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RECENT NEW ZEALAND DEVELOPMENTS ON BRIDGE SEISMIC DESIGN

DÉVELOPPEMENTS RÉCENTS EN NOUVELLE ZÉLANDE DU DESSEIN SÉISMIQUE DES PONTS

NEUE NEUSEELÄNDISCHE ENTWICKELUNGEN ÜBER DEM ERDBEBENANSCHLAG DER BRÜCKEN

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SUMMARY

Information is presented on recent New Zealand developments in the two main approaches for seismic design of bridges. Most commonly, design is based on ductile flexural yielding of specially detailed members, usually the piers. Research data are shown on ductility demand on reinforced concrete piers and their ductility capability. A design office procedure for assessing available bridge structure ductility is discussed. A recent alternative approach involves isolation of the structure from the worst effects of earthquake ground shaking by a combination of flexible mountings and mechanical energy dissipating devices. Details of the devices, examples of their application and a philosophy of design are presented.

RÉSUMÉ

On présente les informations de developpements récents en Nouvelle Zélande concernant les deux procédés principaux pour le dessein séismique des ponts. Très ordinairement, le dessein s'est basé sur le fléchissement ductile à la flexion des membres avec particularités spécialement arrangées, d'habitude les piliers. On montre des faits de recherches concernant la demande de la ductilité sur les piliers de béton armé et leur capacité de la ductilité. On discute un procédé du bureau de dessein pour estimer la ductilité disponible de la structure des ponts. Un procédé récent et alternatif comporte l'isolement de la structure de plus mauvais effets du tremblement de terre par une combination d'affûts flexibles et appareils mécaniques qui dispersent l'énergie. On présent les détails des appareils, exemples de leur application et une philosophie de dessein.

HAUPTINHALT

Informationen wird über neue neuseeländische Entwickelungen von den zwei Hauptmethoden gegenüber dem Erdbebenanschlag der Brücken vorgelegen. Am gewöhnlichsten, wird der Anschlag auf ziehbare krümmende Nachgebendigkeit von Gliedern mit besonders geordneten Details gegründet, meistens die Pfeiler. Erforschungsunterlagen werden über der Anforderung der Biegsamkeit auf Eisenbetonpfeiler und ihre Fähigkeit der Biegsamkeit dargestellt. Eine Handlungsweise des Anschlagbüros, verfügbare Biegsamkeit in Brückenaufbauten einzuschätzen, wird behandelt. Eine neue, alternative Methode hat zur Folge die Absonderung des Aufbaus von den schlimmsten Folgen des Erdbebengrundschüttelns von einer Zusammensetzung biegsamer Vorbauten und mechanische Apparate, die die Energie zerteilen. Die Einzelheiten der Apparate, Beispiele ihres Gebrauchs und eine Anschlagphilosophie werden vorgelegen.

1. INTRODUCTION

In the past, greater attention has generally been given to development of seismic design procedures for buildings than those for bridges. However, some spectacular collapses of several highway interchange bridges during the San Fernando earthquake of 9 February, 1971 highlighted the needs for research into the seismic response of bridge structures and for the development of improved design methods and details. Over recent years, in New Zealand, considerable efforts have been made to satisfy these needs.

It is well known that, when a structure responds to earthquake induced ground motions, very high forces may be generated if the structure is required to remain elastic. Such a requirement is seldom economically justifiable in view of the rapid increase in the cost of the structure, and the foundations in particular, as the design horizontal load increases. The common design approach is, therefore, to limit, or at least reduce, the horizontal forces induced in the structure. Two methods of achieving this are as follows:

(a) *Ductile Design Approach:* Energy dissipating members of a plastic hinge mechanism are designed to yield at an acceptable intensity of earthquake ground shaking and are detailed to deform in a ductile manner. Other structural elements are provided with sufficient reserve strength capacity to ensure that the chosen energy dissipating mechanism is maintained at near full strength throughout the deformations that may occur under a very severe earthquake. In a bridge structure, the chosen energy dissipating members are usually the piers rather than the foundations, because of the greater accessibility for inspection and repair in the former members.

(b) *Application of Mechanical Energy Dissipating Devices:* The structure is isolated from the worst effects of ground shaking by a combination of flexible mountings, usually in the form of elastomeric bearings, and of specially developed energy dissipating devices.

Method (a) above is the most commonly used approach, but a number of disadvantages must be accepted : design procedures may be complicated; details are expensive and sometimes difficult to fabricate; there is a high probability of some form of earthquake induced damage during the life of a structure and the associated requirement for costly repairs; and there may be difficulties in restoring permanent set deflections. The procedure in method (b) of concentrating energy dissipation in special components allows the structure itself to be protected from damage even in severe earthquakes. There is also potential for simplified design procedures and details. However this method is still in its developmental stages. Recent developments in both of these approaches are described in this paper.

2. DUCTILE DESIGN APPROACH

2.1 General

Because the primary energy dissipating members in a bridge designed according to this approach are the piers, satisfactory design depends on an understanding of the ductility demand on the piers and of their ductility capability. Previous buildings-related research is not necessarily applicable. Ductility demand during dynamic seismic response is dependent on aspects peculiar to bridges, such as foundation and elastomeric bearing characteristics. Knowledge of ductility capability has required recently obtained data for axially loaded piers with shapes, sizes and loads commonly adopted for bridge design. Research information must be translated into a form allowing ready use by the designer, and for this purpose a convenient design office procedure for calculating available structure ductility has been developed.

2.2 Ductility Capability of Reinforced Concrete Piers

A programme of testing model bridge piers is proceeding at the University of Canterbury and progress to date has been fully reported by Priestley et al [1]. Five pier units have been tested, all representing variations of the same prototype, namely a single stem octagonal pier 1.5 m wide by 6 m clear height, reinforced vertically with 20 bundles of 3 by 32 mm dia bars giving a steel content of 2.7%.

The first three units were modelled to a scale of $\frac{1}{3}$ full size. Transverse reinforcing was designed to ACI requirements [2], and since the design ultimate axial load was only 0.06 $f_c^{\prime} A_g$ the requirements for flexural members applied. Within the hinge region minimum requirements governed, namely $A_v d/s$ = 0.15 $A_s^{\prime}$ or 0.15 A_s, whichever is the larger, with spacing not to exceed d/4 or 16 bar diameters. In the model this was provided by 6.5 mm dia welded hoops at 65 mm centres, giving a volumetric transverse steel content of 0.44%. The test procedures allowed a variation of the ratio of base moment to shear, and thus piers of different height were simulated in each model. Horizontal load was applied as slow cyclic load reversals of increasing displacement amplitude. The behaviour was generally satisfactory with displacement ductility factors in excess of 5 being achieved for each unit with substantial hysteretic energy dissipation evident. In later cycles hoop steel strains exceeded yield and cover concrete spalled, leading to buckling of compression steel progressively over subsequent cycles with associated moment and stiffness degradation.

The transverse steel of Units 4 and 5 was designed according to the more stringent requirements of ref 3, namely that within the hinge region hoop spacing should not exceed 100 mm, and the volumetric ratio of circular reinforcing should not be less than 0.12 $f_c^{\prime}/f_y$. Unit 4 represented the prototype pier at $\frac{1}{3}$ scale, and hoop steel was 8 mm dia at 34 mm centres. The design office procedure described in Section 2.4[4] results in similar transverse steel requirements. No axial load was applied and horizontal load application was at the equivalent height of 8.25 m on the prototype. This unit behaved exceptionally well. Stable hysteresis loops with only minor load and stiffness degradation were obtained at all displacement levels as indicated in Fig 1. Base moments have been scaled to prototype values and displacements to the effective prototype mass centre. Theoretical ultimate moment capacities based on measured material properties are indicated by dashed lines marked M_u. Displacement ductility factors (DF) were based on a nominal experimental yield displacement found by extrapolation of the post-cracking elastic moment-displacement curve to the theoretical ultimate moment capacity. Maximum recorded hoop steel strains reached 92% of nominal yield based on f_y = 275 MPa and E = 200 GPa, indicating that the design requirement [3] was realistic. Unit 5 was similar in detail to Unit 4 but at $\frac{1}{6}$ scale. It was subjected to sinusoidal and simulated earthquake base accelerations using a MTS

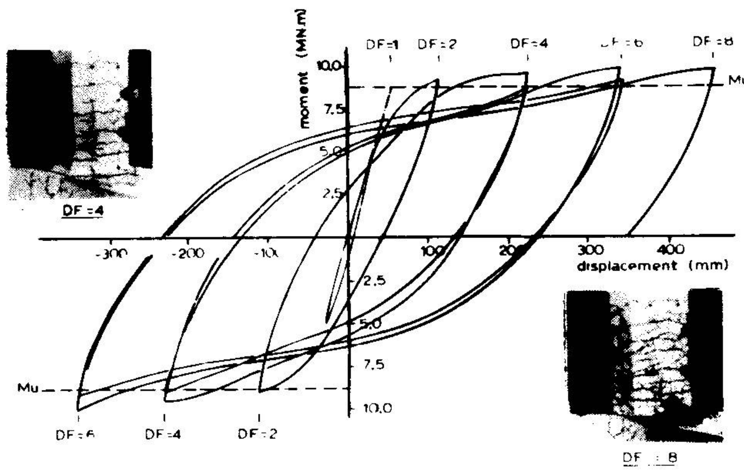


FIG 1: Moment-Displacement Loops for Unit 4, after Priestley et al [1]

electro-hydraulic system coupled to a shaking table. Good agreement was found between moment-displacement curves from Unit 4 tested statically and Unit 5 tested dynamically, giving confidence in the continued investigation of ductility capability using statically tested models.

Research is proceeding at University of Canterbury on ductility of circular and rectangular reinforced concrete bridge piers under combined axial load and bending moment.

2.3 Ductility Demand on Reinforced Concrete Piers

A number of dynamic computer analysis studies have been made [5,6,7,8] of the seismic response of bridge structures, and in particular the ductility demand on reinforced concrete piers. Sharpe [5] used finite element techniques to model a bridge on alluvial soil deposits within a wide valley, and demonstrated the significance of site characteristics on seismic displacement response. Priestley et al [1] computed the theoretical response of Unit 5 described in Section 2.2 above to the El Centro 1940 N-S earthquake record. The results were compared with the experimental response of the model to a displacement time-history form of that record applied to the shaking table, as shown in Fig 2. The theoretical curve was based on the experimentally observed elastic stiffness, a bilinear moment-curvature relationship and a viscous damping of 7%. Agreement is good for the 10 seconds shown and was adequate for the remainder of the earthquake record, although experimental displacements exceed theoretical values, indicating a lower viscous damping at lower levels of response. The corresponding maximum displacement ductility factor demand on the prototype pier was 2.6, and section curvature ductility factor demand was 5.8.

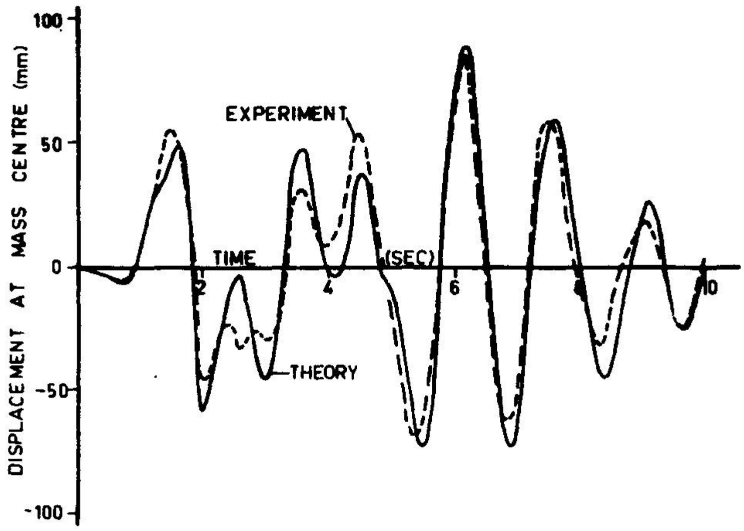
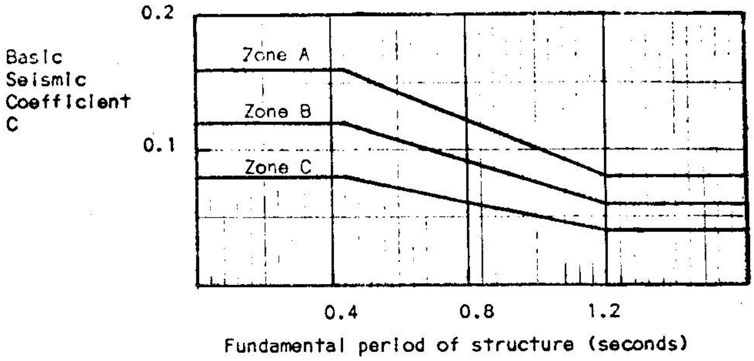


FIG 2: Response of Pier to El Centro 1940 N-S, after Priestley et al [1]

Cameron [7] investigated the sensitivity of the computed response of a reinforced concrete pier to several parameters; namely, the shape of the moment-curvature loop, the contribution of the vertical component of the earthquake ground accelerations, foundation flexibility and different ground acceleration records. Ductility demand decreased moderately with increasing post-yield hardening ratio; inclusion of vertical ground accelerations caused little difference to calculated response values, introduction of foundation flexibility as compared with a fixed base condition caused, in general, a significant reduction in displacement ductility demand but an increase in pier section curvature ductility demand; and ductility requirements varied considerably with different earthquake acceleration records. Analyses of two bridges designed according to current New Zealand requirements [3] and subjected to the El Centro 1940 N-S record showed displacement ductility factor demands of 2 for a portal frame pier and 5 for a single stem pier. Another study [8] of a three-span twin overpass with slab-type piers, analysed with allowance for pier foundation translation and rotation, showed longitudinal displacement ductility factor demands of 2 for the El Centro 1940 N-S record and 4 for the artificial A1 record. Studies are proceeding at University of Canterbury and in Ministry of Works and Development with the objective of defining design ductility requirements taking account of important parameters, in particular foundation and elastomeric bearing flexibilities.

2.4 Design Office Procedure

Design of state highway bridges in New Zealand is governed by the Highway Bridge Brief [3]. Fig 3 shows the recommended seismic design loadings, including importance factors and the basic seismic coefficients applicable to ductile structures for each of the three seismic zones subdividing the country. The seismic coefficients of Fig 3 for Zone A correspond approximately to an elastic response spectrum for the El Centro 1940 N-S earthquake and 5% equivalent viscous damping, divided by a design ductility factor of 6. The corresponding required structure ductility of 6 is regarded as a target and a lower value, with a minimum of 4, is accepted where economics have a strong influence. Less structure ductility is acceptable when the structure has a yield strength exceeding the design value for geometric or other reasons. Research on ductility demand, as discussed in Section 2.3, indicates that the design ductility factor of 6 may be conservatively large, that is that the equal displacement criterion implicit in the assumptions gives an overestimate of displacement response. The figure will be amended, if appropriate, in the light of current research.



Importance Factors F

Category 1	Motorway under or overpass or bridge carrying more than 2500 vehicles per day.	1.0
Category 2	Bridge carrying between 250 and 2500 vehicles per day.	0.85
Category 3	Bridge carrying less than 250 vehicles per day.	0.7

NOTE: Seismic base shear = V = $C \times F \times M_D \times g$ (gravity) where M_D = mass of structure without live load.

Minimum ultimate strength is required to be adequate to resist 1.35xV

FIG 3: Seismic Design Loadings [3]

The ductile design approach requires for completeness an assessment of the available structure ductility. A convenient design office procedure for this purpose has been developed and is more fully described elsewhere [4,9]. The procedure involves, first, assessment of the ductility capability of each section of pier in which plastic hinging is intended to form and, second, calculation of the consequent structure ductility capability. The design charts cover a range of concrete cylinder strengths from 25 to 35 MPa, and apply to mild steel reinforcement with a yield stress of 275 MPa as appropriate for flexurally yielding piers. The stress/strain idealization used for concrete [10] takes account of the degree of confinement. Reinforcing steel strain hardening is assumed to commence at 11 times yield strain. Both circular and symmetrically reinforced rectangular sections are considered.

The section ductility is defined as the ratio of limit curvature of the section to curvature at yield, ϕ_u/ϕ_y, where deformation at yield is nominated as illustrated in Fig 1. In preparation of the charts, the value of ϕ_u was calculated as the ratio of the limit concrete strain in compression to the depth from the compressive face to the neutral axis when the limit strain is reached. The value of ϕ_y was calculated from the curvature at first yield, being the ratio of the steel tensile strain at first yield in the reinforcement closest to the tensile face to the distance from that steel to the neutral axis, multiplied by a factor depending mainly on the section shape, reinforcement layout, and average axial stress on the section. The steps required for calculation of ϕ_u/ϕ_y by the designer using the charts are shown in Fig 4. An example calculation for a circular section is shown in Fig 5. It may be seen that the designer can quickly determine section ductility capability taking account of the relevant variables; namely, section shape, concrete strength, reinforcement percentage, volume of confining reinforcement and average axial stress on the section.

The structure ductility, μ, is defined as the ratio of limit displacement of centre of mass of structure to displacement at yield, δ_u/δ_y, where deformation at yield is nominated as illustrated in Fig 1. Structure and section ductilities are related using the concept of equivalent plastic hinge lengths. From a known moment/curvature relationship and section ductility capability for the piers, a force/displacement curve for the centre of mass of the structure may be derived. Examples of the analysis of various structural forms of bridge and suitable design charts are given in ref 4. It is important that due account is taken of the flexibility of foundations and elastomeric bearings.

At present the method incorporates a number of approximations and it is intended that these be gradually reduced by continuing research. At this stage it should not be used for other than basically single degree-of-freedom structures. A future development of the approach could involve an alternative method for defining and predicting limit curvature [10], being the curvature achieved when the moment of resistance has reduced to 85% of the maximum value. Further test results are needed to support the previous analytical studies before the approach can be developed for design office use.

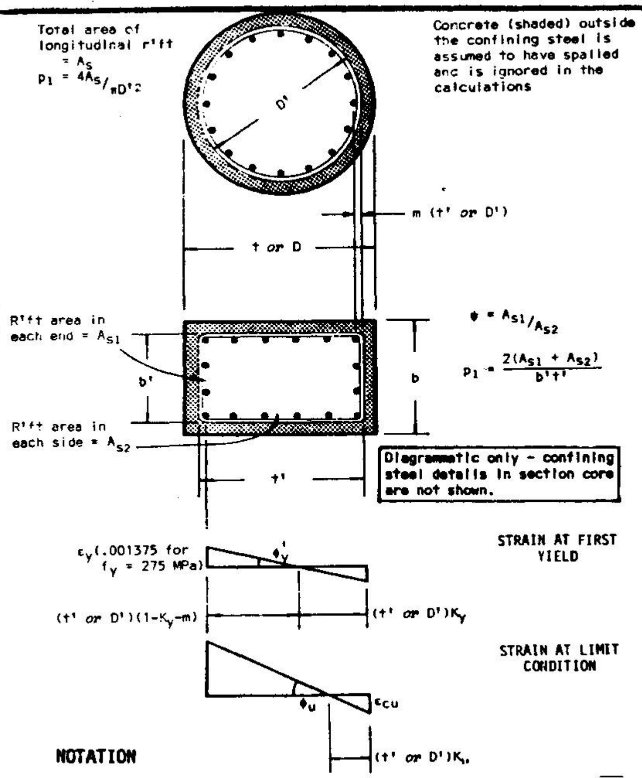
STEPS FOR CALCULATION OF SECTION DUCTILITY

1. Determine Section Properties
   - D or b' and t'
   - m
   - f'_c
   - f_y (normally 275 MPa)
   - P
   - ψ (rectangular sections)
   - p_1
   - Determine p" (chart or calculation)

2. Determine ϕ_u
   - Calculate $P/f'_cD'^2$ or $P/f'_cb't'$
   - Determine K_u (chart)
   - Determine c/d (chart)
   - Calculate d/c
   - Determine ε_{cu} (chart)
   - Calculate $\phi_u = \dfrac{\varepsilon_{cu}}{(t' \text{ or } D')\,K_u}$

3. Determine ϕ_y
   - Calculate $P/f_yD'^2$ or $P/f_yb't'$
   - Determine K_y (chart)
   - Calculate ε_y (0.001375 for Grade 275)
   - Determine ϕ_y/ϕ'_y (chart)
   - Calculate $\phi_y = \dfrac{\varepsilon_y}{(t' \text{ or } D')(1-K_y-m)} \times \dfrac{\phi_y}{\phi'_y}$

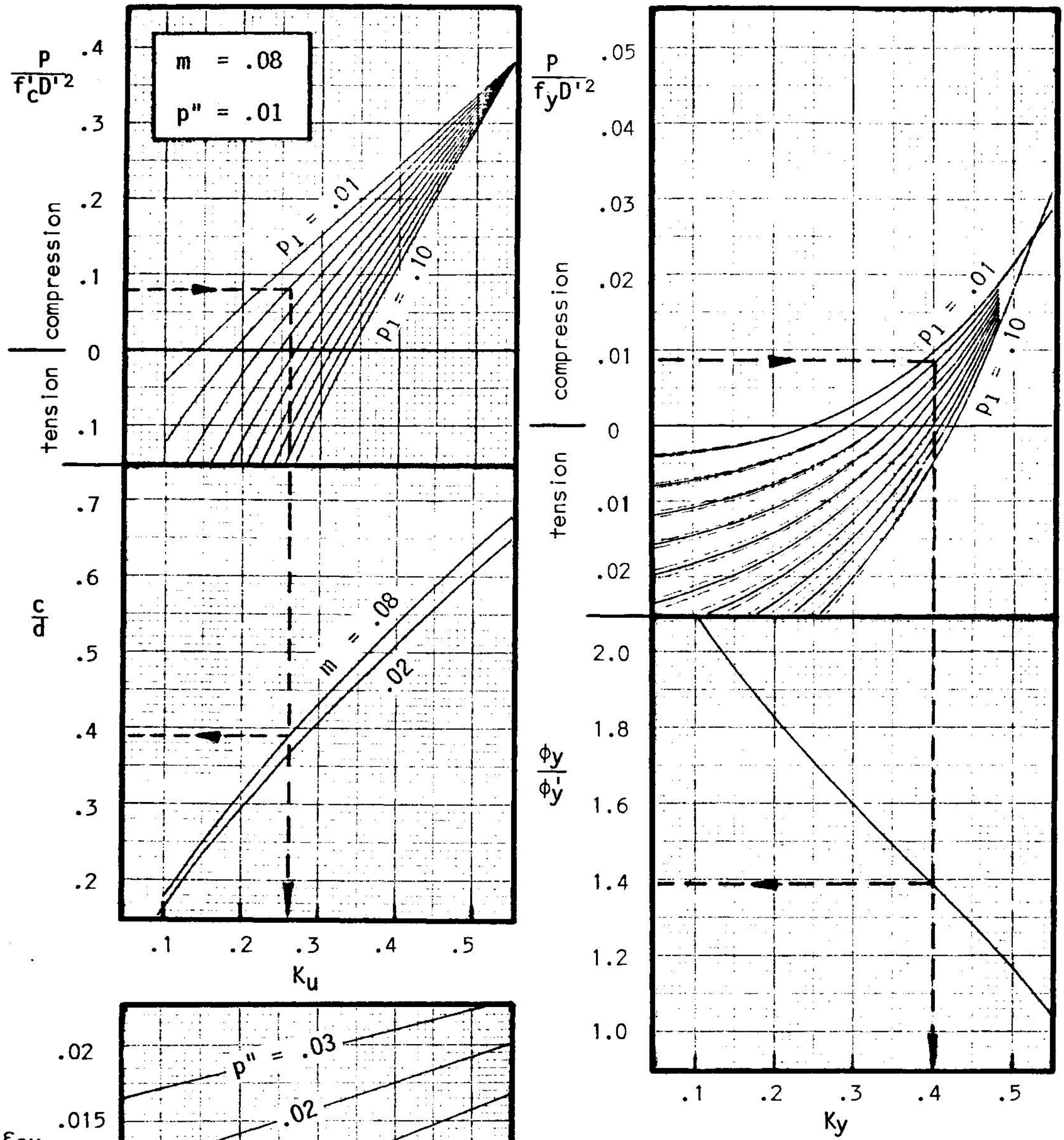
4. Then section ductility = ϕ_u/ϕ_y

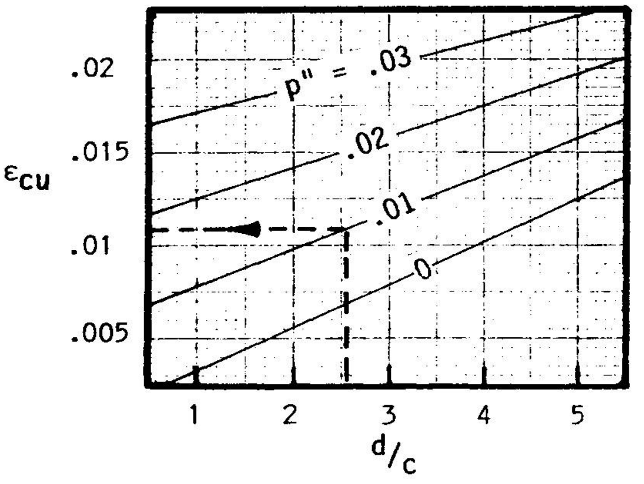


NOTATION

- f'_c = specified compressive strength of concrete
- f_y = specified yield stress of reinforcing steel
- P = total axial force on pier section
- p" = ratio of volume of confining hoops to volume of confined concrete measured to outside of hoops
- c = distance from extreme compression fibre to neutral axis at limit condition
- d = distance from extreme compression fibre to centre of force of tensile reinforcement (effective depth)

FIG 4: Summary of Procedure for Calculating Section Curvature Ductility.





Example:

m = .08 p" = .01 $\frac{P}{f'_c D'^2}$ = .08 p_1=.02

$$\phi_u = \frac{\varepsilon_{cu}}{D'K_u} = \frac{.0108}{.26D'} = \frac{.0415}{D'}$$

$$\phi_y = \frac{\varepsilon_y}{D'(1-K_y-m)} \times \frac{\phi_y}{\phi'_y} = \frac{.001375}{D'(1-.4-.08)} \times 1.39$$

$$= \frac{.00368}{D'}$$

Section Ductility = ϕ_u/ϕ_y = 11.3

FIG 5: Example Charts for Calculation of Curvature Ductility. Circular Section, f'_c = 30 MPa, f_y = 275 MPa

3. APPLICATION OF MECHANICAL ENERGY DISSIPATING DEVICES

3.1 Details of Devices

In recent years a number of methods of isolating a structure from the effects of ground shaking have been proposed, but have not met general acceptance because of practical deficiencies, in particular the expected excessive displacements under wind or earthquake. However, a system overcoming these deficiencies has been made possible by the development of practical mechanical devices which act as hysteretic dampers. Detailed information on development and testing of these devices is given by Skinner, Robinson et al [11,12].

A number of the devices developed to date are illustrated in Fig 6. Four of the devices shown dissipate energy through cyclic yielding of mild steel elements, in either torsion or flexure. The other two devices rely on "hot working" of lead; during its deformation either in extrusion or in shear the lead recovers most of its mechanical properties almost immediately, and exhibits "coulomb damping" characteristics. All devices have been tested at earthquake-like frequencies and displacement amplitudes and have exhibited stable hysteretic characteristics for several hundred cycles of loading. Although energy is eventually dissipated in the form of heat, the temperature rise in the devices was only of the order of 5-10°C when subjected to the loading expected in a severe earthquake. The devices have been patented by the New Zealand Department of Scientific and Industrial Research and are marketed in this country.

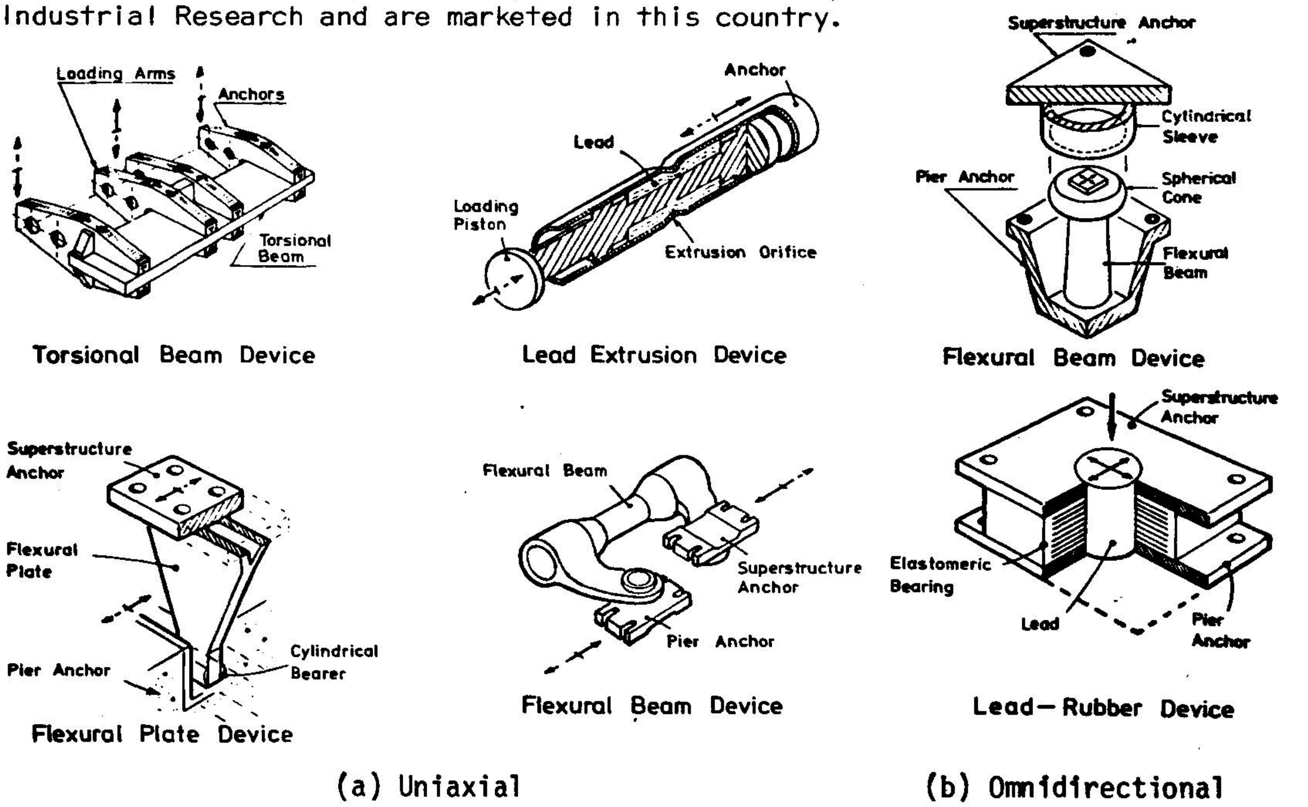


FIG 6: Schematic Drawings of Mechanical Energy Dissipating Devices

3.2 Practical Applications

The application of energy dissipating devices to bridges is shown in Fig 7 with respect to four examples. In general the cost of the devices and associated details is of the order of 1% of the bridge cost. Details are as follows:

(a) This six span prestressed concrete box girder railway bridge on tall reinforced concrete piers is currently under construction. The design concept is

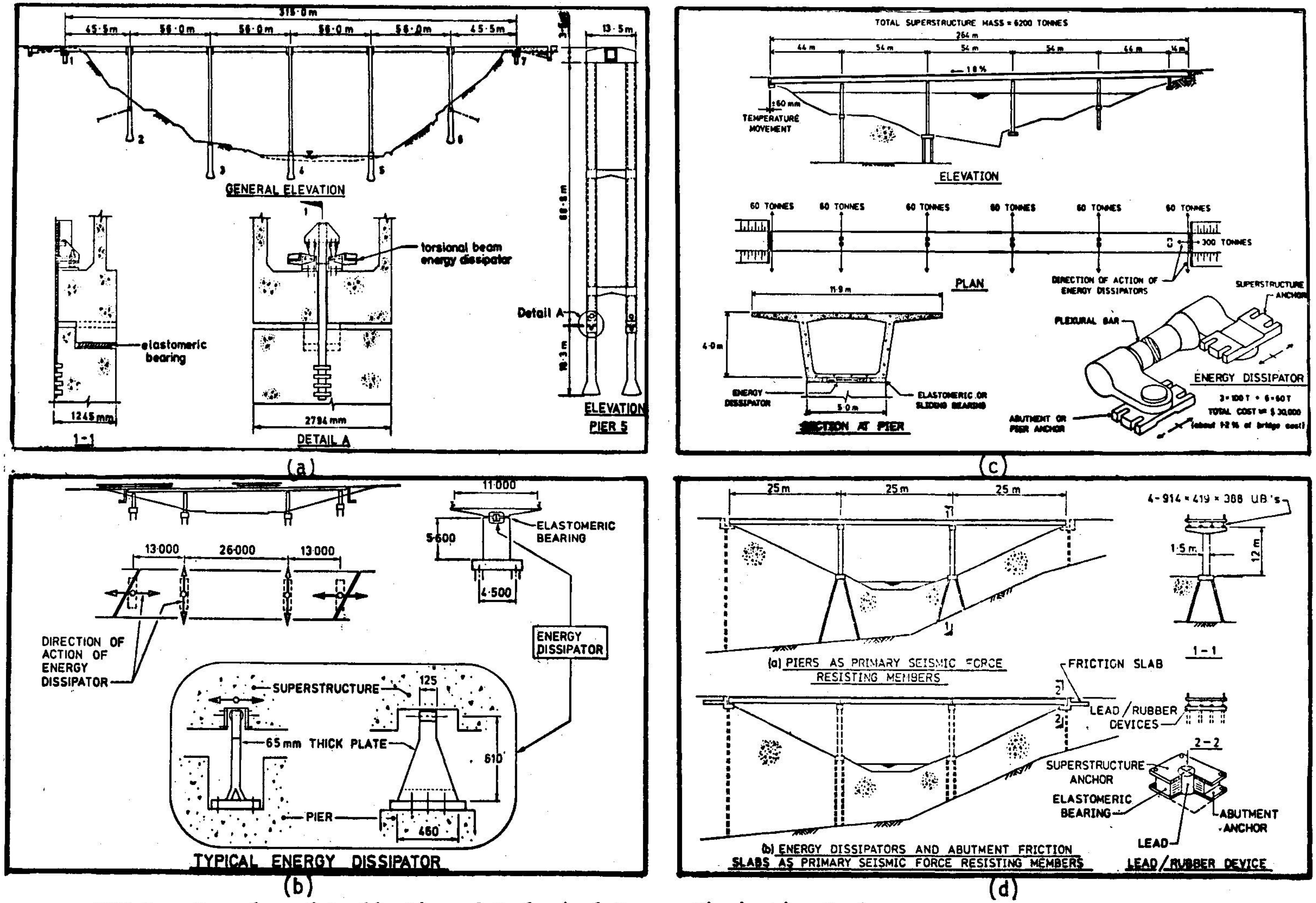


FIG 7: Examples of Application of Mechanical Energy Dissipating Devices.

that under earthquake attack the piers will "step", that is rock transversely to the bridge axis with each leg alternately lifting from the foundations. The lateral displacements are limited to acceptable values by energy dissipation in devices of the 'torsional beam' type at the base of the piers. The cost of the structure with the "stepping" details is comparable to that without, but the benefits lie in protecting the structure from earthquake induced damage and in limiting the axial forces induced in the piers.

(b) A three-span twin overpass with a prestressed concrete hollow cell superstructure and reinforced concrete piers and abutments was designed with uniaxial action energy dissipating devices of the steel "flexural plate" type. Those at the tops of the piers act in the transverse direction of the superstructure and those at the abutment act longitudinally. Provision is made in the latter location for accommodation of lengthening and shortening effects, such as thermal and creep movements, before the devices become effective. The design forces chosen for the structure were similar to those for a conventional design; the benefit was that without any cost penalty the degree of protection against damage was dramatically increased.

(c) A conceptual design of this six-span prestressed concrete box girder bridge on tall reinforced concrete piers incorporated energy dissipating devices of the uniaxial "flexural beam" type. Longitudinal seismic forces are resisted by devices at one rock abutment. Lateral forces are resisted by devices at the tops of the piers. The advantage in this case is that it is economic to design the piers not to yield even under a severe earthquake, and therefore damage may be avoided below the waterline where it is difficult to inspect and repair.

(d) As a result of preliminary studies on this three-span steel universal beam and concrete deck bridge, the seismic design concept was changed from a conventional approach with ductile reinforced concrete piers to a system with energy dissipators and abutment friction slabs as primary seismic force resisting members. The devices adopted were of the "lead/rubber" type, which are simple in application being positioned in the same manner as a normal elastomeric bearing, omnidirectional in action and can creep to accommodate lengthening and shortening effects. The benefits were substantial construction economies for piers and foundations and a large reduction in superstructure movements to be allowed for at the abutments.

3.3 Dynamic Response Analysis Results

Results of extensive numerical integration time-history dynamic analyses in the bridge shown in Fig 7(b) are described more fully elsewhere [8,13]. Fig 8 plots the relationship between maximum force imposed on the substructure of this bridge and period of vibration of the structure. Allowance was made for flexibility of the foundations and a vibrating mass of soil. Damping ratios of 4% and 5% equivalent viscous damping were assumed for modes 1 and 2 respectively. The yield level of the combination of dissipators acting in each principal direction was chosen as 0.05 times the weight of the superstructure. The properties of the elastomeric bearings were chosen after sensitivity analyses. The bearings require sufficient shear stiffness to provide an adequate centring force, but should not be so stiff as to impose excessive forces on the substructure at design earthquake displacements. The structure incorporating energy dissipating devices is compared in Fig 8 with an elastic single mass resonator with 5% equivalent viscous damping responding to three earthquakes, namely El Centro 1940 N-S and the artificial earthquakes B1 and A1. The curves show the dramatic reduction that may be achieved in response forces by the incorporation of energy dissipating devices, particularly for stiff structures. This is accomplished without requiring the structural members to sustain inelastic deformations and damage.

Some results of the time-history of displacement response of the structure shown

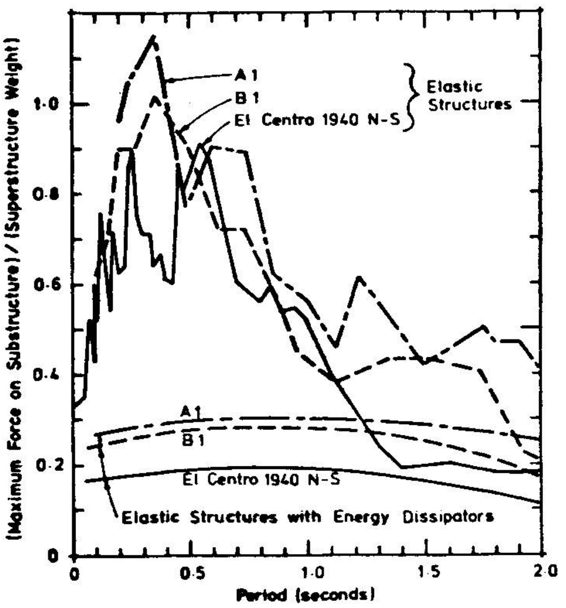


FIG 8: Bridge Substructure Forces for Elastic Structures with and without Dissipators

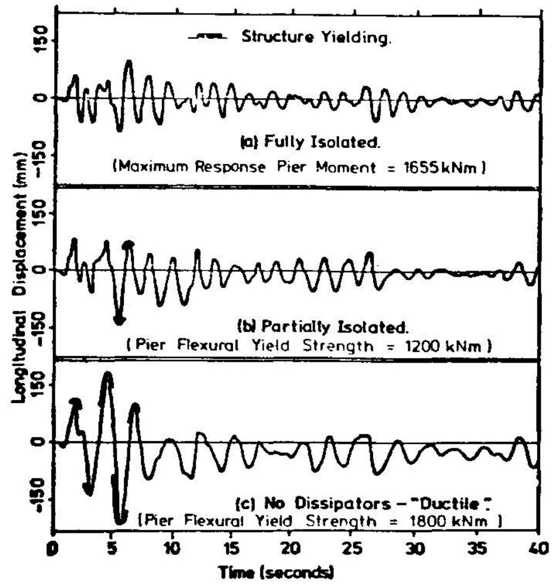


FIG 9: Longitudinal Response of Bridge for Three Structural Alternatives (El Centro 1940 N-S)

in Fig 7(b) to the El Centro 1940 N-S earthquake are shown in Fig 9. Cases (a) and (b) incorporate energy dissipating devices of equivalent characteristics. However, in case (a) the piers have been reinforced so that they would not flexurally yield during an earthquake of this intensity, whereas in case (b) the strength of the piers has been reduced to achieve construction economies. Case (c) corresponds to the conventional ductile design approach without energy dissipators. The intervals of the response during which the piers were yielding is shown. Comparison of (a), (b) and (c) shows that incorporation of energy dissipating devices has the potential for reduction of response displacements, reduction or elimination of damage at the "design" earthquake intensity and construction economies through reduced design forces on structure and foundations.

3.4 Design Philosophy

3.4.1 Advantages

The use of mechanical energy dissipating devices offers a number of advantages for the design of earthquake resisting bridges and other structures such as buildings, nuclear power plants and offshore gravity platforms.

(a) *Conceptual Simplicity:* Earthquake energy dissipation is concentrated in specially developed components which may be readily replaced if necessary. By effectively increasing the structural damping, a desirable seismic response may be achieved.

(b) *Design Simplicity:* The potential for development of standardised solutions for design of bridges incorporating these devices is an advantage relative to the substantial design effort required in the conventional ductile design method.

(c) *Structural Optimisation:* The designer is free to adjust the principal variables, being yield level of the dissipators, yield level of the structural members and intensity of design earthquake, to achieve an optimum solution in terms of construction costs and an acceptable frequency of earthquake induced damage.

(d) *Use of Non-Ductile Forms or Components:* There is new scope for economic or aesthetic advantage through use of non-ductile structural forms or components with sufficient strength to remain elastic under the expected forces imposed by yielding of the dissipators.

(e) *Superior Performance:* Response displacements may be reduced and structural damage minimised or eliminated at the "design" earthquake intensity.

3.4.2 Philosophy

The following is a suggested philosophy based on varying levels of earthquake attack:

(a) *Moderate Earthquake:* For a moderate earthquake, such as may be expected 2 or 3 times during the life of a structure, energy dissipation should be confined to the devices and there should be no damage to structural members.

(b) *"Design" Earthquake:* For a "design" earthquake, for example one with a return period one or two times the anticipated life of the structure, the designer may adjust the strength levels in the structural members to achieve an optimum solution between construction economies and anticipated frequency of earthquake induced damage, with regard to the client's wishes. However, the degree of protection against yielding of the structural members should be at least as great as that implied in relevant codes relating to the conventional seismic design approach without dissipators.

(c) *Extreme Earthquake:* For an extreme earthquake there should be a suitable hierarchy of failure of the structural and foundation members that will preclude a brittle collapse. This may be achieved by appropriate margins of strength between non-ductile and ductile members and with attention to detail.

Although the above philosophy encompasses three earthquake levels, the design practice need only be based on the "design" earthquake. In the course of that design, the implications of yield levels on response to the "moderate" earthquake would have to be considered, as would also the implications of strength margins and detailing for an "extreme" earthquake.

4. CONCLUSION

There have been recent developments in New Zealand regarding the seismic design of bridges both for the conventional approach, based on ductile flexural yielding of the piers, and an alternative approach using mechanical energy dissipating devices. For the conventional approach, research has given useful information on the ductility demand on reinforced concrete piers, and their ductility capability. Also, a design office procedure has been developed for ready assessment of the ductility capability of a bridge structure. The development of practical, low cost, low maintenance mechanical energy dissipating devices has fostered the alternative approach of isolating the structure from the worst effects of ground shaking. This approach has potential for construction cost savings and a greater degree of protection against damage to structural members, with particular benefits for structures in the more seismically active areas.

5. ACKNOWLEDGEMENTS

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