealth of data available in USA. In this International Decade for Natural Disaster Reduction it is appropriate that the international earthquake engineering community seeks to address this imbalance. Quite apart from the direct benefit to the Philippines, information would advance overall knowledge of the earth's seismic activity.

**INDICATIONS FOR THE FUTURE**

In 1991, the Association of Structural Engineers of the Philippines (ASEP) published a Guide on the Earthquake Resistant Design of Structures (5). Significantly, local engineers have written a special introduction to this Guide which is well written, illustrated and presented and covers all major issues.
The ASEP Guide puts seismic design to the fore and it is hoped that a new structural code will give equal prominence to it. Up till now Philippine codes relegated seismic provisions to an Appendix in the National Structural Code, which is based on the Uniform Building Code of the USA. Quite often local engineers were unaware or unfamiliar with its provisions. Frequently stirrup/tie spacings were allocated according to the non-seismic provisions, resulting in double the required spacings. Once again, code writers please take note.

The publication of the Guide is encouraging, though the proof will be in its effective application. Most encouraging is the first section of the ASEP Guide which lays down the principles in a succinct and straightforward way. It highlights the special demands of earthquake motion, the variability of response of elements attached to the building and importance of sound structural concepts.

It cautions on the application of computer analysis with the following apt words:

> Although contemporary structural analysis programs permit the study of complex structural models, these nevertheless are simplified abstractions of the real structure. Thus, not all characteristics can be included in the model. The more complex the structure, the greater is the possibility of a less reliable model.

The desirability of structural symmetry is stressed and the section concludes with a fitting summary of the key issues:

> The design engineer must be aware that a building does not merely consist of a summation of parts such as walls, columns, trusses, and similar components but is a complete integrated system or unit which has its own properties with respect to lateral force response. The designer must follow the forces through the structure into the ground and make sure that every connection along the path of stress is adequate to maintain the integrity of the system. It is necessary to keep in mind that the real forces involved are not static but dynamic; are usually erratically cyclic and repetitive; and can cause deformations well beyond those determined from the elastic design.
RECOMMENDATIONS FOR FUTURE DEVELOPMENT

Based on experience with the Philippines Earthquake Reconstruction Project, the following recommendations were made by the author to local engineers:

- **Surviving Buildings** -- Study them with a view to developing some empirical design rules which work. This has been done in other countries.

- **Masonry** -- Research and develop techniques to provide reliable (and conservative) design criteria. Improve quality and quality control. (It is hoped that ASEP and the Philippine Chapter of ACI will lead the way in achieving this.)

- **Standard Designs** -- Review all standard designs for public infrastructure components, so that they fit the best principles of seismic resistant design. Promulgate them widely so that the old designs are phased out quickly.

- **Education and Training** -- Instigate an intense series of seminars to promote the ASEP Guide and particularly its principles. Place due emphasis on these principles in all undergraduate courses.

- **Architectural Awareness** -- Encourage architects to become more aware of seismic resistant design. This is a welcome trend in USA, New Zealand and elsewhere. Both professions must recognise the need to work together against the earthquake threat to life and property.

- **Structural Features Reports** -- All structural engineers involved in buildings of significant size whether public or private, should adopt a means of recording basic design assumptions. The discipline of filling in the various sections is well worthwhile, and a properly completed form is invaluable when referring back months or even years later.

- **Check List of Key Points** -- Table 2 summarises some of the main points requiring attention in the planning, design and construction phases of building, bridge and roading projects in the Philippines. The coverage is not intended to be fully comprehensive, but gives an insight in the problems faced in the Philippines, and no doubt has relevance to us all, albeit to varying degrees.

These recommendations represent a considerable challenge for Philippine engineers. Hopefully, researchers and practitioners elsewhere will be motivated to take up some of these challenges as a means of assisting their counterparts to reduce exposure to seismic hazards in the Philippines.
LESSONS FOR US ALL

Damage in the Philippines' earthquake of July 1990 was similar to that caused in many earthquakes in other parts of the world. The number of low rise buildings and facilities affected should be a reminder that simple structures deserve attention from earthquake engineers in all earthquake prone countries. We all like to exercise our minds on the 'higher' aspects of more complex buildings and structures, but the fact remains that a substantial part of the world's seismic risk is tied up in low rise structures.

Codes are getting thicker and more complex, equations longer and more numerous. All this is the result of very worthwhile research, generally targeted at the larger more complicated buildings. But earthquakes haven't changed and continue to cause damage and distress to the thousands of simple structures, time after time. It may not be perceived as the cutting edge of earthquake engineering to improve the resistance of these buildings, but it is effective in reducing seismic risk overall. Emphasis is needed on the practical implementation of established concepts and details. There is a real need which outsiders can help address.

For example, there is a need to address the educational and professional development of earthquake engineering in the Philippines. This will be most effective when done in context in the Philippines where it can reach a large number of engineers. A programme currently underway is a series of training seminars sponsored by the United Nations Centre for Regional Development in Japan. These seminars are targeted at engineers in government agencies and local authorities. Leading engineers have been chosen to be trained to train others, which should ultimately help overcome implementation problems.

Surviving buildings represent more of a challenge than a lesson. Many of the buildings which survived would not have passed a designer's appraisal beforehand - a reminder of our incomplete understanding of earthquake phenomena. Can we be satisfied with this level of understanding?

Perhaps the most important lesson is the wisdom of the KISS principle - Keep it Structurally Simple. To achieve this requires a well developed insight into the fundamental behaviour of structures in earthquake. Developing such insight requires that the fundamentals be instilled in engineers during their education and early training, much as Tom Paulay did in his teaching days at Canterbury.

Increasingly over recent years, the world earthquake engineering community has become more close knit. Codes are becoming more similar, the events and problems in the Philippines must be seen by us all to be part of our problem, not just theirs - that is what the International Decade for Natural Disaster Reduction is all about.
REFERENCES


5. ASEP Guide - "Earthquake Resistant Design of Structures" 1991 Edition Published by the Association of Structural Engineers of the Philippines.
<table>
<thead>
<tr>
<th>SECTOR/ITEM</th>
<th>IMPLEMENTING AGENCY</th>
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DPWH = Department of Public Works and Highways  DOH = Department of Health  
WB = World Bank  NHA = National Housing Authority  
ADB = Asian Development Bank  DECS = Department of Education Culture and Sports  
LGU = Local Government Units  
NIA = National Irrigation Administration  
LWUA = Local Water Utilities Administration
### TABLE 2 — KEY ISSUES IN EARTHQUAKE RECONSTRUCTION

<table>
<thead>
<tr>
<th>Component</th>
<th>Planning</th>
<th>Concept</th>
<th>Details</th>
<th>Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roads</td>
<td>Realignment to avoid known hazards.</td>
<td>Engineered measures for stabilisation.</td>
<td>Careful attention to drainage measures</td>
<td>Achievement of all specification requirements.</td>
</tr>
<tr>
<td></td>
<td>Soil boring tests to determine hazards.</td>
<td>Upgrade drainage standards.</td>
<td></td>
<td>Achievement of effective drainage.</td>
</tr>
<tr>
<td></td>
<td>Hydrological studies.</td>
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<tr>
<td></td>
<td>Introduce environmental improvements, e.g. afforestation.</td>
<td></td>
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<tr>
<td>Bridges</td>
<td>Site selection to minimise exposure to earthquake (and other) risks.</td>
<td>Provision of continuous deck.</td>
<td>Ductile detailing of piles, piers and beams</td>
<td>Flexibility under contract to vary foundations to match conditions encountered.</td>
</tr>
<tr>
<td></td>
<td>Soil boring tests to determine appropriate foundations.</td>
<td>Maximum redundancy in pier support system.</td>
<td>Provide details which will achieve continuity and integrity.</td>
<td>Close supervision of key seismic details, especially piles and piers.</td>
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<td>Realistic modelling for structural analysis including allowance for pile/soil flexibility.</td>
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<tr>
<td>Hospital and School</td>
<td>Siting to minimise exposure to known hazards.</td>
<td>Regular shape.</td>
<td>Ductile design and detailing of columns.</td>
<td>Verify soil conditions. Vary accordingly.</td>
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<tr>
<td>Foundations</td>
<td>Soil boring tests for two-storeys or more, and for sites with liquefaction potential.</td>
<td>Symmetrical placement of lateral resisting elements.</td>
<td>Detailing of CHB to take load or be separate from structure.</td>
<td>Ensure CHB is fully reinforced with dowels to columns, floor and beams.</td>
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<tr>
<td>Buildings</td>
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<td>Consideration of the positive and negative effects of Concrete Hollow Block (CHB) walls.</td>
<td>Consideration of loads normal to the face of the wall.</td>
<td>Ensure stirrups bent 135° not 90° to anchor in the concrete core.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Viable and continuous load path from roof down.</td>
<td>Proper anchorage of stirrups in columns and beams.</td>
<td>Ensure roof trusses property secured to concrete columns/beans.</td>
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<tr>
<td></td>
<td></td>
<td>Ensure lateral systems has sufficient stiffness to prevent CHB taking excessive load.</td>
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</table>
Fig. 1—Map of earthquake effected area
Fig. 2—Organization chart for earthquake reconstruction
Fig. 3—Soft-story effects — FRB Hotel
Foundation Plan

Fig. 5—Reinforced concrete shear walls in a typical school
Deformation and Force Capacity
Assessment Issues in
Structural Wall Buildings

by F. Seible, G. R. Kingsley, and A. G. Kürkchübasche

Synopsis: The difficulties in assessing the probable deformation and force states of structural wall buildings under lateral earthquake forces are evaluated by means of laboratory test results from a five-story full-scale reinforced masonry structural wall research building tested to failure at the University of California, San Diego under simulated seismic loads. The individual structural components and actions which contribute predominantly to the seismic response characteristics of a structural wall building such as axial loads on walls, coupling between structural walls, lintel beam force and deformation capacities, as well as floor and wall-flange participation are evaluated based on individual component tests, the full-scale prototype test, and parallel diagnostic analyses. The importance of a realistic assessment of these parameters in a capacity design approach for structural wall buildings is evaluated. The outline for rational design models which allow a prediction of the complex behavior characteristics of structural wall buildings for all design limit states is presented.

Keywords: axial forces; buildings; capacity; coupling agents; deformation; flanges; frames; loads (forces); walls
APPRECIATION

The only way in which Tom Paulay can be fully appreciated is by simultaneous appreciation of three distinct yet inseparable personalities, namely (1) Thomas Paulay and his outstanding extraordinary contributions to earthquake engineering, (2) Tom and his always warm exorbitant humor, and (3) Tommy, the devoted ever-flirting Hungarian cavalry officer and his general. It is safe to say that Thomas wouldn't be Tom, wouldn't be Tommy, without Herta's generous and loving support of Tom's gregarious ways and professional dedication. So my appreciation and thanks go to both Tom and Herta for the invaluable professional inspiration and their continued friendship ever since a German graduate student in Canadian Commonwealth disguise tried to sneak into New Zealand.

Hálásan, köszönöm kitartó szeretetedet és barátságodat!

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**INTRODUCTION**

The general understanding of how building systems perform under lateral earthquake loads has seen significant evolution over the past decades based on data from post earthquake assessment of damaged and undamaged buildings, extensive small and large scale laboratory experiments, prototype field testing, as well as continually improving predictive and diagnostic analytical tools. The research community has long recognized that seismic design for forces alone is mostly inadequate since few building systems can be designed under economic and aesthetic constraints to withstand the actually unknown maximum seismic load input without damage, and that therefore deformation based design procedures are more appropriate to provide the necessary assurance for collapse prevention and serviceability. On the other hand, the practicing design profession is still largely focused on force based equivalent lateral load design provisions which rarely reflect realistic seismic force or deformation limit states in individual structural members or the overall building system, since very few rational design or analysis models exist which predict the actual force and deformation states in complete building systems under specific earthquake loads.

To compensate for the lack of realistic predictive performance characterization and the unknown nature of the actual seismic event, the concept of capacity design was introduced [1] to achieve the most desirable behavior of the
structure under general seismic or lateral load conditions rather than a
deterministic response to a specific set of events or loads. The capacity design
philosophy assumes that a predetermined global ductile failure mechanism
under general lateral earthquake loads can be achieved through the formation
of local mechanisms detailed for ductility and energy dissipation, and that the
fully plastic capacity of these mechanisms can be used in the design of all
other structural components to prevent their non-ductile inelastic action and
brittle failure modes.

This capacity design approach is ideal for frame type structures with well
defined local force and deformation capacities, but it can also be extended to
structural wall systems [2], particularly when clear and simple geometric
principles prevail. Unfortunately, many actual structural wall systems make
capacity determination of individual members difficult due to (1) rather
complex geometries with walls with large eccentric flanges, (2) floor systems
which provide an unknown amount of coupling between structural walls due
to their unknown participating width, (3) doorway lintels which contribute to
the coupling capacity, but limit deformation capacities due to early damage, (4)
torsional characteristics due to the plan arrangement of the structural walls, as
well as (5) dynamic force amplification effects resulting form higher mode
participation. All of these effects have been addressed by the research
community either separately or in combinations [3,4,5] but mostly on idealized
symmetric test specimens or two dimensional analytical models where the
actual participation of these parameters is clearly identifiable. For actual
prototype structural wall systems the participation of the above parameters can
be increasingly complex as will be shown in the following, requiring detailed
component evaluations prior to application in a capacity design approach.

A five-story prototype structural wall building as shown in Figure 1 is used to
show the influence of the above parameters on the capacity determination for
individual structural components. A subsection of the prototype building, (see
Figures 1, 2, and 3), was tested at full-scale under simulated seismic loads at
UCSD (University of California, San Diego) [6,7] as part of the NSF (National
Science Foundation) sponsored TCCMAR (Technical Coordinating Committee
for Masonry Research) program for analysis and design model verification.
Overall dimensions of the five-story full-scale research building are depicted
in Figure 2 together with questions of possible load paths and deformation
mechanisms. The research building consisted of coupled T-shaped (short) and
L-shaped (long) structural walls with wide eccentric flanges. The desired
global failure mechanism for the structure entailed the formation of flexural
plastic hinges in all of the coupling members and at the base of each wall. The
walls were constructed with fully grouted 6-inch (150 mm) concrete block
masonry. Horizontal and vertical reinforcement in both long and short walls
was distributed uniformly, without special confinement or boundary members.
In the first story, where plastic hinges were anticipated, the vertical and
Fig. 1—Prototype for the 5-story reinforced masonry research building

Fig. 2—The question of load paths and actions in the structural system
Fig. 3—Elevation of 5-story RM Research Building

Fig. 4—Effect of coupling forces on wall and structure capacity

Fig. 5—Analytical base moment versus top displacement envelopes showing individual wall contributions and coupling effect
horizontal reinforcement ratios in the wall webs were $\rho_v = 0.0034$ and $\rho_h = 0.0044$ respectively, and in the wall flanges $\rho_v = 0.0022$ and $\rho_h = 0.0011$. No lap splices were permitted in the first story vertical reinforcement. The structural walls were coupled by a precast prestressed hollow-core plank floor system with a 2 inch (50 mm) reinforced concrete cast-in-place topping. Doorway lintels were decoupled from the structural walls by vertical joints and the integral 6 inch (150 mm) wide cast-in-place beam in the 8 inch (200 mm) deep floor system was designed for ductility with over 3% volumetric stirrup reinforcement. Bond beam concrete masonry units allowed full grout flow both vertical and horizontal. Wall to wall connections had, in addition to full grout flow, the full horizontal reinforcement with 90° hooks in L- or 180° hooks in T-shaped wall intersections. Floor to wall connections consisted of full vertical reinforcement penetration, floor topping cast monolithically with 8 in. (200 mm) edge beams over the concrete block walls and shear keys spaced every 16 in. (400 mm) between the top of the floor slab or footing and the concrete block walls to minimize sliding problems. The test building was subjected to sequential earthquake segments recorded during the 1971 San Fernando and the 1979 Imperial Valley earthquakes in an on-line pseudodynamic test program, thus, allowing the lateral force distribution, including higher mode effects, to be determined by the changing dynamic characteristics of the test structure and the earthquake ground motion input. Torsional displacements were eliminated in the test structure due to the symmetric arrangement of structural walls in the loading direction in the prototype structure, see Figure 1. A detailed account of the building geometry, design, detailing, and testing is provided in [6,7] and only relevant data for the capacity determination of individual components and the overall system are discussed in the following.

**GENERAL CHARACTERIZATION OF PROTOTYPE AND COMPONENT BEHAVIOR**

The basic problem in the determination of realistic capacities for individual components and the complete structural wall building is depicted in Figures 4 and 5, and is expressed by the relationship between the individual wall moment capacities and the structure overturning moment capacity [2]:

$$M_{ot} = M_1 + M_2 + AI$$

The sensitivity to changes in the coupling force $A$, which in turn affects the axial force state in the structural walls, is depicted in Figure 4 for the push direction only. In this simplified analysis, the floor dead load is assumed to be introduced to the wall flange due to the plank orientation, while the coupling forces are assumed to act through the elastic neutral axis of the
proportional to the effective slab width), the moment capacity in the short wall increases due to the increased axial compression, whereas the long wall shows a significant decrease associated with coupling induced axial tension. Despite the decrease in the long wall capacity, the force couple $A_1$ dominates the structure overturning moment capacity, which increases with increasing coupling force. The predicted force $A$, shown as a dotted line in Figure 4, was evaluated as the sum of the shears developed in the coupling slab system with flexural plastic hinges occurring at all five floors, assuming a participating effective floor width of two planks or 80 inches (2 m). The effective width was based on the ultimate load capacity and observed strains and crack patterns in preliminary coupling slab tests [8]. Note that a decrease in the assumed effective width, and thus in the coupling force $A$, results in an increase in the moment capacity of the long wall. However, the critical shear in this wall is not controlled by the maximum moment capacity at the base of the wall, but by the moment gradient over the height of the wall which in tum depends on the magnitude of the coupling.

Figure 5 shows the complete predicted response for the research building based on a simple rigid-zone model and the assumed coupling force $A$ indicated by a dashed line in Figure 4. The short wall ($M_1$) contributes little to the overall moment capacity of the structure, while the long wall ($M_2$) dominates in the push direction. In both directions the contribution of the force couple $A_1$ constitutes almost half of the overturning or base moment capacity. The same phenomenon of axial load dependence of the long wall is shown in Figure 6 in the form of complete moment curvature relationships for varying coupling forces. Note that in Figure 6 the dead load force application through the floor planks into the flange of the flanged wall system causes a slight dead load offset for the moments in the two loading directions.

In addition to the direct dependence on the effective floor width, the structural wall capacities also depend on the tributary or effective flange width, particularly with the flange in tension. Various recommended design relationships relating the effective width of the flange in tension of a symmetric flanged wall to the flange aspect ratio are depicted in Figure 7. All have been normalized to the total flange length, and modified where required to express the flange length consistently in terms of the dimensions inset in Figure 7. The New Zealand code for masonry design [9] specifies a relatively small effective width, with an upper limit of eight times the flange thickness:

$$\frac{l_e}{l_f} = \frac{h}{6l_f} \leq 8t_f$$

(2)
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Fig. 6—Moment-curvature relationship for the long wall under varying levels of coupling induced axial forces.

Fig. 7—Effective flange width related to building height.
The NEHRP standard for masonry buildings [10] recommends a greater effective width, limited only by the total flange width:

$$\frac{l_e}{l_f} = \frac{h}{3l_f}$$  \hspace{1cm} (3)

Recent research has suggested that both of the above provisions may underestimate the total effective width which can be developed in tension. It has been suggested that, for general concrete or masonry structural walls, the effective width be taken approximately as half the wall height [2]:

$$\frac{l_e}{l_f} = \frac{h}{2l_f} + \frac{t_w}{2}$$  \hspace{1cm} (4)

Tests at UCSD on flanged reinforced concrete masonry walls [11] showed that, even for aspect ratios close to unity, the full flange width can be considered effective at the ultimate limit state, and a trilinear relationship which provides a reasonable upper bound for the effective flange width of:

$$\begin{align*}
\frac{l_e}{l_f} &= \frac{h}{2l_f} & 4 \leq \frac{h}{l_f} \\
\frac{l_e}{l_f} &= \frac{3h}{8l_f} + \frac{1}{2} & 4 \leq \frac{h}{l_f} \leq \frac{4}{3} \\
\frac{l_e}{l_f} &= \frac{5h}{4l_f} & \frac{h}{l_f} \leq \frac{4}{7}
\end{align*}$$  \hspace{1cm} (5)

was recommended [11]. Individual test results from single-story flanged walls [11] and the five-story building (depicted as discrete points in Figure 7) indicate that equations (4) and (5) provide a realistic measure of the effective width which can be engaged in masonry walls with the flange in tension. The flange aspect ratio for the five-story research building was derived as the height to the center of seismic load, or 2/3 of the building height, divided by the length of the flange. Figure 7 shows that significantly different effective widths and component capacities need to be considered in a capacity design for seismic loads than those currently recommended in force based designs of many building codes for general lateral loads or serviceability conditions.

A realistic capacity assessment of the individual structural component mechanisms requires, in addition to the above influences of effective floor width and wall flange contribution, the evaluation of overstrength due to actual
material properties (yield strength and strain hardening), possible confinement
effects, and dynamic amplification effects due to higher modes. Deformation
issues critical to a capacity design approach for the five-story prototype
building under investigation include the available deformation capacity in the
critical structural walls (i.e. their plastic hinge length, their lateral stability, and
their ultimate compression strain), the deformation capacities of the doorway
lintels in both the decoupled nonstructural lintel as well as the confined internal
beam regions, and base sliding of structural walls and the consequences on wall
toe integrity.

To further quantify some of these parameters, results from full-scale structural
component tests on flanged walls and coupled structural walls are evaluated
and compared with analytical simulations in the following.

**COMPONENT TESTS AND ANALYSES**

The principal objective of the five-story full-scale building test was to verify
analysis and design models developed under the TCCMAR program. As part
of this research effort, a nonlinear finite element program [12] was developed
with a special masonry wall element and a precast plank floor element, see
Figure 8, to predict and diagnose the five-story full-scale building test.

The wall element is a 4 to 8 node membrane element in which the
reinforcement is overlaid (smeared or discrete) onto the concrete. The slab
element is a 9 node flat shell element which accounts for reinforced topped
overlays, prestressing with elasto-plastic slip characteristics and unidirectional
voids and associated orthotropic shear and flexural deformations.

![Fig. 8—Finite element idealization of reinforced masonry walls and precast prestressed hollow-core plank floor system](image-url)
In Figures 9 and 10, this analytical model was employed to capture the behavior characteristics of a flanged wall tested at UCSD [11]. Based on reported measured material properties, the analytical load-deflection envelope, see Figure 9, captures the key flanged wall characteristics of significantly different stiffness, capacity, and failure mode in opposite loading directions observed in the flanged wall test. The analysis was based on no tension stiffening, and no corrections for the geometric domain violations in modeling the flange as a membrane element displacement slaved along the connection to the web. This affects in particular the capacity with the flange in compression since with the narrow compression zone the flange reinforcement will contribute additional tension capacity. The same analytical model was subsequently used to study the shear lag or contributing effective width in the flange with the flange in tension shown in Figure 10 for a top displacement of 0.9 inches (23 mm). At this displacement level both experiment and analysis showed that the flange reinforcement had just reached the yield strain in the bottom bedjoint, whereas at a height of 16 inches (408 mm) uniform strain levels of approximately 500 μstrain were recorded. Fairly uniform strain profiles in the flange were recorded for all load levels, and at ultimate the entire flange width contributed with reinforcement at or above yield, see Figure 7. Thus, for seismic capacity design considerations, flanged walls with aspect ratios greater than one or two should be considered fully effective, particularly in the calculation of flexural overstrength. With the flange in compression, the compression zone will be small for any realistic effective width with little influence on stiffness or capacities.

Due to the importance of the coupling forces from the floor system, both experimental and analytical investigations on coupling slab systems with precast prestressed and topped hollow-core plank floors were conducted, see Figures 11 and 12 [8]. Both experimental and analytical strain distributions in the topping reinforcement along one side of the door opening indicated that only the first plank on either side of the walls contributed to the coupling, a finding which was supported by the encountered crack patterns and the calculated ultimate strength of the system [8]. In a similar experiment on cast-in-place concrete coupling slabs [13], the development of a wider effective width was reported, but the use of an effective width equal to the coupling span length was recommended for design, since, due to significant stiffness degradation, large rotations were necessary to develop the ultimate strength under cyclic loads. For the precast plank floor system, stiffness degradation was not severe for drift levels less than 1.0%, so no decrease in effective width is recommended for determining the moment capacity.

It should be noted that, in order to achieve this ductile behavior, special reinforcement with tightly spaced stirrups was required in the door region [8]. Additional measures had to be taken to inhibit brittle shear failure of the 16 inch (406 mm) deep lintel beams spanning the 40 inch (1 m) wide door
Fig. 9—Cyclic load-displacement response of single-story flanged masonry wall (He and Priestley), and finite element analysis

Fig. 10—Experimental and analytical distribution of vertical reinforcing strains across the flange for single-story flanged wall
Fig. 11—Experimental and analytical distribution of topping reinforcement strains, Section A-A

Fig. 12—Experimental and analytical load-displacement curves for the slab coupling test (Seible et al)
opening. Because of the physical restrictions of the concrete masonry units and the narrow wall width (6 inches, 150 mm), specialized diagonal reinforcing schemes [2] were impossible. Instead, the lintels were separated from the adjacent walls by a 1/2 inch (12 mm) fire-rated expansion joint, thus making the lintels nonstructural, but eliminating the danger of their brittle failure, and reducing the total coupling force.

Although the behavior exhibited in the experimental coupling slab test was stable and ductile, the failure mode was very complex, showing evidence of (1) topping delamination, (2) slip of prestressing strands, (3) horizontal shear failure of the hollow-core plank webs, and (4) vertical cracking of the plank adjacent to the wall-slab interface [8]. Additional problems were encountered due to base slip of the north pin support. All these parameters make an analytical model verification extremely difficult, as shown in Figure 12. However, these phenomena were not observed during the five-story building test which makes analytical correlation studies with the above model more appropriate [12]. Following the calibration with the full-scale component tests, the analytical model was then used to analyze the five-story full-scale research building behavior.

**FIVE-STORY FULL-SCALE BUILDING TEST**

The results of the five-story full-scale research building test were evaluated to assess the interaction of the building components as part of a complete structural system with particular attention to the mechanisms discussed above. Figure 13 summarizes the system behavior in terms of the base shear vs. top displacement response, showing the occurrence of key events in the formation of the global collapse mechanism, and also the corresponding analytical response envelopes from the finite element model and a structural component model discussed below. The first visible damage to the structure occurred at low displacement levels when the construction joints between the walls and floor slabs opened up. This was followed by the development of flexural cracks and subsequent yielding in the first story walls and flanges. The slabs developed significant cracks in the coupling (doorway) region at low drift levels, eventually forming the expected flexural hinges at all floor levels. With the formation of the hinge mechanism in the slabs and extensive yielding in the first story wall reinforcement, the structure reached its predetermined global failure mechanism, showing considerable ductile displacement capacity in both directions. The structure achieved a total displacement ductility of approximately 6 in the push direction and 9 in the pull direction (see Figure 13). Near a drift level of 0.5%, the nonstructural lintels engaged with the adjacent walls, and immediately developed diagonal cracks and localized compression failures. The ultimate displacement limit state of the structure was reached with the development of crushing due to flexure in the wall toes, and
Fig. 13—Experimental base shear versus top displacement response of the five-story research building with key events, and finite element analysis.

Fig. 14—Contribution of four primary modes of deformation to the horizontal displacement of floors 1 and 5.
sliding of the first and second story walls at the floor/wall construction joint. Despite large inelastic tensile strains in the vertical reinforcement of the long wall, lateral instability of the slender walls was not encountered.

Both analytical models represented in Figure 13 showed satisfactory agreement with the experimental response. The initial stiffness of the component model was considerably less than the observed stiffness, since the model was based on the cracked section properties near first yield. The finite element model showed excellent agreement in terms of stiffness. In the push direction, both models underestimated the ultimate displacement defined by first crushing in the wall toe, and the component model overestimated the ultimate strength. In the pull direction, both models captured the ultimate strength satisfactorily, though the finite element model featured larger ultimate displacements. It should be noted here that the experiment was terminated in the pull direction due to stroke limits in the actuators and not due to capacity degradation in the building. Discrepancies in the ultimate displacements predicted by the finite element model are related to the analytical definition of crushing failure employed, while the discrepancies in the component model can be attributed to the over-simplified characterization of the moment-axial load interaction.

The changing deformation modes of the long wall with increasing building drift are illustrated in Figure 14. Here the lateral floor displacement at the first and fifth floors are decomposed into four primary modes: (1) shear, (2) flexure, (3) base rotation, (flexural deformation measured across the construction joint), and (4) horizontal sliding. The modes are plotted, as a proportion of their sum, against the total floor displacement. The error in the decomposition (not shown) was typically 5-10%, with extreme values as high as 40% at very low drift levels; at these low levels, the floor displacements were due primarily to the opening of construction joint cracks, and these were only measured at the bottom of the wall. The unmeasured joint opening at the top of the wall accounts for the error which decreased steadily with increasing building drift. Even in view of measurement errors, the development of distinct deformation mechanisms is evident. At low displacements, most of the lateral floor displacement is caused by opening of the floor/slab construction joints. With the development of well distributed flexural cracks in the first two stories (see Figure 15), the influence of flexural deformations grows, and the opening of the construction joint becomes a small part of the total deformation. Likewise, the shear deformations in the first story grow with the increasing number of diagonal cracks there. At the end of the test, horizontal sliding of the walls at the construction joints becomes a dominant displacement component. Because of the isolated inelastic action in the first floor, the displacement at the fifth floor level (Figure 14) is almost entirely due to flexural action in the walls and at the construction joints.

Crack patterns in the first two stories of the long wall at the maximum building
Fig. 15—Interior of crack patterns in first and second story walls (left) long wall flange; (right) long wall webs

Fig. 16—Overview of exterior crack patterns in both loading directions
drift of approximately 1.5% are shown in Figure 15. Numerous horizontal and diagonal cracks formed in the first and second story walls and continued into the flange. Diagonal crack widths remained small throughout the test, with horizontal reinforcing just reaching yield at the maximum displacement limit, so the inelastic shear deformation mechanism in the walls was well controlled. Figure 16 shows an overview of the building crack patterns for each load direction separately. This figure illustrates the continuity of both horizontal and diagonal cracks around the corners of the walls, due to the continuous reinforcement and grout at the wall intersections. The conventional view of an effective flange width (see inset in Figure 7) implies a steadily decreasing band of influence up the building height; however, the continuous diagonal cracks across the full flange width suggest that the tension shift phenomenon present in diagonally cracked, planar structural walls may extend into the plane of the flanges, and that wall vertical reinforcement may have yielded as high as half the structure height. Measured strains approaching yield at the far end of both flanges at the third floor level confirm this point.

As noted above, the nonstructural lintels suffered no damage until a building drift of 0.5% was reached, at which point the local beam rotations were approximately three times the total wall rotations. Because they were so lightly reinforced, damage occurred almost immediately upon contact of the lintel with the walls, thus these members did not increase the strength of the coupling system, and had no influence on the complete structure load-displacement characteristics.

The appearance of full width horizontal cracks across the wall flanges, as illustrated in Figures 15 and 16, suggests that the entire flange participated in the development of the flexural strength of the walls at the base. Measured and analytical reinforcing strains (see Figure 17) indicate the presence of a clear shear lag effect at low drift levels, but at a drift level of 0.17%, a full width crack appeared, and the gages indicate a nearly uniform yield strain across the flange.

Another key parameter in the capacity design of structural walls is the dynamic shear amplification factor. This factor defines the increase in the base shear due to higher mode force distributions over that which would be expected under a first mode force distribution [5]. During the five-story building test, this factor was evaluated continuously by decomposing the seismic lateral load distribution into two orthogonal components representing the first mode distribution, and all higher mode contributions. The ratio of the total base shear to the first mode component base shear represents the amplification factor. In Figure 18, the amplification factors attained at the peak of each displacement cycle are plotted against the total base shear. It is shown that, in its initial stiff state, the structure was subjected to significant higher mode distributions, with amplification factors as high as 1.5; with the increasing
Fig. 17—Experimental and analytical distribution of vertical reinforcing strains across the long wall flange in the 5-story building

Fig. 18—Amplification of base shear due to higher mode force distributions
development of wall flexural plastic hinges and the resulting dominant flexural deformation pattern, the effect of higher modes decreased, with an upper limit of 1.2 as the structure attained its maximum base shear. Analytical studies [5] suggest that, under different seismic input, higher values can be expected.

**STRUCTURAL COMPONENT MODELS FOR DESIGN**

In the above experimental and analytical evaluations, the nonlinear finite element model employed was able to reproduce satisfactorily the load histories and strain distributions in complex structural wall systems, and was thus a useful research tool. For design, simplified models are needed which can capture all essential aspects of structural wall system behavior, while maintaining the simplicity that is essential for application to design. To provide realistic force and deformation assessments, key components of such a model are: (1) Each structural component (story-height wall, coupling beam, joint, etc.) should be modeled by a single element whose geometry is consistent with the true geometric domain. (2) The essential local nonlinear characteristics of the structural components (flexural hinging in walls and coupling elements) should be incorporated in a simple way to allow the prediction of global structural failure modes under static, monotonically increasing design load distributions. (3) Wall elements should be capable of modeling the asymmetric response of flanged walls. (4) Wall elements should account for the changing axial load effects associated with coupling beams or slabs. (5) Coupling beam or slab elements should be capable of modeling the asymmetric flexural behavior of prestressed floor systems such as those discussed above. This implies that the inflection point in coupling beams may not be assumed to occur at midspan. (6) Input parameters for the model should be limited to basic material properties, section definition, and structure geometry readily available to the designer. Empirically derived parameters should be minimized or eliminated. Such a structural component model is currently being developed at UCSD to satisfy these objectives.

**CONCLUSIONS**

The five-story full-scale reinforced concrete masonry research building test under simulated seismic loads showed that, in order to apply general capacity design principles to a prototype structural wall building, parameters such as the effective flange width, the coupling forces from the participating floor area and the shear force amplification due to higher mode effects need to be considered to protect nonductile structural components against inelastic action and brittle failure limit states. For the investigated hollow-core plank systems, the discontinuities between the planks effectively limited the participating width to one floor plank on each side of the door opening. On the other hand, in the
structural walls the full flange width was effective in the five-story full-scale building test, and the tension shift phenomenon continued from the wall into the flange with approximately 45 degree crack patterns, thus extending yield in the wall flange to over half the building height.

Dynamic amplification or higher mode effects encountered in the five-story full-scale building tests showed that for small levels of base shear the amplification can be up to 1.5, whereas for higher levels of base shear the maximum recorded amplification factor was 1.2.

Horizontal displacement contributions of the individual mechanisms at each story showed that first story displacements were dominated by shear and base sliding while upper stories showed predominantly flexural behavior modes. Achieved displacement ductilities of 6 in the push and 9 in the pull direction showed that ductile structural wall systems are possible. The first significant damage encountered was in the lintels at 0.5% drift, but the overall structural response was unaffected due to the nonstructural nature of the lintels.

The analytical models developed for and calibrated with the five-story full-scale building test can be used as valuable diagnostic tools to identify problem areas and to conduct parametric studies, whereas, for design purposes, simplified component models need to be developed which allow easy sequential hinge formation analyses including the influence of changing axial load effects and eccentric flange geometries.

The above evaluations have shown the importance of complete structural systems testing and analysis in determining the complex interaction of building components, and that this interaction can have a significant effect on the component characteristics in a capacity design for seismic loads.

DEDICATION

This paper is dedicated to Professor Emeritus Tom Paulay from the University of Canterbury in Christchurch New Zealand in honor of his seventieth birthday. Professor Paulay has provided continued inspiration to the authors through his pioneering contributions on shear and seismic design of buildings and his visualization of these complex concepts by means of simple engineering models.
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REFERENCES


Errata by the authors of the paper entitled "Deformation and Force Capacity Assessment in Structural Wall Buildings."

Please replace Fig. 2 in the paper with the following:

Fig. 2—Direct load path [in case of doubt, see ref. (Tom Paulay, 1993)]