Paper No. 2111. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

NEW ZEALAND PERSPECTIVES ON SEISMIC DESIGN OF BRIDGES

R. PARK

Department of Civil Engineering, University of Canterbury
Christchurch, New Zealand

ABSTRACT

The paper summarises the current seismic design philosophy for concrete highway bridges in New Zealand. The background developments and the seismic design provisions are outlined. The seismic design actions at the ultimate limit state are obtained from the response spectrum appropriate to the site conditions and the structure displacement ductility factor appropriate to the bridge substructure. Capacity design is used to ensure that most desirable energy dissipating mechanism forms in the substructure in the event of a severe earthquake. Members and joints are detailed to ensure that the required ductility is available and that the bridge structure behaves as intended.

KEYWORDS

Bridges; columns; confinement; ductility factor; energy dissipation; flexural deformations; flexural strength; piers; seismic design criteria; shear strength.

BRIDGE DESIGN SPECIFICATIONS

In New Zealand the design of highway bridges on public roads is conducted using a Bridge Manual (Transit New Zealand, 1994) prescribed by Transit New Zealand, which is the authority set up by Government to administer the funding, construction and operation of the highway system.

The principles of the seismic design approach of the Bridge Manual have evolved from the recommendations of a study group of the New Zealand National Society for Earthquake Engineering (Study Group, 1980). The seismic design loadings for bridges in the Bridge Manual are those recommended in the New Zealand loadings standards for buildings (Standards New Zealand, 1992), modified appropriately to apply to bridges. The concrete design is conducted in accordance with the New Zealand concrete design standard (Standards New Zealand, 1995).

Significant information on the seismic design of bridges has been published in recent years in papers in the Bulletin of the New Zealand National Society for Earthquake Engineering, in the papers and summary volumes prepared for Bridge Design and Research Seminars organised by the Road Research Unit of the National Roads Board in Auckland in 1984, and more recently by Transit New Zealand in Christchurch in 1990 (Park and Paulay, 1990; Wood and Elms, 1990; Chapman and Kirkaldie, 1990; Jennings et al, 1990; Summary papers and other technical papers, 1990).

This paper summarises the current seismic design provisions for concrete bridges in New Zealand and outlines some of the background to those provisions.

BRIDGE STRUCTURAL SYSTEMS

A range of highway bridge types are in use in New Zealand. Most bridge superstructures constructed in New Zealand in recent years utilise standard precast prestressed concrete units in the form of I or T sections, or hollow core or U or W sections, or solid rectangular sections, depending on the span. Pretensioned or post-tensioned, or combined pre and post-tensioned, units have been used. Generally the precast units act compositely with cast in situ concrete slab or topping. Precast segmental construction has also been used. Totally cast in situ concrete bridge superstructures are more economical and appropriate for long spans, usually continuous, especially where shallow construction depths are necessary, or where the geometric shape of the structure is complicated. Steel superstructures have also been used for longer spans, mainly in the form of modular steel truss form, but also of steel box girder form with composite concrete deck.

Superstructure spans are always connected at intermediate supports, either by continuity, by deck linkage slab or by linkage bolts.

The substructures of bridges tend to be constructed of reinforced concrete and to be relatively simple structures, involving few structural members. A typical bridge substructure may consist of one or more columns, or a wall, supported by either a spread footing or a pile cap connected to driven or cast in situ piles. At the top of the columns or wall is a cap which supports the superstructure. Sometimes the cap is a diaphragm within a box girder.

Recently constructed highway bridge substructures frequently consist of single or multiple steel encased cast in situ concrete piles beneath ground level with a reinforced concrete column and column cap above supporting the bridge superstructure. The end of the bridge superstructure can be designed to be free to rotate and translate relative to the substructure, or designed with integral or semi-integral abutments.

According to the Bridge Manual (Transit New Zealand, 1994), an integral abutment is defined as one which is built integrally with the end of the bridge superstructure and with the supporting piles. The abutment therefore forms the end diaphragm of the superstructure and the retaining wall for the approach filling. The supporting piles are restrained against rotation relative to the superstructure, but are free to conform to superstructure length changes by pile flexure. A semi-integral abutment is defined as an integral abutment which contains provision for relative rotation, but not translation, between the superstructure and the supporting piles.

SEISMIC DESIGN PHILOSOPHY

According to the Bridge Manual (Transit New Zealand, 1994) a bridge shall be useable after being subjected to the design ultimate limit state earthquake, which has a return period of 450 years, although some damage may have occurred and some repairs may be necessary.

The Bridge Manual defines categories of structures depending on the manner in which the structure will respond to earthquake shaking. A bridge substructure may be designed for a horizontal earthquake loading which is somewhat less than the elastic response inertia force induced by a major earthquake, providing that the substructure is designed so as to be capable of dissipating energy by plastic deformations to the required level of ductility in regions of the structure chosen by the designer. Sufficient strength is provided elsewhere in the structure to ensure that the chosen energy dissipating mechanism does develop in the event of a major earthquake. A key aspect of this capacity design approach is the choice of an intended mode of structural behaviour during strong earthquakes, and then the design and detailing of members to ensure that the structure will behave as intended.

When the chosen energy dissipating mechanism for single or multiple column substructures involves plastic hinges in the substructure, the plastic hinges should preferably form in the columns rather than in the foundations (footings or pile caps or piles), because of the greater accessibility for inspection and repair of the columns.

Base isolation and energy dissipating devices, in the form of lead/rubber bearings or steel torsional or flexural devices or lead extension devices, have been incorporated in bridges in New Zealand since 1973. They are normally used to lengthen the natural period of the structure and to increase the damping, thereby modifying the seismic response. The resulting benefits generally include the feasibility of designing the structure to remain elastic at high levels of seismic shaking (even up to the design ultimate limit state earthquake) thereby reducing or eliminating damage in strong earthquakes.

The procedures of structural analysis (equivalent static force, response spectrum and inelastic time history) are set out in the Bridge Manual, including recommendations for member properties for analysis and the manner of calculating displacements of bridge structures due to earthquake shaking.

SEISMIC ACTIONS

Seismic Design Loads

The procedure for determining the equivalent static horizontal seismic loading in the Bridge Manual (Transit New Zealand, 1994) follows the approach recommended by the New Zealand loadings standard for buildings (Standards New Zealand, 1992). That approach determines the basic acceleration coefficient from uniform seismic hazard acceleration spectra for the applicable site subsoil category and an assumed structure displacement ductility factor. Fig. 1 shows uniform seismic hazard spectra for intermediate soil sites. Spectra are also given in the Bridge Manual for rock or very stiff soil sites and flexible or deep soil sites. As shown in Fig. 1, the spectra are presented for a range of values of the structure displacement ductility factor μ. The structure displacement ductility factor is defined as $\mu = \Delta_{max}/\Delta_y$, where Δ_{max} is the maximum horizontal displacement and Δ_y is the horizontal displacement at first yield, both measured at the centre of mass of the superstructure (see Fig. 2). Hence the designer can select the basic design acceleration coefficient C_μ corresponding to the required ductility. The lower the design seismic load the greater the required ductility. The ductility values presented range from $\mu = 1$ (elastic response) to $\mu = 6$ (the maximum allowable design value).

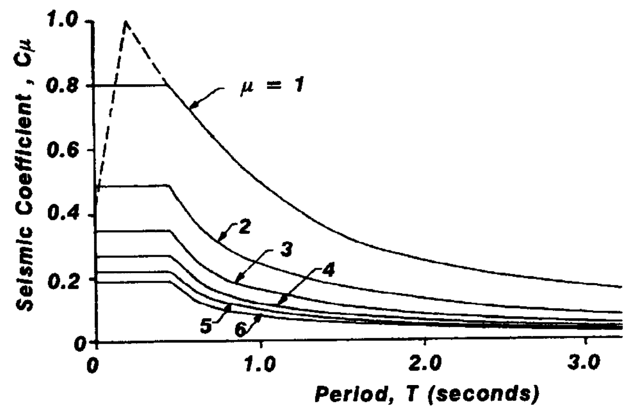

Fig. 1 Uniform seismic hazard spectra for intermediate soil sites (Transit New Zealand, 1994)

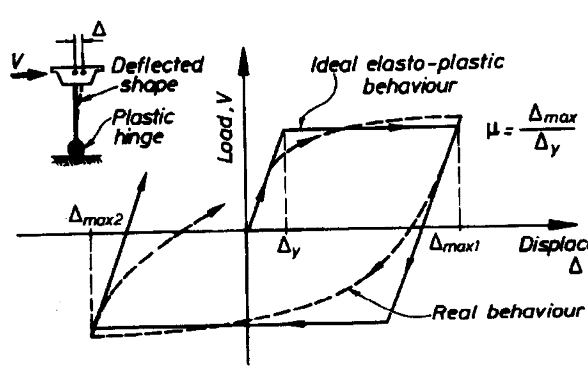

Fig. 2 Illustration of ductile behaviour of a cantilever bridge structure during cyclic horizontal loading

The ordinates of the inelastic ($\mu > 1$) spectra were obtained from the corresponding curves for the 5% damped elastic response ($\mu = 1$) by considering the yield levels of single degree of freedom oscillators required to obtain various displacement ductility factors as a ratio to the maximum elastic response values for each elastic period. For short periods, the inelastic design spectra show a plateau from 0 to 0.2 seconds for stiff subsoils, 0 to 0.45 seconds for intermediate subsoils, and 0 to 0.6 seconds for flexible subsoils. The spectra for rock and for flexible subsoils were obtained by scaling of the spectra for intermediate subsoils.

The Bridge Manual defines the maximum displacement ductility factor μ for each category of structure that may be designed (see Table 1 and Fig. 3). The preferred mechanisms involve plastic hinges in the columns in regions which are accessible for inspection and repair. For less ductile mechanisms, or where the plastic hinge positions are not readily accessible for inspection and repair, a reduced μ value and hence increased seismic design load is used in design.

The variation of seismicity throughout the country is taken into account by a zone factor Z which ranges from 0.6 to 1.2. The zone factor is the peak 5% damped elastic uniform hazard spectral acceleration coefficient for a 450 year return period earthquake.

To determine the design seismic force a structural performance factor S_p is also introduced. The structural performance factor is defined in the loadings standard (Standards New Zealand, 1992) as "a numerical assessment of the ability of a building to survive cyclic displacements. Its value depends on the material, form and period of the system resisting system, damping of the structure, and interaction of the structure with the ground". The Bridge Manual specifies values of S_p varying from 0.67 for soft ground to 0.9 for hard ground for bridge structures.

In summary, the minimum equivalent static design seismic base shear force V is given by:

$$V = C_\mu Z R S_p W_d \text{, but is limited to not less than } 0.05W_d.$$

where C_μ = basic acceleration coefficient, which varies according to the value of the fundamental natural period of vibration T and the site subsoil category and found from the uniform seismic hazard spectra; Z = zone factor, which varies between 0.6 and 1.2 depending on the seismicity of the site; R = risk factor, which varies between 1.0 and 1.3 depending on the importance of the bridge; S_p = structural performance factor, which varies between 0.67 and 0.9 depending on the site subsoil category; and W_d is the total dead weight plus superimposed dead weight (force units) assumed to participate in seismic movements in the direction being considered.

Two horizontal load cases comprising a combination of orthogonal effects are prescribed by the Bridge Manual. 100% of the seismic effects from load in one direction are to be combined with 30% of the effects of loading in the orthogonal direction, and vice versa.

The Bridge Manual states that equivalent static force analysis can be used when it is appropriate to model the bridge structure as a single degree of freedom oscillator. Dynamic analysis to obtain maximum horizontal forces or displacements or ductility demand should be carried out when that modelling is not appropriate.

The Bridge Manual also specifies vertical seismic accelerations for the design of regular structures, which are taken as ⅔ of horizontal accelerations for elastic response at the natural period of vertical vibration. The bridge superstructure is required to remain elastic under this vertical acceleration.

The Bridge Manual requires that, for sites located within 10 km of an active fault with an average return interval of 1,000 years or less, the design loading shall be derived using a site specific seismicity study. To limit the need for site specific studies to only the more important structure, bridge cost parameters are listed.

Table 1. Maximum allowable values of design displacement ductility factors μ according to the Bridge Manual

Energy Dissipation System:	μ
Ductile or partially ductile structure, in which plastic hinges form at the ultimate design load above ground or normal (or mean tide) water level.	6
Ductile or partially ductile structure, in which plastic hinges form in reasonably accessible positions, eg, less than 2 m below ground, but not below normal (or mean tide) water level.	4
Ductile or partially ductile structure in which plastic hinges are inaccessible, forming more than 2 m below ground or below normal (or mean tide) water level, or at a level reasonably predictable. Spread footings designed to rock (unless a larger value can be specifically justified).	3
Hinging in raked piles in which earthquake load induces large axial forces.	2
"Locked in" structure (T = 0) Elastically responding structure.	1

Note: The design ductility factor for structures of limited capacity or demand is to be determined from actual structure characteristics.

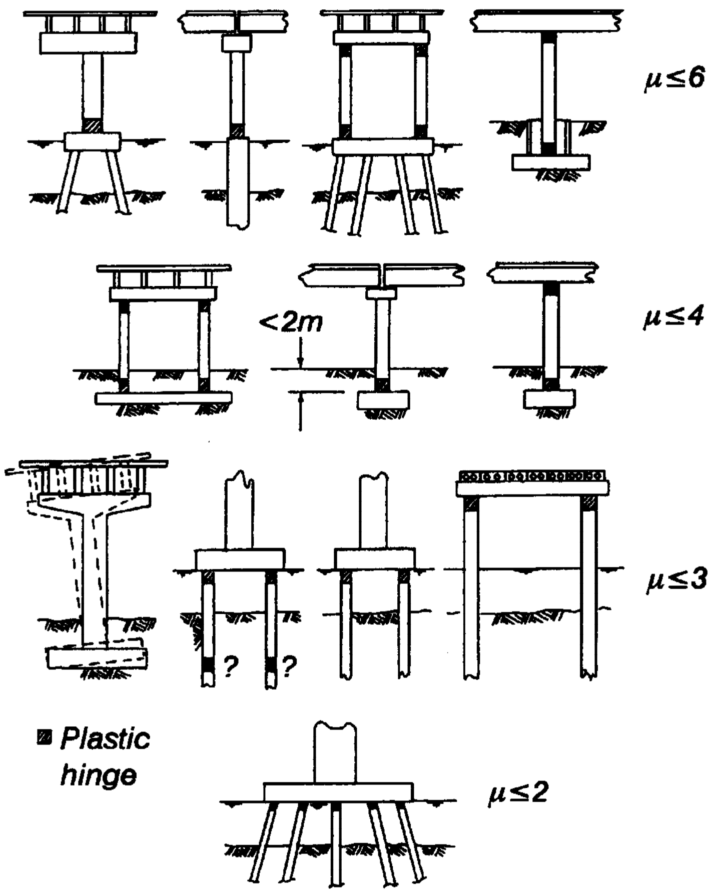

Fig. 3 Examples of maximum values of the structure displacement ductility factor μ permitted by the Bridge Manual when determining the design seismic loading

Combination of Load Effects

Load combinations and load factors for strength requirements involving seismic loading specified by the Bridge Manual are:

$$U = 1.00\ \{kDL + 1.35\ (EP + OW) + SG + ST + EQ + 0.33TP\} \qquad (1)$$

$$U = 1.35\ (DL + EP + OW + SG + 0.33EQ + 1.1CN) \qquad (2)$$

where CN = construction loads, involving loads on an incomplete structure, DL = dead load, including superimposed dead load, EP = earth pressure, EQ = earthquake effects, OW = ordinary water pressure and buoyancy, SG = shortening effects, ST = settlement, TP = temperature effects, overall and differential, U = design load for strength design method, and k = 1.3 or 0.8, whichever is more severe, to allow for vertical acceleration.

MEMBER STRENGTH DESIGN CRITERIA AND FOUNDATION DESIGN

The Bridge Manual (Transit New Zealand, 1994) sets out the acceptable values of member strengths (or relative strengths) to be provided in order to meet capacity design principles. The aim is to ensure that the required yielding behaviour of the structure is achieved, by designing members and joints to have sufficient relative strength so that yielding only occurs in the intended plastic hinge positions. The strengths specified are chosen to provide sufficient margins to satisfy this criterion, while avoiding excessive strength and consequent cost.

The design flexural strength (which is equal to the nominal flexural strength multiplied by a strength reduction factor $\phi = 0.85$) of the potential plastic hinge regions should be at least equal to the bending moments obtained from structural analysis for the load combinations given by Eqs. 1 and 2. Then, according to the capacity design approach for ductile substructures, the actions associated with the overstrength flexural capacities of the plastic hinges are matched by (a) the nominal shear strengths of the plastic hinge regions, and (b) the nominal shear and nominal flexural strengths of the resisting members elsewhere. The calculation of the nominal flexural strength of columns could include the effect of the increased compressive strength of the concrete as a result of confinement which causes an enhanced flexural strength. The nominal shear strength is calculated conservatively (Standards New Zealand, 1995) ignoring the concrete shear resisting mechanisms in the plastic hinge regions if the axial load ratio $N^*/f'_c A_g$ is less than 0.1 and assuming that the shear is resisted by a 45° truss mechanism, where N^* = axial compressive load on column, f'_c = concrete compressive cylinder strength, and A_g = gross area of the column.

The Bridge Manual includes consideration of the ability to soils to resist forces from foundation members. It also deals with structures anchored by friction slabs, structures on rocking foundations and structures with energy dissipating devices.

DETAILING FOR ADEQUATE DUCTILITY

The potential plastic hinge regions of the substructures need to be carefully detailed to ensure that the plastic rotations occurring in the chosen plastic mechanism, as a result of the deformations reached during the design ultimate limit state earthquake, can be sustained. A refined approach is recommended whereby the curvature ductility factor associated with the chosen plastic mechanism and displacement ductility factor is calculated by static push-over analysis or dynamic analysis. Then sufficient transverse confining reinforcement is provided for the column plastic hinge regions to achieve that curvature ductility factor demand (see Park, et al, 1991). The New Zealand concrete design standard (Standards New Zealand, 1995) gives the following equations, derived by Watson et al, 1994, for determining the quantity of transverse reinforcement required to confine the compressed concrete so as to achieve the required curvature ductility factor:

For columns with rectangular hoops and cross ties:

$$A_{sh} = \frac{\left(\dfrac{\phi_u}{\phi_y} - 33p_t m + 22\right) s_h h''}{111} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N^*}{\phi f_c' A_g} - 0.006 s_h h'' \tag{3}$$

For columns with circular hoops or spirals:

$$A_{sh} = \frac{\left(\dfrac{\phi_u}{\phi_y} - 33p_t m + 22\right)}{79} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N^*}{\phi f_c' A_g} - 0.008 \tag{4}$$

where A_{sh} = area of transverse bars in direction under consideration within centre to centre spacing of hoop sets s_h, h'' = dimension of core of column at right angles to direction to transverse bars under consideration, A_g = gross area of column, A_c = core area of column, ϕ_u/ϕ_y = curvature ductility factor, ϕ_u = ultimate curvature, ϕ_y = curvature at first yield, $p_t = A_{st}/A_g$, A_{st} = total area of longitudinal column reinforcement, $m = f_y/0.85f_c'$, f_y = yield strength of longitudinal steel, f_{yt} = yield strength of transverse steel, f_c' = concrete compressive cylinder strength, N^* = axial compressive load on column, ϕ = strength reduction factor and p_s = ratio of volume of transverse circular hoop or spiral steel to volume of concrete core of column.

In Equations 3 and 4 a value of $\phi_u/\phi_y = 20$ may be assumed for calculating the required transverse reinforcement for routine cases.

Equations 3 and 4 give only the transverse reinforcement required for concrete confinement. The transverse reinforcement provided must also be checked to ensure that the tie requirements for preventing premature buckling of longitudinal compression bars, and the requirements for shear reinforcement, are satisfied. This check may lead to more transverse reinforcement being required to prevent bar buckling and/or to prevent shear failure than required for concrete confinement.

Within the potential plastic hinge regions of ductile columns the vertical spacing of transverse reinforcement is not permitted to exceed the smaller of 6 longitudinal bar diameters or one-quarter of the least lateral dimension of the column section, and the horizontal spacing of transverse reinforcement in rectangular columns is not permitted to exceed the larger of 200 mm or one-quarter of the adjacent lateral dimension of the column section. The length of the potential plastic hinge region of the column is one to three times the column lateral dimension, depending on the axial load on the column and the gradient of the bending moment diagram (Standards New Zealand, 1995).

MAXIMUM DISPLACEMENTS

The Bridge Manual (Transit New Zealand, 1994) gives methods for estimating seismic displacements. A very important design criterion is that the bridge superstructure must be adequately supported so as not to become dislodged during a major earthquake when significant displacement of the bridge substructure occurs. It is required that allowance should be made at superstructure movement joints for out-of-phase response of two adjacent sections of the structure. Design provisions for relative displacements and for maintaining structural integrity by linkage across movement joints are also given.

ASSESSMENT AND RETROFITTING OF EXISTING BRIDGES

70% of New Zealand's state highway bridge stock was built in the period 1930 to 1970 and therefore has not been designed using capacity design principles nor deliberately detailed for ductility. Also, the linkages between simple spans of those bridges may be inadequate. A recently developed screening procedure is being trialled by Transit New Zealand to identify bridges which are in need of retrofitting. The 1.3 km long Thorndon overbridge which carries motorway traffic over railway and harbour facilities in the city of Wellington, and which was completed in 1969, has also been assessed and has been shown to need seismic upgrading. The upgrading will cost in the order of $20 million NZ. The retrofitting procedure for the single stem piers of the Thorndon overbridge will involve enclosing the potential plastic hinge regions of the existing circular columns with grouted thin steel jackets and casting an overlay of new reinforced concrete over the existing pile caps.

CONCLUSIONS

Seismic design procedures for bridges have been developed in New Zealand. The seismic design actions at the ultimate limit state are obtained from a response spectrum appropriate to the site conditions and the structure displacement ductility factor appropriate to the bridge substructure. Capacity design is used to ensure that the most desirable energy dissipating mechanism forms in the substructure in the event of a major earthquake. Members and joints are detailed to ensure that the available strength and ductility is as required are that the structures behaves as intended.

ACKNOWLEDGEMENT

The contributions of Mr H E Chapman of Works Consultancy Services, Wellington, New Zealand to the development of the Transit New Zealand Bridge Manual are gratefully acknowledged.

REFERENCES

Chapman, H.E. and D.K. Kirkcaldie, (1990). Seismic design of base isolated bridges incorporating mechanical energy dissipating dissipators, RRU Bulletin 84:3. Transit New Zealand, Wellington.

Jennings, D.N., S.A.L. Read, P.J. Millar, F. Huppert, P.M. Black, I.M. Parton and A.J. Olsen (1990). Road engineering in soft rock materials, RRU Bulletin 84:4. Transit New Zealand, Wellington.

Park, R., H.E. Chapman, L.G Cormack and P.J. North, (1991). Papers on New Zealand Practice. Proceedings of the International Workshop on the Seismic Design and Retrofitting of Reinforced Concrete Bridges, Bormio, Italy.

Park, R. and T. Paulay (1990). Strength and ductility of concrete substructures of bridges, RRU Bulletin 84:1. Transit New Zealand, Wellington.

Standards New Zealand (1992). General Structural Design and Design Loadings for Buildings, NZS 4203:1992, Standards New Zealand, Wellington.

Standards New Zealand (1995). The Design of Concrete Structures. NZS 3101:1995, Standards New Zealand, Wellington.

Study Group (1980). Papers resulting from the deliberations of the Study Group on the seismic design of bridges. Bulletin of the New Zealand National Society for Earthquake Engineering, 13, 3, 226-309.

Summary Papers and Other Technical Papers (1990). RRU Bulletin, 84:5. Transit New Zealand, Wellington.

Transit New Zealand (1994). Bridge Manual, ISBN 0-477-01699-9 plus amendment No. 1 June 1995. Transit New Zealand, Wellington.

Watson, S., F.A. Zahn and R. Park (1994). Confining reinforcement for concrete columns, Journal of Structural Engineering of American Society of Civil Engineers, 120, 6, 1798-1824.

Wood, J.H. and D.G. Elms (1990). Seismic design of bridge abutments and retaining walls, RRU Bulletin 84:2. Transit New Zealand.