EARTHQUAKE RESISTANT DESIGN OF BRIDGES AND THE N.Z. MINISTRY OF WORKS BRIDGE DESIGN MANUAL

by

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SYNOPSIS

The New Zealand Ministry of Works Bridge Design Manual is being revised and recommendations for the provision of earthquake resistance will be included. The following paper discusses current thinking on the problem, concentrating on the design of main structural elements.

INTRODUCTION

In common with other structures bridges vary in their purpose and structural characteristics so widely that comprehensive coverage of the problems associated with their design for earthquake resistance would be very lengthy. Furthermore, the number of factors affecting their design and the possible combination of such factors make it desirable to consider each structure's likely seismic behaviour from basic principles so that the best compromise can be reached to satisfy the often opposing design requirements.

The Ministry of Works bridge office is mainly involved in highway bridge design work, a large proportion of which falls into one of two categories. There are many river crossing bridges where multi-spans up to 80 feet and pier heights up to about 25 feet are chosen. For such structures simply supported prestressed concrete beams are commonly used, whilst structural steelwork is used only occasionally. Longer spans may be of steel truss or girder types. The second category comprises motorway bridging, which has received much attention in New Zealand in recent years, particularly in urban areas. The complicated geometry involved and the desire for more aesthetic structures has led to increased use of a wider variety of superstructure types, both in simply supported and continuous form. Much of this work has been in concrete although a few steel superstructures have been used. In virtually all cases reinforced concrete piers are used.

The remaining designs consist mainly of larger projects which usually merit more extensive and individual consideration than a design manual approach can give. Although the principles set out in the following discussion are often applicable to such designs, attention is directed mainly to the types of structure forming the bulk of the work.

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DESIGN CRITERIA

When setting design standards it is necessary to clarify basic performance criteria for bridges to allow reasonable compromises to be made. Whilst it is desirable for all structures to be designed to be damage free in all likely earthquakes, economics do not justify such a policy for the majority of bridging. The main criterion is for total or partial collapse during major shaking to be prevented. In addition it is desirable for the structure to be usable by light traffic as soon as possible after a major earthquake and for damage to be avoided during the more frequent but less severe shakings likely to occur.

In accepting that damage may occur during severe shaking other factors are relevant. Secondary damage to expensive items is likely to be much less of a problem than for buildings, as is occupier reaction or panic. In urban locations services carried by the bridge may require special details to permit generous movements at joints but, otherwise, investment for earthquake resistance is normally protecting only the structure. Consequently, the percentage of total structure cost justifiable for earthquake protection is smaller than that for buildings. The policy adopted by the Ministry of Works in this direction is reflected in the force levels recommended for design and in the stated acceptance of limited damage occurring at predetermined locations.

DESIGN BASIS

The type of bridge under discussion is likely to act essentially as a single degree of freedom oscillator and response spectra therefore provide convenient directly applicable aids to analysis.

Reference 1 presents curves for seismic loading of public buildings which basically represent acceleration response spectra envelopes for use with structures in the country's three zones of assumed seismic activity. The curves are shown in figure 1. The zone A curve envelops the acceleration response spectrum, for 10% damping, of the El Centro 1940 N-S record, reduced by an assumed structure ductility of 4. For longer period structures the curve gives considerable coverage above this spectrum. For ultimate strength a load factor of 1.25 is applied to the loadings[I].

Work reported in references (2) and (3) indicates that a value of critical damping nearer to 5% than 10% is likely to be most generally applicable to the type of structure being considered. On this basis structural response would be greater than implied by the reference 1 curves unless structural ductilities of about 6 were utilised. Considering the two parameters of strength and ductility it is normally

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[I] A draft revision of reference 1 is currently out for comment. Although the method of presentation has been varied, the resultant seismic coefficients for ultimate strength are unchanged except for longer period structures in zones B and C. Recognition of likely amplifying effects of flexible subsoils is also proposed for introduction into the curves.

more economic in bridges to design for a smaller horizontal strength and accept the necessity for greater ductility. Although the plastic hinging usually forms in the piers, average axial stresses are usually relatively small and the piers are therefore able to act as flexural rather than as axially loaded members.

Whilst there is a limit to the structure ductility attainable in practice, increasing design strength increases the structure cost, particularly that of the foundations. A lower design strength reduces resistance to damage during minor shaking. Determination of optimum values for the design basic seismic coefficient to take these factors into account is necessarily a matter of judgement. The N.Z. Ministry of Works is adopting the values shown in figure 1, combined with a structure ductility of 6. Further provisions are made in cases where, because of the size and proportions of the piers adopted, a plastic mechanism cannot form at the specified loading but would occur at greater response accelerations of the structure, up to a theoretical elastic response of 6 times the values shown in figure 1.

Since the specified values are intended for bridges on the most important routes a reduction factor is applicable for the design of other structures, as set out in Table 1. In effect, category 1 includes bridges around most of the main centres of population and all motorway bridges. Category 2 includes practically all major routes between centres and category 3 covers back-country routes. It is intended that the classification be used with judgement in the case of low usage but important structures, or where alternative routing is not readily available.

Design of members for earthquake resistance is carried out by conventional ultimate strength design methods. Attention is also given to checking that the plastic mechanism assumed in derivation of the specified loading will occur as required, with hinges forming in the desired locations.

STRUCTURAL DESIGN

General

With a design philosophy which includes the acceptance of flexural damage occurring during the more severe shakings it is important for such damage to be located in positions which are easily visible and preferably accessible for repair work to be carried out if necessary. This requirement is not difficult to meet except in the frequent case of piers or abutments supported on deep foundations such as piles or bored cylinders. In such a case it is policy to choose a structural form for the piers which can be designed to develop the plastic hinging in the visible part of the pier, while the foundation members are designed to remain elastic under these conditions. Although it must be accepted that such foundations may be subjected to testing conditions because of soil response (4 and 5) this policy does at least lead to reasonably predictable loads being imposed from the superstructure. Protection of foundation elements particularly against shear failure but also preferably against post-yield flexural behaviour is considered important in river situations because pile damage is likely to go undetected until flood and deep scour conditions find such a weakness.

Strength Design

Although elastic analysis under specified loads, followed by ultimate strength design, leads to adequate strength of the sections where plastic hinging is intended, it does not guarantee the mechanism will form as desired. Complications may occur in this respect because:

(a) The actual yield strength of reinforcement may be up to 25% above the minimum specified[6] and the actual section may also justify use of an undercapacity factor $\phi = 1$.

(b) Strain hardening may occur to increase still further the actual flexural capacity of the plastic hinge areas. This effect occurs after smaller post yield strain in the 60 ksi yield strength steel available in New Zealand than in the mild steel of 40 ksi yield strength. Use of the higher strength steel in members intended to form plastic hinges is therefore not recommended, while its use elsewhere can be advantageous[6].

(c) Design of a pier to give adequate restraint in one direction may result in its actual yield strength in the perpendicular direction being higher than the minimum required.

Thus, the structural design process is one where the plastic hinge areas are designed to have the minimum required strength and the remaining members are designed to be compatible with the most unfavourable condition which can apply in forming a plastic mechanism. Above all, protection against brittle shear failure is required. In an attempt to avoid excessive compounding of safety factors, the following minimum design standards are adopted for members in which plastic hinges are to be prevented:

(a) Earthquake induced member forces should be 1.15 times those obtained from a frame analysis assuming all plastic hinges to have formed. Plastic hinge moments should be calculated assuming longitudinal steel in the section to be stressed 25% above minimum specified yield and undercapacity factor $\phi = 1$.

(b) Member flexural reinforcement may be calculated assuming its yield strength to be 15% above minimum specified yield and undercapacity factor $\phi = 1$.

(c) Member shear design should be calculated assuming shear steel yield strength as minimum specified and undercapacity factor $\phi = 1$. Concrete may be assumed to carry some shear except if member average axial stress is less than .12 f'_c. All shear to be carried by transverse reinforcement where plastic hinging may occur. Confining steel to be placed in such areas.

(d) Overall structure stability should be based on the forces calculated in (a) above.

Confining reinforcement in plastic hinge areas should normally be in accordance with the ACI 318-71 provisions. Caution is necessary in designing piers with low average axial stresses as flexural members in accordance with ACI 318-71. In bridges with relatively high piers the shear necessary to cause plastic hinging may create quite a low shear stress with a consequently apparently small quantity of transverse steel necessary. Reference 7 suggests that in such conditions more confining steel may be necessary and at present therefore, it is considered advisable in such cases to comply at least with the requirements for confinement, but without applying the alternative minimum of $P_s = .12\ f'c/fy$, which governs for circular or square piers larger than about 3 ft.

The design approach discussed above can lead to very large forces for foundation elements, up to a maximum equivalent to elastic response of the structure. The choice then lies between providing strong foundations or using pier systems with lower yield strengths. For this reason, the 'slab' type of pier is currently less favoured for structures founded on piles. The problem does not arise with spread footings except in design of the superstructure/pier connections. At this point suitably increased design loads are usually easily and cheaply accommodated.

Ductility Check

Structure ductile capacity is checked using the approach outlined in reference 8. Although incorporating several approximations it does form a basis for judging structure capability. The relatively simple plastic mechanisms likely to form in bridges help to simplify the ductility check calculations.

Superstructure Inertia Forces on Abutments

Another area in which non-ductile behaviour can lead to unpredictably high response accelerations is at abutments. Problems can result from their stiff nature:

(a) Because the lack of plastic action requires larger design forces to prevent shear failures. Considerable damping is likely to prevent much dynamic amplification and design for maximum ground acceleration is therefore considered reasonable (say 3 times the values shown in figure 1). It is however difficult to predict the mass to which such an acceleration should be applied because:

(b) The relative transverse stiffnesses of piers and abutments are very different and transverse analysis, taking post-yield deflections of the piers into account, usually indicates that the abutments would provide most of the seismic resistance. In simply supported structures with spans linked by rubber-packed linkage bolts, this effect is reduced, though indeterminate.

Because of uncertainties associated with such cases it is normal policy, on structures with 2 or more piers and with continuous deck diaphragm action available, to separate superstructure from abutments transversely and to carry all transverse loads on the piers. To minimise horizontal rotation of the deck about a vertical axis pier stiffnesses are chosen with the aim of making the centre of mass of the superstructure coincide with the centre of rigidity of the pier tops.

It is also necessary to consider the share of any such induced rotational energy resisted by each of torsion and shear in the piers. Too close a longitudinal pier spacing can lead to high torsion stresses, whilst resistance by flexure is preferable. When pier spacing depends on other considerations and is unsuitable from this point of view, transverse superstructure freedom from abutments may therefore not be possible.

Choice of Structural Form

The choice of structural form at the scheme stage can obviously have considerable bearing on the subsequent seismic behaviour and costs involved. Although the type of superstructure is usually chosen according to factors other than seismic, a range of possibilities is open to the designer in the treatment of the chosen superstructure to produce optimum earthquake resistance. Table 3 summarises the possibilities. Desirable seismic features unfortunately lead to complications from other sources - for example it is instinctively preferable to introduce redundancy into the structure by making the piers and superstructure monolithic but superstructure shortening effects can induce large pier moments with the reduced pier flexibility. The compromise solution depends mainly on the balance of pier heights, foundation conditions and spans. Types 1a(iii) 1b(ii) and 1c(iii) listed in table 3 all have the disadvantage of being fixed to a usually stiff abutment with a consequent lack of ductile capacity to justify lower design forces. In addition the concentration of unpredictably large forces from a long length of bridge results in the need for a substantial anchorage structure. Types 1a(i), 1b(i) and 1c(i) may be given flexibility to accommodate shortening and thermal effects by introducing rubber linings into shear key pockets and rubber packings between linkage bolt washers and superstructure diaphragms. Satisfactory results under service conditions have been obtained with this solution.

Movement Joints

In recent designs more attention has been paid to providing for seismic movements. It is usually not difficult to protect main structural members from impact damage by allowing large clearances at movement joints. Minimum required clearances are calculated as $1\frac{1}{2}$ times the elastic response displacement (i.e. 9 times the elastic deflection under the loading shown in figure 1), assuming that separate adjacent structural sections may oscillate out of phase. Although usually easy and inexpensive to provide these clearances it is expensive to continue such movement provisions into the deck surface with compatible deck movement joints. A policy of permitting secondary damage to occur at deck level is therefore followed, providing only a gap to cater for moderate shaking. A seismic displacement of 2 times elastic deflection under the loading shown in figure 1 is adopted for this case. So as to concentrate the secondary damage occurring, a 'knock-out' wedge of concrete, tied to the main structural concrete only with light reinforcement, is inserted as part of the deck on one side of the movement joint. At abutments the approach settlement slab is constructed to rest on top of and project approximately a foot beyond the face of the abutment backwall. Wheel loads are spread by locating the slab approximately 1 foot below road level and the deck joint is incorporated into an upstand built on the slab at the end adjacent to the bridge superstructure. Impact from the bridge during severe longitudinal motion is intended to displace the settlement slab and create the required clearance, whilst, with approach fill settlement likely, ramped access onto the bridge would be maintained.

Holding down bolts

Associated with provisions for the movements discussed above are those necessary to avoid damage to holding down bolts. The basis of design for these is usually arbitrary at points of relative movement and a figure of 0.1 times the dead load reaction is commonly used. Anomalies can occur however (e.g. at the ends of cantilever structures) and judgement is therefore necessary. In the normal case it is considered preferable to provide tie-down for spans during earthquake and long bolts are normally used with adequate freedom of movement provided for the bolt at least to yield in flexure rather than to shear. Rubber packing is provided under washers in cases where horizontal movements would otherwise impose large strains in the bolts. Provision is also made for nuts to be readily accessible to allow for slackening so that bearings can be replaced. Where convenient, bolts are detailed to allow for withdrawal and replacement.

Stiff Non-ductile Structures

Although many structures can be designed to be isolated from the stiff members which induce unpredictable behaviour, this is not possible in some cases. These include:

(a) Simply supported spans restrained transversely at the abutment.
(b) Short bridges of one or two spans where restraint at the abutment is unavoidable.
(c) Bridges with short piers where minimum practical pier proportions and strength prevent ductile flexural behaviour.

Ideally the design should be based on maximum predicted elastic response but the economic justification is doubtful, particularly in the case of (b) and (c), where the cost of what would normally be a minor structure would be considerably increased. Moreover, such cases would not normally represent collapse risks if brittle failure occurred but behaviour would more likely be one where failure of superstructure restraint would lead to sliding on its supports. The policy for such structuresis therefore one of accepting that damage is likely to occur in severe conditions and providing the structure with horizontal capacity according to Table 2, increased by a load factor of 1.3. Horizontal restraint maybe by shear keys or bolts in tension with clearances left between members to reduce the chances of hammering damage during severe conditions. It is still an aim to minimise inertia forces developed in foundations and it is therefore desirable to design these to resist the predictable maximum which can be transmitted to them. Work is at present in progress on means of connecting superstructures to supports with a ductile force limiting device, but until this is available and is used with sliding bearings, prediction of maximum inertia forces likely to be imposed on supporting members in such situations is very uncertain.

CONCLUSION

Attention has been directed to discussing problems associated with designing small and medium sized bridges to absorb seismic energy by ductile flexural action of bridge members. Whilst alternative methods of increasing the damping are likely to become more widely used in future it will still be necessary, for economic reasons, to approach the design with an aim of encouraging any likely damage to occur in predetermined locations. At such positions concrete will still be required to behave in a ductile manner.

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Figure 1 - Basic seismic coefficient C

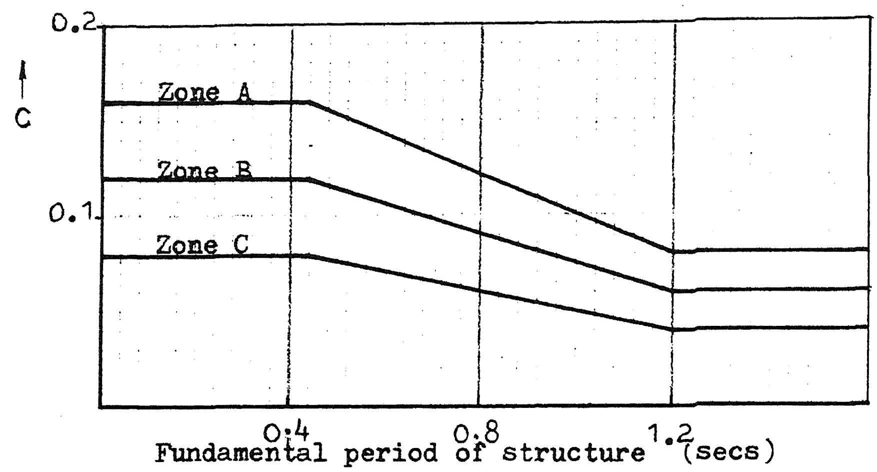

Table 1 - Importance factors F

Category 1	Motorway under or overpass or bridge carrying more than 2,500 vehicles per day	1.0
Category 2	Bridge carrying between 250 and 2,500 vehicles per day	0.85
Category 3	Bridge carrying less than 250 vehicles per day	0.7

Table 2 - Minimum horizontal seismic coefficient for design of non-ductile structures - equivalent to C x F

	Zone A	Zone B	Zone C
Category 1	.24	.18	.12
Category 2	.20	.15	.10
Category 3	.17	.13	.09

NOTE: Seismic base shear = $V = C \times F \times W_t$ where W_t = dead load of the structure

Minimum ultimate strength is required to be adequate to resist $1.3 \times V$.