PROGRESS REPORT OF EARTHQUAKE ENGINEERING RESEARCH AT THE UNIVERSITY OF CANTERBURY

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SUMMARY

This papers reviews aspects of earthquake engineering research conducted in the Department of Civil Engineering of the University of Canterbury, New Zealand, during approximately the last four years. This work generally falls into the following categories: earthquake code factors and ground motions, dynamic analyses of structures responding elastically and inelastically to seismic ground motions, experimental and analytical investigations of the behaviour and design of structural concrete elements and sub-assemblages subjected to earthquake type loading, and experimental and analytical investigations of earth retaining structures.

INTRODUCTION

The Department of Civil Engineering of the University of Canterbury, New Zealand, has continued its earthquake engineering research during the period since the Sixth World Conference on Earthquake Engineering. This progress report will outline the work which has been completed since early 1977, and mention will be made of work which is currently in progress. The work will be reported under four headings: code factors and ground motions, dynamic analyses, behaviour and design of structural concrete, and earth retaining structures. A list of departmental research reports and published papers covering this work in more detail is given at the end of the paper under the above headings.

CODE FACTORS AND GROUND MOTIONS

Code Factors

The determination of code factors involves two main problems: obtaining an overall level, and deriving relative values between factors.

The overall level of the New Zealand seismic code requirements has been investigated (1.1, 1.5) by deriving the total expected cost (including all costs of failure) of reinforced concrete ductile frame buildings for different nominal design levels. It was found that the cost was very insensitive to the base shear level chosen, and that of far greater influence on cost was the decision as to whether or not to require adequate design detailing for earthquakes.

A rational approach to the determination of the correct relative values of load factors has been developed based on the classical economic problem of the optimal distribution of a fixed amount of a resource. First, a general approach was developed which balances initial costs, failure costs and probabilities of failure for structures with a finite number of failure modes

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and for which money could be spent on safety in a discrete number of ways (1.7). The technique was applied to the derivation of code structural type factors. Subsequently, the same basic method is being used, in a considerably modified form involving the use of reliability indices, for the derivation of risk or importance factors (1.9). Examination of a particular case showed that the appropriate risk factor for an essential facility whose cost consequences of failure was 1000 times that of a normal building should be about 1.7.

Earthquake Ground Motions

A number of very strong accelerograms have been studied (1.4, 1.6) in the light of earthquake source theory to obtain an understanding of their characteristics and to determine the effectiveness of simple source models in predicting strong, near field motions. A suite of strong accelerograms has been selected and used to study the behaviour of building frames and bridge piers during time-history analyses under near "maximum credible" shaking (2.4, 2.20, 3.4).

Design ground motions for New Zealand are currently being estimated using both conventional empirical techniques and a more physically based approach through source and attenuation models (1.2). As well as suggesting code revisions, this work is providing input to a broad study of measures to reduce the risk of earthquake damage to Wellington city (1.3).

Earthquake engineering in New Zealand is hampered by a lack of records of local strong ground shaking. The effectiveness of the New Zealand accelerograph network has been examined (1.8) and the University of Canterbury is installing accelerographs in some of the more active regions not covered by the present network.

DYNAMIC ANALYSES

General Approach

Work has continued during the last few years on the development of a computer program for large scale dynamic analyses of inelastic frame and shear wall structures. The program developed integrates the dynamic time-history response of the structure to an arbitrary earthquake acceleration utilizing one of the Newmark family of time-wise integration of the equations of motion. Several different inelastic frame member type models are incorporated in the program together with a variety of choices for the viscous damping model (2.4, 2.5, 2.10, 2.13, 2.15).

One of the recent extensions of the program capability was the development of a two-node finite element model suitable for the analysis of shear wall type members. This model numerically integrates the wall stiffness properties across the wall section as well as along the member axis. The behaviour of inelastic coupled shear wall structures (2.3) has been studied and now work is continuing into the behaviour of combined frame and shear wall structures (2.7).

In the analysis of many inelastic framed structures little attention has been given to the effects of the choice of the damping model used. It

has been observed that a poor choice of model and/or damping parameters, as they affect the higher modes of free vibration, can have a pronounced effect on the computed inelastic response of the structure (2.8). This choice of damping characteristics also tends to have a marked effect on the response of structures free to rock on their foundations and in particular on the behaviour after the impacts between the structure and the ground (2.9).

These same inelastic analyses have been extended to allow for the effects of large displacements on the response of the structure. The analyses may range from a full nonlinear geometric analysis to a simplified P-Δ analysis suitable for tall regular framed structures.

Elastic finite element models suitable for shells and other three-dimensional space structures have been used for the dynamic analyses of bridge structures. The analyses have been carried out with a view to correlating the analytical and experimental results of vibration studies of prototype bridges (2.6) and this work is continuing. Further, as a continuation of earlier work on soil-structure interaction, two-dimensional finite element models have been applied to identify some of the effects of valley topology on the likely seismic behaviour of bridge sites (2.12).

Column Design Actions

The previously developed dynamic analysis program was used to study the inelastic response of a number of ductile frames to different selected earthquake motions. The frames were designed to possess high (2.4) and low (2.10) lateral load resistance, corresponding with New Zealand seismic code and zoning requirements. All frames were proportioned in accordance with "capacity design" procedures, described later in this paper. The analyses verified that the intended frame behaviour can be attained (2.19, 2.21). For 12 and 18 storey frames column hinging was restricted to the column bases, even during the largest (Pacoima Dam 1971 and Parkfield 1966) input motions. For 6 storey frames some yielding in upper storey columns was encountered. It was found that the ductility demands in plastic hinges of beams and columns were within the capabilities obtained experimentally. Some of the continuing work is applying a probabilistic approach to the determination of the design actions for columns using dynamic analyses and capacity design procedure (2.11).

In some of the analyses the influence of P-Δ effects (2.17) was also studied. For 6 storey frames P-Δ consideration affected insignificantly the response, even for the Pacoima Dam excitation (2.10). For 12 and 18 storey frames no significant change in response was observed as long as storey drift did not exceed 1/100 of storey height (El Centro 1940 record). For larger (Pacoima and Parkfield records) inelastic drifts, however, ductility demands were found to increase by as much as 50%. It was concluded that only in exceptional cases do P-Δ effects need to be considered in design.

Base Isolation and Rocking Structures

The behaviour of structures designed to limit seismic forces by base isolation or rocking has been studied in a number of projects (2.1, 2.2, 2.9, 2.13, 2.15, 2.16, 2.18, 2.22).

The fundamental action of base isolation is to lengthen the natural period of vibration of the structure and to provide additional damping by separating the main structural mass from the supporting system by means of the base isolation system. Analyses (2.16, 2.22) have shown that the most important aspect was the lengthening of the natural period. Substantial reduction in seismic forces (but with a corresponding increase in seismic displacements) results if the natural period is shifted to a region of lower response, as will generally be the case. However, if the response spectrum of the earthquake under consideration is such that considerable energy exists in the long period range (for example, Bucarest 1977), then the increase in natural period associated with base isolation may shift the structure into a region of high response, resulting in an amplification rather than a reduction in seismic forces. Analyses of base isolated shear wall structures (2.1, 2.13, 2.15) have shown that shear force and moment distributions were greatly influenced by higher mode effects. These were difficult to predict by hand analyses, and at present it would appear that dynamic inelastic time-history analyses is essential to provide realistic design data for a given structure.

Theoretical and experimental studies (2.2, 2.9, 2.18, 2.22) have investigated the response of structures free to rock on their foundations. Although it is generally tall slender structures such as elevated water towers that may be expected to rock, analyses (2.18) indicate that more squat structural elements, such as shear walls in buildings and some bridge piers, may also rock under seismic attack. Seismic forces are limited by the rocking action, and failure rarely occurs as overturning moments exceed gravity restoring moments for only short periods of time. The research has been directed towards developing an appropriate inelastic time-history analytical method, and also a simplified hand method, to predict rocking response. Predictions have been correlated with results of shake-table testing. Influence of foundation flexibility and partial elastic restraint of rocking have been considered.

Storage Tanks

The seismic response of cylindrical tanks containing fluids of low viscosity has been studied theoretically and experimentally (2.14). Theoretical analyses of surface fluctuations, forces and moments were based on a full mathematical description of the fluid, and avoided incon istent assumptions made in some earlier research. Predictions were compared with experimental work from shake-table tests, with good correlation being obtained. The analyses are currently being extended to deal with fluids of high viscosity.

Bridges

Theoretical analyses have been carried out on the seismic response of bridge piers (2.16, 2.20, 3.4) using dynamic inelastic time-history analyses. Ref. 2.16 compares the response of alternative designs, with and without base isolation, for a planned bridge. Refs. 2.20 and 3.4 present results of a series of analyses designed to investigate the significance of foundation flexibility on the seismic response (particularly ductility demand) of bridge piers. Bridge piers with a range of natural periods were supported on foundations of variable flexibility and subjected to a range of earthquake

motions. The results indicated that foundation flexibility only increased the curvature ductility demand of short period structures.

BEHAVIOUR AND DESIGN OF STRUCTURAL CONCRETE

A summary of the research work which has been conducted on the behaviour and design of structural concrete up to early 1977 has been published (3.21). More recent research work is described below. Many of the design procedures evolved have been implemented in the new draft New Zealand Concrete Design Code. A short summary of these code developments in structural concrete design is given elsewhere (3.38). The emphasis on concrete has been because this is the predominant structural building material used in New Zealand.

Structural Design Philosophy

To ensure the survival of major earthquakes by ductile reinforced concrete frames it is essential that undesirable failure mechanisms are avoided. Therefore a procedure has been developed in New Zealand, referred to as "capacity design", in which energy dissipating elements (plastic hinges) are chosen while other parts of a frame are given sufficient strength to ensure that only the chosen and specially detailed mechanisms will be utilized (3.20, 3.21, 3.25). First the moments and forces on frames are derived for an equivalent lateral static load, using elastic analysis. Subsequently significant moment redistribution is used in determining the minimum beam strength at each floor. Next the flexural overstrengths that might be developed at the chosen beam plastic hinges, as detailed, are evaluated and these are considered as moments applied to columns. To allow for inelastic dynamic response the beam input moments at overstrength are further magnified. Similarly,maximum earthquake induced beam shear forces are summed over the height to determine the maximum likely axial load. From an estimate of the maximum likely moment gradient the shear forces across the columns are evaluated.

It is claimed that in frames so designed by the capacity design procedure the possibility of sidesway mechanisms developing by the formation of plastic hinges in the top and bottom of the columns of one storey during the largest seismic excitation is eliminated, and the likelihood of column hinging in the upper storey is minimized (3.28). Columns of tall frames so designed require full confining reinforcement at the base of the lower storey columns where plastic hinge formation with significant ductility demand cannot be avoided. At upper floors column confinement can be reduced because very limited yielding, if any, is to be expected. Controlled plastic hinging in columns may be acceptable in low rise frames in which gravity loads dominate (3.32).

Sufficient research has now been conducted into the behaviour of prestressed concrete subjected to seismic type loading (3.23, 3.33) to enable seismic design provisions for prestressed concrete frames to be developed. These provisions are being included in the new draft New Zealand Concrete Design Code. This research work has involved both experimental and analytical studies of prestressed and partially prestressed concrete beam-column subassemblages. Partially prestressed concrete is seen to have excellent potential for use in earthquake resistant frames. The use of prestressing tendons enables precast elements to be post-tensioned together

to form continuous frames and gravity loads to be balanced, and the presence of nonprestressed reinforcement increases the ductility and energy dissipation of the plastic hinge regions.

Research has also been conducted into seismic design procedures for bridge supporting structures (3.36, 3.39, 3.41). Capacity design procedures to ensure ductile behaviour of bridge piers have been considered and recommendations for detailing potential plastic hinge regions have been made.

Stress-Strain Behaviour of Steel and Concrete

Cyclic load tests have been conducted on typical samples of reinforcing bar and prestressing steel used in New Zealand to obtain data on which to model the stress-strain curves (3.22, 3.34, 3.37). A modified form of the Ramberg-Osgood function was fitted to the experimental results to follow the cyclic stress-strain behaviour in the plastic range. The empirical constants in the function were found to depend mainly on the strain imposed in the previous loading run and on the characteristics of the monotonic stress-strain curve. The proposed cyclic stress strain models were found to predict the experimental curves for Grade 275 reinforcing steel and high strength prestressing steel with good accuracy. The moment-curvature characteristics of cyclically loaded structural concrete members are strongly dependent on the steel stress-strain characteristics.

The installation at the University of Canterbury in 1978 of a 10 MN DARTEC electro-hydraulic universal testing machine, with dynamic loading capability and sophisticated load and displacement control, (3.18) has enabled the testing of large scale specimens of confined concrete. Recently tests have been conducted on reinforced concrete column elements confined either by spiral reinforcement (3.13) or various arrangements of overlapping rectangular hoop steel (3.8). The specimens have typically a cross section dimension of about 0.5 m. The test results have enabled stress-strain models for confined concrete to be refined and information to be obtained on the effect of strain gradient across the section, loading rate, and cyclic loading. Previously used models for the stress-strain behaviour of confined concrete have been based on limited test data (3.21, 3.29, 3.39). Well confined concrete shows a significant enhancement of strength and ductility. The useful limit of concrete compressive strain appears to be the stage at which the transverse reinforcement fractures due to the lateral pressure from the concrete. The main variables involved in defining the complete concrete stress-strain curve are the spacing, size, arrangement and yield strength of the transverse steel, the arrangement of longitudinal steel, and the strength of the concrete. Rate of loading and strain gradient can also be significant variables.

Fibre reinforced concrete may have a useful application in the plastic hinge regions of beams and columns. Compression tests conducted on fibre reinforced concrete cylinders, containing various quantities of enlarged-end steel fibres, have shown that the fibres result in a less steep "falling branch" of the stress-strain curve for the concrete (3.9). Thus the fibres improve the ductility of the concrete. The possible use of fibre concrete, with lesser quantities of conventional transverse reinforcement, in plastic hinge regions is being examined (3.9).

Columns and Piers

The amount and distribution of transverse reinforcement necessary to ensure adequate curvature ductility in the potential plastic hinge regions of reinforced concrete building columns and bridge piers is still a matter of some controversy. Significant differences exist between the recommended quantities of confining steel in the design codes of various countries. A commonly used design criteria has been to provide sufficient transverse confining steel to ensure that the axial load strength of the column is maintained after the concrete cover has spalled away. Research has been conducted at the University of Canterbury using a more logical approach which aims at providing sufficient transverse confining steel to maintain a satisfactory moment-curvature relation during earthquake loading (3.17, 3.21, 3.36, 3.39, 3.41).

Analytical moment-curvature studies have been conducted using idealised stress-strain curves for concrete confined by circular spirals which took into account the increased strength and ductility due to the spiral (3.29). This work concluded that the required content of spiral steel was a function of the level of compressive load on the column, and that high load levels required high spiral steel contents. This general conclusion is similar to that determined earlier for columns with rectangular hoop steel (3.21). Code recommendations for confining steel for adequate ductility were made on the basis of these analyses (3.17).

Experimental work had been conducted on a range of small scale spiral columns (3.4, 3.36) and the results had shown reasonable agreement with the trends predicted analytically. More recently, with the installation of the 10 MN DARTEC testing machine, tests on near full scale columns have been possible. Four spiral columns and four columns with various arrangements of rectangular hoops (3.6, 3.7) have been tested under simulated seismic loading. These columns were subjected to axial load at various levels and to cyclic horizontal loading causing flexure up to displacement ductility factors of at least 6. The columns were designed according to the previously made recommendations (3.17). The tests showed that the confinement necessary in the plastic hinge regions of columns with high load levels enhances the concrete strength and ductility significantly and causes the plastic hinge region to spread along the column. Hence for heavily loaded columns a longer length of confined column is necessary than for lightly loaded columns. When failure outside the confined plastic hinge region did not occur in the tests, the columns demonstrated very stable horizontal load-displacement hysteresis loops and very little strength degradation during cyclic loading. Very high concrete compressive strains were measured on the surface of the confined core concrete (up to 0.074 for the columns with spirals and up to 0.026 for the columns with rectangular hoops). The flexural strength of the columns, particularly of the columns with high axial load, was significantly greater than that calculated using normal code assumptions for the concrete compressive stress block. However, calculations which took into account the complete stress-strain curve for confined concrete, and the effect of strain hardening of the longitudinal reinforcement, predicted the measured maximum flexural strength of the columns with very good accuracy.

Beam-Column Joints

The critical aspects of the behaviour of reinforced concrete beam-

column joint cores in earthquake resisting ductile frames are those associated with anchorage and shear (3.21, 3.35, 3.38). In design the possibility of bond failure in the joint core can be reduced by limiting the diameter of the longitudinal bars in the beams and columns to a certain proportion of the column and beam depths (3.11, 3.24, 3.30, 3.35).

As a result of test results and analysis (3.1, 3.3, 3.9, 3.11, 3.30, 3.35, 3.42) it has been postulated that the shear force acting on a beam-column joint core is resisted partly by a concrete compression strut which acts between diagonally opposite corners of the joint core and partly by a truss mechanism formed by the reinforcement in the joint core and the concrete. It has been observed that when plastic hinges develop adjacent to the joint core during cyclic loading the contribution of the concrete diagonal compression strut to the joint core shear resistance diminishes. This is because full depth cracks exist in the beams at the column faces and also because yield of beam steel penetrates into the joint core. Hence significant shear force cannot be introduced at the corners of the joint core. It has been found that unless the column carries a significant compressive load, after a few large load reversals the shear resistance of the joint core is provided mainly by the truss mechanism. It has been shown that shear resistance by the truss mechanism requires both horizontal and vertical shear reinforcement (3.11, 3.35, 3.42). This shear reinforcement can be provided by horizontal ties and by vertical column bars around the perimeter of the column section, passing through the joint core. Horizontal shear reinforcement alone does not provide effective shear reinforcement unless the axial compression on the column is high. Hence for most frames, columns with four (corner) bars only should not be used.

Tests have also shown that the shear strength of a beam-column joint core is much improved by the presence of a prestressing tendon passing through the joint core in the mid-depth region of the beam (3.24, 3.30, 3.33) Where such prestressing tendons exist they can be assumed to carry some of the horizontal shear stress across the joint core. A recent test (3.9) has also demonstrated that the presence of steel fibres in the joint core concrete can aid the transfer of joint core shear by providing some control of diagonal tension cracking.

A beam-column joint assembly, in which beams existed in two directions at right angles, has been tested (3.11) with simulated earthquake loading applied concurrently in both principal directions. The joint core had been designed for loading applied to the beams in one direction only, but it was found to possess sufficient strength for concurrent loading, no doubt due to the confining action of the beams on the four sides of the joint core. On the basis of this test result it is recommended that only plane frame actions (that is from loading in only one direction at a time) need to be considered in the design of space frame joints.

An attractive approach to beam-column joint design is to use suitable bar curtailment or haunching of beams to enforce the plastic hinges in the beams to form away from the column faces. If the beam bars do not yield at the column faces the bond conditions in the joint core are much less severe, and the concrete diagonal compression strut across the joint core can continue to transfer shear throughout the loading reversals. Tests have verified the improved shear resistance for such a design (3.3),which can

lead to a significant reduction in the required joint core shear reinforcement.

Shear in Plastic Hinge Regions

The shear strength of plastic hinge regions, and the effect of shear displacements on the hysteretic response, of reinforced concrete beams has been studied (3.2). With nominal shear stresses greater than $0.4\sqrt{f'_c}$ MPa across the plastic hinge regions of conventionally reinforced concrete beams, very significant loss of energy dissipation during inelastic cyclic response was observed. This is due to sliding along wide full depth cracks across beams, developed after considerable yielding of both the top and bottom flexural reinforcement. In such cases the use of diagonal reinforcement across the plastic hinge region was found to improve the energy dissipation very significantly. Reinforced concrete beams in which the plastic hinge region was diagonally reinforced and relocated from a column face were found to have an excellent response (3.40), similar to that of a structural steel member. Suitable design recommendations have been published (3.19).

The spiral column tests described previously (3.7) showed that well distributed longitudinal reinforcement and closely spaced spiral steel increased the shear capacity of the concrete in the plastic hinge regions to well above that normally assumed in design.

Shear Walls

As a sequence to the study of reinforced concrete shear walls coupled by deep diagonally reinforced coupling beams (3.27), the coupling of walls solely by slabs has been investigated (2.3). Slabs were specially reinforced where, due to earthquake loading, moments and shear forces are to be transmitted between walls. Slab-wall subassemblages were subjected to cyclic loading with progressively increasing ductility demand. Dramatic loss of stiffness characterised the inelastic seismic response of the slab coupling. It was concluded that such slabs are not contributing significantly to energy dissipation.

Excellent measured load-displacement response was obtained for shear walls coupled with frames (3.12) provided that careful detailing was used. To attain significant plastic rotation at the base of cantilever walls, in certain cases confining reinforcement over parts of the cross section is required (3.26).

Preliminary tests have shown that significant flexural ductility can be attained also in squat shear walls, and that it is possible to control diagonal tension failures in accordance with capacity design procedures. With the cyclic loading of such walls, loss of energy dissipation is mainly due to sliding shear displacement at the base (3.10), which will lead eventually to failure. Diagonal reinforcement crossing the potential failure planes has been found to significantly improve the energy dissipation characteristics of the wall (3.10).

Masonry

Experimental research in structural masonry (3.5, 3.14, 3.16) has aimed

to better define the strength and ductility capacity of masonry shear wall elements, and to improve design details. The walls were constructed of proprietary concrete blocks. The vertical reinforcement was placed in the hollow cells and the cells filled with grout. Shear reinforcement was also present. The testing of well designed masonry shear walls under cyclic load reversals (3.16) has demonstrated that high displacement ductilities can be obtained in the deformation mode involving yielding of the flexural reinforcement. Provided a capacity design approach is adopted, shear failures can be inhibited for even very squat walls (3.5) or walls with very high shear stresses (3.16). The influence of workmanship (3.5) and the seismic performance of masonry veneer panels (3.43) have also been studied.

Design philosophy for structural masonry has been discussed (3.31). The need for simplicity of concept and suitable location of plastic hinge regions was emphasised. Simple cantilever wall units are preferred to pierced walls or infilled frames because of the susceptibility of the latter structural types to shear failure and rapid strength degradation.

EARTH RETAINING STRUCTURES

Work on the seismic behaviour of gravity retaining walls began in 1976 following a suspicion that gravity walls failed in practice at lower levels of earthquake excitation than that predicted by the classical Mononobe-Okabe analysis for earth pressures. It was shown that, while the Mononobe-Okabe results were essentially correct for earth pressures, the inertia forces acting on the wall itself were of the same order of magnitude as the earth pressures so that this effect should be included in analysis (4.1). Design including wall inertia forces led to uneconomically large wall masses. A model was then developed by Richards and Elms (4.1, 4.5, 4.7, 4.9) for the incremental failure of a wall. This approach was similar to Newmark's sliding block method, though developed independently, and provided a means for calculating the displacement of a wall for any given earthquake record. Further work produced design recommendations for the US (4.6, 4.7) and New Zealand (4.9). A stability criterion against tilting was developed (4.9) and the approach was extended to the design of some bridge abutments (4.6).

An experimental program on retaining walls was carried out to verify the Richards-Elms model using small scale prototype tests on a shaking table (4.2, 4.8). The tests showed that the wall closely followed the behaviour predicted by the Richards-Elms model in qualitative terms, but that the final displacement was only approximately 60% of that predicted by theory. The theoretical model thus appears to be conservative, though it should be pointed out that vertical and lateral accelerations, not included in the experiments, would cause appreciably greater displacements. However, later work (4.3) has shown that a refinement of the Richards-Elms model carried out by Zarrabi and Whitman at MIT shows very close agreement with the experimental results: this model takes into account the vertical acceleration of the soil wedge due to its horizontal relative motion.

Further experimental work has investigated the basic mechanism of failure by examining the response of a retaining wall to a smooth sinusoidal acceleration input, as well as to scaled earthquake records (4.3).

In addition to the above work on gravity retaining walls, some insitu lateral load tests have been conducted on a 1.37 m diameter, 24 m long, reinforced concrete foundation cylinder. The cylinder was constructed and tested as part of a comprehensive foundation investigation for a major bridge (4.4). Strain gauge readings from the cylinder enabled the vertical distribution of lateral subgrade coefficient to be estimated.

CONCLUSIONS

The Department of Civil Engineering of the University of Canterbury, New Zealand, has actively continued its earthquake engineering research during the four years since the Sixth World Conference on Earthquake Engineering.

The work completed on earthquake code design factors has involved an investigation of the optimum level of the seismic requirements for reinforced concrete framed buildings. Currently, design spectra for earthquake code loading appropriate to New Zealand are being evolved.

Dynamic analyses of a range of structures subjected to severe earthquake ground motions have been conducted. Time-history analyses have been used in most studies. The inelastic dynamic response of frames has been extensively investigated to determine the ductility demand and the appropriate design actions for columns taking into account higher mode effects, geometry, material properties and design approach. Shear wall structures and bridge piers responding inelastically to severe earthquake motions have been analysed to determine ductility demand and other design requirements as a function of a number of variables. Circular tanks containing liquids of low viscosity have been examined. Base isolated structures and structures with rocking foundations have been studied.

A range of structural concrete elements and subassemblages have been subjected to static cyclic loading simulating seismic loading. These laboratory tests involved reinforced concrete, prestressed concrete and partially prestressed concrete beam-column subassemblages, masonry shear walls, reinforced concrete bridge piers, and reinforced concrete columns. The test results give design information on methods for detailing members for ductility, methods for detailing beam-column joint cores and adjacent members to prevent joint core shear failure and bond failure, and methods for detailing reinforcement in shear walls. Shake-table tests have been conducted on model bridge piers and circular tanks.

The seismic behaviour of retaining walls has been studied analytically and a simple theoretical model proposed for a sliding displacement mechanism. Model walls have been subjected to shake-table tests.

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LIST OF DEPARTMENT RESEARCH REPORTS AND PUBLISHED PAPERS

1. Earthquake Code Factors and Ground Motions

Research Reports of Department of Civil Engineering, University of Canterbury

1.1 Silvester, D. and Elms, D.G., "Optimal Level for New Zealand Earthquake Code", No. 77-3.

1.2 Braithwaite, A.J. and Berrill, J.B., "A Geophysical Procedure for Estimating Strong Ground Motion", No. 79-10.

1.3 Darwin, D.J., Elms, D.G. and Berrill, J.B., "Earthquake Hazard Reduction in Wellington", No. 80-1.

Papers

1.4 Berrill, J.B., "A Note on the Gazli Accelerogram", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 4, Dec. 1977, pp.238-239.

1.5 Elms, D.G. and Silvester, D., "Cost Effectiveness of Code Base Shear Requirements for Reinforced Concrete Frame Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 11, No. 2, June 1978, pp.85-93.

1.6 Berrill, J.B., "Properties of Strong Ground Motions", National Roads Board Bridge Seminar, Auckland, Vol. 4, Nov. 1978, pp.35-40.

1.7 Elms, D.G., "Risk Balancing for Code Formulation and Design", Proceedings of 3rd International Conference on Applications of Statistics and Probability in Soil and Structural Engineering, Sydney, Jan. 1979, pp.701-713.

1.8 Berrill, J.B., "Suggested Extensions of the New Zealand Strong Motion Accelerograph Recording in New Zealand", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1979, pp.264-268.

1.9 Elms, D.G., "Rational Derivation of Risk Factors", Proceedings of the Seventh Australasian Conference on the Mechanics of Structures and Materials, Perth, Western Australia, May 1980.

2. Dynamic Analyses

Research Reports of the Department of Civil Engineering, University of Canterbury

2.1 Crosbie, R.L., Priestley, M.J.N. and Carr, A.J., "Base Isolation for Brick Masonry Shear Wall Structures", No. 77-2.

2.2 Evison, R.J., Priestley, M.J.N. and Carr, A.J., "Rocking Foundations", No. 77-8.

2.3 Taylor, R., Paulay, T. and Carr, A.J., "The Non-Linear Seismic Response of Tall Shear Wall Structures", No. 77-12.

2.4 Jury, R.D., Paulay, T. and Carr, A.J., "Seismic Load Demands on Columns of Reinforced Concrete Multistorey Frames", No. 78-12.

2.5 Carr, A.J., Moss, P.J. and Pardoen, G.C., "Imperial County Services Building - Elastic and Inelastic Response Analyses", No. 79-15.

2.6 Moss, P.J. and Carr, A.J., "Vibration Tests on Buller River Bridge at Westport", No. 79-16.

2.7 Carter, B.H.P., Paulay, T., Carr, A.J. and Berrill, J.B., "Earthquake Response of Shear Walls", No. 80-2.

2.8 Chrisp, D.J., Carr, A.J. and Moss, P.J., "Damping Models for Inelastic Structures", No. 80-3.

2.9 McManus, K.J., Priestley, M.J.N. and Carr, A.J., "Seismic Response of Structures Free to Rock on Their Foundations", No. 80-4.

2.10 Tompkins, D.N., Paulay, T. and Carr, A.J., "Seismic Demands on Columns of Reinforced Concrete Multistorey Frames of Low Seismic Resistance", No. 80-5.

2.11 Bryson, S.J., Elms, D.G. and Park, R., "Probabilistic Approach to Capacity Design of Reinforced Concrete Structures", (to be printed).

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