

RECENT DEVELOPMENTS IN LATERAL FORCE TRANSFER IN BUILDINGS

THOMAS PAULAY
SYMPOSIUM

EDITORS

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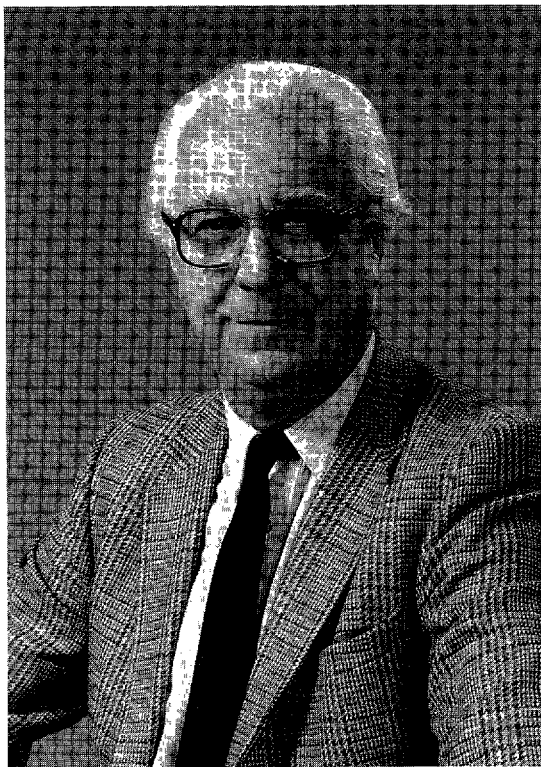
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- President of the International Association for Earthquake Engineering (1993)

PREFACE

Thomas Paulay, Professor Emeritus at the University of Canterbury, New Zealand, is widely known and respected for his deep involvement and contributions to modern seismic design. He has had a long and extremely active career, and his many technical publications and books have had a great impact on seismic design of concrete structures. His research has been recognized with numerous awards and prestigious appointments, including his current position as President of the International Association for Earthquake Engineering, and Honorary Member of the American Concrete Institute.

In 1993, Professor Paulay celebrated his 70th birthday. To recognize this event, and his illustrious career, a special symposium in his honor was organized. Invited addresses on topics related to lateral force transfer in buildings, included in these proceedings, were prepared by invitation by eminent researchers and practitioners who have had personal involvement with Professor Paulay and his research.

The symposium, sponsored by the American Concrete Institute, the Earthquake Engineering Research Institute and the University of California, San Diego, was held September 20-22, 1993, in La Jolla, California. The proceedings are published in this symposium volume.

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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.

R. Park, FACI, is Deputy Vice-Chancellor and a Professor of Civil Engineering at the University of Canterbury, Christchurch, New Zealand. He has teaching and research interests in reinforced and prestressed concrete structures, particularly in the design for earthquake resistance. He is the author or co-author of more than 200 technical papers and two books. He has received 16 awards from North America, United Kingdom and New Zealand for his published papers.

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.

*"Code of Practice for the Design of Concrete Structures, NZS 3101:1982", Standards Association of New Zealand, Wellington, 1982.

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 concrete 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

**Hollings, J P, "Reinforced Concrete Seismic Design", Bulletin of New Zealand National Society for Earthquake Engineering, Vol. 2, No. 3, 1969, pp 217-250.

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, $T\ell > M_1 + M_2$ in Fig. 15(b) and (c), noting that the total overturning moment on the wall is resisted by $M_1 + M_2 + T\ell$) 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.

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- B3 Paulay, T and Priestley, M J N, "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley & Sons, New York, 1992, 767 pp.

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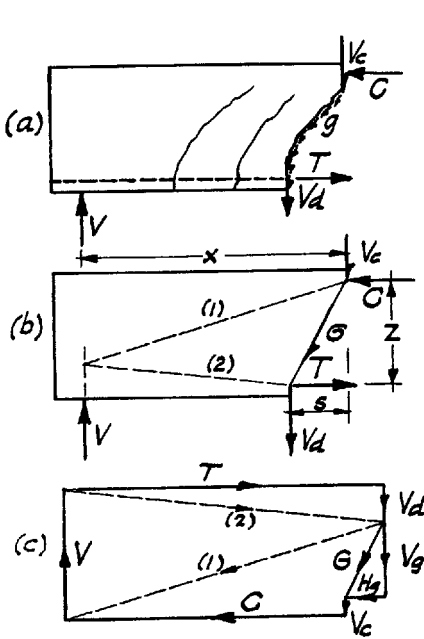


Fig. 1—Equilibrium condition for the shear span of a beam. (a) Free body; (b) and (c) force equilibrium requirements (J1)

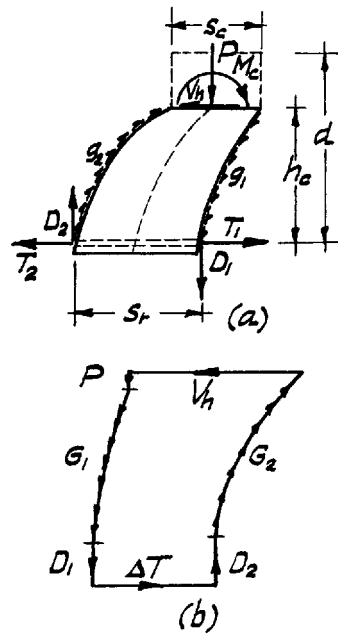


Fig. 2—Action on a concrete cantilever. (a) The forces; (b) force equilibrium requirements (J1)

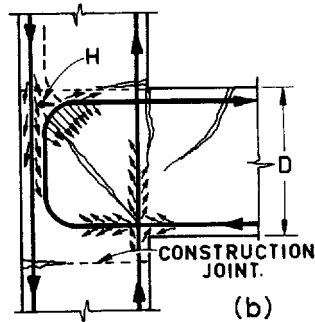
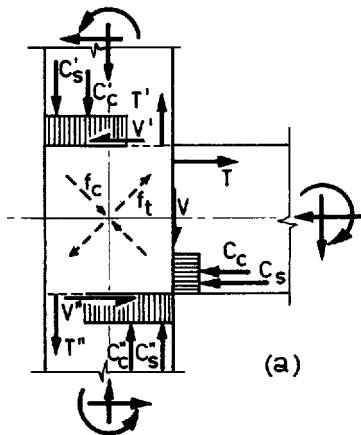


Fig. 3—Force transfer across an exterior beam-column joint. (a) Diagonal tension and compression induced by stress resultants in beam and columns; (b) diagonal tension cracking and bond forces (C1)

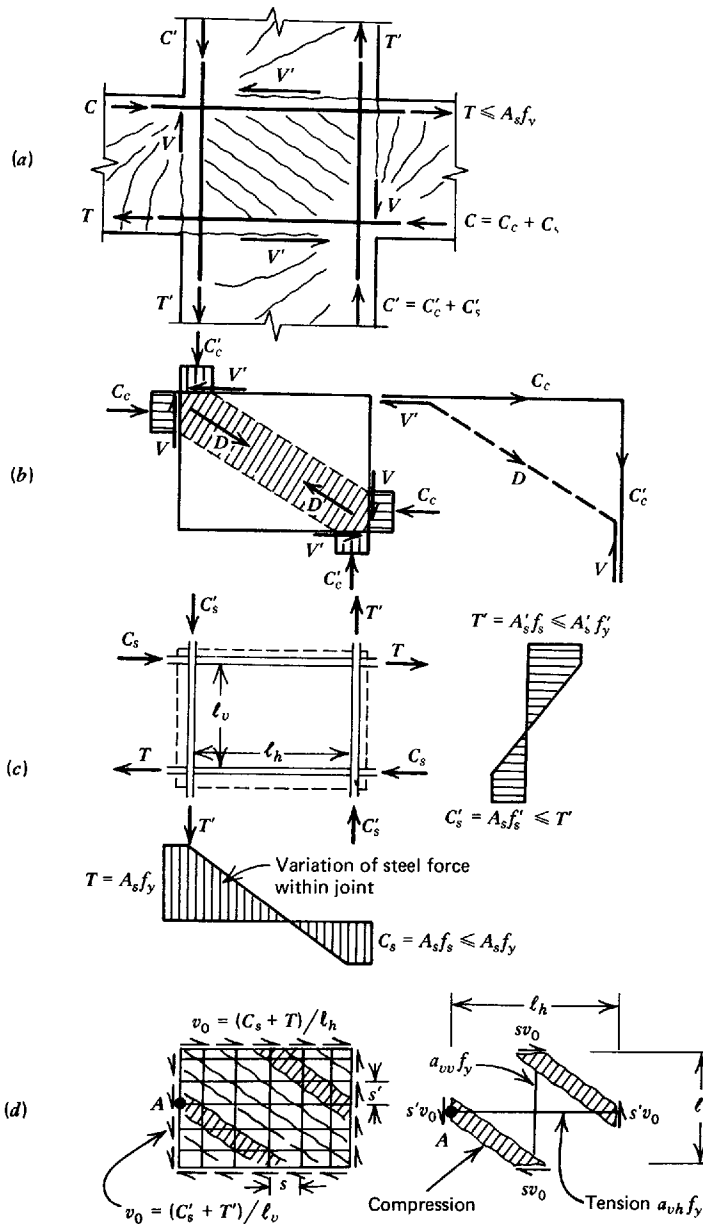


Fig. 4—Idealized behavior of interior beam-column joints (a) Internal actions and crack pattern; (b) shear transfer by diagonal compression strut mechanism; (c) forces in the reinforcement only; and (d) shear transfer by truss mechanism (B1)

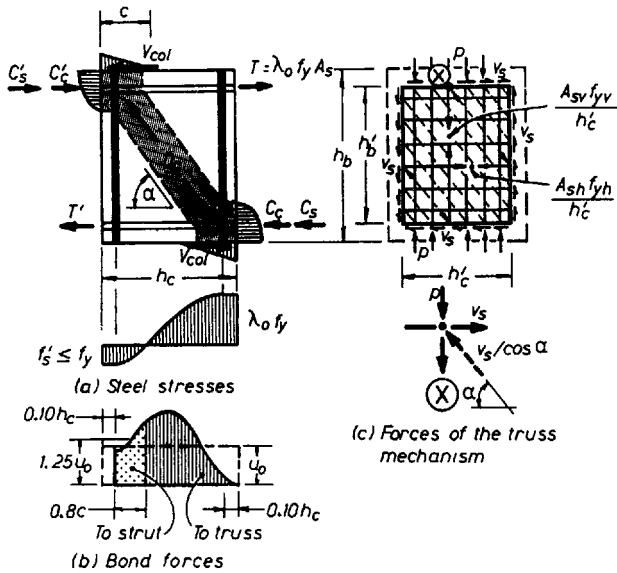


Fig. 5—Principal mechanisms of shear transfer in interior beam-column joint (B3)

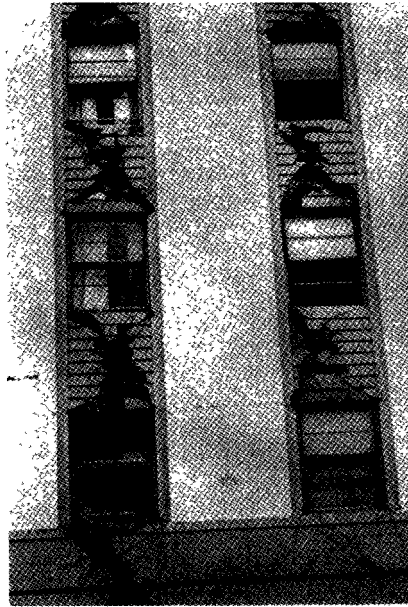


Fig. 6—Damage to coupling beams of structural walls of Mt. McKinley building, Anchorage, Alaska, 1964



Fig. 7—Three-quarter scale reinforced concrete coupling beam in loading frame after testing illustrating diagonal tension cracking (T1)

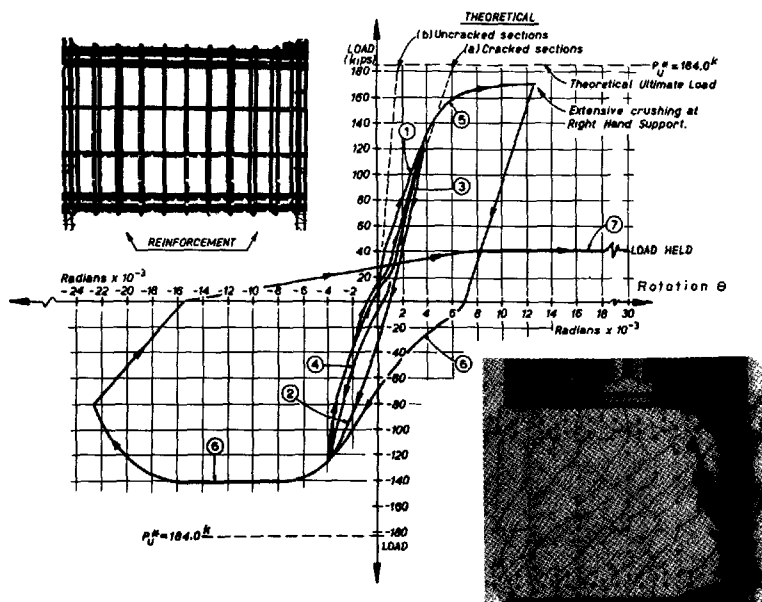


Fig. 8—Load-rotation relationship for coupling beam containing shear reinforcement in excess of that required by truss action and the sliding shear failure that occurred at the end of the test (T1, J7, B1)

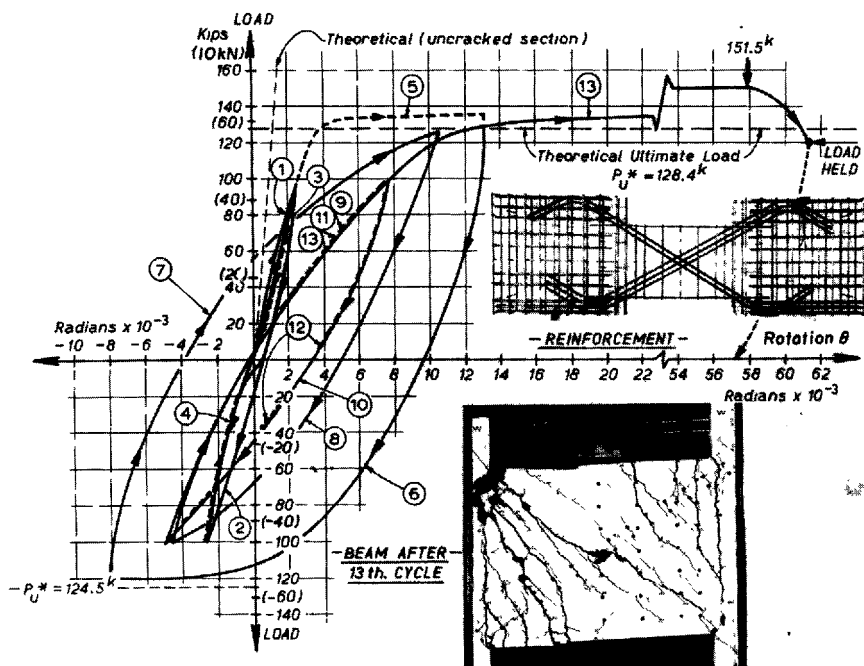


Fig. 9—Load-rotation relationship for diagonally reinforced coupling beam and diagonal tension cracking at end of test (J10, B1)

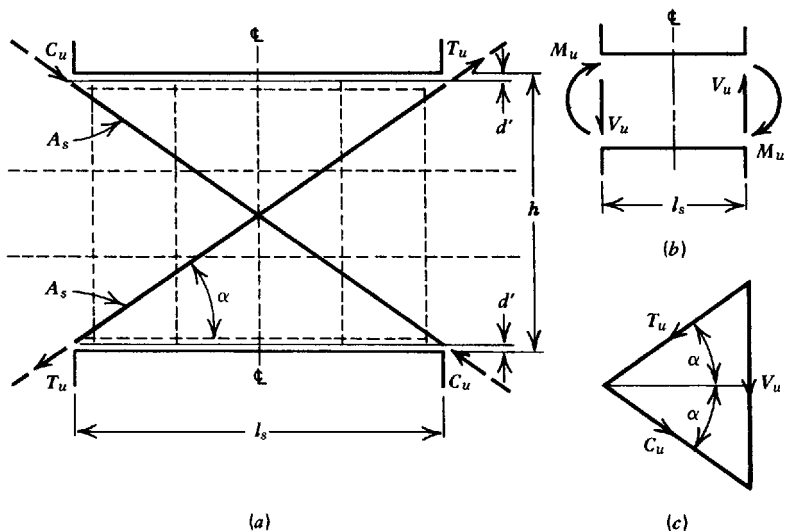


Fig. 10—Model of diagonally reinforced coupling beam. (a) Geometry of reinforcement; (b) external actions; and (c) internal forces (B1)

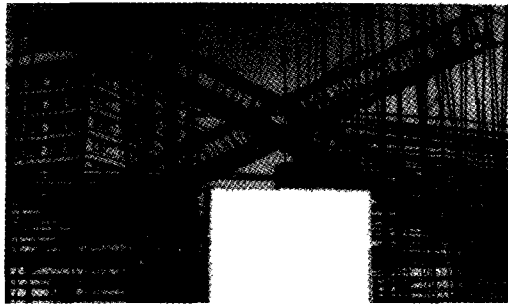


Fig. 11—Diagonal reinforcement for a coupling beam of a structural wall (photo courtesy of New Zealand Ministry of Works)

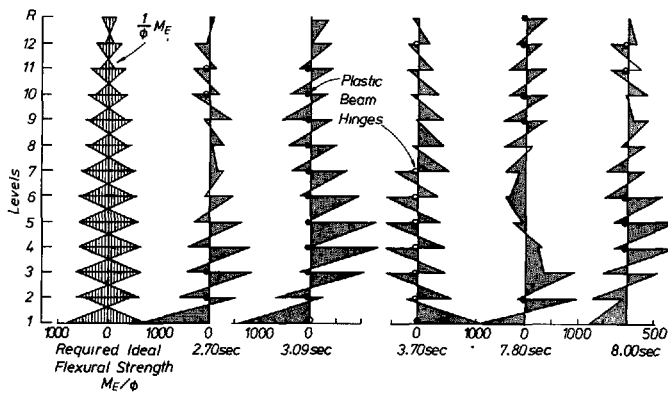


Fig. 12—Comparison of column moments due to horizontal equivalent static and dynamic forces (P13, B3)

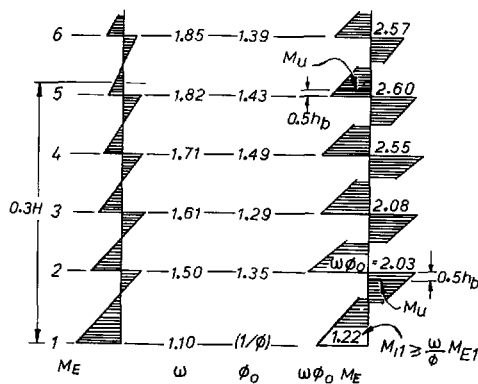


Fig. 13—Moment magnifications for column in lower stories of 15-story building (C6, B3)

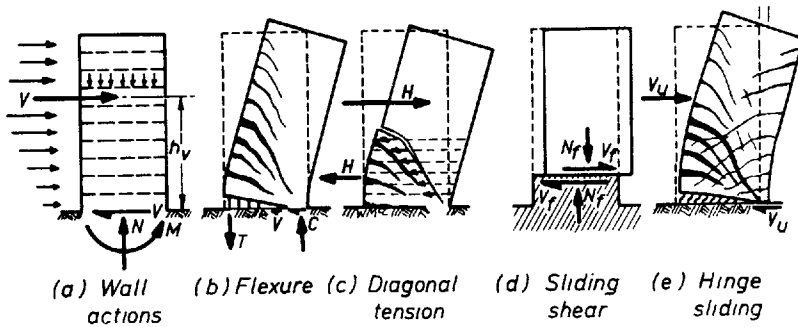


Fig. 14—Failure modes of cantilever structural walls (P6, B3)

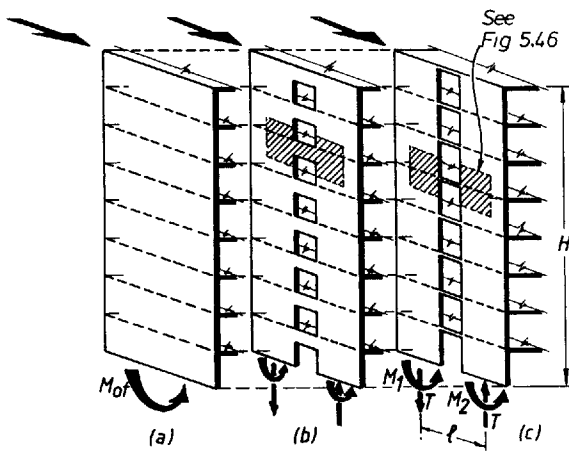


Fig. 15—Flexural resisting mechanisms of structural walls (J29, B3) B3)

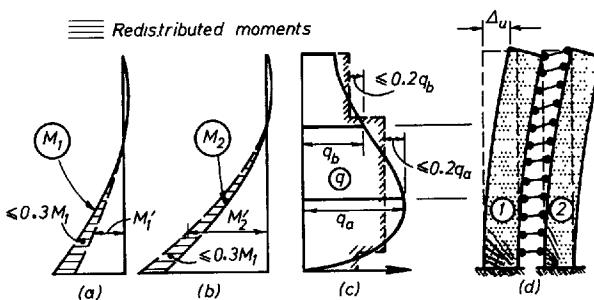


Fig. 16—Ductile response of coupled structural wall (P23, B3)

SP 157-2

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

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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),

$$v_c' = (1.08 - 0.0242f_c')f_c' \quad (1)$$

where v_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.

$$v_c' = (0.65 - 0.014f_c')f_c' \quad (2)$$

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).

$$v_c = (0.5 - 0.01f_c')f_c' + 0.86\sqrt{p_w f_y} \quad (3)$$

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.

$$v_c = (0.78 - 0.016f_c')f_c' + 0.5p_w f_y \quad (4)$$

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. Hirose, 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

$$\tau_{\max} = 1.3 \sqrt{f_c} \quad (5)$$

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 α 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

$$V_{jh} = (A_{s1} + A_{s2}) \alpha f_y - V_c \quad (6)$$

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

$$V_{ch} = D_c \cos \beta = C_c + \Delta T_c - V_c' \quad (7)$$

where ΔT_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 ΔT_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

$$V_{sh} = D_s \cos \beta' = T' + C_s - \Delta T_c \quad (8)$$

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

$$V_{jh} = V_{ch} + V_{sh} = T' + C_c + C_s - V_c' \quad (9)$$

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 ΔT_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 ΔT_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 ΔT_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 ΔT_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 Hognestad 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.

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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.

NZ EFFORT FOR CODE REVISION

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 αf_y 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 αf_y , 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 ΔT_c may be determined by the bond within the column compression zone $0.8c$. The bond stress, u_0 in Fig. 10 (e) may be obtained by dividing the total steel force $(T_1 + C_{s2})$ by $0.6h_c$. Thus ΔT_c will be found by

$$\Delta T_c = 0.8c (T_1 + C_{s2}) / 0.6h_c \quad (10)$$

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