# SEISMIC DESIGN OF BRIDGES

## SECTION 3

## CAPACITY DESIGN PRINCIPLES AND PRACTICE

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3.1 PHILOSOPHY:

3.1.1 The capacity design approach may be defined as being a design procedure intended to ensure that various members of a structural frame form a desired hierarchy of strengths.

3.1.2 The aim is twofold:

(a) To ensure that plastic hinges intended to develop during strong ground motions have a specified minimum 'dependable' strength. This prevents undue damage occurring during the more frequent moderate earthquakes.

(b) To recognise that plastic hinges developing are likely to possess flexural strengths in excess of the 'dependable' values. Other members in the structure intended to remain elastic are designed on the basis of the plastic hinges developing their overstrength flexural capacities.

3.2 DEFINITIONS:

3.2.1 The following definitions apply to sections of a structural member:

The 'ideal strength' is the theoretical limit strength, based on the section geometry as detailed and on the nominal minimum material strengths.

The 'dependable strength' is related to the 'ideal strength' by the strength reduction factor $\phi$

$$\text{'Dependable strength'} = \phi \times \text{'ideal strength'} \quad (3.1)$$

where $\phi$ is less than 1.

The 'overstrength' takes account of all the possible factors that may contribute to section strength, such as overstrength reinforcement, increased reinforcement stress due to strain hardening at large deformations and a concrete strength exceeding the specified minimum, due to age, confinement or other clauses.

$$\text{'Overstrength'} = \phi_0 \times \text{'ideal strength'} \quad (3.2)$$

where $\phi_0$, the overstrength factor, is greater than 1.

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3.3 DESIGN PROCEDURE:

3.3.1 It is recommended that for structures in which a plastic mechanism is intended or likely to develop, the structural analysis and design procedure comprise two stages:

(a) Design plastic hinge sections to have the minimum required flexural strengths.

(i) Decide structural form and choose desired location of plastic hinges to allow a plastic mechanism to develop.

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(ii) Carry out elastic analysis under specified loads included in the specified load effect combinations.

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(iii) Hence determine minimum flexural strengths required for plastic hinges. Design these sections to have 'dependable' strengths to match the requirements.

(b) Design all sections other than the plastic hinges for shear and flexure. Design plastic hinges for shear.

(i) Calculate 'overstrength' flexural capacities of plastic hinges as designed in (a) above.

↓

(ii) Analyse structure assuming all plastic hinges to have developed their 'overstrength' flexural capacities. Hence determine shear and moment capacities required for all sections other than the plastic hinges, and design sections accordingly. Design plastic hinges for shear.

3.3.2 For structures such as abutments anchored to the approaches by a friction slab, it is recommended that the design procedure used ensures that the strength of the connection exceeds the anticipated force by which the friction slab will be mobilised within the approach soils. Alternatively a suitable cyclically ductile connection should be used.

3.4 DERIVATION OF DESIGN CONDITIONS FOR STRUCTURAL MEMBERS

3.4.1 Bending Moment

3.4.1.1 Plastic hinging intended to occur as a primary energy dissipating mechanism should have a flexural strength derived as follows:

The structure should be analysed elastically under the specified design load effect combination.

The 'dependable' flexural strength of the plastic hinges should be not less than the bending moments assigned to their locations from this elastic analysis.

3.4.1.2 Every member resisting the moments from plastic hinges (both intended or potential), should have a flexural strength derived as follows:

The structure should be analysed as a plastic mechanism, assuming all intended or potential plastic hinges to have developed their overstrength capacities. The 'ideal' flexural strength of the resisting member being designed should be not less than the bending moment assigned to the member from this analysis.

3.4.1.3 Every member intended to remain elastic under 'design' earthquake conditions, and resisting the moments caused by frictional forces in sliding bearings, should have a flexural strength derived as follows:

The 'ideal' flexural strength of the pier members, and the 'dependable' flexural strength of foundation members should be not less than the moment induced in the member by sliding of the bearing during earthquake motions. An upper limit to the coefficient of friction of at least 0.15 should be assumed for PTFE/stainless steel sliding bearings. Adequate account should also be taken of any additional moments which may be induced in the member as a result of earthquake motions along both major axes of the structure concurrently. Such additional moments can arise from friction or shear stiffness of devices intended to prevent horizontal movement in a direction perpendicular to that being considered.

3.4.1.4 Members resisting the moments caused by shear forces induced in elastomeric bearings during earthquake motions equivalent to the 'design' earthquake, may be divided into two categories:

(a) those in which plastic hinging is intended to occur as part of the energy dissipating mechanism;

(b) those intended to remain elastic.

Members in category (a) should be designed as in 3.4.1.1. Members in category (b) should have a flexural strength derived as follows:

The structure should be analysed elastically under the combination of external loading effects specified in the appropriate specification.

(i) If the structure is in the 'partially-ductile' category the increase in forces induced in the elastic members should be calculated equivalent to the horizontal limit displacement appropriate to the design. Then, the 'ideal' pier member flexural strength and the 'dependable' foundation member flexural strength should be not less than the moment from this analysis.

(ii) If the structure is in the 'non-ductile' category, no increase in force is necessary beyond the initial elastic analysis. The 'dependable' member flexural strength should be not less than the moment from the elastic analysis.

3.4.2 Shear

3.4.2.1 Every member in which plastic hinging is intended to occur or could not potentially occur as part of a primary energy dissipating mechanism and in which the shear force able to develop is limited by formation of plastic hinges, should have a shear strength derived as follows:

The member should be analysed assuming all plastic hinges governing its shear to have developed their overstrength capacities. The 'ideal' shear strength of the member should equal or exceed the shear force from this analysis.

3.4.2.2 Every member (except for those designed under 3.4.2.1) resisting the moments from plastic hinges (both intended or potential), should have a shear strength derived as follows:

The structure should be analysed as a plastic mechanism, assuming all intended or potential plastic hinges to have developed their overstrength capacities. The 'ideal' shear strength of the member at any location should equal or exceed the shear force assigned to that location from this analysis.

3.4.2.3 Every member intended to remain elastic under 'design' earthquake conditions, and resisting the shear forces caused by friction in sliding bearings, shall have a shear strength derived as in 3.4.2.1.

3.4.2.4 Members resisting the shear forces induced in elastomeric bearings during earthquake motions equivalent to the 'design' earthquake, may be divided into two categories:

(a) those in which plastic hinging is intended to occur as part of the energy dissipating mechanism;

(b) those intended to remain elastic.

Every member in category (a) should be designed as in 3.4.2.1.

Every member in category (b) and part of a structure in the 'partially-ductile' category should be designed as in 3.4.2.1.

Every member in category (b) and part of a structure in the 'non-ductile' category, should have a shear strength derived as follows:

The structure should be analysed elastically under the combination of external loading effects specified in the appropriate specification. The 'dependable' shear strength of the resisting member should be not less than the shear force from this analysis.

3.4.2.5 When there is a relatively greater degree of uncertainty in the structural analysis (eg, in foundation piles) members resisting plastic hinge moments should be desinged to take account of the possibility of shear forces exceeding those derived from the plastic analysis required in 3.4.2.2. Where possible, the 'ideal' shear strength of such members should be not less than the shear forces which would develop if overstrength plastic hinges developed in the member instead of at the intended locations. Design judgement should be exercised, taking into account the economic effect on the structure of such provision.

3.4.3 Tension in Connections to Friction Slabs

3.4.3.1 The tension connection between a structure and a friction slab intended to act as its anchorage, should be designed so that the yield force of the connection at minimum specified yield stress is not less than the ideal resistance of the friction slab.

COMMENTARY:

C3.2.1 Calculation of member strengths should be based on appropriate Codes of Practice for the material being considered. In particular, the following standards should be used:

New Zealand Standard DZ 3101:1980 "Code of Practice for the Design of Concrete Structures".

New Zealand Standard NZS 3404:1977 "Code for Design of Steel Structures". This is for use in conjunction with Australian Standard AS 1250:1975 "SAA Steel Structures Code". It should be noted that NZS 3404 and AS 1250 are written for use in design of buildings and are not intended for bridge superstructure design. They do, however, cover strength design of members in which plastic hinging may occur.

C3.3.1 Adoption of the philosophy of capacity design relates to experimental evidence that suitably detailed structures have considerable capacity to undergo post-elastic displacements, provided they can do so by flexure rather than by shear. The object of the design procedure is to ensure that this is achieved.

In most cases the members involved in the capacity design procedure will be the piers, foundation footings or pilecaps, and the foundation piles or cylinders. Gravity load requirements usually lead to superstructures having adequate strength to prevent seismic plastic hinging occurring in superstructure members. Designers should, however, be aware that this may not always be so. For example, when a concrete deck slab is not used it may be necessary to provide additional strength to prevent plastic hinging in main longitudinal members under transverse loading.

C3.3.2 In structures requiring stabilisation under seismic loading, usually in a longitudinal direction, friction slabs offer a solution. They may be constructed to act as a 'fuse', sliding within the approach filling while anchoring the abutment, and thereby dissipating seismic energy. Current knowledge does not, however, extend to the reliable design of such energy dissipating mechanisms.

It should also be remembered that appreciable displacement of the abutment would be associated with such energy dissipation and that such a structural arrangement would also lead to ground motions being fed into the total structural mass anchored to the friction slab, up to the acceleration equivalent to the sliding force of the slab. It is therefore advisable to use friction slabs only for stabilising small bridges or abutments free to move horizontally relative to the main structural mass.

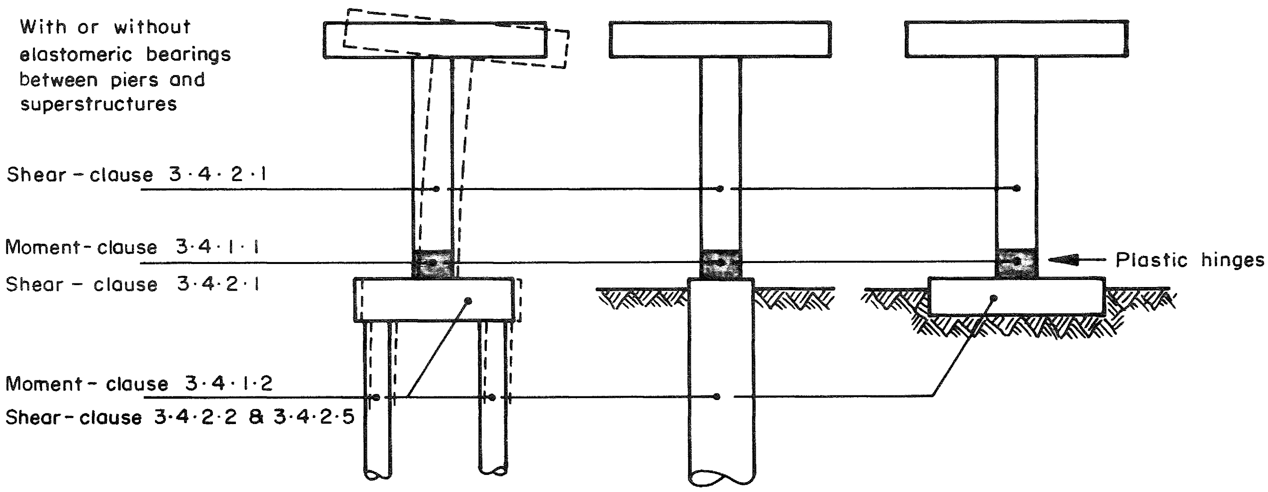
Ductile devices offer a method of connecting a superstructure mass to an abutment with sufficient strength to resist service loads, but with the capability of limiting the inertia force developing in the connection during earthquakes (see Section 6).

C3.4 Design conditions for various structural members can be found in the clauses noted on figures C3.1 (a) and (b).

C3.4.1 Recommendations for member strengths and their relative values necessarily involve applying engineering judgement. The philosophy adopted for flexural strengths in this section is as follows:

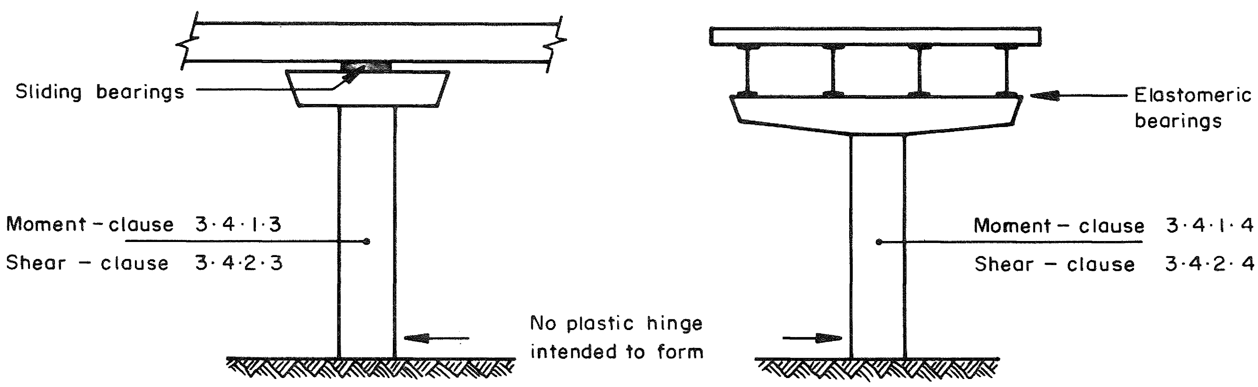
(a) For members in which primary seismic energy dissipation is intended or likely to occur - a positive margin is specified for the resisting member strength over the plastic hinging member, to take account of the effect of large, likely member curvature ductility demands and consequent strain hardening effects.

(b) For members in which primary seismic energy dissipation is not intended to occur, and for which calculations are made to quantify seismic effects, such as effects of dynamic displacements - a small margin is specified for




a) With plastic hinge intended to form in the pier.

FIG. C3·1




b) With no plastic hinge intended to form in the pier

FIG. C3·1

resisting member strength over the potential plastic hinging member. This is on the basis that curvature ductility demands should, in any case, be small and consequent damage of little significance.

In intermediate cases in which some seismic energy dissipation is intended in pier members, but less than would be the case in 'fully-ductile' structures, the designer should provide a larger margin of strength in the foundation, even if it is not up to the standard set out in 3.4.1.2.

C3.4.1.2 It is not intended that this clause be applied to a member (e.g. a pier stem) in which plastic hinging forms. It is normal practice to carry the flexural reinforcement required in the plastic hinge to a point past that to which the length of the plastic hinge will extend. No extra provision of flexural reinforcement is necessary beyond the plastic hinge.

C3.4.1.3 A small margin of strength for the foundation member over the pier it supports is specified in this case since, theoretically, plastic hinging cannot occur in the pier. In practice, many cases would include the need to provide such a strength margin due to other considerations - for example the pier could be required to act in a ductile manner transversely. In such a case clause 3.4.1.2 would govern foundation design.

C3.4.1.4 For the purposes of design, structure types are defined with reference to the relationship between the applied horizontal force and the resultant displacement ($\delta$) of the centre of mass of the structure.

'Fully-ductile' structures are those in which a plastic mechanism can form in the structure. The relationship is essentially one where, after 'yield', the resultant displacement increases without appreciable increase in applied horizontal force (see Figure C3.2). In addition, the relationship must apply for reversing loads and over at least several cycles, to ensure hysteretic dissipation of energy.

'Partially-ductile' structures are those where some of the earthquake resisting elements (eg, piers in flexure) yield while others (eg, elastomeric bearings at abutments) remain elastic. With increasing displacement the applied force increases, although at a decreasing rate (see Figure C3.3).

The title does not imply that there is a limit to the ductility available relative to a 'fully-ductile' structure, but refers to part of the structure being ductile and part remaining elastic. As for 'fully-ductile' structures the relationship must apply for reversing loads to ensure hysteretic dissipation of energy.

'Non-ductile' structures are those where no earthquake resisting elements yield and the force/displacement relationship develops neither a yield 'plateau' nor a hysteretic energy dissipating capability. The force/displacement relationship includes elastic behaviour leading to sudden and irreversible reduction of load capacity (see Figure C3.4).

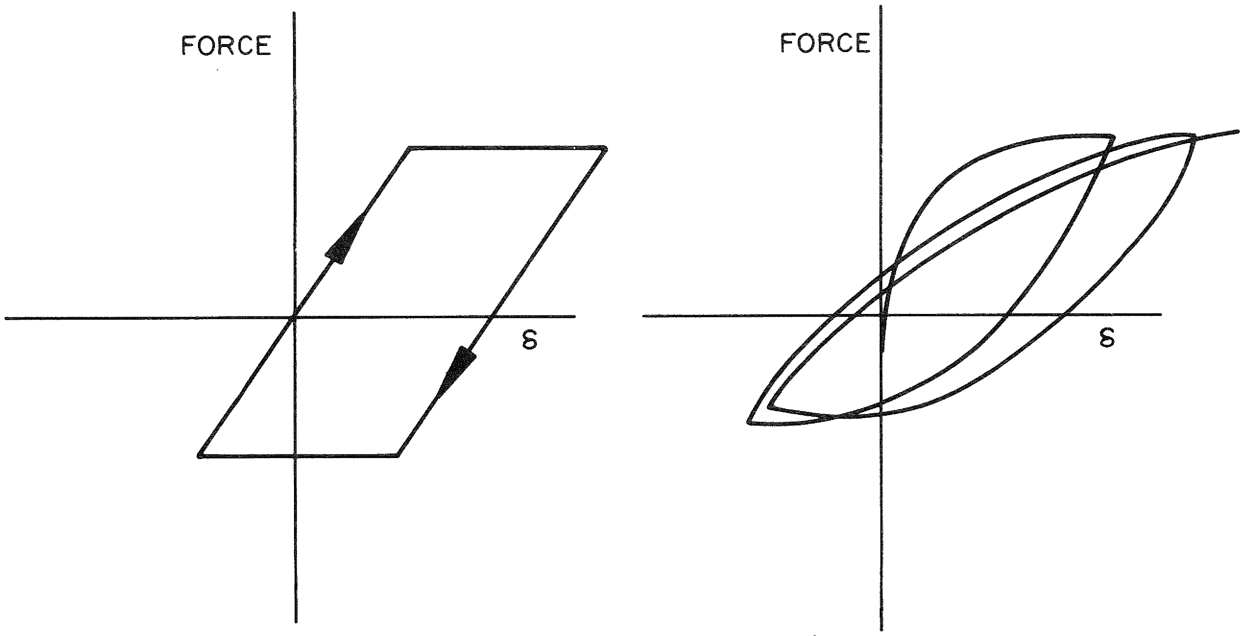
C3.4.2 Design of members to resist shear forces should be conservative, and this is reflected in the recommendations in this section. Because shear failures are not of a ductile nature nor reversible in behaviour, it is important that flexural yield of members be assured before shear failure occurs. Furthermore, it is usually not very expensive to provide added shear strength to members.

In addition to the predictable shear forces which would develop through a structure forming a plastic mechanism, there are several other sources of unpredictable shears or torsional effects which must be provided for by an adequate safety margin. These include such possibilities as torsional ground motions or structural response, skewed bridge effects torsion due to out-of-phase transverse response at adjacent piers, and concurrent longitudinal and transverse motion causing friction to be mobilised in restraints which are intended to prevent horizontal movement in a direction perpendicular to that being considered.

Thus the design basis for shear set out in 3.4.2 should be regarded as the least conservative which is acceptable. In structures where some of the unpredictable effects listed above are likely to be of consequence, extra shear resistance should be provided.

C3.4.2.5 Dependable analysis and determination of a heirarchy of structural member strengths is difficult for some structures. For example, when piles, either raked or vertical, pass through soft soil layers and appreciable water depths before entering the pilecap, the structural simulation and dynamic representation of the structure become more approximate. Dependable prediction of the location of the plastic hinges becomes more difficult. Precautions to ensure that foundation members can resist the shear forces arising from a plastic mechanism inadvertently forming within the foundation members should be considered.

C3.4.3.1 Recommendations for calculating the slidng resistance of friction slabs are included in Section 4 - Bridge Foundations.




a) Idealised b) Actual

FIG. C3·2 : 'FULLY DUCTILE'

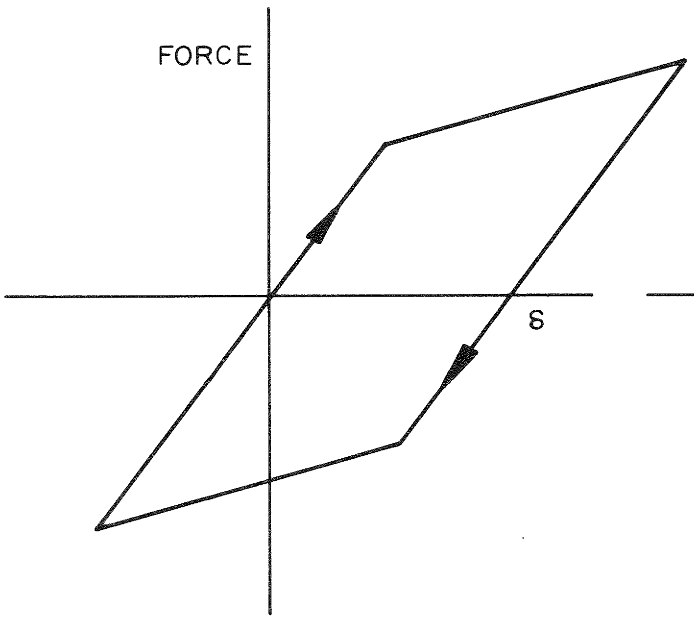



FIG. C3·3
'PARTIALLY - DUCTILE'

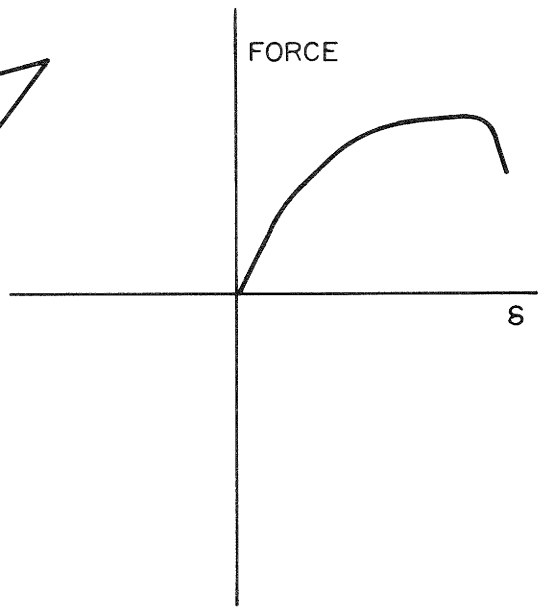



FIG. C3·4
'NON - DUCTILE'