This paper istheresult of deliberations of the Society's discussion group on
SEISMIC DESIGN OF DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES

SECTION A

INTRODUCTION AND PHILOSOPHY

R.J. Burns*

INTRODUCTION

The new loadings code NZS 4203:1976 requires designers to consider 'concurrent effects' and 'capacity design' in the earthquake resistant design of structures. Many engineers have been confused on how to achieve these aims in practice. In May 1976 the Management Committee of the New Zealand National Society for Earthquake Engineering called together a group of engineers to try to resolve the differences in interpretation and to produce a set of design guides which would form the basis of a series of workshops to be held throughout the country. As much of the material produced by this study group is new and perhaps controversial, it was decided to publish the findings as they became available to give other engineers an opportunity to test the proposed methods and to report back to the group. This issue of the Bulletin contains six papers from a set of eleven which deal with the seismic design of ductile moment resisting reinforced concrete frames. The remaining papers will be published in the September or December issues.

The original aim of the Management Committee was to hold workshops in the main centres during 1977 and to cover a variety of structural types at each one. It soon became apparent, however, that this expectation was too optimistic. The scope was then limited to the design of frame structures in reinforced concrete to resist earthquake induced forces. The new loadings code modifies design procedures for this class of structure more than it does for other types. Moreover, it seems that more capital is invested in them than any other type. The Society plans to consider other structural systems such as concrete shear wall buildings and structural steel frames, and study groups will be brought together for this purpose at a later date.

Emphasis has changed from holding workshops to publishing findings of the study group. This will expedite the distribution of material to a greater number of designers. It is proposed that workshops be held in 1978 following receipt of comments from designers, and these will probably take the form of a step by step analysis and design of typical frame buildings.

The Standards Association of New Zealand is at present drafting a new concrete design code (NZS 3101), which should be issued in draft form for comment in September or October of this year. Four members of the study group are also members of the SANZ committee and we understand that many of the study group's recommendations will be incorporated in the draft code.

PERSONNEL

An attempt was made to obtain a balanced representation of interests within the group, which at the present time, consists of the following members:-

Prof. R. Park (Chairman)	Univ. of Canterbury
Prof. T. Paulay	" " "
Dr. M.J.N. Priestley	" " "
Dr. D. G. Elms (Co Chairman)	" " "
Dr. H. M. Irvine	Univ. of Auckland
Dr. R.W.G. Blakeley	Min. of Works & Dev.
Dr. C. D. Mathewson	" " " " "
Messrs. O. A. Glogau	" " " " "
G. H. McKenzie	" " " " "
B. W. Buchanan	" " " " "
R. Williams	" " " " "
H. E. Chapman	" " " " "
N. W. Allardice	Consulting Engineer
A. L. Andrews	" "
J. R. Binney	" "
J. P. Hollings	" "
R. A. Poole	" "
G. K. Sidwell	" "
I. C. Smith	" "
K. E. Williamson	" "
R. J. Burns (Group Secretary)	" "

PAPERS

The study of the seismic design of reinforced concrete frame buildings was divided into eleven sections as follows :-

Section	Title	Author
A	Introduction and Philosophy	R. J. Burns
B	Analysis	A. L. Andrews
C	Analysis for Torsion	R. A. Poole
D	Foundations	Subcommittee
E	Beam Flexure	I. C. Smith & G. K. Sidwell
F	Shear Strength Requirements	T. Paulay
G	Columns - Evaluations of Actions	T. Paulay
H	Columns Subjected to Flexure and Axial Load	R. Park
J	Beam-Column Joint Design	R.W.G. Blakeley
K	Parts and Secondary Elements	N. W. Allardice
L	Low Ductility Frames	K. W. Williamson

Each section, except sections A and B,

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is written in two parts: the first a recommended method for analysis or design; and the second a commentary containing background information, explanations of methods used and similar data.

Before any section is published it is discussed by the group and amended where necessary until general agreement is reached. Controversial issues were debated and changed until an acceptable solution (to most) was found.

Sections B C D J and L are still under discussion.

PHILOSOPHY

The purpose of this series of papers is to describe a practical method of satisfying the seismic provisions of NZS 4203 as they apply to framed structures in reinforced concrete. Information from observations of the behaviour of buildings in intense earthquakes and from research studies is continuously accumulating. The study group has endeavoured to use the most up to date information available in the design scheme it has devised.

The following notes apply to reinforced concrete frames designed for flexural yielding.

Three basic requirements of the new loadings code are:-

i) that ductility demand be recognised and members designed accordingly,
ii) that concurrent biaxial earthquake excitation be considered, and
iii) that capacity design principles apply.

a) Ductility

Ductility requirements were recognised in our previous loadings code, NZS 1900, Chapter 8 : 1965. Since 1971 Appendix A of ACI 318-71 has been widely used and deemed to satisfy these early ductility requirements. The draft concrete design code will replace this appendix with an updated version better suited to New Zealand needs. Methods of designing and detailing for ductility are contained in the design sections of this series and we understand that these are similar to corresponding sections in the new concrete design code.

The study group believe that potential hinge zones at the ends of all columns require detailing for ductility. Capacity design procedures reduce the probability of hinges forming in columns but yield strains and hinging may occur in these regions for a short period during a severe earthquake.

b) Concurrent Effects

It is recognised that conventional code earthquake simulating loads as applied to framed buildings represent only a portion of the inertia which would be generated in those buildings in a major earthquake were the buildings to respond elastically. Post elastic yield behaviour can be expected to control the development of inertia forces by dissipating energy inelastically rather than storing it. A skew earthquake, at some angle to either principal axes of a building, could develop post-elastic response in frames parallel to each principal axis.

Skew attack has little effect on beam design but has a significant effect on any column which is part of a longitudinal and a transverse frame. Such a column must be capable of resisting moments in both directions concurrently together with axial load resulting from various combinations of gravity loads and shears induced by the formation of hinges in beams framing into columns from both directions.

Since acceptance of this principal in the early 1970's columns of two-way frames and corner columns of perimeter framed structures have been designed for biaxial loading. This presented a major design task to engineers who had no access to a suitably programmed computer. It is proposed in Section G that two-way columns be designed for uniaxial bending about both axes separately with design moments factored to allow for possible concurrent action. The possibility of maximum moments occurring at the same section, about both axes simultaneously is remote and this has been taken into consideration. A dynamic magnification factor, ω, is introduced to allow for concurrent effects and also for higher mode effects which cause the column bending moment pattern to depart from that obtained from analysis with a static earthquake simulating load.

c) Capacity Design

An elastic analysis of a frame (either a dynamic analysis or a static analysis using the lateral code load), will give a reasonable description of actions within the frame up to the point of first yielding. From commencement of yielding, actions on the frame and the distribution of internal forces vary significantly from those indicated by the elastic model. This has been verified from studies of buildings damaged in the San Fernando earthquake of 1971 and also by numerous computer studies of model buildings. Therefore, to prevent collapse when ductile yielding occurs, the structure must be capable of resisting these unknown and varied forces.

The purpose of capacity design is to tell the structure how to behave when it must deflect well beyond limits set by the elastic properties of the frame. A hierarchy in the development of energy dissipating mechanisms is established to minimise the likelihood of brittle failure in members and the formation of storey collapse mechanisms.

Flexural yielding with consequent hinge formation should ideally be restricted to locations in framing beams at or near column faces. These hinges are required to have capacity to dissipate energy throughout repeated cycles of alternate straining at amplitudes of many times yield strain. Columns should be strong enough to remain essentially elastic as beam plasticity developes fully. Reasonable precautions taken to ensure that column strength has sufficient margin in hand to allow beam hinges to form may not prevent limited post yield strain from occurring in columns; but an acceptable design procedure should control the strains and prevent full hinge development. In particular it is essential to guard against simultaneous formation of hinges at the top and bottom of every column in any storey of any building of more than one or two storeys. There are locations in the columns of most framed structures where hinge formation is either unavoidable (as at the springing of columns from flexurally stiff restraints at foundations) or unobjectionable (as at the soffits of roof beams). When column hinging is contemplated, suitable detailing is needed.

Restricting the potential for column hinge formation dictates the use in design of moments that are significantly larger than column moments found in analyses. Appropriate factors to modify analysis moments so that the design accounts for all reasonably foreseeable moment augmenting influences are suggested in this series of papers.

CONCLUSION

Ductile reinforced concrete frame design is now far more complex and time consuming than it was. There is now better understanding of the requirements of frames to resist severe earthquakes, and current codes insist that this knowledge be applied in practice. Application of capacity design principles is the single biggest change but in almost every other aspect of design there have been advances and refinements to established techniques : beam-column joints require careful design and detailing to prevent bond and shear failures, greater attention must be given to parts and secondary elements, analysis for torsion is more 'refined', and individual members must be carefully designed for possible ductile yielding.

It is hoped that this series of papers fulfills a need with practising designers - to understand and implement the philosophies of the new loading code and also to meet specific requirements of the proposed new concrete design code, in so far as these codes relate to ductile reinforced concrete frames.

At this stage the study group and papers produced by members of the group do not have official backing by SANZ. However, membership of the study group includes some who served on the committee responsible for NZS 4203, and others are at present preparing the concrete design code.

The Management Committee of the Society is grateful to members of the study group for the time and effort they have made available in producing this series.

NOTATION

Symbols used in these papers are those defined in Appendix B, Notation, of ACI 318-71, and also clause 1.1.4 of NZS 4203 : 1976. Additional terms were found to be necessary, particularly for capacity design considerations and these are listed below.

Where other symbols are used, these are defined immediately adjacent to the formulae or diagram where they first appear.

d_{co} = Dimension of column core in direction of load.

b_{co} = Dimension of column core perpendicular to direction of load.

k = Relative stiffness of member.

M_{code} = Moment at section derived from analysis for code loading (V_{code} similar).

n = No. of beam levels above section considered.

n = No. of sets of ties over specified distance.

n_j = No. of sets of ties within joint.

P_{eq} = Axial load due to earthquake loading only.

S_i = Ideal or nominal strength, $M_i = \frac{M \text{ dependable}}{\phi}$

S_o = Overstrength ($S_o = \psi o\ S_i$)

T_n = Period of nth mode of vibration.

T = Tensile force in reinforcing steel.

V_c = Shear force carried by concrete.

V_e = Shear force in member due to earthquake loading only.

V_j = Design shear force within a joint.

V_{oe} = Earthquake induced beam shear at development of flexural overstrength.

V_s = Shear force carried by web reinforcing.

ω (omega) = Dynamic magnification factor

θ (theta) = Used as subscript for angle to a defined axis e.g. M_θ.

ψo (psi nought) = Overstrength factor (>1.0).

λ (labda) = $\frac{P_{eq}}{\Sigma V_{oe}}$