Behaviour of Concrete Bridge Piers under Seismic Attack

Comportement de piles de pont sous l'effet de charges sismiques

Verhalten von Stahlbeton-Brückenpfeilern unter Erdbebeneinwirkungen

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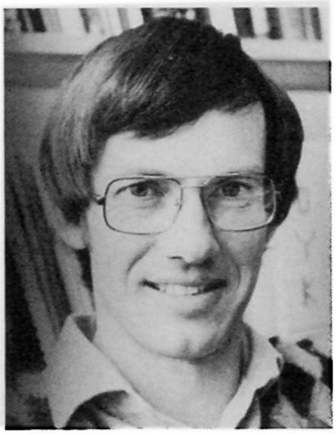

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SUMMARY
The paper summarises recent research in New Zealand into the seismic behaviour of bridge piers. Results are reported for a wide variety of section shapes tested under axial load and bending moment at high displacement ductilities. Design recommendations for bridge piers under seismic loading are presented.

RESUME
Cet article donne un résumé des recherches récentes en Nouvelle Zélande sur le comportement sismique des piles de ponts, et présente les résultats d'expériences sur des piles de formes diverses, soumises à des charges axiales et latérales, avec des grands déplacements ductiles. Des recommandations pour le calcul parasismique des piles sont présentées.

ZUSAMMENFASSUNG
Der Artikel fasst die jüngsten in Neuseeland durchgeführten Untersuchungen des Verhaltens von Brückenpfeilern unter Erdbebeneinwirkung zusammen. Es werden die Ergebnisse von Versuchen an einer Reihe Brückenpfeiler mit verschiedenen Querschnittsformen wiedergegeben, die auf Biegung und Normalkraft bis zum Erreichen grosser plastischer Verformungen belastet wurden. Es werden ferner Empfehlungen für die Bemessung von Brückenpfeilern unter Erdbebenbelastung gemacht.

1. INTRODUCTION

The seismic performance of bridges is a matter of special importance. For effective post-earthquake rescue operations to be mounted, access to the affected area by road or rail is essential. This access can be completely cut by the failure of one or two critical bridges, particularly when rough terrain does not facilitate cross-country mobility. On economic grounds, bridges also assume special importance. The costs of having a rail bridge out of action may exceed US$20,000/hr to the railroad operator. Similar costs, though generally less direct, can be assigned to the loss of a major road bridge, where alternative routes are not available, or involve lengthy detours.

Despite these observations, performance of bridges in severe earthquakes has not been good, with extensive damage being sustained by bridge piers, in particular, and it is only comparatively recently that detailed research has been undertaken to improve seismic design of bridging. This paper summarises an extensive research programme into the seismic performance of bridge piers carried out at the University of Canterbury, New Zealand, over the past 10 years.

2. SEISMIC LOADING: STRENGTH VS DUCTILITY

It is generally uneconomic to design structures to withstand lateral forces corresponding to full elastic response to design-level earthquakes. The alternative, and widely accepted approach is to design for a lower force level, and detail the structure for ductility to ensure it can sustain the inelastic displacements at the design level of seismic attack without significant strength degradation. Frequently, however, codes are not explicit about the interaction between strength and ductility, and the ductility capacity imparted by the specified detailing requirements (if any exist) are not stated.

An approach recently proposed in New Zealand [1] involves specification of inelastic response spectra for clearly identified levels of structural ductility, such as those shown in Fig. 1. The 5% damping spectra apply for a design earthquake of 150 year average return period for the most seismically active regions of New Zealand, and correspond to an expected peak ground acceleration of 0.5g. For periods greater than $T = 0.7s$, inelastic spectra are obtained from the elastic spectrum by dividing the latter by the ductility factor μ, that is, the 'equal-displacement' principle is applied. However, for shorter period structures, the equal-displacement principle is non-conservative, and inelastic spectra C_μ are found from the elastic spectrum C_1 using the expression

$$C_\mu = \frac{0.7\ C_1}{(\mu-1)T + 0.7} \qquad (1)$$

which provides a gradual transition from the equal-displacement approach at $T = 0.7s$, through the equal-energy principle at about $T = 0.3s$, to the equal-acceleration requirement at $T = 0$. The result is inelastic spectra for short period structures that substantially exceed those resulting from codes requiring force-reduction coefficients which are independent of period.

Use of detailed spectra such as those of Fig. 1 implies an ability to assess the ductility capacity of a bridge structure with some accuracy. Design requirements will normally dictate that any inelastic action occurs in the bridge piers, since it is both impractical and undesirable to design for plastic hinges in superstructures, and plastic hinges in piles should be avoided because of difficulties in assessing and repairing damage after an earthquake. Fig. 2 illustrates an idealisation of ductility by representing as equivalent elasto-plastic behaviour. For the majority of bridges, where ductility is provided by flexural plastic hinging of the piers, the ductility cap-

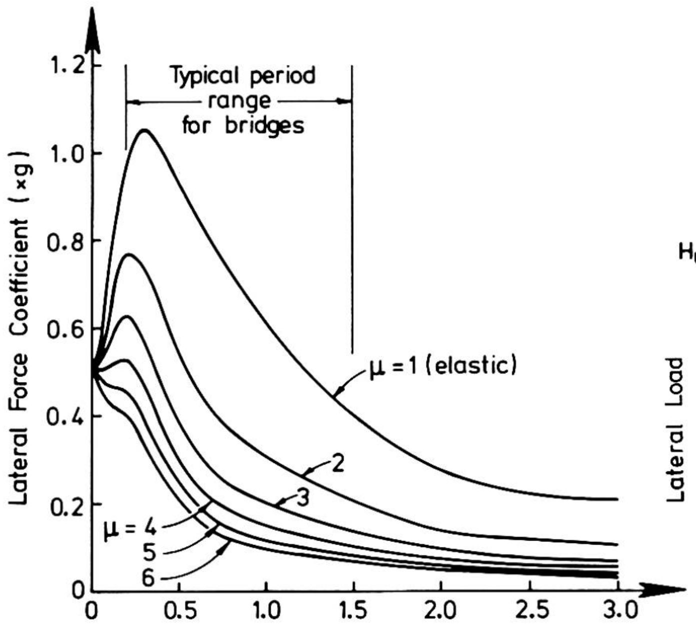

Fig.1 INELASTIC DESIGN SPECTRA FOR 0.5g GROUND ACCELERATION

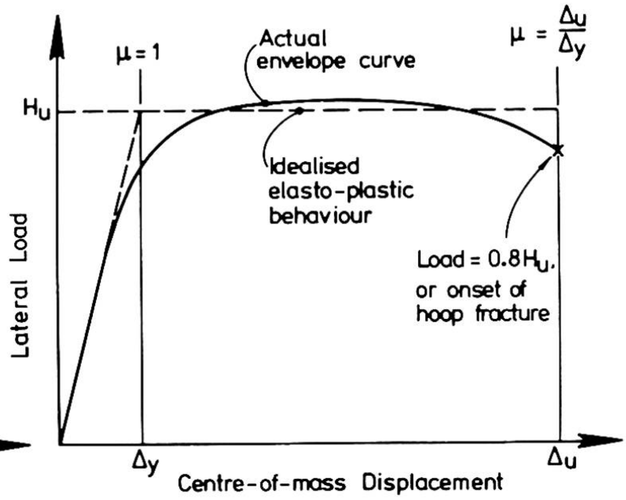

Fig. 2 DEFINITION OF DUCTILITY CAPACITY

acity will be limited by the ultimate displacement Δ_μ of the bridge pier. Definition of Δ_μ is somewhat subjective, but New Zealand practice is to take the displacement corresponding to either first fracture of confining steel (which results in rapid degradation of performance), or to a 20% drop of lateral load capacity as the limit. In assessing the overall structure ductility capacity from the pier inelastic displacement capability, consideration of elastic deformations occurring in foundations and bridge bearings must be made, as these reduce the structure ductility [2].

3. EXPERIMENTAL RESEARCH INTO BRIDGE PIER BEHAVIOUR

In order to obtain the ductilities implied by Fig. 1, compression strains at the extreme compression fibre as high as 2 to 4% may be necessary. As this vastly exceeds the unconfined compression strain of concrete, commonly taken as 0.3-0.4%, confinement in the form of transverse hoops or spirals is required. Research in New Zealand into the strength and ductility of bridge piers has over recent years been directed towards an assessment of the effectiveness of different amounts and configurations of confining reinforcement. The research has involved two phases of testing: axial load testing to investigate the compression stress-strain characteristics of confined concrete, and flexural testing to investigate ductility capacity of sections designed on the basis of stress-strain curves developed in the first phase.

Fig. 3 shows some of the sections that have been tested in the two phases. Section sizes have generally been as large as possible within the load and physical size limitations of a 10 MN capacity DARTEC servohydraulically controlled testing machine used to apply axial load, to facilitate realistic modelling of both concrete and reinforcement.

3.1 Stress-Strain Characteristics of Confined Concrete

For sections tested under axial load only (Fig. 3a) variables have included section shape, longitudinal reinforcement content, lateral reinforcement (confining steel) content and configuration, and loading rate. The relationship between amount and distribution of confining reinforcement and the confined

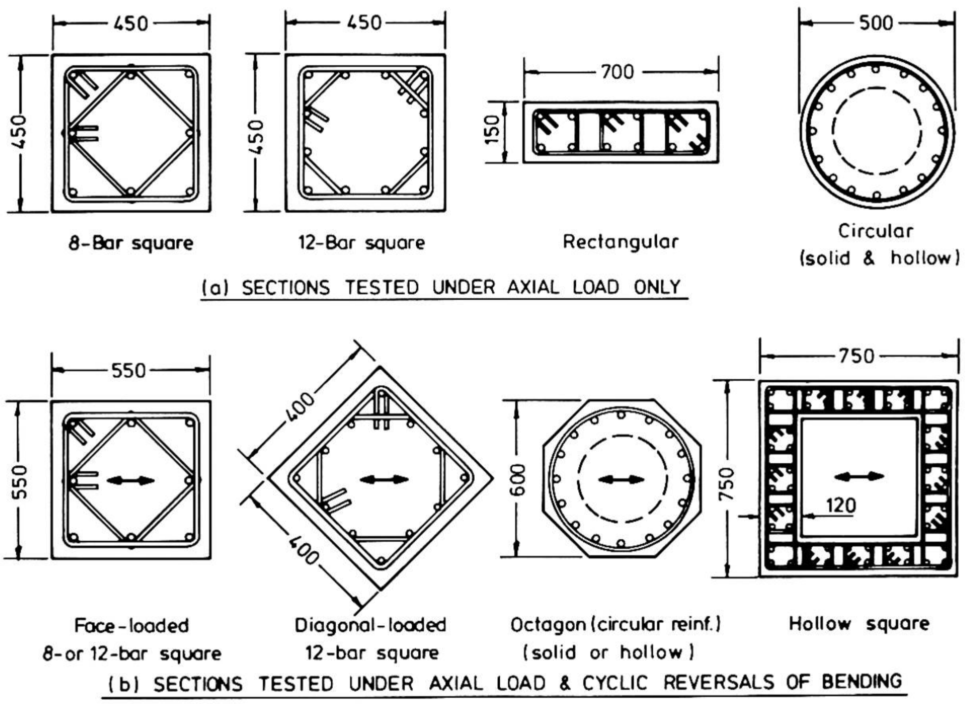

Fig. 3 SECTIONS TESTED TO ESTABLISH DUCTILITY OF BRIDGE PIERS

stress-strain curve has been of particular interest, as has been the influence of loading rate. The design of the DARTEC machine is such that strains can be applied at seismic rates, with the maximum load capacity of 10 MN being attainable in less than 0.3s.

Typical results are shown in Fig. 4 for circular and rectangular sections containing different amounts of confining reinforcement, expressed in terms of volumetric ratio, ρ_s , related to the volume of the confined core [3]. Comparison of confined stress-strain curves with unconfined curves ($\rho_s = 0$) indicates significant increases in compression strength with amount of confinement, and more importantly, very substantial increases in concrete ductility, apparent in the reduced slope of the falling-branch section of the stress-strain curves, and the high strain at which hoop fracture first occurred. Fig. 4a indicates that ultimate compression strains exceeding 5% are attainable, with first hoop fracture indicated by a sudden drop in load capacity near the end of the stress-strain curve.

Mander showed [3] that the experimental curves could be adequately predicted by an analysis based on a '5-parameter' multiaxial failure criteria developed by William and Warkne [4]. This approach resulted in a prediction for maximum confined concrete strength f'_{cc} to be related to the unconfined strength f'_{co} by the expression

$$\frac{f'_{cc}}{f'_{co}} = -1.254 + 2.254\sqrt{1 + 7.94\,\frac{f'_{\ell}}{f'_{co}}} - 2.0\,\frac{f'_{\ell}}{f'_{co}} \qquad (2)$$

where f'_{ℓ} is the effective lateral pressure exerted on the core concrete by the confining steel at yield stress. Mander also showed that longitudinal strain at first fracture of confining reinforcement could be predicted by energy considerations, relating the increase in strain energy in compression of the confined concrete to that provided by tensile straining the confining steel to fracture. Theoretical stress-strain curves based on this approach are included for comparison with experimental curves in Fig. 4. It will be observed that very good agreement results.

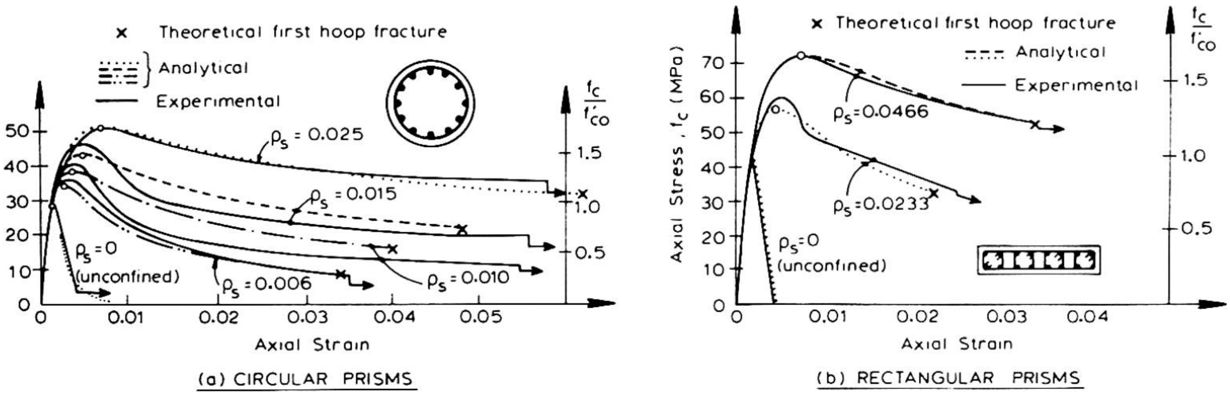

Fig.4 AXIAL STRESS-STRAIN CURVES FOR CONFINED CONCRETE

3.2 Flexural Ductility Under Cyclic Loading

Sections subjected to combined axial load and lateral bending are shown in Fig. 3b. In addition to these sections, smaller octagonal and rectangular sections have been tested to investigate the influence of aspect ratio [column height divided by section thickness or diameter]. Testing involved cyclic reversals of lateral displacement to successive limits of $\mu = 0.75$, 2, 4, 6 and 8, with two full cycles at each level of ductility [5-8].

Fig. 5 shows typical results for a rectangular column and an octangonal column confined in accordance with provisions presented in section 4 of this paper. It will be noted that the load-displacement hysteresis loops are very stable, with insignificant strength or stiffness degradation between cycles of displacement to the same ductility limit. The results support the use of the $\mu = 6$ inelastic spectrum of Fig. 1 for fully confined columns. Results for square columns loaded parallel to a diagonal, and for hollow rectangular columns have also been satisfactory, but it has been found that thin-walled hollow circular columns with only one ring of longitudinal reinforcement have suffered rapid strength degradation at comparatively low ductilities, particularly when axial load levels were high. Columns with longitudinal reinforcement lapped at the critical (maximum moment) section tended to have reduced ductility capacity because of concentration of plasticity over a very small hinge length adjacent to the critical section.

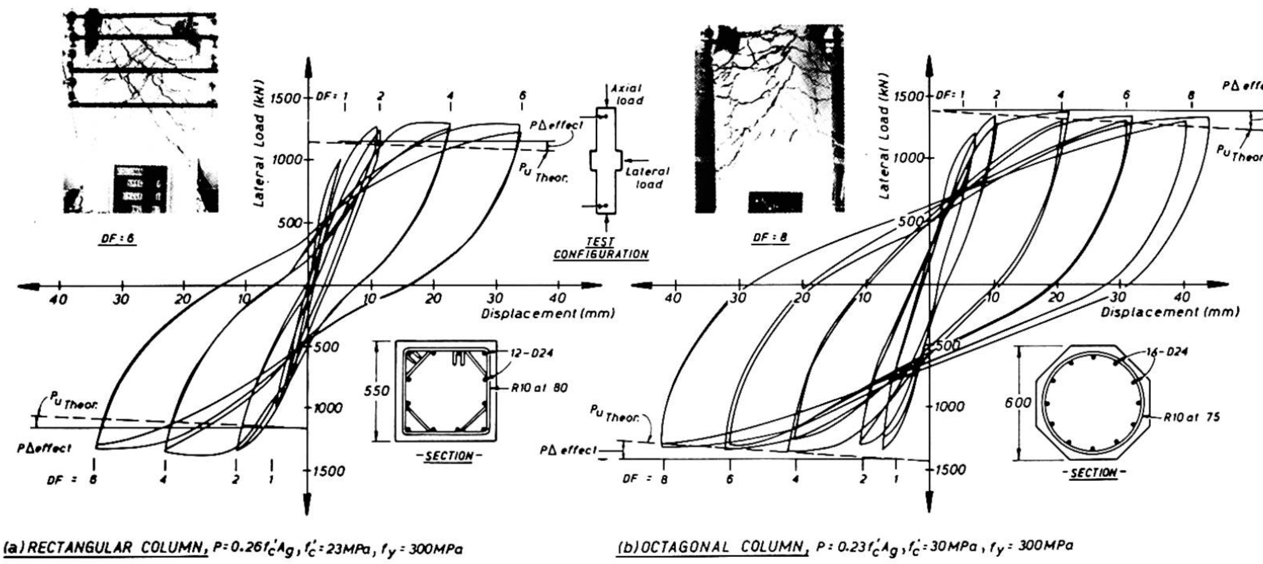

FIG. 5 - LOAD-DISPLACEMENT LOOPS FOR FULLY CONFINED COLUMNS

3.3 Shear Strength Under Cyclic Loading

All columns tested under combined axial load and cyclic reversals of bending have attained flexural strengths exceeding 'ideal' capacity based on measured strengths, normal flexural strength equations for concrete, and an ultimate compression strain of 0.3%. This is primarily a result of enhanced concrete compression strength, and strain at maximum moment, resulting from confinement (see Fig. 4). Fig. 6 shows the extent of strength enhancement for a large number of columns as a function of axial load level. Despite considerable scatter, the trend is obvious, with strength enhancement between 50% and 100% for high axial load ratios.

This has significance for shear design. Since the enhanced flexural strength will be developed at the design level earthquake, albeit at a somewhat reduced ductility, the column must be able to support the correspondingly enhanced shear force without developing a brittle shear failure, which exhibits very limited ductility. Current research [9] is investigating the suitability of different approaches, including the ACI method, and a modified form of compression field theory, to predict behaviour of bridge columns failing in shear. Results to date have indicated that the ACI approach is conservative for elastic response, or for very low ductility, but does not give sufficient protection at high ductility levels, when concrete shear resisting mechanisms break down under the cyclic reversals of loading. Typical results are presented in Fig. 7, which compares degradation of shear strength V expressed as a fraction of shear corresponding to flexural hinging, V_{if}, for circular columns identical except for the amount of shear reinforcement, expressed in terms of the volumetric ratio ρ_s. It will be seen that column 12, with $\rho_s = 0.0102$ contained sufficient shear reinforcement to develop the flexural strength (i.e. $V/V_{if} > 1$) but eventually suffered a shear failure at $\mu = 4$. Column 18, with half as much shear reinforcement also initially developed the flexural strength, but suffered shear failure, and rapid strength degradation at $\mu = 1.5$. Column 19, with a lesser amount of shear reinforcement was unable to develop the flexural strength before suffering shear failure. The results indicate that for a given amount of shear reinforcement, the shear strength is a function of the required flexural ductility level.

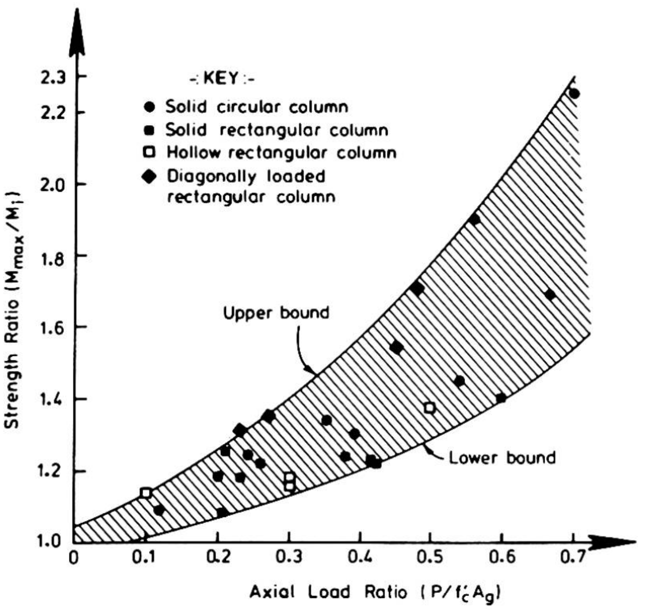

Fig. 6 FLEXURAL STRENGTH ENHANCEMENT OF CONFINED COLUMNS

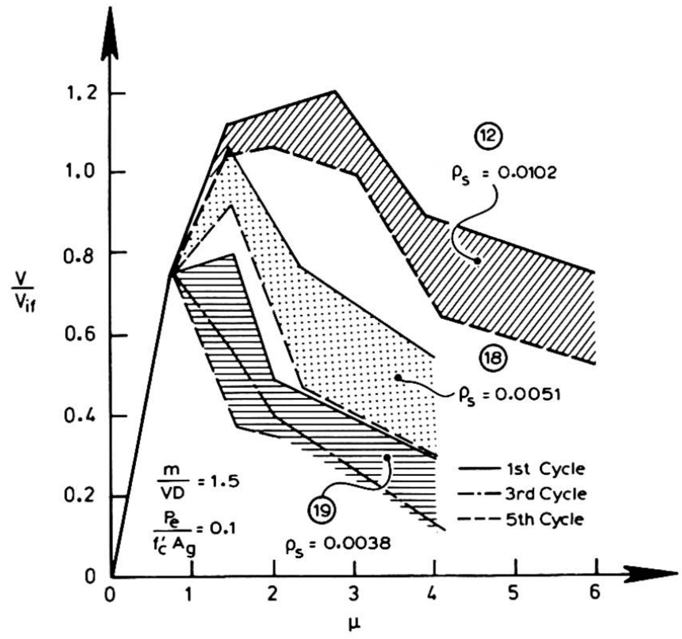

Fig. 7 DEGRADATION OF SHEAR STRENGTH WITH DUCTILITY

4. DESIGN RECOMMENDATIONS

On the basis of the experimental research summarised in the previous section, New Zealand practice allows design for a ductility of $\mu = 6$, provided the following requirements are met.

1. For column with axial load less than $P = 0.3\ f'_c A_g$, confining reinforcement must be provided for the portion of column subject to more than 80% of the maximum moment, or for a length equal to the section depth, whichever is larger. For axial loads greater than $P = 0.3\ f'_c A_g$, the extent of confinement is increased by 50%.

2. Anchorage of confining steel must be by welding stirrups closed, or bending back into the confined core. Lapping in the cover concrete is not permitted.

3. The maximum spacing of the confining reinforcement along the member axis must not exceed 6 times the longitudinal bar diameter, nor 1/5 of the section width, nor 200 mm. The first requirement is the most important, as longitudinal bar buckling invariably develops at moderate ductility levels when confinement is more widely spaced.

4. The amount of confinement required for full confinement is given by the greater of the two following requirements:

$$A_b = F(0.3s(\frac{A_g}{A_c} - 1)\ \frac{f'_c}{f_{yh}}\ (0.5 + 1.25\ \frac{P_e}{f'_c A_g}\ )) \tag{3}$$

or

$$A_b = F(0.12s\ \frac{f'_c}{f_{yh}}\ (0.5 + 1.25\ \frac{P_e}{f'_c A_g}\ )) \tag{4}$$

where A_g and A_c are the gross and core section areas respectively, s is the axial spacing of hoop sets, or spiral pitch, and f_{yh} is the yield strength of the confining reinforcement. For circular hoops or spirals of bar area A_b , F = one quarter the diameter of the confined core. For rectangular hoops with total bar area A_b [i.e. the sum of areas of all parallel confining legs] in the direction being confined, $F = h$, the width of the confined core.

Expressions (3) and (4) will provide member ductilities of at least $\mu = 8$. For lower ductility demands, the amount of confinement required, As_μ , is

$$As_\mu = As_8\ \frac{(\mu - 2)}{6} \tag{5}$$

where As_8 is the amount of confinement given by the more stringent of eqns. 3 or 4.

5. Shear strength must exceed the maximum feasible flexural strength, to avoid shear failure.

5. CONCLUSIONS

Extensive testing of large scale models of reinforced concrete bridge piers has established that ductilities of the order of $\mu = 6$ to $\mu = 8$ can be dependably obtained from well confined columns. The design requirements listed in this paper are felt to be the minimum necessary to obtain satisfactory performance at a design ductility level of $\mu = 6$.

6. ACKNOWLEDGEMENTS

Financial support by the Road Research Unit of the N.Z. National Roads Board, and by the University of Canterbury is gratefully acknowledged.

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