5.0 Earthquake resistant design of structures

In this section

5.1 Scope and design philosophy

5.1.1 Scope and terminology

This section applies to the structural design of structures for earthquake resistance where the structures are composed of reinforced or prestressed concrete, steel or aluminium, timber, or other advanced engineering materials such as fibre reinforced composites, and include bridges, culverts, stock underpasses, subways and retaining walls. This section excludes the design of earth embankments and slopes for earthquake resistance, which is covered by section 6.

For the structural design of bridge abutments, both integral and non-integral, and retaining walls this section shall apply. The earthquake resistant design of non-integral bridge abutments and retaining walls for overall stability and for the limitation of displacements due to sliding and soil deformation, which are primarily geotechnical design in nature, shall comply with section 6.

The terminology: damage control limit state (DCLS) and collapse avoidance limit state (CALS) are adopted in this manual as more appropriately representing the limit states applicable to seismic resistance. The intensity of seismic response corresponding to the damage control limit state largely equates to the ultimate limit state as specified by NZS 1170.5 *Structural design actions* part 5 Earthquake actions – New Zealand[(1)], except that the upper bound and the seismic intensity for Northland have been amended by 5.2.2, and for geotechnical structures covered by section 6 magnitude weighting is not applied. The intensity of seismic response corresponding to the collapse avoidance limit state is taken to be 1.5 x DCLS subject to the upper bound limit specified by 5.2.2 and may be considered to be what is commonly referred to as the maximum considered earthquake (MCE). It should be noted that the performance requirements associated with these limit states for highway structures differ from those for other structures covered by NZS 1170.5[(1)].

5.1.2 Objective

The primary objectives of seismic design shall be to ensure life safety and that the structure can safely perform its function of maintaining communications after a seismic event. The extent to which this is possible will depend on the severity of the event, and thus by implication on its return period.

For design purposes, structures shall be categorised according to their importance or the importance of the highway on which they are located, and assigned a return period factor related to the seismic ground motion return period. This will define the design earthquake hazard and the earthquake loading as defined in 5.2.

Performance expectations at the three earthquake intensity levels outlined below and in table 5.1 require philosophic consideration in selecting the structural form and in detailing, and should be discussed in the structure options report and structure design statement.

a. The damage control limit state (DCLS) event: After exposure to a seismic event of design (DCLS) severity, the structure shall be usable by emergency traffic within three days, although damage may have occurred, and some temporary repairs may be required to enable use by vehicles. Permanent repair to reinstate capacity for all design actions including for at least one subsequent seismic event of design (DCLS) severity shall be economically feasible. (The performance expectations for any such subsequent event may be considered as being those for a CALS event as described in item (c).) Where settlement is expected to occur, reinstatement of the structure's geometry to provide an acceptable level of service for traffic and to reinstate required clearances shall also be economically feasible. (Refer also to 5.1.3 for further explanation of this limit state.)

5.1.2 continued

b. The serviceability limit state (SLS) event: After an event with a return period significantly less than the design (DCLS) value, damage should be minor, and there should be no more than minimal disruption to traffic (eg temporary speed restrictions and temporary lane closures to facilitate repairs such as the reinstatement of deck joint seals).

c. The collapse avoidance limit state (CALS) event: After an event with a return period significantly greater than the design (DCLS) value, the structure should not collapse, although damage may be extensive. It may be usable by emergency traffic after temporary repairs such as propping. The structure may be capable of permanent repair, although a lower level of loading may be acceptable.

Table 5.1: Seismic performance requirements

	Earthquake severity		
	SLS earthquake (as 5.1.2(b)) Return period factor = $R_u/4$	DCLS earthquake (as 5.1.2(a)) Return period factor = R_u	CALS earthquake (as 5.1.2(c)) [1] Return period factor = $1.5R_u$
Post-earthquake function - immediate	Minimal disruption to traffic. Operational functionality is maintained	Usable by emergency traffic[2] within three days	May be usable by emergency traffic[2] after temporary repair
Post-earthquake function – after reinstatement	Minimal reinstatement necessary to cater for all design-level actions. All damage to be repairable within a period of one month	Feasible to economically reinstate to a capacity sufficient to avoid collapse under a repeat design-level earthquake, and for serviceability for traffic	May be capable of permanent repair, but possibly with reduced load capacity
Acceptable damage	No damage to primary structural members. Damage to secondary and non-structural elements shall not be such as to impede the operational functionality of the structure. Deck joint seals may be dislodged but should be readily reinstated. Knock-off elements should not be damaged or dislodged	Damage may be significant; temporary repair may be required	Damage may be extensive; collapse prevented

Notes

1. The CALS earthquake event need not be taken to exceed an upper bound limit event corresponding to Z x Return Period Factor = 1.05
2. Usable by emergency traffic means able to carry two lanes of HN (normal) loading utilising load combination 4A with an impact factor appropriate for a speed of 30 km/h (see 7.2.2), in conjunction with the predicted post-earthquake settlements, including differential settlements.
3. The operational functionality and extent of damage incurred at each limit state shall be treated as absolute requirements. Repairability and the time taken to accomplish repairs may be regarded as aspirational.

In general, design of structures for the DCLS is expected to result in satisfactory performance at the SLS without need for specific consideration of the SLS except as specified in 5.7 for the design of movement joints. Exceptions may arise when between the SLS and DCLS a significant reduction in the structure's initial stiffness occurs reducing the structure's seismic response, eg due to liquefaction.

5.1.2 continued

Note that the performance of a structure in a particular earthquake cannot be deterministically predicted because of the high variability and uncertainty associated with earthquake ground movements and the response of structures to these movements. However the available empirical damage data indicates that the likelihood of structures designed in accordance with this *Bridge manual* not achieving the performance standards is low.

5.1.3 Maintenance of structural integrity

Bridge, culvert, stock underpasses and subway structures shall be designed to remain stable against excessive sliding or overturning in earthquakes of CALS magnitude. Limited sliding of retaining walls is permitted as specified in section 6 but they shall remain stable against overturning in CALS events.

All elements of a structure shall be detailed to maintain their integrity and to continue to perform in either an elastic or ductile manner under earthquake response exceeding the DCLS, and up to the CALS. Providing that the conditions of support of the structure and the nature of loading on the structure do not change as earthquake response increases from the DCLS up to the CALS (eg as may arise due to liquefaction of the ground or due to slope failure or movement), for the strength design of reinforced concrete or steel structural elements in ductile and limited ductile structures this may be assumed to be satisfied by designing for the DCLS design loading, applying the capacity design requirements of 5.6.1 and the ductile detailing requirements specified herein, and avoidance of situations that may lead to brittle failure of elements or connections developing. However, the degradation of the concrete contribution to the shear strength at plastic hinge locations with increasing curvature ductility demand shall be taken into account and adequate total shear strength provided for the CALS as specified by 5.6.3.

For other elements, such as elastomeric bearings, mechanical energy dissipation devices, and soil structures providing support and restraint, their ability to maintain their integrity and stability under response to earthquakes exceeding the DCLS event, shall be considered and collapse avoidance of the structure as a whole confirmed. Where the mode of behaviour of the structure changes, or loading conditions acting on the structure change, from that applying at the DCLS in events greater than the DCLS event, collapse avoidance under the CALS event shall be confirmed. Such situations may arise, for example, when:

- an elastic structure develops in-elastic behaviour (as discussed below)
- when supporting ground undergoes some degree of failure altering the support provided to the structure and/or imposing soil lateral spread loading onto the structure
- when bearings or base isolation devices exceed their displacement capacities.

In considering the performance of the structure at the CALS, the factor of safety of the structure against overturning shall equal or exceed 1.0. Sliding of the structure is permissible providing it does not pose a risk to human life. The structure as a whole, based on probable material strengths and capacity reduction factors of ϕ=1.0 for steel and concrete elements, and for soil elements supporting or restraining the structure a capacity reduction factor of ϕ=0.75 (allowing for the greater uncertainty associated with the strength of soil elements) shall retain sufficient integrity against collapse, though individual elements within the structure may fail with the forces that they were carrying being redistributed within the structure.

5.1.3 continued

Care shall be taken to ensure that detailing practices recognize the potential for inelastic response even when the bridge is designed to respond elastically. When plastic hinges do not form at the DCLS event, in order to ensure collapse avoidance in earthquakes exceeding the design DCLS event, bridge elements likely to develop plastic hinging in such events shall be detailed for ductility as required by the referenced materials design standards NZS 3101 *Concrete structures standard*(2) and NZS 3404:1997 *Steel structures standard*(3) for structures of limited ductility and capacity design shall be applied to ensure shear failures are avoided.

The requirements of section 6 shall be complied with in considering the interaction between the structure and its supporting ground under earthquake response and in situations where ground instability may arise.

The design of any structure located in an area which is susceptible to earthquake induced liquefaction, or which is located within 200m of an active fault with a recurrence interval of 2000 years or less, shall recognise the large movements which may result from settlement, rotation or translation of substructures. To the extent practical and economic, and taking into consideration possible social consequences, measures shall be incorporated to mitigate against these effects. Mitigation measures may include:

- alteration of the route to enable relocation of the structure to another less vulnerable site with better ground conditions
- at locations where crossing of a fault is unavoidable, or liquefaction is likely, adoption of a structural form more tolerant to fault movement or lateral spreading and able to be more rapidly reinstated should fault or lateral spreading movement occur (eg a reinforced soil embankment in preference to a bridge).

5.1.4 Background and commentary

The earthquake provisions included in this edition of the *Bridge manual* have been developed with reference to:

- AS/NZS 1170.0 *Structural design actions* part 0 General principles(4)
- NZS 1170.5 *Structural design actions* part 5 Earthquake actions – New Zealand(1)
- *Seismic design and retrofit of bridges*(5)
- *Displacement-based seismic design of structures*(6)
- NZS 3101 *Concrete structures standard*(2).

In the specification of design earthquake loadings, extensive reference is made to NZS 1170.5(1) in this section of the *Bridge manual*. Where appropriate, text has been included but generally only where modification has been made. The reader is referred to NZS 1170.5 supplement 1 *Structural design actions* Part 5 Earthquake actions – New Zealand – Commentary(7) for background information relating to NZS 1170.5(1).

Within this section of the *Bridge manual*, the term "damage control limit state" (DCLS) is equated to the term ultimate limit state (ULS) as applied by NZS 1170.5(1), but is adopted to more appropriately represent the limit state under consideration. The following extract from the Commentary to NZS 1170.5(7) explains what this limit state represents:

> Given the current state of knowledge of the variables and the inherent uncertainties involved in reliably predicting when a structure will collapse, it is not currently considered practical to either analyse a building to determine the probability of collapse or base a code verification method around a collapse limit state. It is therefore necessary to adopt a different approach for the purposes of design.

5.1.4 continued

It is possible to consider a limit state at a lower level of structural response, at a level where structural performance is more reliably predicted, and one that is familiar to designers and then rely on margins inherent within design procedures to provide confidence that acceptable collapse and fatality risks are achieved. In this Standard [NZS 1170.5(1)] this limit state is referred to as the ultimate limit state (ULS).

It is inherent within this Standard [NZS 1170.5(1)] that, in order to ensure an acceptable risk of collapse, there should be a reasonable margin between the performance of material and structural form combinations at the ULS and at the collapse limit state. For most ductile materials and structure configurations it has been assumed that a margin of at least 1.5 to 1.8 will be available. This is intended to apply to both strength and displacement.

In this edition of the *Bridge manual* the following significant departures from the previous edition and from the approaches advocated by the NZS standards have been introduced:

- Where bridges are to be designed to be ductile or to possess limited ductility, displacement-based design, as presented in the book *Displacement-based design of structures*(6) has been adopted as the preferred method of analysis, unless other design analysis procedures are agreed to by the road controlling authority.
- The use of a structural performance factor (S_p) to modify the design seismic loading has been discarded.
- The design of potential plastic hinges within structures for flexure under seismic actions is to be based on the use of expected material strengths with no strength reduction factor applied. In the design of potential plastic hinge regions for flexure, seismic moment demands need not be combined with non-seismic demands when determining the required flexural capacity. For the assessment of overstrength actions to be designed for in applying capacity design, maximum feasible material strengths to be adopted are specified.
- A consequence of the above approach to the design of plastic hinges is that three scenarios need to be considered in the design of plastic hinges and of non-yielding elements:
  i. Non-seismic load combination actions as they act prior to any seismic response.
  ii. DCLS earthquake response with plastic hinges designed for horizontal earthquake response moments, axial loads and shears and permanent load axial loads and shears. Non-yielding elements are capacity designed for earthquake overstrength actions plus permanent load actions. The redistribution of permanent load actions as a result of relief of their moments at plastic hinge locations needs to be taken into account.
  iii. Post DCLS earthquake response with elements (including plastic hinge regions) designed for the non-seismic load combinations but taking into account the redistributed permanent load actions as a result of yielding during earthquake response.

  For the design of ductile, yielding, reinforced concrete elements, the strain limits, confinement reinforcement requirements, and method of design for shear outlined in *Seismic design and retrofit of bridges*(5) have been adopted in preference to the requirements specified by NZS 3101(2) though there are exceptions where the NZS 3101(2) requirements have been retained. Capacity design has been specified to be required whenever ductile yielding is expected, regardless of the level of structure ductility, to ensure shear failure, other types of brittle failure, and plastic hinging in non-ductile members does not occur.

5.1.5 Assessment of the earthquake performance of existing bridges

In general, this section 5 of the *Bridge manual* does not include assessment of the earthquake performance of existing bridges. It is recommended however that those undertaking earthquake assessments should avoid unnecessary conservatism and make use of simplified analysis procedures based on displacement-based design where possible. Detailed assessment analyses can be very time consuming and may not provide a more reliable prediction of performance than simplified methods.

5.2 Design earthquake loading and ductility demand

5.2.1 Design earthquake loading

a. DCLS design earthquake loading

The design earthquake loadings are defined by response spectra appropriate to the site location, including proximity to major active faults, the site subsoil conditions, the specified annual probability of exceedance of the design earthquake determined from 2.1.3, and the modification factors for damping and ductility.

For force-based design, design spectral accelerations for horizontal earthquake response are derived from the site hazard elastic spectra determined in accordance with 5.2.2 or 5.2.5, factored as specified by 5.5.1 and 5.5.3 for foundation damping and structural ductility respectively.

For displacement-based design, design spectral displacements for horizontal earthquake response are derived from the site hazard elastic acceleration response spectra determined in accordance with 5.2.2 or 5.2.5, factored as specified in 5.2.4(a) and (b), and factored as specified by 5.4.2 to take into account damping.

Vertical earthquake response is determined in accordance with 5.2.3 for force-based design or 5.2.4(c) for displacement-based design.

The need to increase the design earthquake loading due to possible local site effects or location shall be considered. Where significant these aspects and their implications for the design shall be discussed in the structure design statement.

b. CALS earthquake loading

Where consideration of the CALS is required the CALS earthquake loading shall be taken as 1.5 times the DCLS design acceleration or displacement response spectra determined as outlined in 5.2.1(a), but with an upper bound limit as specified in 5.2.2(c).

5.2.2 Site hazard elastic acceleration response spectra for horizontal loading

The site hazard elastic response spectrum for horizontal loading shall be determined in accordance with section 3.1 of NZS 1170.5[(1)] as modified herein, for the annual probability of exceedance corresponding to the importance level of the structure specified in table 2.1, or by a site-specific seismic hazard study in accordance with 5.2.5.

(Note that over the period range from T=0 to T=0.5 seconds, the NZS 1170.5[(1)] elastic site hazard spectra incorporate magnitude weighting, and the spectra correspond to 5% damping.)

a. Hazard factor (Z)

The hazard factor (Z) shall be derived from NZS 1170.5[(1)] figures 3.3 and 3.4, and/or table 3.3 with the hazard factor values for Auckland and Northland replaced by table 5.2 and figure 5.1 of this manual, north of the 0.15 contour of figure 5.1.

For the damage control limit state, the product ZR_u shall not be taken as less than 0.13.

5.2.2 continued

Figure 5.1: Hazard factors (Z) for Northland

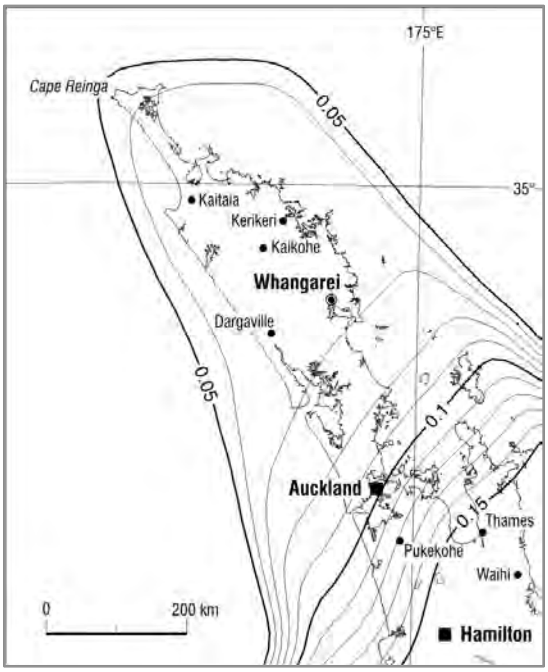

Table 5.2: Hazard factors (Z) for Northland

Location	Z Factor
Kaitaia	0.06
Paihia / Russell	0.06
Kaikohe	0.06
Whangarei	0.07
Dargaville	0.07
Warkworth	0.09
Auckland	0.10
Manakau City	0.12
Waiuku	0.11
Pukekohe	0.12

b. Return period factor (R_u)

NZS 1170.5[1] clause 3.1.5, Return period factor, shall be amended to read as follows:

The return period factor R_u for the damage control limit state, shall be obtained from table 3.5 of NZS 1170.5[1], or table 5.3 of this manual as appropriate, for the annual probability of exceedance appropriate for the importance level of the structure as prescribed in tables 2.1 to 2.3 of this manual.

Table 5.3: Return period factor (supplemental to table 3.5 of NZS 1170.5[1])

Required annual probability of exceedance	R_u
1/1500	1.5
1/700	1.15

c. Upper bound for ZR_u

The limit placed on ZR_u in NZS 1170.5[1] clause 3.1.1 of $ZR_u \leq 0.7$ shall be disregarded, as for bridges it is desirable to not only avoid collapse, but also to limit the extent of damage at the DCLS. For both the DCLS and the CALS, the upper bound earthquake event shall be taken to be one corresponding to $ZR_u = 1.05$.

d. Spectral shape factor $C_h(T)$

For periods from 3s up to a long-period corner period (T_L) the spectral shape factor $C_h(T)$ for a period of T seconds shall be determined from:

$$C_h(T) = C_h(3s) \times \left(\frac{3}{T}\right) \qquad 3s \leq T \leq T_L \qquad (5\text{-}1)$$

$$C_h(T) = C_h(T_L) \times \left(\frac{T_L}{T}\right)^2 \qquad T > T_L \qquad (5\text{-}2)$$

5.2.2 continued

Where:

$C_h(3s)$ = as given by section 3.1.2 of NZS 1170.5(1) for the corresponding site class or site period T_{site}

$C_h(T_L)$ = as determined from equation (5–2) for the period T_L specified for the location in table 5.4.

Table 5.4: Corner periods T_L throughout New Zealand for assigned moment magnitude M_w

Region	Assigned M_w[1]	Corner period T_L(s)
Northland, Auckland	6.5	3
Waikato, Taranaki, Western Bay of Plenty, Tauranga, Rotorua	6.9	5
Elsewhere in New Zealand	7.5	≥ 10

Notes

1. Magnitudes based on those recommended for consideration in determining collapse-avoidance motions in figure 6.3, with consolidation of some regions.

5.2.3 Site hazard elastic acceleration response spectra for vertical loading

The elastic site hazard spectrum for vertical loading shall be determined in accordance with section 3.2 of NZS 1170.5(1).

5.2.4 Design displacement response spectra for displacement-based design

a. Elastic displacement spectral shape factor $\Delta_h(T)$*

The period-dependent elastic displacement spectral shape factor $\Delta_h(T)$ shall depend on the site subsoil class defined by NZS 1170.5(1), clause 3.1.3, unless determined by an approved site-specific earthquake hazard study.

The displacement spectral shape factor may be obtained from the acceleration spectral shape factor of NZS 1170.5(1), table 3.1 by use of equation (5–3) when approved site-specific hazard studies defining the displacement spectral shape are not available:

$$\Delta_h(T) = \frac{T^2}{4\pi^2} C_h(T) g \quad \text{(mm)} \qquad (5\text{–}3)$$

Where:

$C_h(T)$ = the acceleration spectral shape given in table 3.1 of NZS 1170.5(1) for modal response spectrum and numerical integration time-history methods as modified by 5.2.2(d).

T = period (seconds)

g = gravitational acceleration (9807mm/s^2).

The displacement spectral shapes for different subsoil classes resulting from equation (5–3), with minor rounding, are listed in table 5.5 and are plotted in figure 5.2.

* 5.2.4(a) was adapted from a 2017 draft of *Guideline for the design of seismic isolation systems for buildings*(8) with the permission of the New Zealand Society for Earthquake Engineering, GNS Science and the Ministry of Business, Innovation & Employment.

5.2.4 continued

Table 5.5: Elastic displacement spectral shape factor $\Delta_h(T)$ (mm)

Period T (sec)	Displacement spectral shape factor $\Delta_h(T)$ (mm)			
	Site subsoil class			
	A strong rock, B rock	C shallow soil	D deep or soft soil	E very soft soil
0.0	0	0	0	0
0.05	1	1	1	1
0.075	3	4	4	4
0.1	6	7	8	8
0.2	23	29	30	30
0.3	53	66	67	67
0.4	75	94	119	119
0.5	99	124	186	186
0.56	114	143	232	232
0.6	125	156	254	268
0.7	151	189	308	365
0.8	179	224	364	477
0.9	207	259	421	604
1.0	236	295	481	745
1.5	391	490	797	1240
2.0	522	656	1060	1650
2.5	652	820	1330	2060
3.0	783	984	1590	2470
3.5	913[1]	1150[2]	1860[3]	2890[4]
4.0	1040[1]	1310[2]	2130[3]	3300[4]
4.5	1170[1]	1480[2]	2390[3]	3710[4]
5.0	1300[1]	1640[2]	2660[3]	4120[4]
6.0	1570[1]	1970[2]	3190[3]	4950[4]
7.0	1830[1]	2300[2]	3720[3]	5770[4]
8.0	2090[1]	2620[2]	4250[3]	6600[4]
9.0	2350[1]	2950[2]	4780[3]	7420[4]
10.0	2610[1]	3280[2]	5320[3]	8250[4]

Notes

1. Need not exceed $\Delta_{h\_A,B}(T_L)$ for the T_L value applicable for the location
2. Need not exceed $\Delta_{h\_C}(T_L)$ for the T_L value applicable for the location
3. Need not exceed $\Delta_{h\_D}(T_L)$ for the T_L value applicable for the location
4. Need not exceed $\Delta_{h\_E}(T_L)$ for the T_L value applicable for the location

5.2.5 addendum

Recent research (Cubrinovski et al[35] and Bradley et al[36]) indicates that for some locations the magnitude-weighted peak ground accelerations (PGA) from the hazard presented in NZS 1170.5[1] are lower than the magnitude-weighted PGAs obtained from site-specific probabilistic seismic hazard analysis (PSHA). A comprehensive study to update the seismic hazard of NZ is currently underway. This will include the changes to the response spectral values, not only PGA. The PGA hazard presented in Cubrinovski et al[35] cannot be used to derive spectral accelerations, and scaling of the spectral shape factor from NZS 1170.5[1] using the PGAs recommended in Cubrinovski et al[35] is not appropriate.

Consequently, the following interim measures shall be adopted whilst relevant research continues.

For projects in the six "principal locations" (ie Gisborne, Wellington, Palmerston North, Napier, Whanganui and Blenheim) and their neighbouring regions ("locations") identified in Cubrinovski et al[35] figure 11 where the project structures to be designed for earthquake resistance have a total value greater than or equal to $10 million or the value of any individual structure is greater than or equal to $3.5 million a site-specific seismic hazard study shall be undertaken as a special study, as per this clause 5.2.5. Otherwise the elastic hazard spectra and procedures specified in NZS 1170.5[1] for the appropriate site class shall be used, unless a site-specific seismic hazard study is undertaken.

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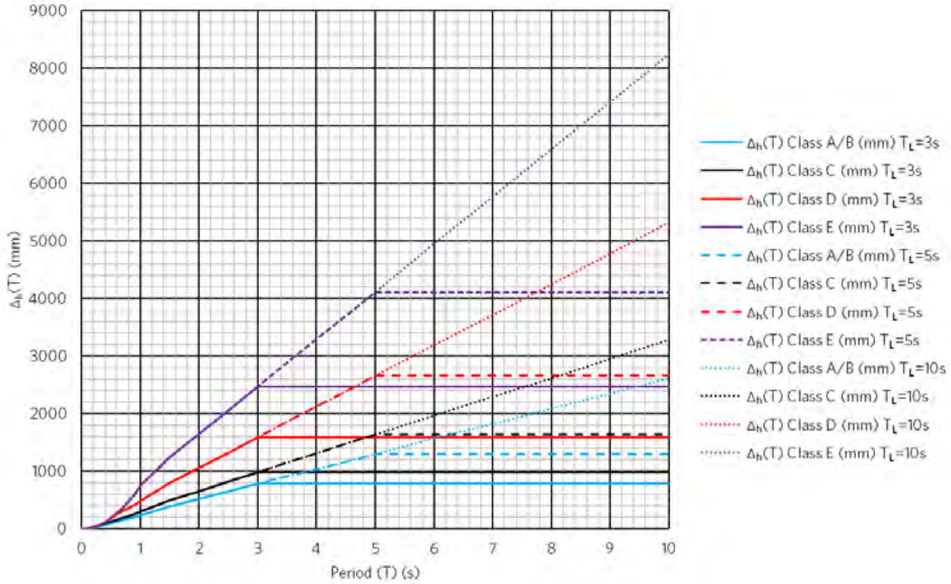
5.2.4 continued

Figure 5.2: Displacement spectral shapes ($\Delta_h(T)$) for the four subsoil classes and corner periods T_L=3s, 5s and 10s



b. Elastic design spectra for horizontal earthquake response

The design elastic displacement spectrum for horizontal earthquake response shall be defined by the product of return period factor R_u, seismic hazard factor Z, and the spectral shape factor $\Delta_h(T)$ in accordance with equation (5-4). The requirements of 5.2.2 in respect to the hazard factor, Z, the return period factor, R_u, and the upper bound for ZR_u apply also to the derivation of displacement spectra.

$$\Delta(T) = R_u Z N(T,D) \Delta_h(T) \qquad (5\text{-}4)$$

Where:

R_u	=	DCLS return period factor from 5.2.2(b)
Z	=	design seismic hazard factor, given in 5.2.2(a)
$N(T,D)$	=	near fault factor given in NZS 1170.5[(1)] clause 3.1.6
$\Delta_h(T)$	=	displacement spectral shape factor, given in table 5.5.

The corner-period elastic spectral displacements $\Delta(3.0)$, $\Delta(5.0)$ and $\Delta(10.0)$ are defined as the value of equation (5-4) at the periods of 3.0 seconds, 5.0 seconds and 10.0 seconds respectively.

c. Elastic design spectrum for vertical response

The displacement spectrum $\Delta_v(T)$ for vertical response shall be obtained by use of equation (5-3) with $C_h(T)$ replaced by $C_v(T)$ obtained from 5.2.3. (Note: $C_v(T)$ is derived from $C(T)$ and so incorporates the factors R_u, Z and $N(T,D)$ and is the acceleration response spectrum and not a spectral shape factor as $C_h(T)$ is.)

5.2.5 Site-specific seismic hazard studies

a. Basis for site-specific seismic hazard studies

The intensities of design ground motion specified by NZS 1170.5[(1)] and adopted by this manual have been derived from hazard analysis and are generally applicable to the design of bridges and other roading structures. However, at any given site the actual seismic hazard based on a probabilistic seismic hazard analysis may vary somewhat from the spectra specified by the standard due to a variety of factors.

5.2.5 continued

It should also be noted that the results of the hazard analysis have undergone modification in both regions of low seismicity and regions of high seismicity. In regions of low seismicity, the possibility of moderate magnitude earthquakes within 20km has been considered and results in the specified minimum ZR_u combination value of 0.13. The maximum considered earthquake (MCE) in the zone of highest seismicity represents the maximum motions considered by the NZS 1170.5[1] standard committee as likely to be experienced in New Zealand. The limit placed on ZR_u in NZS 1170.5[1] clause 3.1.1 of $ZR_u \leq 0.7$ shall be disregarded, as for bridges it is desirable to not only avoid collapse, but also to limit the extent of damage at the DCLS.

Special studies may be carried out to justify departures from the specific provisions of this manual and from NZS 1170.5[1]. All such studies shall be undertaken in a manner consistent with the principles upon which NZS 1170.5[1] was developed and in accordance with the special studies principles outlined in AS/NZS 1170.0[4] appendix A. In all cases the minimum provisions stated elsewhere, either below or in NZS 1170.5[1] shall still apply unless they too are included within the special study.

In addition to the NZS 1170.5 Commentary[7], the following papers and publication provide an outline of the basis of the NZS 1170.5[1] provisions and more recent developments:

- *Probabilistic seismic hazard assessment of New Zealand: New active fault data, seismicity data, attenuation relationships and methods*[9]
- *New national probabilistic seismic hazard maps for New Zealand*[10]
- *A new seismic hazard model for New Zealand*[11]
- *From hazard maps to code spectra for New Zealand*[12]
- *New Zealand acceleration response spectrum attenuation relations for crustal and subduction zone earthquakes*[13]
- *New national seismic hazard model for New Zealand: Changes to estimated long-term hazard*[14]
- *National seismic hazard model for New Zealand: 2010 update*[15]

Where a special study is undertaken to develop site-specific design spectra or for the selection of earthquake records for time history analysis, then the following limitations shall apply:

- The site-specific hazard study shall be peer reviewed by reviewers acceptable to the road controlling authority.
- Historical catalogue based seismicity and fault models shall be used.
- The modelling of the distributed seismicity component shall consider earthquakes of magnitude down to at least as low as 5.5.
- The site hazard spectra shall be based on a seismic hazard model that reflects New Zealand seismic and attenuation conditions.
- The site hazard used for the DCLS event shall be for the acceptable annual probability of exceedance based on the importance level of the structure.
- The site hazard spectrum for survival-level motions under which collapse is to be avoided (ie the CALS event) shall be scaled up by a factor of 1.5 from the DCLS design level motions corresponding to an assumed margin of safety of 1.5 resulting from the design procedures for ductile structures.
- The CALS and DCLS spectra need not exceed that calculated for 84th percentile motions from a magnitude 8.1 strike slip earthquake at zero distance.

5.2.5 continued

- Adopted 5% damped spectra shall be within ±30% of the design spectrum determined for the specific site from NZS 1170.5(1) combined with this manual, but for the CALS event, shall not be less than the 84th percentile motion resulting from a magnitude 6.5 earthquake located 20km from the site, nor for the DCLS less than ⅔ of the 84th percentile motion resulting from a magnitude 6.5 earthquake located 20km from the site.
- The common practice of truncating peaked acceleration response spectra over the short period range may be applied with such truncation to be limited to not exceed 25% of the peak spectral values nor to be below the 0.4 second spectral ordinate for rock or shallow soil sites, the 0.56 second ordinate for deep or soft soil sites, or the 1.0 second ordinate for very soft soil sites as defined in NZS 1170.5(1) section 3.1.3.
- Possible shortcomings at long periods of the ground-motion prediction equations (GMPEs) used in the hazard analyses should be recognised*. In particular, hazard spectra determined from site-specific studies should recognise the increase in the velocity-displacement corner period T_L from the default value of 3s assumed in NZS 1170.5(1). Depending on the GMPE models used, the site-specific spectra may not have sufficiently long T_L. If the corner period of the hazard spectrum is shorter than the T_L value given for the site location in table 5.4, it should be shown that the GMPEs have corner periods for magnitudes and distances that make sizeable contributions to the estimated hazard at the site that are at least as long as that given by the expression:

$$\log_{10}\left(\frac{T_L}{3}\right) = 0.5229(M_w - 6.5) \qquad (5\text{–}5)$$

The corner period of the hazard spectrum for the return period of interest should be compared to the corner period given by equation (5–5). The magnitude M_w to be used in equation (5–5) shall be such that larger magnitudes produce no more than 20% of the estimated exceedance rate of the spectral acceleration for the period closest to the effective period T_{eff} of the system. Note that the moment magnitude M_w used to calculate T_L may be smaller than that given in table 5.4 if it is justified by deaggregation analysis, except that it may not be taken as less than 6.5 anywhere (ie T_L cannot be reduced below 3s). If the GMPE model does not satisfy this requirement on its corner period, the resulting hazard spectra should be modified by increasing spectral values for periods longer than 3s by the expressions:

$$SA(T) = SA(3s) \times \left(\frac{3}{T}\right) \qquad 3s \le T \le T_L \qquad (5\text{–}6)$$

$$SA(T) = SA(T_L) \times \left(\frac{T_L}{T}\right)^2 \qquad T > T_L \qquad (5\text{–}7)$$

If a site-specific seismic hazard study is undertaken and results in a higher design seismic hazard spectrum than that derived from this manual together with NZS 1170.5(1), then the site-specific hazard spectrum shall be adopted.

b. Documentation of site-specific seismic hazard studies

The results from any special study undertaken shall be presented in an appendix to the structure design statement in accordance with 2.7. The minimum details required to be included within the appendix are:

- the project geo-referenced coordinates

* The final bullet point of 5.2.5(a) was taken from a 2017 draft of *Guideline for the design of seismic isolation systems for buildings*(8) with the permission of the New Zealand Society for Earthquake Engineering, GNS Science and the Ministry of Business, Innovation & Employment.

5.2.5 continued

- the organisation/individual who has undertaken the special study
- a brief outline of the experience and capability of the agency and personnel undertaking the special study
- details of the seismicity model used as the basis of the study within which the seismic signature of faults of significance to the study are to be prescribed
- a description of how background seismicity has been incorporated in the model
- the attenuation relationships used within the model and, when international attenuation relationships are used, an explanation of their appropriateness for the New Zealand setting
- details of the site classification chosen together with justification
- the raw spectral results of the study together with an explanation of any adjustments or spectral smoothing that may have been applied to arrive at the proposed design spectra
- the proposed design spectra, compared with the requirements of this manual, and
- where the study provides earthquake ground motion records that may be used for time history analysis, the basis upon which these records have been selected, how any record scale factors have been devised and the resulting spectra relating to these records, together with comment on the presence or otherwise of forward-directivity effects in any records selected.

5.2.6 Structural performance factor (S_p)

In the design of roading structures a structural performance factor shall not be applied to reduce the design actions. Where in the materials design standards a structural performance factor is permitted to be applied, the structural performance factors shall be taken as S_p=1.0.

5.2.7 Designing and detailing for ductility

a. Section ductility and strain limitation

At all locations of plastic hinging or other inelastic behaviour the strain limits specified by 5.3.5 shall be satisfied, irrespective of whether displacement-based or force-based design is adopted.

Where the foundations and/or bearings provide additional flexibility to the structure to that of the piers alone, the flexibility of foundations and bearings should be considered when calculating compliance with strain limits. (See *Influence of foundation on the seismic response of bridge piers*(16) or *Seismic design and retrofit of bridges*(5) for further explanation).

a. Classification of structures, and nominally ductile structures

This manual considers only three classes of structure according to their behaviour at the DCLS: fully elastic structures, structures of limited ductility, and ductile structures, as outlined in 5.6.4(a).

Nominally ductile structures, as referred to in the referenced material design standards, shall as a minimum be designed and detailed in accordance with the requirements for structures of limited ductility, with capacity design applied to protect elements not intended to yield against brittle failure.

b. Freedom to displace

All structures designed on the basis of developing ductility (ie ductile structures, and structures of limited ductility) shall possess sufficient freedom to displace to enable their design level of ductility to develop and their ductile behaviour to be maintained in earthquake events exceeding the DCLS, up to the CALS event.

5.3 Analysis methods – general

5.3.1 General

Design forces on members shall be determined from analyses that take account of the stiffness of the superstructure, bearings, piers and foundations. The design load shall be applied to the whole structure. Consideration shall be given to the effects on structural response of likely variation in both structural and foundation material properties. Consideration shall also be given to the consequences of possible yielding of components of the foundation structure or soil and of rocking or uplift of spread footings on the response and energy dissipation characteristics of the structure. The type of analysis used shall be appropriate to the form of structure being designed.

Analyses shall be undertaken for two orthogonal horizontal directions and the vertical direction. For horizontally curved bridges, one of the horizontal directions shall be the chord between the two abutments. Where appropriate, alternative orientations for the two orthogonal horizontal directions shall be considered in order to capture the maximum effects on individual structural elements. For example, where piers are founded on groups of piles, the effect of the earthquake loading acting along the diagonal of the pile group should be considered. For skewed bridges, one orientation for the two orthogonal horizontal axes to be considered should be parallel and perpendicular to the skewed alignment of the piers and abutments.

5.3.2 Combination of seismic actions from elastic analyses

For elastically responding or brittle structures, a combination of the effects of orthogonal seismic actions shall be applied to the structural elements to account for the simultaneous occurrence of earthquake shaking in two perpendicular horizontal directions. Seismic forces and moments on each of the principal axes of an element shall be derived as set out below. The absolute values of effects (forces or moments) resulting from the analyses in two orthogonal directions shall be combined to form two load cases as follows:

LOAD CASE 1: 100% of the effects resulting from analysis in direction x (eg, longitudinal) plus 30% of the effects resulting from analysis in the orthogonal direction y (eg transverse).

LOAD CASE 2: 100% of the effects resulting from analysis in direction y (eg transverse) plus 30% of the effects resulting from analysis in the orthogonal direction x (eg longitudinal).

Concurrency of loading in two perpendicular horizontal directions need not be considered for structures that are ductile (including those detailed for limited ductility) in both directions but shall be considered for structures that are ductile in only one of the two directions.

For further explanation refer to NZS 1170.5 Commentary(7), clause C5.3.1.)

5.3.3 Vertical seismic response

The vertical seismic response shall be considered to act non-concurrently to horizontal seismic response.

Bridge superstructures shall be designed to remain elastic under both positive and negative vertical acceleration induced by DCLS seismic response, while collapse is to be avoided under the CALS seismic response.

A span-by-span static analysis may be used, where the span under consideration is modelled together with adjacent continuous spans, if any, at either end of the span. End support conditions at the far end of the adjacent span shall be considered fixed, if continuous over the support, or pinned, as appropriate (eg if the end of the adjacent span is simply supported at an abutment).

5.3.3 continued

Vertical seismic response moments shall be determined from the spectrum defined by 5.2.3, 5.2.4(c) or 5.2.5 which are based on a damping ratio of 0.05. The damping ratio for vertical seismic response shall be taken as 0.02 for structural steel girder superstructures (including steel truss and steel arch supported superstructures), 0.03 for prestressed concrete superstructures, and 0.05 for reinforced concrete girder superstructures. The spectrum shall be modified for structural steel girder and prestressed concrete girder superstructures by multiplying by the appropriate damping modifier M_{ξ} determined from equation (5–17).

5.3.4 Pier elastic flexibility

a. Pier yield curvature

Yield displacements and elastic stiffness of piers shall be based on the pier yield curvature. The yield curvature of the bi-linear representation of the pier moment-curvature relationship depends on the section depth D in the direction considered, and the flexural reinforcement (or structural steel) yield strain ε_y and may be approximated by equation (5–8) for most common pier shapes. For specific pier shapes a more accurate equation may be obtained from *Displacement-based seismic design of structures*[6], section 3.4.2:

$$\phi_y = \frac{2.15\varepsilon_y}{D} \qquad (5\text{–}8)$$

Where:

ε_y = yield strain of flexural reinforcement or structural steel based on the probable (expected) yield strength

D = section depth in the direction considered.

For piers with non-prismatic or complex prismatic section shapes the yield curvature may be determined by finite-element analysis or other means recognizing the non-linear behaviour of materials and the influence of cracking, where appropriate.

The effective elastic stiffness of a pier may be approximated by equation (5–9):

$$EI = \frac{M_N}{\phi_y} \qquad (5\text{–}9)$$

Where:

M_N = moment capacity of the critical section determined in accordance with 5.6.2

ϕ_y = yield curvature defined by equation (5–8).

b. Yield displacement of piers

The yield displacement of a pier will depend on the yield curvature and the end fixity conditions at base and top, and for piers of constant section over their height may be expressed as:

$$\Delta_y = C_1\phi_y\left(H + L_{sp}\right)^2 + \Delta_{yf} \qquad (5\text{–}10)$$

Where:

C_1 = coefficient dependent on the end fixity conditions (refer to C5.7.2 in *Bridge manual commentary*)

Δ_{yf} = displacement at the pier top resulting from foundation deformation

ϕ_y = yield curvature at the pier critical section

H = effective column height (see figure C5.2 in *Bridge manual commentary*)

5.3.4 continued

L_{sp} = strain penetration length for reinforced concrete piers

= $0.022 f_{sye} d_{bl}$ at top and/or bottom of pier

Where, for reinforced concrete piers:

d_{bl} = diameter of flexural reinforcement

f_{sye} = probable (expected) yield strength of the flexural reinforcement.

Where bearings are provided, the bearing deflection does not affect the pier yield displacement but will modify the displacement of the superstructure at the pier position.

(For guidance and worked examples for other pier forms, eg pile columns with plastic hinges forming below ground, refer to *Displacement-based seismic design of structures*(6), section 10.3.)

5.3.5 Strain limits for the damage control limit state and the collapse avoidance limit state

Material strains at plastic hinges are due to the combination of both permanent and earthquake actions. They shall be assessed based on load combination 5A of table 3.3.

The limiting strains specified below for fully ductile structures shall be complied with at the DCLS. For reinforced concrete structures classified as limited ductile, strain limits of 0.58 times those for fully ductile structures shall apply. Where there is a need to consider strain limitation at the CALS, the strain limit adopted shall be the DCLS strain limit for concrete increased by a factor of 1.5 and for reinforcing steel the strain shall not exceed 0.08.

At potential plastic hinge locations that are inaccessible for inspection and repair (eg below water level or >2m below ground level) the allowable strain limits shall be reduced by a factor of 0.7.

a. Reinforcing steel:

Limit strain in flexural reinforcing steel in plastic hinges shall be related to the volumetric ratio of transverse reinforcement (ρ_s) in accordance with equation (5–11), and shall not exceed 50% of the strain ε_{sul} at maximum stress:

$$\varepsilon_{sd} = 0.015 + 6(\rho_s - 0.005) \leq 0.5\varepsilon_{sul} \quad (5–11)$$

Where:

ρ_s = volumetric ratio of transverse reinforcement

ε_{sul} = strain at maximum stress of longitudinal reinforcement, not to be taken as larger than 0.10 for 500E reinforcement or 0.12 for 300E reinforcement.

b. Concrete compression strain:

Limit compression strain of concrete in plastic hinges shall be related to the volumetric ratio of transverse reinforcement (ρ_s) and shall not exceed the value given by equation (5–12):

$$\varepsilon_{cd} = 0.004 + 1.4\frac{\rho_s f_{sy.t}\varepsilon_{sut}}{f'_{cc}} \quad (5–12)$$

Where:

f'_{cc} = confined compression strength of the concrete, which may be taken as $1.5f'_{ce}$ (f'_{ce}= expected concrete compression strength – see C5.13.1 in *Bridge manual commentary*) if not calculated by a rational analysis

$f_{sy.t}$ = lower characteristic yield stress of the transverse reinforcement

5.3.5 continued

ε_{sut} = strain at maximum stress of transverse reinforcement, not to be taken as larger than 0.10 for 500E reinforcement or 0.12 for 300E reinforcement.

c. Structural steel strain:

Limit compression and tension strain in ductile structural steel piers shall not exceed values corresponding to the onset of buckling under cyclic reversals of stress. In the absence of definitive design information, a value of ε_{sd}= 0.02 may be assumed.

d. Hollow concrete piers:

The maximum concrete compression strain for hollow reinforced or prestressed piers shall not exceed the value given by equation (5–12) or 0.005.

e. Prestressing steel:

Tensile strain in prestressing steel shall not exceed the limit-of-proportionality strain (ie 0.1% proof strain).

f. Concrete infilled steel tubes:

Concrete infilled steel tubes which satisfy the steel encasement thickness requirements of NZS 3101[2] clause 10.3.11.6.1 to ensure that buckling of the steel tube is precluded may use the strain limits for concrete compression strain and for reinforcing steel and structural steel strain nominated elsewhere in this clause 5.3.5.

Alternatively concrete infilled steel tubes which satisfy the casing width to thickness ratio requirements of AS/NZS 5100.6 *Bridge design* part 6 Steel and composite construction[22] clause 10.6.1.5 may be designed according to AS/NZS 5100.6[22] provided the displacement ductility demand on that member is no greater than 1.0 at CALS.

More slender concrete infilled steel tubes (ie with increased aspect ratio of the casing) may be used when supported by authoritative research acceptable to the road controlling authority, and in conjunction with limits for concrete compression strain and for reinforcing steel and structural steel strain which capture the test results reported by that research. The approach shall be documented in the structure design statement.

Where a permanent steel casing is to be used to contribute to the axial and/or flexural strength of the pile or cylinder, allowance shall be made for the casing section loss due to corrosion and the casing thickness after section loss shall comply with the requirements of NZS 3101[2] or AS/NZS 5100.6[22] as appropriate.

5.3.6 Limitations on displacement

Deflections of the structure under the effects of the design DCLS earthquake shall not be such as to:

a. endanger life

b. cause loss of function

c. cause contact between parts if such contact would damage the parts to the extent that persons would be endangered, or detrimentally alter the response of the structure or reduce the strength of structural elements below the required strength

d. cause loss of structural integrity.

Deflections of the structure under the effects of the design serviceability limit state earthquake shall not be such as to cause loss of function.

5.3.7 P-delta effects

An analysis for P-delta effects at the DCLS shall be carried out where either the displacement-based design method, the equivalent static force method or modal response spectrum method of analysis are used unless any one of the following two criteria are satisfied:

5.3.7 continued

a. the largest translational period based on the initial elastic stiffness of the structure, given by equation (5–9), is less than 0.4 seconds

b. the height of the structure measured from the point of fixity of its foundation is less than 15m and the largest translational period based on the initial elastic stiffness of the structure is less than 0.6 seconds.

When the numerical integration time history method of analysis is used, P-delta effects shall be incorporated into the analysis for the DCLS.

To avoid the "ratcheting" of structural displacements leading to a large residual displacement and possible instability, the unfactored P-delta moments at the DCLS shall not exceed 25% of the pier-base moment capacity.

For concrete piers, the required seismic design moment at potential plastic hinge locations shall be increased by 50% of the calculated P-delta moment when the P-delta moment exceeds 10% of the pier-base moment capacity.

For steel piers, the required seismic design moment at potential plastic hinge locations shall be increased by 100% of the calculated P-delta moment when the P-delta moment exceeds 5% of the pier-base moment capacity.

The criteria above are presented for plastic hinging at the base of a pier. Similar criteria apply for plastic hinging in other locations (eg in piles or pile-columns).

5.3.8 Distribution of structural mass

a. General

Either a distributed mass model or a lumped mass model may be adopted to represent the distribution of structural mass. For displacement–based analysis the structure's mass is usually modelled as lumped masses at nodes throughout the structural model.

Lumped mass modelling shall comply with the following:

- As a minimum representation of the seismic mass distribution, tributary superstructure mass, including mass of superimposed dead load, mass of pier caps and effective mass of piers shall be combined as a single mass acting on the axis of the pier at the height of the centre of gravity of the combined masses. In this context the effective mass of the piers may be taken as 33% of the total mass of the pier columns or wall (excluding the mass of the pile-cap and pier foundations), with this effective mass positioned at the base of the pier-cap or at the superstructure soffit if there is no pier cap.
- Where the superstructure mass is supported on bearings whose flexibility in the direction considered is such that the superstructure seismic response displacements are expected to significantly exceed pier cap displacements the tributary mass should be represented by a two-mass model separately comprised of the superstructure tributary mass, and the combined pier and pier cap beam mass, separated by a flexible element representing the bearing.
- For bridges with spans longer than 40m and with significant lateral flexibility of superstructure, the superstructure mass distribution should be represented by at least four masses along the length of each span.
- For bridges with tall piers (25 to 40m height) of significant mass, the pier mass distribution should be represented by at least four concentrated sub-masses up the pier height.
- For the analysis of vertical seismic response, the mass of the span under consideration, and of the adjacent spans, if any, should be distributed to not less than four locations along each of the spans.

5.3.8 continued

b. Horizontal torsion

Provision for variation in the seismic effect at supports, due to the centre of resistance and/or the centre of mass of the bridge not being in their calculated horizontal positions need not be considered.

c. Rotational inertia effects

For superstructures supported on single-stem piers with wide hammerheads, the effects of superstructure and hammerhead rotational inertia in generating additional moments in the pier shall be considered, and the additional moments provided for by appropriate detailing.

5.3.9 Member properties for analysis

In calculating natural period, forces and deflections under seismic loading the following values shall be used:

a. Concrete member section properties

For highly-stressed cracked sections (eg piers and piles), the sectional rigidity EI values appropriate to the damage control limit state shall be adopted. Guidance on appropriate values and their application may be found in *Seismic design and retrofit of bridges*(5), and *Displacement-based seismic design of structures*(6). The effective elastic sectional rigidity may be determined as outlined in 5.3.4.

For uncracked sections (eg prestressed concrete superstructures), the gross uncracked section value shall be assumed.

b. Sliding bearings

The coefficient of friction to be used for analysis shall be assessed on a conservative basis for the situation being considered. Unless otherwise justified for the specific materials and applications proposed, noting that the coefficient of friction can increase significantly with high velocities, and taking into account deterioration and maintenance requirements over time, 0.02 shall be assumed as the coefficient of friction for situations where a minimum frictional force is appropriate. For situations where a maximum frictional force is appropriate, a coefficient of friction of at least 0.15 shall be used.

c. Variation of material properties

The effects of actual material stiffness properties varying significantly from those assumed for analysis and design shall be taken into account. The likely variation in foundation soil stiffness properties in particular shall be considered. Default limits that should be considered, as a minimum, are 0.5 to 2.0 times the best estimate of soil stiffness, and for concrete 1.0 to 1.3 times the best estimate of the elastic modulus E_c.

5.3.10 Single degree of freedom methods of analysis

Both the equivalent static force method of analysis, based on using the initial tangent stiffness, and the simplified displacement-based design method, based on using the inelastic secant stiffness, as promoted in *Displacement–based seismic design of structures*(6) are approximate methods of analysis. The equivalent static force method of analysis relies on an assumption that the structure can be appropriately modelled as a single degree of freedom responding structure. The simplified displacement-based design analysis method outlined in 5.4 also makes the same assumption though this is not a general restriction on the method. Structures to which these methods of analysis, assuming single degree of freedom response, may be applied shall exhibit the following features:

- conventional superstructures, eg slab, beam and slab, box girder or truss superstructures

5.3.10 continued

- supported by single column or multi-column or wall piers and/or abutments of reinforced concrete, steel, or segmental precast concrete connected by either bonded or unbonded prestressing
- spans ≤100m
- pier heights ≤40m
- subtended angle between the abutments of ≤90°
- concrete strength $f_c{}'$≤65 MPa.

Structures designed by these methods but meeting the criteria of 5.3.11(a) under which dynamic analysis is recommended shall have their designs confirmed by an appropriate method of dynamic analysis.

5.3.11 Dynamic analysis

Where specified as required, the dynamic analyses procedures described in this section apply to both forced-based and displacement-based design.

a. Criteria under which dynamic analysis is recommended

Dynamic analysis to obtain maximum horizontal forces and displacements or ductility demand, should be carried out where it is not appropriate to represent the structure as a single degree of freedom oscillator. Such cases are:

i. Bridges where the mass of any pier stem (including any allowance for hydrodynamic effects) is greater than 20% of the mass of that part of the superstructure assumed to contribute to the inertia loading on the pier.
ii. For transverse analysis, where the bridge or an independent length of bridge between expansion joints has abrupt changes in mass distribution, horizontal stiffness or geometry along its length, or is substantially unsymmetrical.
iii. Bridges which describe a horizontal arc subtending more than 45°.
iv. Bridges in which the seismic load resistance is provided by structural systems other than conventional piers and abutments.
v. Suspension, cable-stayed and arch bridges.
vi. Bridges with piers designed to rock, for which inelastic time history analysis is preferred (see 5.6.12).

b. General

Consideration shall be given to the regularity of the structure and what directions of seismic attack are likely to yield the greatest demand on the structure. Dynamic analysis shall be undertaken for at least two orthogonal horizontal directions. For horizontally curved bridges one of these directions shall be the chord between the two abutments. Concrete member section properties shall be as defined in 5.3.9(a).

c. Modal response spectrum analysis

Modal response spectrum analysis shall comply with the requirements of NZS 1170.5[(1)] clause 6.3, as appropriate to the analysis of bridges.

The horizontal design response spectrum, $C_d(T)$, shall be given by:

$$C_d(T) = \frac{C(T)M_\xi}{k_\mu} \qquad (5\text{–}13)$$

Where:

$C(T)$ = the elastic site response spectrum determined from 5.2.2 or 5.2.3

k_μ = the modification factor for ductility, determined as set out in 5.5.3

5.3.11 continued

M_ξ = the modification factor for damping determined as set out in 5.5.1.

For each direction of earthquake attack considered, the combination of modal action effects shall be carried out using the complete quadratic combination (CQC) technique.

Where the base shear derived from the modal response spectrum analysis is less than the corresponding base shear derived from an equivalent static analysis the design seismic actions and displacements shall be scaled by the ratio of V_e/V where:

V_e = the base shear found from the equivalent static force method

V = the base shear found from the modal response spectrum method.

The vertical design response spectrum $C_v(T)$, is the elastic design spectrum as determined from 5.2.3 or 5.2.4 without modification for ductility but modified for damping in accordance with 5.3.3.

In displacement based analysis the horizontal design spectrum is not modified by the ductility reduction factor and effective member stiffnesses at expected maximum displacement demand are used together with appropriate damping levels. Refer to 5.4.1.

d. Numerical integration inelastic time history method

Inelastic time history analysis shall comply with the requirements of NZS 1170.5(1) clause 6.4, as appropriate to inelastic analysis and excluding requirements in respect to inter-storey deflection. Ground motion records shall comply with the requirements of NZS 1170.5(1) clause 5.5. In NZS 1170.5(1) clause 5.5.2(a), S_p shall be taken as 1.0.

In addition, the records shall contain at least 15 seconds of strong ground shaking or have a strong shaking duration of 5 times the fundamental period of the structure, whichever is greater.

Inelastic moment curvature and force displacement idealisations shall be appropriate to the materials being considered and the likely structural performance.

Where soil-structure interaction damping is not included in the modelling of hysteretic damping, the overall damping in the bridge system expressed as a percentage of critical equivalent viscous damping shall take into account both the structural damping and the additional damping arising due to the soil-structure interaction of the foundations.

Hysteretic rules adopted for non-linear time-history shall be appropriate for the materials and sections modelled.

The strain in inelastically deforming elements computed from an inelastic time history analysis and accepted for the design shall not be greater than that permitted by 5.3.5.

5.3.12 Seismic displacements

a. In displacement-based design determination of the seismic displacement is an inherent component of the design process.

b. In force based design, where the structural system can be simulated as a single-degree-of-freedom oscillator, the maximum seismic displacement (Δ) of the centre of mass shall be taken from the elastic displacement response spectrum for T_1 in cases where on site subsoil classes A, B, C and D T_1 exceeds 0.7 second or on site subsoil class E T_1 exceeds 1.0 second or calculated as follows, unless a more detailed study is undertaken:

$$\Delta = \frac{\mu C_d(T_1) g {T_1}^2}{4\pi^2} \qquad (5\text{–}14)$$

5.3.12 continued

Where:

Δ = seismic displacement in metres

T_1 = the fundamental natural period, in seconds

g = gravity, 9.81m/s^2

$C_d(T_1)$ = as defined in 5.5.3

μ = as defined in 5.5.2 and figure 5.3.

Where required to be assessed by 5.3.7, displacement due to P-delta effects shall be added to the displacement determined as above.

c. Where a modal response spectrum analysis is used, displacements derived from the analysis based on the design seismic response spectrum specified in 5.3.11(c) shall be factored by μ.

d. Where time history analysis is used, displacements may be taken directly from the analysis results.

5.3.13 Coherence of longitudinal seismic response

For bridges longer than 200m in length the effects of spatially varying ground motion may be significant and should be considered in design. There are three main causes of spatial variability:

- the incoherence effect, which represents random differences in the amplitudes and phases of seismic waves due to refractions and reflections of seismic waves during wave propagation from the seismic source
- the wave passage effect which describes the difference in arrival times at different locations
- the site-response effect, which accounts for the differences in the surface motion caused by variable soil profiles at the foundations of the piers and abutments.

The solution of the equations of motion of a multi-degree-of-freedom structure subjected to variable input motions at it supports can be separated into a pseudo-static part produced by the static differential displacements at the restrained joints and a dynamic part produced by the dynamic response of the structure.

A pseudostatic response analysis shall be carried out for bridges longer than 200m by estimating the non-synchronous displacements at the pier and abutment locations. The maximum longitudinal differential displacement can be estimated by:

$$\Delta_m = V_m \frac{L}{2V_a} \qquad (5\text{–}15)$$

Where:

V_m = the peak ground velocity (PGV),

L = the distance between the foundation locations

V_a = the apparent wave speed between the foundation locations.

The transverse differential displacements can be estimated from:

$$K_g = \frac{A_m}{V_a^2} \qquad (5\text{–}16)$$

5.3.13 continued

Where:

K_g = the maximum ground curvature

A_m = the peak ground acceleration (PGA).

The PGV for a specific site and earthquake event can be estimated using ground motion prediction equations (GMPEs). However, there is a moderately strong correlation between PGA and PGV allowing the PGV to be estimated with sufficient accuracy for this application from the PGA specified for the design ground motion. The ratio of PGV in m/s to PGA in g units is approximately 1.0 for soil category classification A and B (rock), 1.3 for soil category C (intermediate soils) and 1.7 for soil category D (soft and deep soils).

The apparent wave velocity between the foundation points is influenced by the wave propagation speeds in the underlying strata as well as the shear wave velocity in the upper layers. Surface waves may also have a significant influence on the apparent wave velocity. A value of 1,000m/s is recommended for design ground displacement and curvature estimates. This relatively low value will give an upper bound to the relative displacements between the foundation locations.

For bridge frames longer than 400m a direct numerical integration analysis using displacement input motions varying between the support points should be performed to obtain the dynamic part of the displacement of the structure displacement solution. Ground displacement time histories for the support points along the length may be estimated using simplified procedures suggested in *Seismic design and retrofit of bridges*(5). For bridge frames less than 400m in length the dynamic component can be estimated assuming uniform input motions at the support locations unless the local ground conditions vary significantly over the length. For bridge frames longer than 200m and where the local ground conditions vary significantly a numerical integration analysis should be carried out or an average response spectrum based on the different ground conditions used (see *Seismic design and retrofit of bridges*(5)).

Psuedostatic and dynamic displacement components should be combined using the square root of the sum of the squares (SRSS) rule.

Because of the difficulties in adequately representing the influence of ground conditions along the bridge together with uncertainties related to the lack of coherence and synchronism of the input ground motions at the piers it is recommended that bridges longer than 200m be subdivided into frames extending between movement joints in the superstructure or between abutments and movement joints. Each frame can be considered to respond independently of the rest of the bridge with the input spectrum (or time histories) based on the local soil conditions for each frame. Where the frames have no interconnection the relative displacement between them can be estimated using the displacement-based design method. For frames interconnected by linkages or restrainers, the charts presented in *Seismic design and retrofit of bridges*(5) and *Performance of linkage bars for restraint of bridge spans in earthquakes* (17) can be used to make predictions of the relative movements and linkage forces.

(Equation (5–15) and its application to bridge analysis is presented in *Seismic design and retrofit of bridges*(5). Background information on both equations (5–15) and (5–16) and the apparent wave speed parameter is given in *Seismic design of buried and offshore pipelines*(18). A detailed derivation of the equations is given in *Problems in wave propagation in soil and rocks* (19).)

5.4 Displacement-based analysis methods

5.4.1 General

Displacement-based design based on using the inelastic secant stiffness as promoted in *Displacement-based seismic design of structures*(6) provides an alternative approach to the force-based approach in determining earthquake actions for bridges. For design and construct type projects, where the lateral load resisting elements are designed to be ductile or to possess limited ductility, displacement-based design shall be adopted unless other methods are specified in the principal's (or minimum) requirements for the project or are otherwise agreed to by the road controlling authority. Seismicity is represented by displacement, rather than acceleration spectra, and is completely compatible with the seismic hazard as defined in 5.2.

Strength requirements for seismic resistance are based on strain limits defined for the damage control limit state. Strength so determined is taken to be adequate for the serviceability limit state and for the collapse avoidance limit states provided that there is no change in the mode of structural behaviour under seismic loading greater than the DCLS event and that there are no additional loads applied to the structure during seismic response, eg due to soil lateral spread.

The final design of all bridge structures of importance level 4 and of all bridges with significant irregularity of structural form resulting from high horizontal curvature and/or adjacent piers of significant difference in stiffness, should be verified by modal response spectrum analysis using effective member stiffness at expected maximum displacement demand together with appropriate damping levels, or non-linear time-history analysis, in accordance with 5.3.11. The selection of earthquake records for time history analysis shall comply with 5.3.11(d). (Use of modal analysis requires substitute structure modelling. Refer to *Seismic design and retrofit of bridges*(5) sections 4.4.3 and 4.5.2(b) for guidance on undertaking this.)

For a bridge to be considered to be regular it shall satisfy the requirements of the AASHTO *Guide specifications for LRFD seismic bridge design*(20), table 4.2-3 and should also satisfy the relative stiffness between bridge elements requirements of clauses 4.1.2 and 4.1.3 of the *Guide specifications for LRFD seismic bridge design*(20).

The procedure for displacement-based earthquake design will generally proceed in accordance with the following steps. However, when it is obvious that specific seismic design will be required, steps (iii) to (vi) may be omitted.

i. Determine the site seismicity in terms of the elastic design displacement spectrum.
ii. Determine the yield displacements of all piers.
iii. Check whether yield displacements exceed the elastic corner-period displacement of the elastic design displacement spectrum. If so, standard detailing of the load resisting members appropriate to an elastically responding structure in accordance with this manual and the referenced materials standard will be adequate subject to the requirements of strength, ductility (if mobilised) and stability under response to the CALS event being confirmed.
iv. If the check in step (iii) fails, determine the fundamental period of bridge in the direction considered.
v. Determine elastic displacement response at fundamental period.
vi. Check whether yield displacements exceed the displacement requirements of the elastic design displacement spectrum for fundamental period. If so, standard detailing of the load resisting members in accordance with this manual and the referenced materials standard will be adequate, subject to the requirements of strength and stability under response to the CALS event being confirmed.

5.4.1 continued

vii. If ductile earthquake design is indicated by the above steps, carry out displacement-based earthquake design, in accordance with the following provisions, to determine required lateral strength of piers and abutments.

More complete information on the procedure is available in *Displacement-based seismic design of structures*[6].

5.4.2 Reduced design displacement spectrum for ductile response

The equivalent viscous damping ξ_e for the bridge or bridge sub-frame, corresponding to the design ductility level of response (SLS, DCLS or CALS) shall be calculated in accordance with 5.4.3(f). Allowance shall be made for elastic and hysteretic damping associated with pier ductility, superstructure flexure, foundation flexibility and abutment displacement, as appropriate, in accordance with 5.4.3(g).

The reduced design displacement spectrum $\Delta_d(T)$ for ductile response shall be found by multiplying the elastic displacement spectrum given by equation (5–4) by the damping modifier M_ξ defined by equation (5–17).

$$M_\xi = \left(\frac{0.07}{0.02+\xi_e}\right)^\alpha \tag{5–17}$$

Where:

α = 0.25 for near-field situations, within 10km of a major active fault shown in NZS 1170.5[1] figure 3.5 or of faults with a recurrence interval of less than 2000 years; or

= 0.5 for all other situations

ξ_e = equivalent viscous damping ratio, given in equation (5–23).

Thus the design displacement

$$\Delta_d(T) = M_\xi RZN(T,D)\Delta_h(T) \tag{5–18}$$

Where:

R shall be taken as $\frac{R_u}{4}$ for the SLS, R_u for the DCLS and $1.5R_u$ for the CALS.

Z, $N(T,D)$ and $\Delta_h(T)$ are as given in 5.2.4(b).

5.4.3 Seismic analysis for design strength of plastic hinges

a. Design lateral earthquake force

The design lateral earthquake forces shall be determined in accordance with the provisions of this section.

The design lateral earthquake force, F_F, for a bridge frame shall be determined from equation (5–19):

$$F_F = k_e\Delta_d \tag{5–19}$$

Where:

Δ_d = the characteristic design displacement of the frame, defined in 5.4.3(b)

k_e = the effective stiffness of the frame, defined in 5.4.3(d).

Abutment design lateral forces shall be calculated in accordance with 5.4.8.

b. Frame characteristic design displacement

The characteristic design displacement of the frame is defined by equation (5–20):

$$\Delta_d = \frac{\sum_{i=1}^{n}(m_i\Delta_i^2)}{\sum_{i=1}^{n}(m_i\Delta_i)} \tag{5–20}$$

5.4.3 continued

Where:

Δ_i = design displacements of the n masses describing the frame given in 5.4.4

m_i = the masses at the n mass locations describing the frame.

c. Frame effective stiffness

The frame effective stiffness is defined by equation (5–21):

$$k_e = \frac{4\pi^2 m_e}{T_e^2} \tag{5–21}$$

Where:

T_e = period, defined in 5.4.3(e)

m_e = the effective mass of the frame defined in 5.4.3(d).

d. Frame effective mass

The frame effective mass is defined by equation (5–22):

$$m_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i)}{\Delta_d} \tag{5–22}$$

Where:

Δ_d = the characteristic design displacement defined by equation (5–20).

e. Frame effective period

The frame effective period (T_e) at DCLS displacement response is found from the displacement spectra defined in equation (5–18) corresponding to the characteristic design displacement defined by equation (5–20), and the calculated equivalent viscous damping defined in 5.4.3(f).

f. Frame equivalent viscous damping ratio

The frame equivalent viscous damping ratio shall be related to the shear force (V_i), the displacement (Δ_i), and the damping ratios (ξ_i) of the structural components (piers, abutments, superstructure, foundations, bearings) of the frame according to equation (5–23):

$$\xi_e = \frac{\sum_{i=1}^{n}(V_i \Delta_i \xi_i)}{\sum_{i=1}^{n}(V_i \Delta_i)} \tag{5–23}$$

Where:

V_i = shear force in structural components of the frame at design response

Δ_i = design displacement of structural components of the frame (5.4.4)

ξ_i = damping of structural components of the frame given in 5.4.3(g).

g. Equivalent viscous damping ratio of component actions

Within this sub-clause (g) μ_m is the member displacement ductility assessed over the member height or length.

i. Reinforced concrete piers

The equivalent viscous damping ratio of reinforced concrete piers shall be related to the pier member displacement ductility (μ_m) by equation (5–24):

$$\xi_i = 0.05 + 0.444\left(\frac{\mu_m - 1}{\mu_m \pi}\right) \tag{5–24}$$

5.4.3 continued

ii. Structural steel piers

The equivalent viscous damping ratio of structural steel piers shall be related to the pier member displacement ductility (μ_m) by equation (5–25):

$$\xi_i = 0.02 + 0.577\left(\frac{\mu_m - 1}{\mu_m \pi}\right) \quad (5\text{–}25)$$

iii. Foundation rotation effect

In lieu of more accurate determination, the equivalent viscous damping associated with rotation of spread footings on dense sand and alluvium of greater than θ=0.00182 or on medium-dense sand greater than θ=0.00172 shall be given by equation (5–26) and (5–27) respectively. For rotations less than these values of θ a value of ξ_i=0.05 may be used.

For dense sand and alluvium: $\xi_i = 0.365 + 0.115\log_{10}\theta$ (5–26)

For medium dense sand: $\xi_i = 0.52 + 0.17\log_{10}\theta$ (5–27)

Where:

θ = the foundation rotation in radians.

For foundations on rock of essentially zero deformation no additional damping shall be assumed and the equivalent viscous damping shall be taken as ξ_i=0.05.

iv. Superstructure transverse flexural deformation

When a reinforced concrete superstructure is subjected to lateral deformation involving abutment reactions without significant abutment displacement, the superstructure damping ratio shall be taken as ξ_i=0.05. The value to be taken for prestressed concrete or structural steel superstructure shall be 0.03 and 0.02 respectively.

v. Abutment deformation

The equivalent viscous damping ratio associated with soil deformation at an abutment will depend on the abutment soil material and shear strain.

For abutment foundations, not supported by piles, and where significant sliding on the ground occurs ξ_i=0.25. For abutments not supported on piles and fitted with a friction slab ξ_i = 0.30.

Where the abutment is supported by piles, behaviour is further complicated. In lieu of a more accurate determination, a conservatively low value of ξ_i=0.12 may be adopted for analysis. If the piled abutment is fitted with a friction slab a value of ξ_i = 0.25 may be used.

vi. Bearings

- Elastomeric bearings: In lieu of specific manufacturers' data, use ξ_i=0.05
- Friction slider bearings: In lieu of specific manufacturers' data use:

$$\xi_i = 0.05 + 0.67\left(\frac{\mu_m - 1}{\mu_m \pi}\right) \quad (5\text{–}28)$$

- Elastomeric bearings in conjunction with lead plug: use manufacturers' data
- Steel damping elements: use equation (5–25)
- Friction pendulum bearings: use manufacturers' data.

vii. Piled foundations where hinges develop in piles

$$\xi_i = 0.10 + 0.565\left(\frac{\mu_m - 1}{\mu_m \pi}\right) \quad (5\text{–}29)$$

5.4.3 continued

viii.Pile/column designs

In lieu of detailed studies the following conservative values may be used.

- Column fixed to superstructure:

  Sand: $\xi_i = 0.075 + 0.03(\mu_m - 1) \leq 0.135$ (5–30)

  Clay: $\xi_i = 0.12 + 0.03(\mu_m - 1) \leq 0.18$ (5–31)

- Column pinned to superstructure:

  Sand: $\xi_i = 0.10 + 0.04(\mu_m - 1) \leq 0.18$ (5–32)

  Clay: $\xi_i = 0.15 + 0.04(\mu_m - 1) \leq 0.23$ (5–33)

ix. Friction slabs

A conservative value of ξ_i=0.25 may be used, independent of displacement level.

x. Segmental piers connected by un-bonded post-tensioning

ξ_i=0.05

xi. Segmental piers connected by bonded post-tensioning

ξ_i=0.05, provided tendon strain does not exceed the limit of proportionality.

5.4.4 Design displacement profile

The design displacement profile (Δ_i) shall be related to the normalized fundamental displacement mode shape (δ_i) scaled to fit the displacement capacity (Δ_c) of the critical inelastic structural element, measured at the appropriate mass location by the relationship:

$$\Delta_i = \delta_i\left(\frac{\Delta_c}{\delta_c}\right) \quad (5\text{–}34)$$

Where:

δ_i = the normalized fundamental displacement mode shape at location i

δ_c = value of the normalized fundamental displacement mode shape at the critical location c

Δ_c = design displacement capacity of the critical inelastic structural element at location c.

In equation (5–34), the displacement capacities of inelastic structural elements shall be based on the strain limits defined in 5.3.5, and shall include effects of foundation and bearing flexibility, where appropriate.

5.4.5 Displacement capacity of piers

The structural component of displacement capacity of a pier corresponding to the damage control limit state depends on the plastic hinge length (L_p) the limit state strains in the plastic hinge (5.3.5) and the pier height (H) and may be calculated from equation (5–35):

$$\Delta_u = \Delta_y + \Delta_p \quad (5\text{–}35)$$

where Δ_y is given by equation (5–10), and the plastic displacement is given by

$$\Delta_p = (\phi_u - \phi_y)L_pH \quad (5\text{–}36)$$

and the plastic hinge length is given by

$$L_p = k_{lp}H_c + L_{sp} \geq 2L_{sp} \quad (5\text{–}37)$$

5.4.5 continued

Where:

$$k_{lp} = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \qquad (5\text{–}38)$$

In equation (5–36), ϕ_u is the lesser of the damage control curvatures corresponding to the limit state strains defined in 5.3.5, ϕ_y is the yield curvature given by equation (5–8), and H is the height of the pier between critical sections of the plastic hinges at top and bottom of the pier (pier in double bending), or the height from the critical section of the plastic hinge to the point of contraflexure at top or bottom of the pier.

In equation (5–37), L_{sp} is the strain penetration length defined in 5.3.4(b) and H_c is the distance from the critical section of the plastic hinge to the point of contraflexure in the pier.

In equation (5–38), f_u and f_y are the ultimate and yield strengths of the pier flexural reinforcement. For reinforcing steel sourced from Pacific Steel, default values for f_u/f_y of f_u/f_y =1.2 for grade 500E and f_u/f_y =1.4 for grade 300E reinforcing steel may be adopted. For reinforcing steel from other sources the ratio of f_u/f_y shall be established from representative data obtained from the manufacturer.

In the case of pile-columns of constant cross-section, plastic rotation shall be assumed to occur at the level of maximum moment in the pile with the plastic hinge length taken to be $L_p = D + 0.1H_c \leq 1.6D$ where H_c in this case is the height from the ground surface to the point of contraflexure in the pier above the ground surface. In the case of pile columns with a fixed connection to the superstructure design will generally be governed by the column top hinge for which L_p shall be taken to be as defined by equation (5–37). (For further guidance in respect to the pile-column case refer to *Displacement-based seismic design of structures*[6] section 10.3 and in particular, the design examples of 10.3.5. This guidance includes methods for determining the depth of the plastic hinge forming in the ground and the C_1 coefficient in equation (5–10) for estimating the yield displacement.)

5.4.6 Distribution of design lateral force

The lateral design force F_F given by equation (5–19) shall be distributed to the n frame mass locations m_i in accordance with equation (5–39):

$$F_i = F_F \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \qquad (5\text{–}39)$$

5.4.7 Design seismic moments in potential plastic hinges

Design seismic moments in potential plastic hinge regions of a frame shall be determined from the lateral frame forces (F_i) using accepted methods of structural analysis, and shall include consideration of P-delta moments in accordance with 5.3.7. Stiffness of ductile elements shall be based on the secant stiffness at the design displacement (Δ_i).

5.4.8 Design abutment forces

Design abutment reactions shall be determined by one of the following approaches:

a. Where initial analysis indicates that elastic response of the frame is assured at the DCLS, design abutment forces may be determined by a static analysis using an assumed first-mode shape or an elastic modal analysis for both the DCLS and the CALS event (ie an event corresponding to a return period factor of 1.5R_u).

b. Where ductile response is adopted for design in accordance with 5.4.1, the abutment forces shall be determined by one of the following procedures:

   – Forces determined by effective modal superposition under the design seismicity, where the stiffness of ductile elements is the secant stiffness at design displacement response and the global damping ratio used in the analysis is the system damping determined in the displacement-based design (equation (5–23)).

   – Inelastic time history analysis under the design seismicity.

5.5 Force based analysis methods

5.5.1 Elastic response spectrum reduction due to foundation damping

Equivalent viscous damping associated with soil structure interaction may be taken into account to reduce the design seismic response spectrum by undertaking an initial seismic analysis using the elastic response spectrum to derive the relative shears and displacements for the above ground structure and the relative foundation shears and displacements; and then by applying the procedures of 5.4.3(f) to derive the equivalent combined structural and soil-structure interaction damping ratio associated with the whole structure, followed by equation (5–17), to derive the damping modifier (M_ξ). The damping modifier may then be applied to factor down the elastic response spectrum prior to applying the procedures of 5.5.2 to derive the modified elastic response spectrum. The damping modifier (M_ξ) shall not be taken to be less than 0.7.

The equivalent viscous damping ratios for the various foundation elements may be conservatively assumed as follows:

- spread footings founded on dense sand or alluvium, or medium dense sand:

Dense sand and alluvium	Rotation > 0.00182 radians $\xi_i = 0.365 + 0.115\log_{10}\theta$	Rotation ≤ 0.00182 radians $\xi_i = 0.050$
Medium-dense sand	Rotation > 0.00172 radians $\xi_i = 0.52 + 0.17\log_{10}\theta$	Rotation ≤ 0.00172 radians $\xi_i = 0.050$

Where:

θ = the foundation rotation in radians.

- spread footings founded on rock of essentially zero deformation: ξ_i=0.05
- pier pile foundations in sands and granular material: ξ_i=0.10
- pier pile foundations in clay: ξ_i=0.15
- abutments supported on piles: ξ_i=0.12
- abutments, unsupported by piles, and with friction slabs sliding on ground: ξ_i=0.25.

For above ground structural elements the equivalent viscous damping ratios may be taken as:

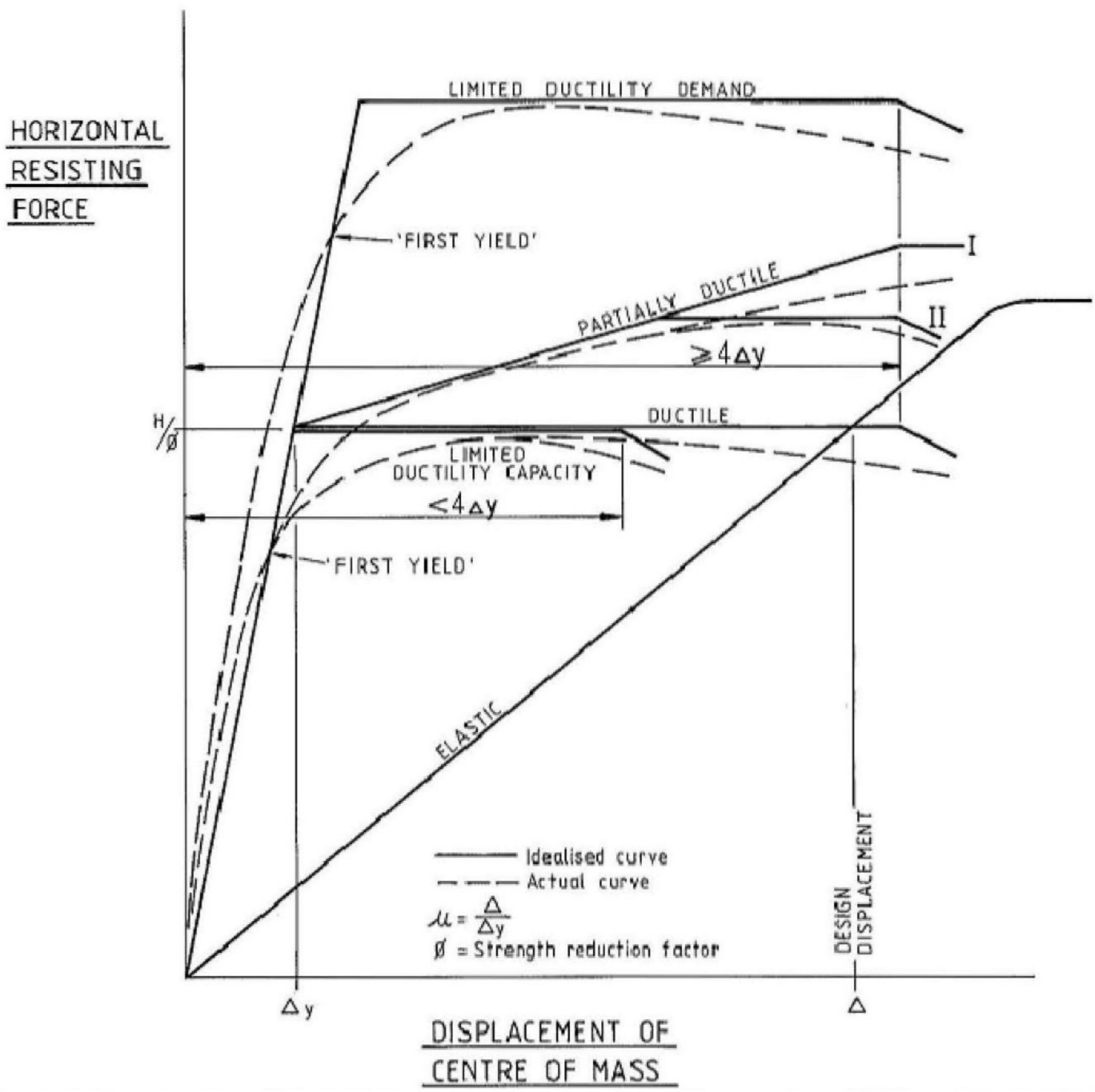
- reinforced concrete elements: ξ_i=0.05
- prestressed concrete elements: ξ_i=0.03
- steel elements: ξ_i=0.02.

5.5.2 Modified elastic response spectrum reduction due to ductility

The structure displacement ductility factor (μ) is defined as the design displacement of the centre of mass under DCLS earthquake response divided by the displacement at yield, as illustrated in figure 5.3, which also illustrates the nature of the force/displacement relationship for structures exhibiting various categories of behaviour.

In figure 5.3, the design force H is the design force derived by the procedures specified by 5.2 and this section, 5.5, including the modifications for ductility and foundation damping. For the flexural design of plastic hinges, the strength reduction factor (ϕ) is taken as 1.0.

Figure 5.3: Idealised force/displacement relationships for various structural categories



5.5.2 continued

For equivalent static force analysis and modal response spectrum analysis, ductility shall be taken into account to derive the design inelastic response spectrum from the site elastic hazard spectrum modified for foundation damping as set out in 5.5.3. The maximum allowable values of μ for various structural forms that may be adopted are listed in table 5.6, and examples for some of the structural forms are shown diagrammatically in figure 5.4. Force-based design is based on assuming a ductility demand at the outset of design. In all cases, the designer shall check the actual ductility demand that will be imposed on their structure by the design actions and ensure that the structure as detailed satisfies the maximum allowable value of μ for its form and is capable of sustaining the actual ductility demand, and that any plastic hinge section curvature ductility limitations imposed by the strain limits specified by 5.3.5 are satisfied.

5.5.2 continued

In the context of table 5.6, ductile, partially ductile and elastic structures are as defined in 5.6.4, 5.6.5 and 5.6.6 respectively. A structure on spread footings designed to rock is as defined in 5.6.12, and a locked-in structure is as defined in 5.6.8.

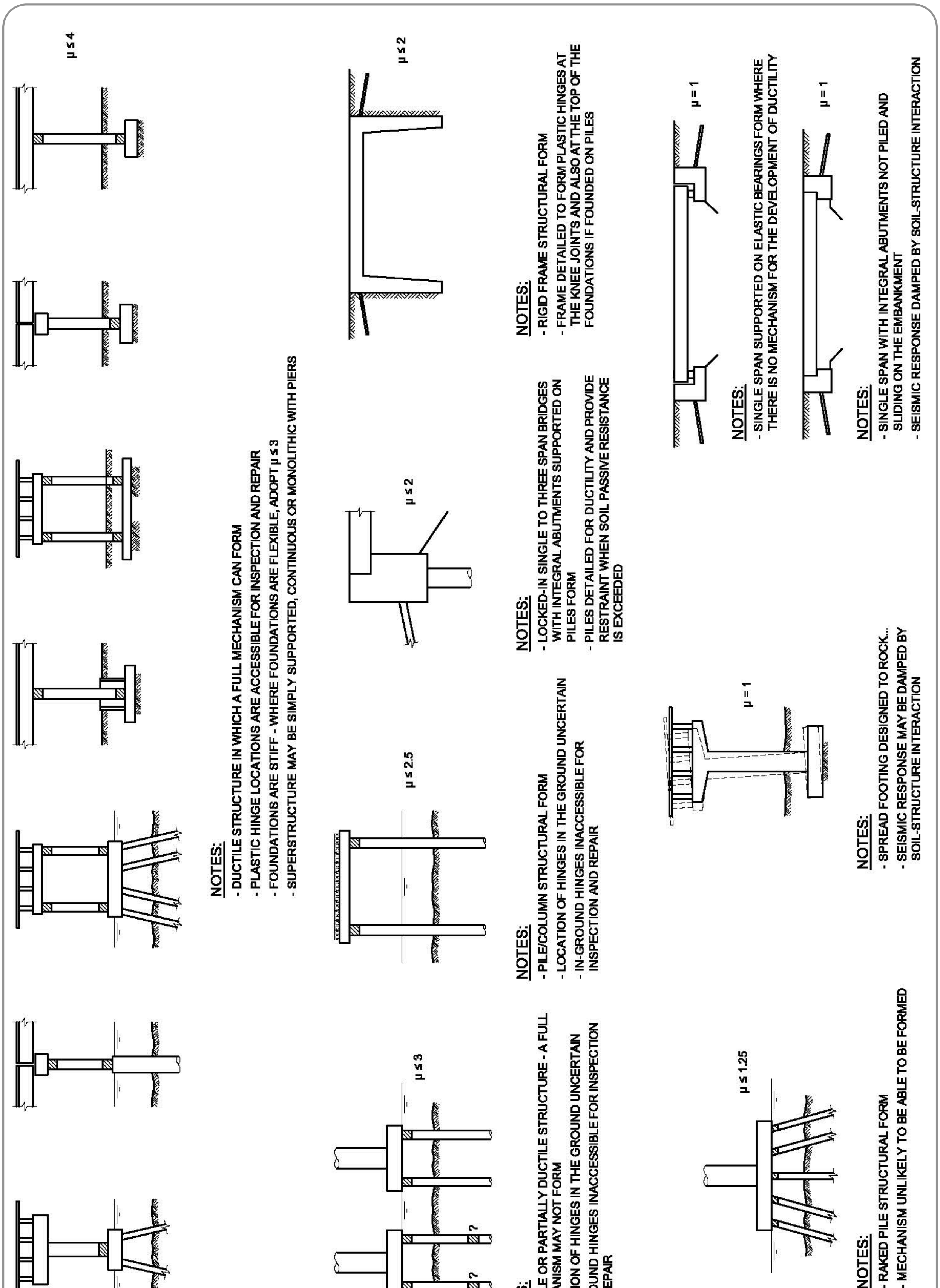
For cantilever columns on flexible foundations, the displacement due to the foundation shall be assessed as the displacement at the base of the column base plastic hinge plus the displacement due to rotation and displacement in the foundation at the level of the base of the plastic hinge projected up to the level of the centre of mass (ie the centre of the seismic inertia).

The structure displacement ductility factors for response in the longitudinal direction and in the transverse direction need not necessarily be the same value.

Table 5.6: Design displacement ductility factor (μ) – maximum allowable values

Energy dissipations system / structural form	μ
• Structures with ductile frame type piers in which plastic hinges form in the columns at design load intensity in reasonably accessible positions, eg less than 2m below ground but not below normal (or mean tide) water level to form a complete mechanism • Monolithic ductile pier (column or wall) – superstructure designs in which plastic hinges form at design load intensity in reasonably accessible positions • Structures with ductile cantilever columns or walls pinned to the superstructure in which plastic hinges form in reasonably accessible positions and where the foundations contribute less than 30% of the yield displacement	4.0
• All of the above types of structures in which plastic hinges are inaccessible, forming more than 2m below ground or below normal (or mean tide) water level, or at a design level that is not reasonably predictable. • Partially ductile structures (types I and II) • Structures with ductile cantilever columns pinned to the superstructure in which plastic hinges form and where the foundations contribute more than 30% of the yield displacement	3.0
• Structures with ductile pile-column piers (ie the pier is composed of continuous pile-column elements of constant cross-section) where plastic hinges form in the pile elements.	2.5
• Structures where plastic hinges form in ductile hollow columns • Structures where the earthquake resistance is provided predominantly by ductile vertical piles at the abutments. • Single span portal frame with ductile walls	2.0
• Structures where piers or abutments are supported on raked piles designed to resist the earthquake loads and where plastic hinges do not form in the columns.	1.25
• Structures with piers supported on spread footings expected to rock • “Locked-in” structure with abutments founded on spread footings • Elastically responding structure • Superstructures subjected to vertical response	1.0

Figure 5.4: Examples of maximum values of μ allowed by table 5.6 for the DCLS



5.5.3 Equivalent static force method of analysis

For a structure represented as a single-degree-of-freedom oscillator, the minimum horizontal seismic base shear force (V) for the direction being considered, shall be calculated as:

$$V = C_d(T_1)W_t \tag{5–40}$$

Where:

$C_d(T_1)$	=	horizontal design action coefficient, determined as set out below
W_t	=	total dead weight plus superimposed dead weight (force units) assumed to participate in seismic movements in the direction being considered.

The horizontal design action coefficient ($C_d(T_1)$) shall be:

$$C_d(T_1) = \frac{C(T_1)M_\xi}{k_\mu} \tag{5–41}$$

For the damage control limit state, $C_d(T_1)$ shall satisfy the following:

$$C_d(T_1) \geq \left(\frac{Z}{20} + 0.02\right)R_u \quad \text{but not less than } 0.03R_u \tag{5–42}$$

Where:

$C(T_1)$	=	the ordinate of the elastic site hazard spectrum determined from 5.2.2 or 5.2.5, for the fundamental translational period of vibration
T_1	=	the fundamental translational period of vibration
Z	=	the hazard factor, determined from 5.2.2(a) and NZS 1170.5(1) clause 3.1.4
R_u	=	DCLS return period factor from 5.2.2(b)
M_ξ	=	the modification factor for foundation damping determined as set out in 5.5.1
k_μ	=	the modification factor for ductility, determined as follows:

For soil Classes A, B, C and D as defined by NZS 1170.5(1) clause 3.1.3:

$$k_\mu = \mu \quad \text{for } T_1 \geq 0.7 \text{ seconds} \tag{5–43}$$

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \quad \text{for } T_1 < 0.7 \text{ seconds} \tag{5–44}$$

For soil Class E as defined by NZS 1170.5(1) clause 3.1.3:

$$k_\mu = \mu \quad \text{for } T_1 \geq 1.0 \text{ seconds or } \mu < 1.5 \tag{5–45}$$

$$k_\mu = (\mu - 1.5)T_1 + 1.5 \quad \text{for } T_1 < 1.0 \text{ seconds and } \mu \geq 1.5 \tag{5–46}$$

provided that for the purpose of calculating k_μ, for all soil types, T_1 shall not be taken less than 0.4 seconds.

The vertical design response spectrum $C_v(T)$, is the elastic design spectrum as determined from 5.2.3 or 5.2.4 modified for damping in accordance with 5.3.3 but without modification for ductility.

5.6 Member design criteria and foundation design

5.6.1 Capacity design

The principles of capacity design apply to both displacement-based design and force-based design and shall be applied to the design of ductile structures and structures of limited ductility as defined by 5.6.4(a). These principles are:

a. That elements of the structure intended to dissipate seismic energy through plastic deformation be designed to possess sufficient strength to withstand the design action, and to maintain their structural integrity sufficient to develop the necessary ductility without undue loss of strength.
b. That other elements and members of the structure, intended to remain elastic during earthquake response, be designed to withstand the forces induced in them through the plastically deforming elements developing their overstrength capacity under response of the structure to strong earthquake motions.

Plastically deforming elements will commonly be plastic hinge zones designed to form at the base and possibly also at the top of columns. However, they also include base isolation devices incorporating mechanical energy dissipation. Development of overstrength in these elements can arise through actual material strengths being greater than assumed in design and through the strain hardening of steel as it is strained plastically. For concrete plastic hinge locations, the overstrength capacity should be determined by moment curvature analysis using the probable ultimate material strengths given in column (2) of table 5.7 and with strain hardening of the flexural steel also allowed for in the analysis.

For wall or single column type piers, where simple analyses that do not take into account strain hardening of the reinforcement are used in lieu of a detailed analysis, using material strengths in accordance with table 5.7, column (2), the flexural overstrength capacity at the plastic hinge locations may be assumed to be 1.5 times the design strength where grade 500E reinforcement is used, and 1.7 times the design strength where grade 300E reinforcement is used.

For a bent containing multiple columns, the axial load in the columns will vary under seismic response acting in the plane of the bent, and this shall be taken into account in assessing the maximum moment capacity of the plastic hinges to be considered in determining the overstrength actions to be catered for.

In the strength design of elements and members intended to remain elastic, and of plastically deforming elements for modes of action other than that of the intended plastic action, shear failure and the formation of unintended plastic hinges shall be avoided.

Where either:

- simple methods of section analysis are used (eg *Gen-Col*[21]), or
- the moment curvature method of section analysis, incorporating reinforcement strain hardening

is used for the DCLS level of earthquake response, the dependable strength of capacity protected actions and locations shall be determined using conservative estimates of material strength in accordance with table 5.7, column (3) and standard strength-reduction factors (eg strength reduction factors of less than 1.0 as specified for reinforced concrete by NZS 3101[2] clause 2.3.2.2 or for structural steel as specified by NZS 3404:1997[3] table 3.3 or AS/NZS 5100.6 *Bridge design* part 6 Steel and composite construction[22] table 3.2). For concrete sections, the flexural strength should be determined at the extreme fibre compression strain of 0.004 or a reinforcement strain of 0.015, whichever occurs first. The capacity design of foundations shall comply with 6.5.4 and 6.5.5.

5.6.1 continued

As required by 5.1.3, where the mode of behaviour of the structure changes, or loading conditions acting on the structure change, from that applying at the DCLS in events greater the DCLS event, the avoidance of brittle failure and/or formation of unintended plastic hinges under levels of earthquake response up to the CALS shall be ensured. For this a moment–curvature analysis shall be undertaken of the plastic hinge overstrength capacity at the CALS level of plastic hinge curvature and reinforcement strain hardening. Material strengths as applied for design at the DCLS but less conservative strength reduction factors of ϕ=1.0 for flexure and axial load and ϕ=0.9 for shear and torsion shall be adopted in assessing the capacity of the capacity protected members and elements to withstand the CALS overstrength actions.

Since there is uncertainty regarding the strength and stiffness properties of the foundation soil or rock, and in the contribution of the soil or rock to either increased loads or increased resistance depending on the case, upper bound and lower bound properties shall be determined and used to assess the performance of the structure, as required by 5.3.9(c), with the most critical combinations of actions and levels of resistance used in the capacity design of the structure. (Refer also to 5.3.9(c).)

5.6.2 Required flexural and axial load capacity for seismic and other actions

a. Non-seismic and vertical seismic response load cases

The structural members, including critical ductile elements, shall be designed, using characteristic material strengths and with the normal strength reduction factors applied, with at least sufficient capacity to resist the factored action demands due to all non-seismic load cases and the vertical seismic response load case. The critical force actions shall be determined from consideration of how they will initially exist prior to any earthquake response resulting in inelastic behaviour, and also from making allowance for the redistribution of permanent load moments arising from any plastic deformation in the critical ductile elements under horizontal seismic loading.

b. Seismic load case

i. At potential plastic hinge locations

The critical ductile elements shall be designed for the following DCLS concurrent actions:

- horizontal seismic response moments or displacement demands (including P-delta effects determined in accordance with 5.3.7)
- horizontal seismic response axial forces
- permanent load axial forces.

Vertical seismic response and permanent load moments need not be combined with horizontal seismic response moments and may be ignored. Axial forces due to horizontal seismic response are to be added to the permanent load axial load effects. Where axial loads arise from soil or water permanent actions, the load factor to be applied to those axial loads shall be taken as 1.0. (Note 5.3.5. While at plastic hinges the moments due to permanent loads may be neglected in the design of the plastic hinge flexural capacity, the strains due to permanent load moments must be taken into account.)

The moment capacity at plastic hinge locations for DCLS horizontal seismic response actions shall be determined using probable material strengths (f'_{ce}, f_{sye} and f_{ye}) in accordance with table 5.7, column (1).

5.6.2 continued

It is recommended that section design of plastic hinges be undertaken using moment-curvature analysis that includes modelling of strain hardening of the reinforcement to achieve a more economical reinforcement design than will be achieved by conventional section design and estimated overstrength capacity demands. (Refer to *Displacement-based seismic design of structures*(6), section 4.5.1, for a design example.)

Flexural strength reduction factors need not be used for determination of seismic moment capacity. The moment capacity, taking into account concurrent axial load effects, shall not be less than the moment demand (including the associated P-delta effects) imposed by horizontal earthquake response determined from analyses in accordance with 5.4 or 5.5.

Table 5.7: Material strengths to be used in seismic design

	Probable (expected) material strength for plastic hinge zone design level flexural capacity (1)	Maximum feasible material strength for plastic hinge zone overstrength capacity evaluation (2)	Material strength for capacity design of non-hinging zones and plastic hinge shear capacity (3)
Concrete compressive strength	$f'_{ce} = 1.3f'_c$	$f'_c{}^\circ = 1.7f'_c$	f'_c
Flexural reinforcement	$f_{sye} = 1.1f_y$	$f_{sy}{}^\circ = 1.25f_y$	f_y
Transverse reinforcement	$f_{sy.te} = f_{sy.t}$	$f_{sy.t}{}^\circ = f_{sy.t}$	$f_{sy.t}$
Structural steel	$f_{ye} = 1.1f_y$	$f_y{}^\circ = 1.3f_y$	f_y

Where:

f'_c = the specified 28 days compressive strength of concrete

f_y = lower characteristic yield strength of longitudinal reinforcement or structural steel

$f_{sy.t}$ = lower characteristic yield strength of transverse reinforcement steel.

Notes:

1. The values for $f_{sy}{}^\circ$, $f_{sy.t}{}^\circ$ and $f_y{}^\circ$ do not include allowance for strain hardening of the steel.
2. For flexural reinforcement the values for the probable (expected) yield strength, f_{sye}, and for the maximum feasible (upper bound) yield strength, $f_{sy}{}^\circ$, have been determined from a review of 2012 Pacific Steel test results. Where reinforcement from other sources of supply is proposed to be used appropriate values for f_{sye} and $f_{sy}{}^\circ$ should be assessed from representative test data.

ii. At other locations

Elements in ductile and limited ductile structures, intended not to yield, shall be designed in accordance with 5.6.1 for the actions (moments and axial loads) acting on them when induced by the horizontal seismic response mobilising the overstrength capacity of the plastic hinges and combined with permanent load actions. The redistribution of permanent load moments due to the plastic hinging shall be taken into account.

The formation of unintended plastic hinges shall be avoided by capacity design in accordance with 5.6.1 unless the structure as a whole is otherwise designed to remain elastic up to the CALS.

5.6.2 continued

Structures responding elastically when subjected to the DCLS design earthquake event shall be provided with sufficient dependable strength (ie based on characteristic material strengths given in column (3) of table 5.7 with capacity reduction factors as specified in 5.6.1) to withstand the load combinations specified in 5.3.2.

(Refer to the *Displacement-based seismic design of structures*(6) sections 3.7 and 4.6, for guidance on the effects of stiffness reduction in the ductile member, the redistribution of the permanent load moments, and equilibrium considerations.)

iii. Serviceability limit state requirements

Prior to an earthquake, during serviceability limit state earthquake response, and following an earthquake where the structure is expected to be repaired and returned to service, the design of the structure shall also satisfy serviceability limit state requirements.

The capacity provided at plastic hinge locations, determined based on characteristic material strengths with normal capacity reduction factors applied, shall also be sufficient to withstand the serviceability limit state seismic actions determined using a return period factor of $R_u/4$ combined with the actions due to permanent loads corresponding to load combination 5A given in table 3.3.

Redistribution of gravity load moments is not permitted at the serviceability limit state unless a detailed study is undertaken to ensure a stable shake down situation arises and the deformations associated with this state do not conflict with serviceability requirements.

iv. Maintenance of stability and avoidance of ratcheting

The stability of the structure must be maintained and ratcheting avoided during and after the earthquake. The stability of beams cantilevering off portal frames as the extension of beams that are plastic hinging must be maintained by their cantilever moment being entirely reacted by their supporting columns. In structures with unbalanced lateral strengths and /or eccentric gravity loading causing ratcheting, the requirements of NZS 1170.5(1) clause 4.5.3 shall be complied with.

5.6.3 Required shear capacity and joint detailing for seismic actions

At all locations, shear forces resulting from seismic response mobilising the overstrength capacity of plastically deforming elements, shall be combined with shear forces resulting from the dead load of the structure and other permanent actions.

The design of the elements listed below for shear capacity shall comply with cited references, using the material strengths given in column (3) of table 5.7 and the strength reduction factors specified in 5.6.1:

- reinforced concrete elements carrying compression (*Displacement-based seismic design of structures*(6) section 4.7.3)
- footings and pile caps (*Seismic design and retrofit of bridges*(5) section 5.6)
- beam-column and footing/pilecap-column joints (*Seismic design and retrofit of bridges*(5) sections 5.4 & 5.6).

Alternatively, the design and detailing of beam-column and footing/pilecap-column joints shall comply with NZS 3101(2) clause 10.4.6.5 and chapter 15. In adopting the approach of *Seismic design and retrofit of bridges*(5), while that approach seeks to alleviate reinforcement congestion at joints, the requirements of NZS 3101(2) clause 10.4.6.5 to terminate the main flexural reinforcement with 90° hooks with the horizontal leg of the bend directed towards the far face of the column should be complied with to the maximum extent practicable.

5.6.3 continued

In the design of plastic hinge regions for shear, allowance shall be made for the degradation of the concrete contribution to shear strength with increasing curvature ductility demand as the seismic response increases from the DCLS up to the CALS. Adequate total shear strength shall be provided to ensure collapse avoidance at the CALS.

(Refer to *Displacement-based seismic design of structures*(6) section 4.7.3 for guidance on the degradation of the concrete contribution to shear strength with increasing curvature ductility demand.)

5.6.4 Ductile structures and structures of limited ductility

a. Classification of ductility level for the application of materials design standards

For the purpose of aspects of earthquake resistant design for which reference to the relevant materials design standard is required, the structure shall be classified into one of the ductility classes given in table 5.8 based on the displacement ductility factor adopted for design:

Table 5.8: Classification of ductility level for design

Adopted structural ductility factor for design for the DCLS	Materials standards Structure classification
$3.0 < \mu \leq 4.0$	Ductile structure
$1.0 < \mu \leq 3.0$	Structure of limited ductility
$\mu = 1.0$	Elastically responding or brittle structure

b. Characteristics of a ductile structure

Under DCLS horizontal loading, a plastic mechanism develops. After yield, increasing horizontal displacement is accompanied by approximately constant total resisting force. A ductile structure must be capable of sustaining the adopted design ductility factor through at least four cycles to maximum design displacement, with no more than 20% reduction in horizontal resistance. For the purpose of determining the design load for force-based design, the design ductility value is restricted to four or less, as specified in 5.5.2 and table 5.6.

A structure of limited ductility as illustrated in figure 5.3, may otherwise qualify as ductile or partially ductile, but its proportions or detailing mean that its ductility capacity at the DCLS is low.

c. General requirements

In a ductile structure, where the ductility is provided by plastic hinges, the hinge probable* flexural strengths shall be at least equal to the moments from an analysis as described in 5.3, 5.4 and 5.5. Hinge shear strength and the design of members resisting the hinge moments shall be according to capacity design principles as set out in 5.6.1. The capacity design requirements of this manual shall take precedence over those of NZS 1170.5(1) and the materials design standards that may be referred to.

Capacity design requirements will be considered satisfied if the overstrength flexural capacity of a hinge is matched by at least its own dependable† shear strength and the dependable shear and moment strength of resisting members forming the balance of the structure.

* Probable strength: The theoretical strength of a member section calculated using the expected mean material strengths as defined in 5.6.2.

† Dependable strength: The theoretical strength of a member section, calculated using section dimensions as detailed and the lower 5 percentile characteristic material strengths, (ie the nominal strength) multiplied by the strength reduction factor specified by the relevant materials code.

5.6.4 continued

Pile analysis shall also consider the flexural and axial load consequences of seismic ground distortions such as lateral spread and settlement resulting from liquefaction. Pile caps and other members shall be designed to resist the vertical shear and other actions resulting from plastic hinging at pile tops, where this is considered likely.

In particular, plastic hinging in piles is to be avoided if practicable and within reasonable cost.

Where seismic design is based on ductile response and it is possible through appropriate design to ensure that plastic hinging will only occur in locations readily accessible for inspection and repair, ie above water level or less than 2m below the ground surface, capacity design as outlined in 5.6.1 shall be applied to ensure this.

d. Column detailing

Special consideration shall be given to the detailing of concrete compression members, bearing in mind the manner in which earthquake-induced energy will be dissipated and the desirability of avoiding brittle failures, especially in shear. In particular, the ultimate shear capacity shall be assessed and additional capacity provided, where necessary, to ensure that premature failure does not occur.

Where specific detailing requirements such as reinforcement anchorage or hook details are not covered in this section, compliance with the appropriate requirements of NZS 3101[(2)] is required.

e. Potential plastic hinge zones

At potential plastic hinge locations, the zone of the plastic hinge, for the purpose of detailing the confining reinforcement, shall be taken to be the ductile detailing length as defined by NZS 3101[(2)] clause 10.4.5. (The ductile detailing length should not be confused with the plastic hinge length to be applied to the calculation of plastic hinge curvatures and displacements, which is specified in 5.4.5.)

f. Longitudinal reinforcement

In reinforced and prestressed concrete compression members the cross-sectional area of the longitudinal reinforcement shall be not less than $4\frac{A_g}{f_{yl}}$ and not be greater than $18\frac{A_g}{f_{yl}}$, except that in the region of lap splices the total area shall not exceed $24\frac{A_g}{f_{yl}}$, where A_g is area of the gross cross-section of the member and f_{yl} is the lower characteristic yield stress of longitudinal reinforcement.

Circular columns shall be provided with a minimum of 8 vertical bars for column diameters larger than 500mm, and a minimum of 6 vertical bars for smaller diameters.

Rectangular columns, shall be provided with a minimum of 8 vertical bars.

Groups of parallel longitudinal bars bundled to act as a unit shall have not more than 3 bars in any one bundle and shall be tied together in contact. This limitation also applies where bars are lapped.

g. Splicing and anchorage of longitudinal reinforcement

The splicing of longitudinal reinforcement shall conform with the requirements of NZS 3101[(2)] clauses 8.9.1.1 and 8.9.1.2. Welded splices shall comply with NZS 3101[(2)] clause 8.7.4. Mechanical coupling of reinforcement shall comply with clause 4.2.1(f) of this *Bridge manual*.

The anchorage of longitudinal reinforcement shall conform with the requirements of NZS 3101[(2)] clause 10.4.6.5.

5.6.4 continued

h. Lateral reinforcement

The lateral (confinement) reinforcement in potential plastic hinge zones shall restrain the longitudinal reinforcement against buckling, confine the core concrete in the event that cover spalling occurs, and ensure that shear (brittle) failure will not occur during the design seismic event.

i. Where spirals or circular hoops are used for the transverse reinforcement of plastic hinge zones, the volumetric ratio of the transverse reinforcement per unit length of member (ρ_s) shall provide adequate displacement capacity in accordance with 5.3.5, but shall not be less than ρ_s=0.005.

The pitch of spirals or circular hoops within potential plastic hinge zones shall be not greater than the smaller of:

$0.15D_c$ and (5-47)

$$0.7\left(3+6\left(\frac{f_u}{f_y}-1\right)\right)d_{bl} \quad (5\text{-}48)$$

Where:

D_c = diameter of column

f_u = ultimate stress of the longitudinal reinforcement

f_y = yield stress of the longitudinal reinforcement

d_{bl} = diameter of longitudinal reinforcement.

Values for the ratio f_u/f_y shall be determined as specified in 5.4.5.

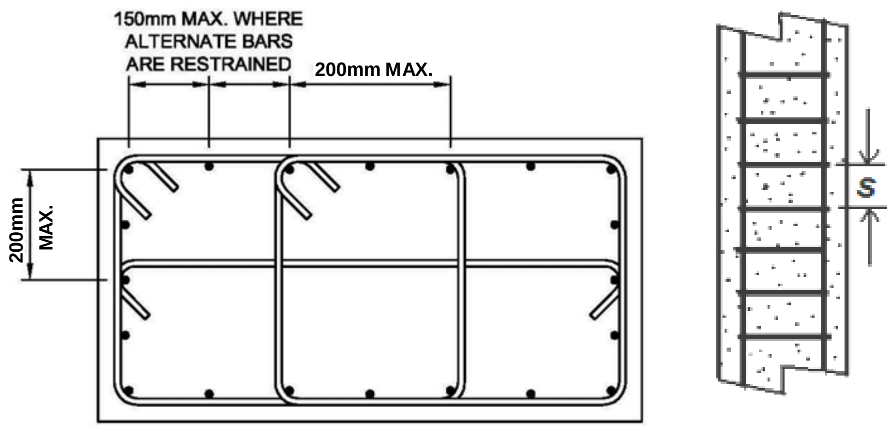
Where analysis indicates that the column will remain elastic under the DCLS design earthquake, the above limits may be relaxed to $0.25D_c$ and $8d_{bl}$.

ii. Where closed rectangular ties are used for the transverse reinforcement of plastic hinge zones (see figure 5.5), the reinforcement shall provide adequate displacement capacity in accordance with 5.3.5, but shall not be less than ρ_s=0.006.

In addition to satisfying NZS 3101[(2)] clause 10.4.7.6, the centre to centre distance of any unrestrained bar to a laterally restrained bar shall not exceed 150mm.

The spacing of the lateral confinement reinforcement shall not exceed the limits given in (i) above, where D_c in this case is the depth of the rectangular column in the direction considered.

Figure 5.5: Rectangular column tie spacings



5.6.4 continued

iii. Outside of potential plastic hinge zones, and for columns expected to remain elastic under the design earthquake, the spacing(s) of the lateral reinforcement shall not exceed the requirements of the relevant NZS 3101[(2)] clauses: 10.3.10.4.3, 10.3.10.5.2, 10.3.10.6.2, 10.4.7.4.5, 10.4.7.5.5.

i. Splicing and anchoring of lateral reinforcement

Splicing and anchorage of lateral reinforcement in plastic hinge zones shall comply with NZS 3101[(2)] section 8.7 for the splicing of lateral reinforcement, NZS 3101[(2)] clause 7.5.7.1 and 7.6.3.6 for the anchorage of stirrups and ties and with the following:

i. Splicing of helices in potential plastic hinge zones shall be avoided. Where helices need to be spliced, splice the helices by lapping the helices one turn and then anchoring each end of the helix bars with a 135° hook, engaging a longitudinal bar. Alternatively, splice each helix bar to the other with a single sided lap weld complying with AS/NZS 1554.3 *Structural steel welding* part 3 Welding of reinforcing steel[(23)].

   Spiral or circular hoop reinforcement shall be anchored using 135° hooks or by welding to itself. (Refer to Practice Advisory 8: *Don't be undone – anchor your spiral*[(24)].)

   Quenched and tempered grade 500E reinforcement shall not be used where spirals and hoop reinforcement is to be anchored or spliced by welding.

   Transverse reinforcement including stirrups, ties, spirals and hoops shall not be anchored by welding to longitudinal reinforcement.

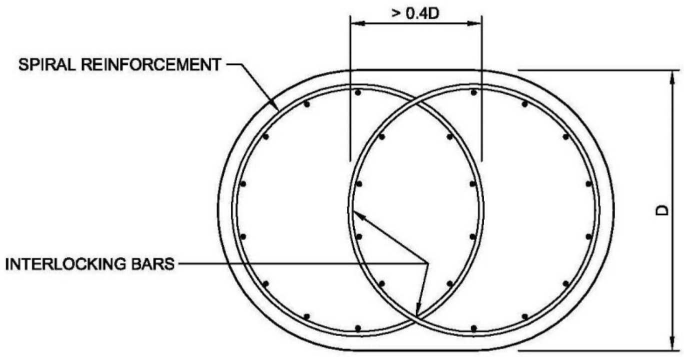
ii. Closed rectangular ties in accordance with 5.6.4(h)(ii) shall be used singly or in sets spaced vertically at not more than the value given by equation (5–48) or one-quarter of the minimum cross-section dimension, whichever is smaller.

   Supplementary ties, of the same diameter as the closed ties, consisting of a straight bar with a 135° minimum hook at each end, may be considered as part of a closed tie if they are spaced horizontally at not more than 350mm centres and secured with hooks on the closed tie to the longitudinal bars.

j. Additional provisions for rectangular / elliptical shaped piers

Where interlocking spirals are used, the overlap of the spirals should be at least 40% of the column diameter, as shown in figure 5.6.

Figure 5.6: Overlapping spiral reinforcement



k. Blade and wall type piers

Blade and wall type piers have a width to thickness ratio of 4 or greater.

The requirements of this sub-clause (k) apply to the design of blade and wall type piers for the strong direction.

The weak direction may be designed as a column in accordance with this sub-clause.

5.6.4 continued

Except in the end region (ie outer edges) of wall type piers the reinforcement ratios ρ_l longitudinally and ρ_s laterally shall not be less than 0.003 and ρ_l must not be less than ρ_s. In the end region, extending for twice the wall thickness from each end, but not less than 1.0m, the longitudinal reinforcement ratio shall be not less than 0.005.

Cross-ties shall be provided in wall type piers. In the end regions, defined above, cross ties shall comply with 5.6.4(h). Between end regions cross-ties shall be provided at spacing not exceeding twice the wall thickness both horizontally and vertically.

For blade or wall type columns, the centre-to-centre spacing between vertical bars shall not be greater than 450mm or 1.5 times the wall thickness, whichever is the lesser.

Outside of potential plastic hinge zones, and for columns expected to remain elastic under the design earthquake, the spacing(s) of the lateral reinforcement shall not exceed the requirements of the relevant NZS 3101(2) clauses: 10.3.10.4.3, 10.3.10.5.2, 10.3.10.6.2, 10.4.7.4.5 and 10.4.7.5.5.

A layer of effectively orthogonal reinforcement shall be provided on each face of the pier.

Lateral reinforcement shall be continuous and uniformly distributed.

Splices for the vertical and lateral reinforcement shall be staggered.

l. Hollow columns

The longitudinal reinforcement for hollow columns shall be not less than $0.01A_g$ and not be greater than $0.06A_g$, where A_g is area of the concrete in the cross-section of the column.

For hollow rectangular columns provide a layer of effectively orthogonal reinforcement on each internal and external face, with detailing in accordance with this clause.

For hollow circular columns, a single layer only of effectively orthogonal reinforcement is permitted.

5.6.5 Partially ductile structure

In a partially ductile structure under DCLS horizontal loading, a plastic mechanism forms in only part of the structure, so that after yield there is a significant upward slope in the force/displacement relationship. As illustrated in figure 5.3 there are two types of partially ductile structure:

- In a type I structure, this continues up to design displacement.
- In a type II structure, a complete mechanism will form after further displacement but the load at which this happens may not be predictable if it is due to hinging in piles.

Potential plastic hinges that form in piers at close to the DCLS design loading, and their resisting members, shall be designed as in 5.6.4. Members that resist forces from plastic hinges that form at greater than design loading shall be designed on the same basis.

The dependable shear strength of piles shall exceed the shear developed by a possible mechanism at overstrength. Sections at potential plastic hinges at depth down the pile shall be detailed to ensure that they can sustain the expected plastic rotations without significant damage.

5.6.6 Structure remaining elastic at design earthquake loading

An elastic structure remains elastic up to or above the DCLS design load based on the elastic spectrum (unreduced by ductility but reduced for damping greater than 5% due to foundation damping). Elastic structures may have little or no reserve ductility after reaching their load capacity, which, while undesirable, may be unavoidable.

5.6.6 continued

The pier and foundation member design forces shall be determined on the basis of an analysis as described in 5.3 and 5.4 or 5.5. If practicable or economically justifiable, damage during seismic overload should occur in accessible locations. The design strengths* of members below ground shall at least match the nominal flexural strengths† of members above ground.

All elastic structures shall be provided with either or both sufficient strength and sufficient ductility to be able to withstand a CALS earthquake event (ie an event corresponding to return period factor of $1.5R_u$) without collapse. For elements not designed on the basis of capacity design for the actions induced by yielding members mobilising their overstrength capacity, design shall be based on characteristic material strengths with capacity reduction factors applied. The flexural members of the substructure shall be detailed for ductility as required for a structure of limited ductility unless it can be demonstrated that plastic hinging is very unlikely. Columns and piles shall be provided with minimum confinement steel ratios (ρ_s) of not less than 0.005 and 0.006 for circular and rectangular sections respectively.

In the design of an elastically responding structure, no moment redistribution shall be applied other than that permitted by NZS 3101[(2)] section 6.3.7 to non-seismic loads.

5.6.7 Structure anchored by a friction slab, deadman anchors, soil reinforcement, abutment piles or base friction

a. Friction slabs, deadman anchors or soil reinforcement, piles (if provided) and friction on the base of abutments (provided they are not piled) may be assumed to provide seismic anchorage to a bridge abutment to resist both inertia loads from the abutment and superstructure only if the integrity of the embankment within which they are located can be relied upon under earthquake conditions (see 5.1.3 and 5.6.13(a)). The effect of seismic load transmitted by the friction slab, deadman anchors or soil reinforcement, piles, or friction on the base of an abutment to the embankment shall be taken into account in assessing the integrity of the embankment.

b. The horizontal restraint provided by a friction slab, deadman anchors or soil reinforcement, piles acting alone or in combination, and abutment (if not piled) base friction shall at least match the design force on the abutment specified in figure 6.4(a).

c. Allowance shall be made for seismic inertia forces arising from the weight of the friction slab and overlying soil, the weight of the deadman and passive soil wedge, or from the weight of the reinforced soil block, reducing the restraint provided to the structure.

d. The design value of horizontal restraint provided by a friction slab shall be calculated as the lesser of the design value of friction between the slab and the underlying bedding, and the design value of friction between the bedding and the underlying natural ground or fill. A strength reduction factor of ϕ=0.8 shall be applied in the determination of the design value of friction.

e. The assessment of the restraint provided by a friction slab shall take into account the extent to which the friction slab maintains contact with the underlying ground or fill in the event of settlement occurring. In general, the friction slab should be detailed in a manner to ensure that it maintains contact with the underlying ground throughout most of its length. Typically, where the abutment is supported on piles, this is usually achieved by detailing the friction slab to hinge at the rear of the abutment sill beam and again at a short distance away from the rear of the abutment sill beam. Friction slabs detailed in such a way would not provide the benefits of a settlement slab.

* Nominal strength: defined as the theoretical strength of a member section, calculated using the section dimensions as detailed and the lower characteristic strengths of the reinforcement and concrete.

† Design strength: defined as the nominal strength multiplied by the appropriate strength reduction factor.

5.6.7 continued

f. The design strength of the connection between the friction slab, deadman anchor or soil reinforcement and the abutment shall be at least 1.3 times the nominal sliding resistance of the friction slab, deadman anchor or soil reinforcement.

g. For multi-span bridges, abutment restraint by friction slabs and other forms of soil anchorage shall be assumed to provide no more than 30% of the total longitudinal and transverse lateral load resistance required. Piles at the abutments and the flexural strength of the piers shall be designed to carry the remaining part of the lateral load.

5.6.8 Structure 'locked in' to the ground longitudinally

Only single span structures, or multi-span structures up to 35m in length, with integral or semi-integral abutments, as described in 4.8, may be treated as structures 'locked-in' to the ground longitudinally. These structures rely on the integrity of the abutment approach material for seismic resistance. Refer to 5.6.13 for the specification of constraints to the restraint that can be assumed to be provided by abutments.

These structures are assumed to move with the ground displacement and for design purposes are assumed to be subjected to ground acceleration without amplification. Longitudinally there are two common cases as outlined in (a) and (b) below.

The forces acting on the locked-in structure, that are to be designed for, are illustrated in figure 5.7. The peak horizontal ground acceleration coefficient (C_o) to be used in computing the seismic inertia force shall be not less than as follows:

$$C_o = C_h(T_0)ZR_u \qquad (5\text{–}49)$$

Where:

$C_h(T_0)$	=	spectral shape factor at T=0 applicable to modal response spectrum and numerical integration time history analysis from NZS 1170.5[1] clause 3.1.2
Z	=	hazard factor from 5.2.2(a) and NZS 1170.5[1] clause 3.1.4
R_u	=	DCLS return period factor from 5.2.2(b).

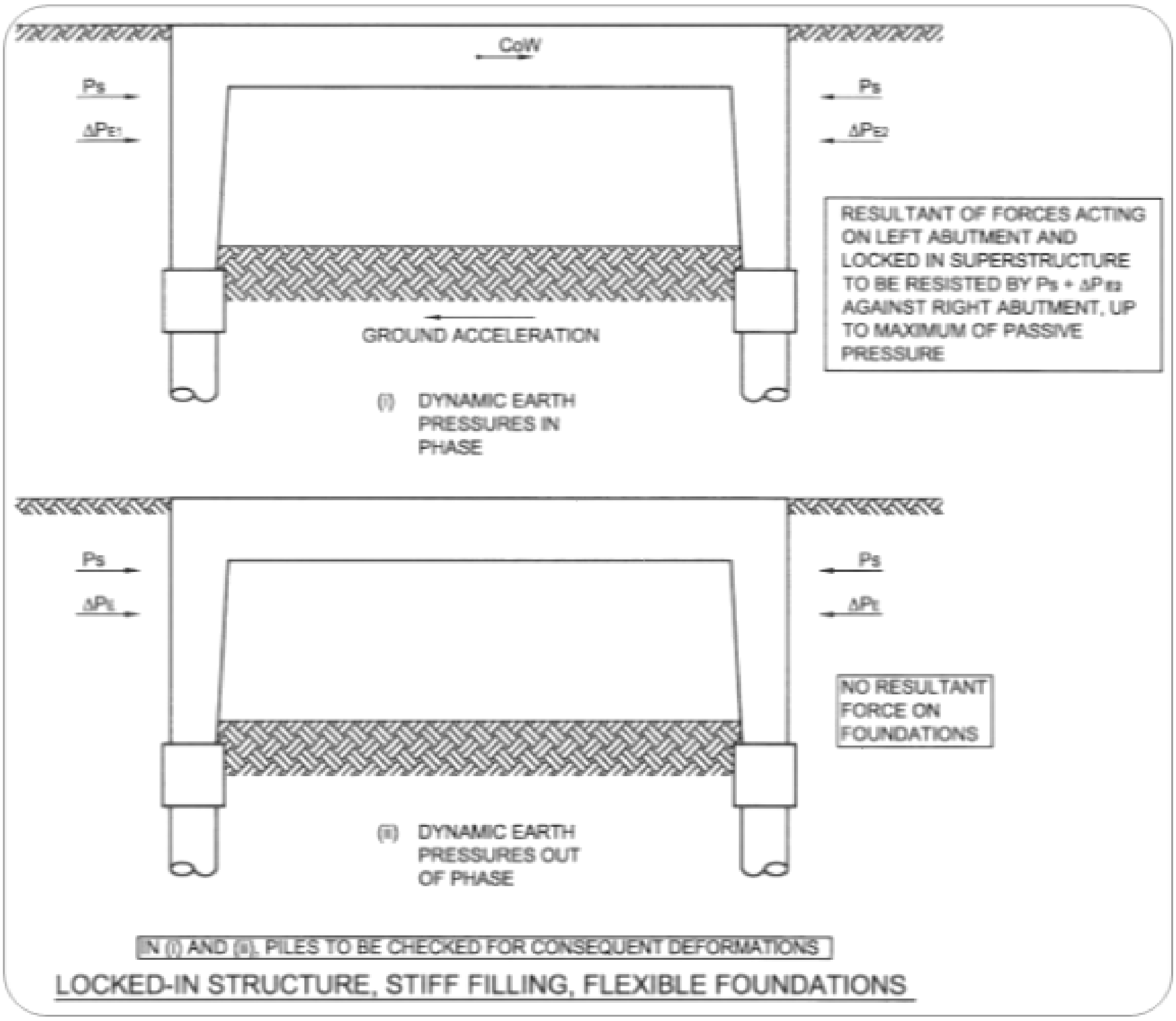
Resistance to longitudinal seismic loads shall be provided by pressure of soil against each abutment alternately. Earth pressure shall be determined as in 6.2.4, but to allow for possible seismic overload, greater pressure shall be allowed for, up to a maximum equivalent to passive pressure based on upper bound soil strength estimates without the application of reduction factors in the design of structural elements such as abutment backwalls. Conservative values of soil strength parameters shall be adopted in assessing the soil passive resistance and the frictional resistance to the structure sliding, and a strength reduction of ϕ=0.8 shall be applied to both forms of resistance.

Forces in the foundations due to consequent soil deformation shall be determined by an appropriate analysis, including the effects of soil stiffness. Such a structure shall not be assumed to be locked-in for transverse earthquake, unless a specific resisting system is designed.

a. Single span with conventional integral or semi-integral abutments on either piles or spread footings

   For this case the resistance shall be provided by piles, passive resistance, or friction from friction slabs (and footings if not piled), or a combination of these resisting components. The passive resistance shall be reduced by the 0.7 PGA acting on the passive wedge. The active earthquake pressure component shall be assumed to act on the abutment moving away from the soil embankment. Where the abutments are not supported on piles and sliding displacements of up to 50mm are considered acceptable the response acceleration assumed to act on the superstructure and abutment structure in either the longitudinal or transverse direction may be taken as 0.7 × PGA.

Figure 5.7: Seismic force combinations acting on a locked-in structure



5.6.8 continued

b. Portal frame structure with abutment walls higher than 3.0m

For this case the resistance shall be provided by passive resistance or piles or a combination of both. Potential plastic hinge areas in the walls and piles shall be detailed to meet the requirements for a structure of limited ductility (as defined in 5.6.4(a)).

The earthquake component of soil pressure acting on the walls shall be assumed to be in-phase increasing the static at-rest pressure on one wall and reducing the static at-rest pressure on the other wall. In lieu of more detailed analysis, the earthquake pressure component coefficient may be taken as 0.5 × PGA with the pressure assumed to be uniform over the height of the wall (stiff wall assumed coefficient of 0.75 reduced by a factor of 0.7 to allowing for damping and wave scattering). The response acceleration acting on the structure (walls and superstructure) shall be taken as the PGA.

As an alternative to the above analysis procedure the structure may be assumed to be subjected to soil shear-strain racking where the shear strain is computed from the free-field shear strain over the height of the structure under the DCLS event and the relative shear stiffness between the soil and structure. The response acceleration on the structure shall be taken as the PGA. Details of this method are described in *Earthquake design of rectangular underground structures*(25).

5.6.9 Multiple span bridges restrained longitudinally by ground passive resistance

This is a structure with integral or semi-integral abutments and without movement joints in the superstructure which relies substantially on the integrity of the abutment approach material for longitudinal seismic resistance although the piers may provide a proportion of the longitudinal restraint. Effectively, due to the spring stiffness of the restraining ground behind the abutments, this structure will exhibit response to an amplified acceleration, greater than the peak ground acceleration, at a period greater than T=0. But also due to the soil-structure interaction, increased damping of the response may arise.

For multi-span bridges the period of vibration shall be assessed as the basis for deriving the design seismic response spectrum. Increased damping may be taken into account to reduce the design elastic response spectrum. Passive and sliding resistance developed at abutment walls and footings may be considered to provide up to 30% of the total longitudinal and transverse resistance provided the stability of the abutment embankments and backfills is assessed and found to be satisfactory at the DCLS. A strength reduction factor of ϕ=0.8 shall be applied to the estimated soil passive and sliding resistance and the remaining resistance shall be provided by the piers and abutment piles (if any).

5.6.10 Structure on pile/cylinder foundations

a. When estimating foundation stiffness to determine the natural period(s) of vibration of the structure and the curvature ductility demand on plastic hinges, a range of soil stiffness parameters typical for the site shall be considered. Allowance shall be made for:
   - residual scour, which shall be taken to comprise thalweg plus general scour under mean daily flow conditions. Allowance shall also be made for any long term degradation that is occurring of the river
   - pile/soil separation in cohesive soils to a depth of two times pile diameter
   - liquefaction of soil layers and the potential for soil stiffness and strength degradation under repeated cyclic loading associated with earthquakes
   - the non-linear stress-strain properties of the resisting ground
   - the contribution of permanent steel casing to the stiffness of the pile, taking into account the extent of bond development between the casing and the concrete core, and section loss due to corrosion.

b. The design of pile foundations shall take account of:
   - pile group action
   - strength of the foundation as governed by the strength of the soil in which the piles are embedded
   - the effect of liquefaction-induced lateral spreading of the ground
   - additional loads on piles such as negative skin friction (down-drag) due to subsidence induced by liquefaction or settlement of the ground under adjacent loads (such as the approach embankment).

   The horizontal support provided to piles by liquefied soil layers and overlying non-liquefied layers shall be assessed using appropriate current methods for determining liquefied or post-liquefied soil strength and stiffness. Alternatively, for liquefied soil layers their horizontal support to piles may be conservatively ignored.

5.6.10 continued

c. The required strength of the piles, pile caps and the connection between these elements to resist the loads induced by seismic action shall be in accordance with the criteria above as appropriate. In addition:

- the design tensile strength of the connection between a pile and the pile cap shall not be less than 30% of the tensile strength of the pile based on probable material strengths. In determining the tensile strength of the pile, the strength of the casing shall be excluded where it is not effectively anchored to the pile cap.
- the region of reinforced concrete piles extending for the larger of the ductile detailing length defined by clause 10.4.5 of NZS 3101.1[2], twice the pile dimension, or 500mm from the underside of the pile cap shall be reinforced for confinement as a plastic hinge. (The pile dimension shall be taken as the diameter of a circle of area equivalent to the pile cross-sectional area.)

d. In the region of a steel pile casing immediately below the pile cap, the contribution of the casing (after deducting corrosion losses) may be included with respect to shear and confinement but shall be neglected in determining moment strength unless adequate anchorage of the casing into the pile cap is provided.

Where plastic hinging may occur in piles at the soffit of the pile cap the casing shall be terminated at least 50mm but not greater than 100mm below the pile cap soffit and any associated blinding concrete. This is to prevent the casing acting as compression reinforcement, which can cause buckling of the casing and enhancement of the pile strength by an indeterminate amount affecting the capacity design of the structure. The plastic hinge length in this situation, arising from strain penetration both up into the pile cap and down into the pile, shall be taken to be:

$$L_p = g + 0.044 f_y d_b \qquad (5\text{–}50)$$

Where:

L_p = plastic hinge length

g = the gap between the pile cap soffit and the top of the casing (mm)

f_y = yield strength of the pile flexural reinforcement (MPa)

d_b = diameter of the pile flexural reinforcement bars (mm).

The reduction in curvature ductility and displacement capability resulting from this limited plastic hinge length shall be taken into account in design.

e. Piles may develop unintended potential plastic hinge positions at the top of the piles and at locations down the pile where there is an abrupt change in soil stiffness. Where plastic hinging may occur at depth in the ground, adequate confinement of the plastic hinge zone shall be provided for a distance of at least three times the pile diameter either side of the level of maximum moment, taking into account the possible variability of this level due to such factors as the variability in the soil stiffness, variability in the depth of scour, and liquefaction of soil layers. Allow also for migration of the location of maximum moment upwards towards the ground surface as plasticity in the pile develops with the final depth being approximately 70% of that predicted by elastic analysis. (Refer *Seismic design and retrofit of bridges*[5] section 5.3.2(b).)

Alternatively, the plastic hinge length of 5.4.5 may be applied provided composite action in the casing is prevented over the potential plastic hinge length, eg by coating the inside of the casing with a bitumen debonding paint.

5.6.10 continued

f. When a permanent steel pile casing is used, even if it is intended to be non-structural, the effect of it acting compositely shall be considered and the steel shall meet the material requirements of AS/NZS 5100.6[22] section 2 or appendix H, or the withdrawn NZS 3404.1:2009 *Steel structures standard*[27] section 2 and all welds shall be full strength butt welds.

g. Analyses of the effect of seismic loading on groups of raked piles shall take account of the simultaneously induced axial forces and flexure in the piles and rotation of the pile cap due to lateral displacements.

5.6.11 Structure on spread footing foundations

The soil stress induced by load combination 5A shall not exceed the product of the nominal bearing capacity of the foundation and the appropriate strength reduction factor derived in accordance with 6.5.3. The foundations shall be considered under the combined static and earthquake loads.

5.6.12 Structure with rocking piers or on rocking foundations

Structures incorporating rocking of substructure elements include the following:

- structures in which spread footings rock on the supporting soils or rock
- structures in which the supporting columns rock on their structural foundations (spread footing or tops of foundation cylinders)
- structures in which the supporting columns rock on their structural foundations (spread footing or tops of foundation cylinders) and which incorporate mechanical energy dissipation devices to dampen seismic response.

Structures in which spread footings rock on the supporting soil or rock, and structures in which supporting columns rock on their structural foundations and without the incorporation of mechanical energy dissipating devices, are special cases of a ductile structure that remains elastic, in which spread footing foundation or column rocking is associated with rotation about one edge of the foundation or column base transferring to rotation about the other edge in a clear stepping action and the deformation of the soil and impact effects provide energy dissipation and increased damping. In using force-based design, for this structural system a value of μ=1.0 shall be adopted and the increased damping may be taken into account.

Where mechanical energy dissipation devices are incorporated the structure will respond inelastically. The mechanical energy dissipation devices are usually positioned distributed around the base of the supporting columns that rock on their structural foundations and are activated by the rocking motion of the columns. A restoring force to promote the column to return to its initial vertical position may also be provided by an unbonded prestressed cable positioned down through the centre of the column and anchored into the structural foundation.

a. Piers founded on spread footings may be expected to rock when the proportions of the spread footings are insufficient to withstand the overstrength moment capacity of a plastic hinge forming in the pier. If pier spread footings are expected to rock under design DCLS earthquake conditions to the extent that complete decompression of the bearing pressure and loss of ground contact beneath an edge of the spread footing occurs, the structure's behaviour shall be studied, preferably by performing a time history dynamic analysis in accordance with 5.3.11(d). The stiffness properties of the soil or rock shall be considered in the analysis.

5.6.12 continued

As an alternative to the dynamic analysis, a simplified analysis based on equilibrium consideration, as described in appendix A of the AASHTO *Guide specifications for LRFD seismic bridge design*[20], may be carried out. Where this simplified method is adopted for design, geotechnical capacities of the foundations, including assessment of potential settlement, shall be assessed to ensure that undesirable systems do not jeopardize the resistance or stability of the bridge system. Overturning shall be prevented under a CALS event. (The simplified method is also outlined in *Seismic design and retrofit of bridges*[5] section 6.4.2.)

b. For piers founded on rocking spread footings, the footing and pier stem shall be designed based on capacity design principles, to ensure that any yielding occurs in the pier stem. Capacity design requirements will be satisfied if the overstrength flexural capacity of the pier hinge is matched by at least its own nominal shear strength, the design moment and shear capacity of the footing and the bearing capacity of the foundation.

   The potential plastic hinge region at the base of the pier stem shall be detailed to ensure that it can sustain the possible limited rotation.

c. The interaction of the structure and foundation during rocking shall be carefully considered in the assessment of a rocking foundation, and the potential for foundation strength and stiffness degradation shall be taken into account.

d. Structures supported on columns that rock on their structural foundations, whether or not they incorporate mechanical energy dissipation, shall have the structure's behaviour studied as specified in (a). Structures incorporating mechanical energy dissipation shall also satisfy the requirements of 5.6.14. Whether or not mechanical energy dissipation devices are incorporated, consideration should also be given to the need or desirability of providing a restoring force that will act to reduce displacement of the structure and promote the displaced column to return to its original vertical position.

e. For all types of structure incorporating the rocking of substructure elements, an assessment shall be made of the performance of both the structural and non-structural components of the bridge as a consequence of the vertical and horizontal movements associated with the rocking motion of the piers, to ensure that structural integrity will be maintained under both DCLS design, and more extreme CALS earthquake conditions. The structure should be proportioned to ensure that displacements under CALS conditions are not sufficient to precipitate instability.

f. Structures founded on piles shall have their foundations proportioned such that rocking through the capacity of piles in tension being exceeded does not occur under the design DCLS intensity of earthquake shaking.

5.6.13 Constraints on the restraint assumed to be provided by abutments

In the assessment of restraint provided by abutments to a bridge, the following constraints shall apply:

a. The embankments may only be relied on to provide restraint provided they will maintain their stability and capability to provide restraint at the DCLS event and that avoidance of collapse of the structure in a CALS event is assured (see 5.1.3).

b. For abutments supported on piles, frictional restraint from the soffit of the abutment bearing against the underlying soil shall not be assumed due to the likelihood of embankment settlement occurring during a major seismic event.

5.6.12 continued

c. Under transverse response, frictional restraint from the embankment backfill acting against the rear face of the abutment or a vertical plane at the rear edge of the settlement slab shall not be assumed due to the likelihood of this restraint being diminished by possible gapping caused by the longitudinal response.

d. Under transverse response, abutment wingwalls bearing against backfill overlying a settlement slab do not mobilise restraint from this backfill as it responds with the abutment.

5.6.14 Structure with energy dissipating devices

A structure incorporating energy dissipating devices shall be designed in a similar manner to a ductile structure, as in 5.6.4. The energy dissipating devices shall be treated similarly to plastic hinges, and members resisting the forces induced in them designed using capacity design principles.

Energy dissipating devices shall have had their performance substantiated by tests. Their long-term functioning shall be assured by protection from corrosion and from water or debris build-up. The devices shall be accessible for regular inspection and maintenance, and to enable them to be removed and replaced if necessary.

Design guidance is provided by the AASHTO *Guide specifications for seismic isolation design*(28) and is also contained in Road Research Unit bulletin 84, volume 3 *Seismic design of base isolated bridges incorporating mechanical energy dissipators*(29).

Base isolation and energy dissipation devices shall maintain their integrity and functionality under earthquake events up to the magnitude of the CALS event.

5.6.15 Provision for foundation settlements

Where foundation settlements due to DCLS earthquake response and any associated liquefaction and/or ground movement of greater than 25mm are predicted, provision shall be made in the bridge detailing for jacking and re-levelling of the bridge superstructure to achieve the original design levels. Major reconstruction of primary substructure elements shall not be required. After reinstatement, the design level actions to be catered for shall include the effects of any permanent seismic settlement of the foundations and any additional actions arising from the re-levelling.

5.7 Provision for relative displacements

5.7.1 Clearances

a. Structural clearances

At locations where relative movement between structural elements is designed to occur, sufficient clearance shall be provided between those elements and around such items as holding down bolts, to permit 2.0 times the calculated relative movement under design DCLS earthquake conditions to occur freely without inducing damage. Similarly, bearings shall be designed to accommodate this range of movement without spans unseating.

Where two components of earthquake movement may be out of phase, the earthquake component of the clearance provided may be based on the square root of the sum of the squares approach. Long-term shortening effects and one half of the temperature induced movement from the median temperature position shall be taken into account as implied by the load combinations in table 3.3.

On short skew bridges, consideration shall be given to increasing the clearance between spans and abutments by up to 25% to counter possible torsional movement of the span with respect to the substructure.

b. Deck joints

At temperature movement deck joints, clearances may be less than specified in (a), provided damage to structural elements due to the design DCLS earthquake is limited to sacrificial devices (knock-up or knock-off devices), which have intentional weakness that permits localised damage to occur in a predetermined manner.

In such circumstances the range of movement to be accommodated by the joint shall not be less than the calculated relative movement under the serviceability limit state design earthquake conditions corresponding to a return period factor of $R_u/4$, plus long-term shortening effects where applicable, and one half of the temperature induced movement from the median temperature position. Damage to deck joint seal elements due to the joint opening under this reduced earthquake movement is acceptable. Mechanical damage, however, is to be avoided under the joint both opening and closing under the DCLS movements (ie damage to the jaws retaining the seals, joint fixings or primary joint elements other than flexible glands).

c. Provision for extreme seismic movements

Where movements outside the range of conventional bearings or clearance provisions are expected, additional devices may be used to limit movements under earthquake loading only. These special devices, such as buffer bearings, shall be designed to be activated only by large displacements, or by high relative velocities. The influence of such devices on the distribution and magnitude of earthquake force in the bridge shall be fully evaluated and considered in the design of all structural elements.

d. Clearance between adjacent structures

The clearance between adjacent structures to be provided shall exceed the desired minimum clearance of the sum of 2.0 times the displacement under the DCLS event of each structure. Additional compensatory clearance shall be provided where there is the possibility of components of displacement arising due to soil lateral spreading, soil cyclic softening or other non-seismic loadings or effects that may reduce the clearance provided.

5.7.1 continued

Where agreed by the road controlling authority, the separation between the adjacent bridges may be reduced, but to not less than an absolute minimum separation of the square root of the sum of the squares of the DCLS deflection of each bridge. Justification for this reduction shall be presented in the structure design statement.

Where less than the desired minimum clearance is provided, measures shall be taken to prevent injury to persons on or below the bridge, caused by falling debris (eg barriers or downstands to barriers). In addition, elements providing restraint to the superstructures of the bridges (eg shear keys) shall be designed to withstand any increased forces they may sustain due to pounding between the structures.

5.7.2 Horizontal linkage systems

a. General

The security of all spans against loss of support during seismic movement shall be ensured. Outlined below, situations are described where a positive horizontal linkage system shall be provided, and other situations are described where, as an alternative to the provision of a linkage system, specific provision for large relative displacements may be provided.

Linkage may be either tight or loose as described in (b) and (c), according to whether relative longitudinal movement is intended.

Requirements for provision of linkage are as follows:

- Longitudinal linkage is required between all simply supported span ends and their piers, and between the two parts of the superstructure at a hinge in the longitudinal beam system. This requirement shall also apply to abutments supported on walls or MSE fills, or with batter slopes in front of them steeper than 1.5 horizontal: 1.0 vertical. Longitudinal linkage is not required at a spill-through abutment with batter slopes flatter than or equal to 1.5 horizontal : 1.0 vertical, provided that the overlap requirements of 5.7.2(d) are complied with.
- Longitudinal linkage is not required at a pier, for a superstructure with full moment continuity, provided the displacement of the reaction point would not cause local member distress.
- Transverse linkage is not required for any type of superstructure, other than multi-beam superstructures with the beams each supported on individual columns, provided that the transverse strength and stability of the span is sufficient to support an outer beam or truss if it should be displaced off the pier or abutment.
- In the case of multi-beam superstructures with the beams each supported on individual columns, transverse linkages shall be provided between the tops of the columns and the superstructure to prevent excessive relative transverse displacement between the columns and the superstructure and the overlap requirements of table 5.9 shall be satisfied.

Linkage elements shall be ductile, in order to ensure integrity under excessive relative movement. Acceptable means of linkage are linkage bars. Requirements for ductile linkage bars are given in appendix C. Elements anchoring linkage bars shall be capacity designed to withstand elastically the forces imposed on them by seismic response loading the linkage bars to their overstrength capacity.

Ductile shear keys (eg concrete infilled steel tubes) are also acceptable provided that they are designed to withstand elastically the force induced in them by plastic hinges or mechanical energy dissipating devices developing their overstrength capacity under seismic response. Bearings, other than fixed pot bearings designed on a capacity design basis, are not an acceptable means of linkage.

5.7.2 continued

Due to the nature of earthquake loads, horizontal restraints shall not rely on any component of friction, unless the surface across which frictional restraint is to be transferred is designed and constructed as a shear-friction concrete construction joint between different stages of construction with reinforcement crossing the interface. Otherwise, for assessment of the structure under any load combination which includes earthquake effects, the friction coefficient between any material types to be used when determining horizontal restraint shall be taken to be equal to zero. However, an upper-bound estimate of the coefficient of friction shall be assumed for determination of maximum feasible force transmitted by friction through material interfaces, when assessing demand on structural elements, such as piers, for capacity-demand conditions in accordance with 5.6.1.

b. Tight linkage

A tight linkage shall be used, where relative horizontal movement is not intended to occur under either service loads or seismic loading. In ductile structures and structures of limited ductility, the linkage system shall be designed to have a design strength not less than the force induced therein by capacity design actions arising under DCLS design seismic conditions. Nor shall the design strength be less than that prescribed below for loose linkage. The linkage system of structures responding elastically at the DCLS shall be provided with either or both sufficient strength and sufficient ductility to prevent span collapses in a CALS earthquake event. Where applicable, rubber pads shall be provided between the two elements of the bridge linked together in this fashion, to enable relative rotation to occur.

c. Loose linkage

At a position where relative horizontal movement between elements of the bridge is intended to occur under DCLS design earthquake conditions, the linkage shall be designed to be `loose', ie sufficient clearance shall be provided in the system so that it does not operate until the relative design seismic displacement plus long term shortening plus one half of the temperature induced movement from the median temperature position is exceeded. Loose linkage is intended to act as a second line of defence against span collapse in earthquakes more severe than the design event, up to the CALS event or in the event of pier top displacement resulting from excessive pier base rotation.

Toroidal rubber buffers as shown in appendix C shall be provided between the elements of the bridge which are loosely linked. The elements of loose linkage between a span and its support shall have a design strength not less than that required to resist a force equal to at least 0.4 times the dead load of the contributing length of superstructure. The contributing length of superstructure shall be not less than the total length of the spans being supported on the abutment or pier fitted with the linkage system.

d. Overlap requirements

Overlap dimensions are defined in figure 5.8. They apply in both longitudinal and transverse directions.

To minimise the risk of a span being displaced off either its bearings or the pier or abutment under earthquake conditions in excess of the design event, the bearing overlap at sliding or potentially sliding surfaces and the span/support overlap of not less than that given in table 5.9 and, at non-integral abutments, by equation (5–51), whichever is the greater, shall be provided.

5.7.2 continued

At non-integral bridge abutments at which linkages are not provided, bearing seats supporting expansion ends of the superstructure shall be designed to provide a minimum support overlap length, measured normal to the face of an abutment, of not less than that required by table 5.9 and also not less than L_{bs}, as expressed in equation (5-51).

$$L_{bs} = \Delta(3.0) + 0.0004L_d + 0.007h_d + 0.005W \geq 0.4\text{m} \qquad (5\text{-}51)$$

Where:

$\Delta(3.0)$ = the displacement at a period of 3 seconds for the design seismicity (5.2.4(b))

L_d = length of the superstructure to the next expansion joint

h_d = average height of the columns or piers supporting the superstructure length L_d

W = width of the seating transverse to the bridge axis.

Figure 5.8: Overlap definition

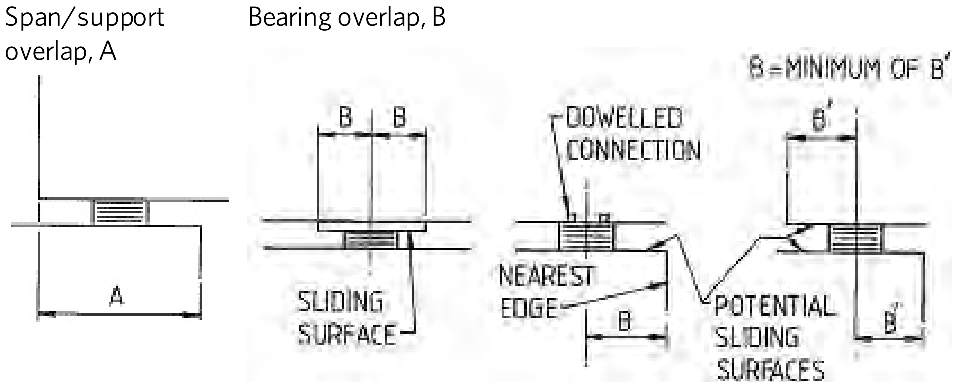


Table 5.9: Minimum overlap requirements

Linkage system	Span/Support overlap (A)	Bearing overlap (B)
No linkage system	2.0*E* + 100mm (400mm minimum)	1.25*E*
Loose linkage system	2.0*E*′ + 100mm (300mm minimum)	1.0*E*′
Tight linkage system	200mm	-

Where:

E = relative movement between span and support, from median temperature position at construction time, under DCLS design earthquake conditions, EQ+SG+TP/3

E′ = equivalent relative movement at which the loose linkage operates, ie *E*′ ≥ *E*.

EQ, SG and TP are displacements resulting from load conditions described in section 3 and combined as in table 3.3.

5.7.3 Holding down devices

See 2.1.7.

5.7.4 Effects of concurrent orthogonal movement

Provision shall be made for the effects on linkage and bearing assemblies of relative horizontal seismic movement between bridge elements occurring concurrently in the longitudinal and transverse directions.

5.8 Tsunami effects on coastal bridges

5.8.1 Introduction

The understanding of tsunami effects on coastal structures is in its infancy. The following outlines provisional requirements for the consideration of tsunami effects on coastal bridges and has been developed from research undertaken by the University of Auckland detailed in their reports *Outline for designing bridges that may be subjected to tsunami loads* stage 1 Literature review: New Zealand's exposure to tsunami hazard, bridge failure mechanisms and existing design guidelines(30) and stage 2 Draft requirements for the consideration of tsunami effects on bridges(31).

This is a topic that is the focus of ongoing research effort and it is expected that these requirements will be modified as the state of knowledge develops.

5.8.2 Consideration of design for tsunami effects

The need for a structure to be designed for the effects of tsunami, and the required design performance level, shall be agreed on a case by case basis with the road controlling authority. The need for and performance level of the design shall be based on:

- an assessment of the capacity of the structure to withstand tsunami effects when there has been no specific design for tsunami. As a minimum requirement, bridges potentially affected by tsunamis shall be designed to ensure that the superstructure is connected to the substructure in a manner that the horizontal and vertical (uplift) capacity of the substructure can be fully developed to resist the tsunami load effects on the superstructure.
- consideration of the incremental cost to increase the capacity of the structure from including no specific design for tsunamis to designing for tsunamis of increasingly lesser annual probability of exceedance down to the annual probability of exceedance of the design earthquake event.
- recognition that the level of tsunami design principally affects the extent of post-tsunami bridge damage rather than road user safety. Road users will generally be adversely affected, regardless of bridge performance.

5.8.3 Design events

The annual probability of exceedance for the full design tsunami event shall correspond to that for damage control limit state earthquake actions given in table 2.1.

The maximum tsunami height at the coastline shall be determined from figures 5.9(a) to 5.9(f)* as appropriate for the design annual probability of exceedance. For each colour band, the maximum tsunami height represented by the colour band shall be adopted (eg for the yellow band, an 8m high tsunami height shall be adopted). For zones colour coded black, a maximum tsunami height of 14m shall be assumed. The maximum tsunami height shall be assumed to be its height above mean sea level (ie its elevation). Future increases in mean sea level should be taken into account, as specified in 2.3.2(c).

(Note: These tsunami heights are relative to the sea level at the time of an event occurring but for the purpose of this consideration these tsunami heights shall be treated as being relative to mean sea level, as an average event.)

5.8.4 Tsunami overland maximum run-up elevation

Coastal bridges will commonly be waterway crossings discharging at bays in the coastline which are likely to have a focusing effect on the impact of the tsunami against the coastline. Taking this effect into account, the maximum elevation above mean sea level that the tsunami shall be assumed to run up to overland and up waterways shall be taken to be twice the maximum tsunami height at the coastline.

* Figures 5.9(a), 5.9(c) and 5.9(e) have been reproduced from *Review of tsunami hazard in New Zealand (2013 update)*(32) with the permission of the Ministry of Civil Defence and Emergency Management and GNS Science.

5.8.4 continued

All bridges sited on ground or in water with a surface elevation lower than this maximum run-up elevation shall be considered to be exposed to the effects of tsunami. In determining the maximum run-up elevation, allowance shall be made for the effects of climate change.

5.8.5 Inland tsunami flow velocity

Typically, tsunami waves break as they reach the coast and will run inland as a 'bore' (broken wave). The flow depth of this bore and its velocity will diminish as the elevation of the surface over which it is flowing increases and both will be zero at the maximum run-up elevation. The flow depth of the bore at the bridge location shall be assumed to be:

$$y_t = H_c\left(1 + \frac{x}{L}\right) - H_b \qquad (5\text{–}52)$$

Where:

y_t = tsunami flow depth at the bridge; ie height of the tsunami surface above the pre-tsunami water level or above the ground surface if the stream bed at the bridge is dry (m)

H_c = maximum tsunami height at the coast (from figure 5.9(a) to 5.9(f) as appropriate) (m)

x = distance of the bridge site from the coast (m)

L = distance from the coast at which the maximum run-up elevation is reached (m)

H_b = elevation of the ground surface or pre-tsunami water level at the bridge site (m)

The tsunami flow velocity of the bore at the bridge shall be assumed to be:

$$V_t = \sqrt{g y_t} \qquad (5\text{–}53)$$

Where:

g = gravitational acceleration (m/s²)

y_t = tsunami flow depth at the bridge as defined above (m).

5.8.6 Hydrodynamic forces acting on the bridge

Hydrodynamic forces acting on the structure shall be treated as an ultimate limit state load case using load factors for load combination 5B in table 3.3. The forces shall be determined from the equation:

$$F = C_d\left(0.5 \rho V_t^2 A\right) \qquad (5\text{–}54)$$

Where:

C_d = a coefficient to be taken as 4.5 for horizontal loading, and, for vertical loading, either 3.0 for vertically upward loading or the appropriate negative value from figure 15.4.3 in AS 5100.2 *Bridge design* part 2 Design loads[33] for vertically downward loading

ρ = the density of the flowing tsunami water, to be taken as 1.100 tonne/m³ unless sediment entrainment is unlikely (tonne/m³)

V_t = the tsunami horizontal flow velocity at the bridge (m/s)

A = the projected surface area of the bridge onto a vertical plane perpendicular to the flow in the case of the horizontal force applied to the bridge, or onto a horizontal plane in the case of the vertical uplift force or downward force applied to the bridge (m²).

5.8.6 continued	Horizontal and vertical loadings shall be treated as concurrent. Vertically upward loadings shall be treated as non-concurrent with vertically downward loadings. The eccentricity of loadings on the superstructure relative to reactions at the supports, inducing moments in the superstructure, shall be taken into account. Where inland flow of the tsunami may carry debris and lodge a debris raft against the bridge the size of debris raft to be allowed for shall be determined in accordance with 2.3.
5.8.7 Bridge scour	Scour effects on the bridge foundations shall be assessed based on 2.3 using the bore flow depth and velocity at the bridge.

Figure 5.9(a): Tsunami height (maximum amplitude) in metres at 50th percentile (2500 year return period)

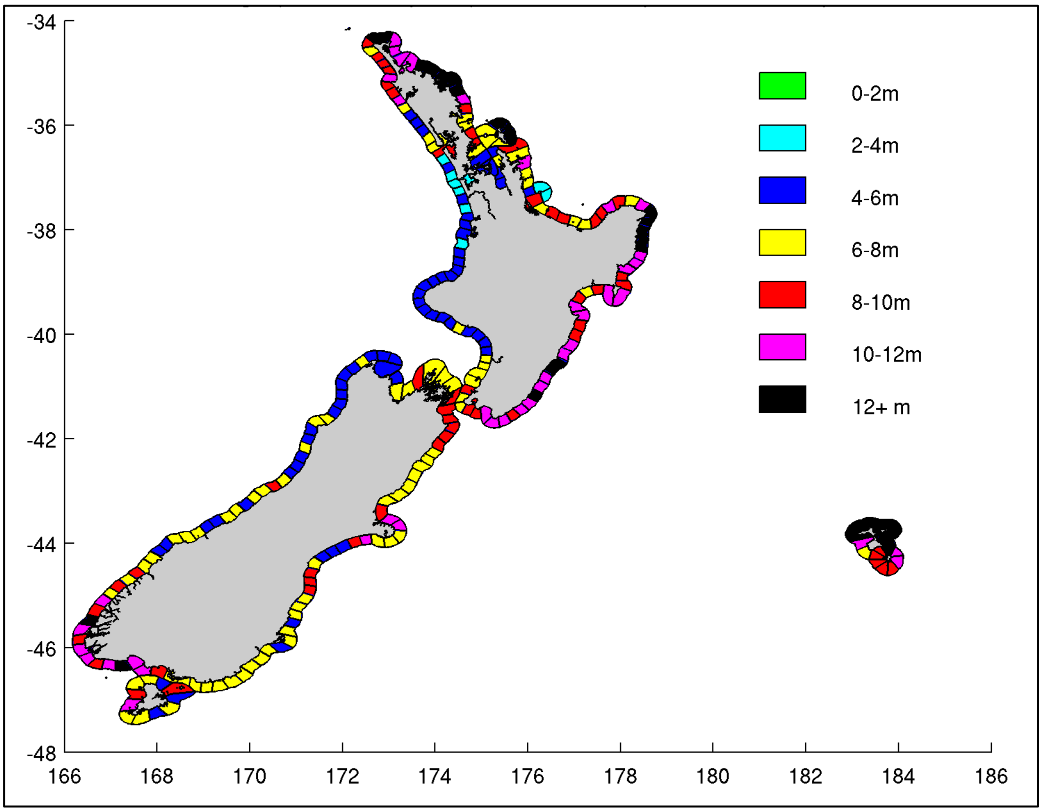


Figure 5.9(b): Tsunami height (maximum amplitude) in metres at 50th percentile (1000 year return period)

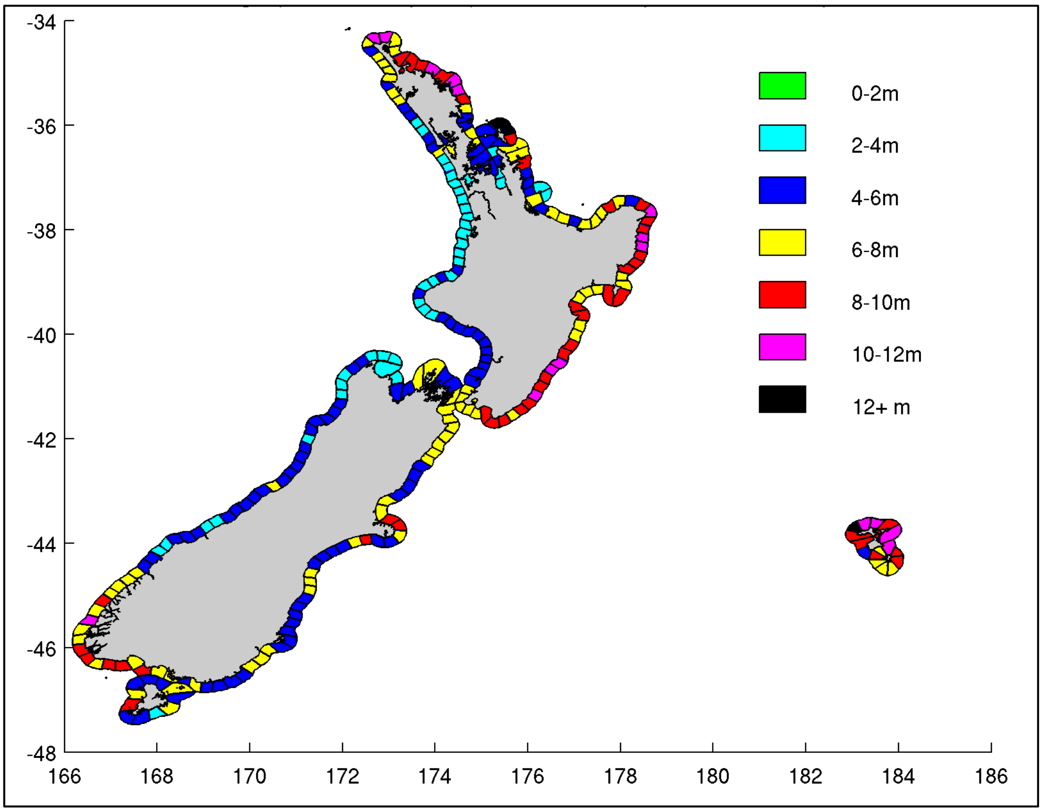


Figure 5.9(c): Tsunami height (maximum amplitude) in metres at 50th percentile (500 year return period)

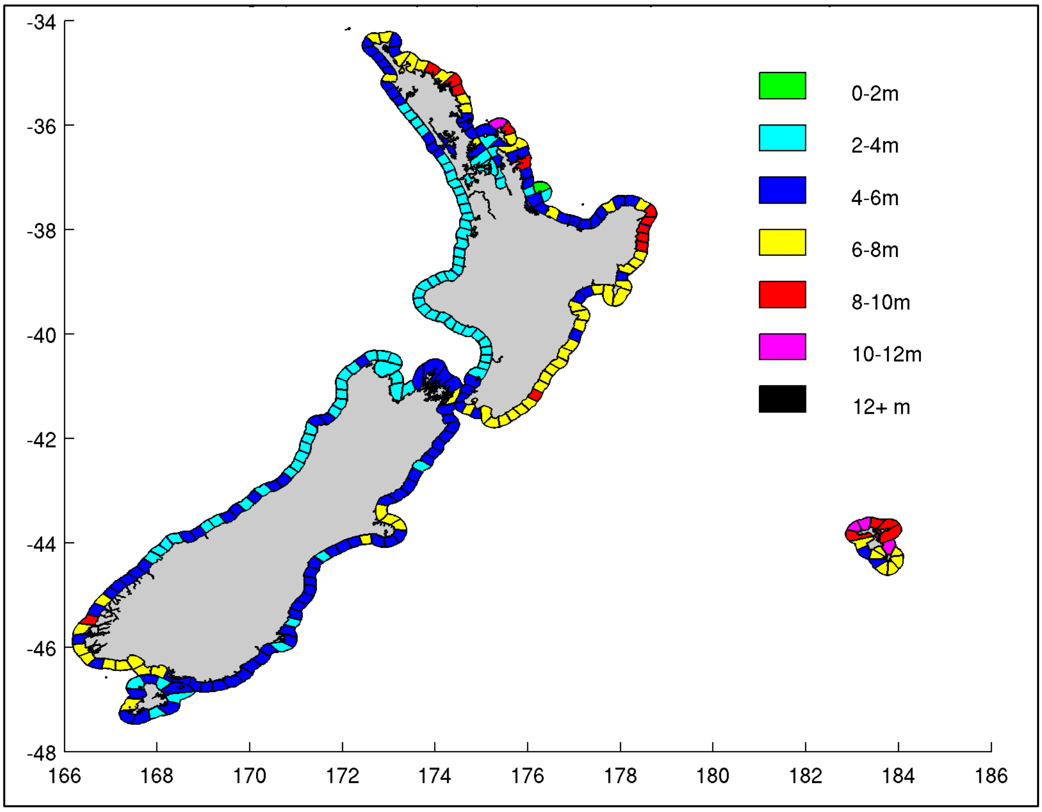


Figure 5.9(d): Tsunami height (maximum amplitude) in metres at 50th percentile (250 year return period)

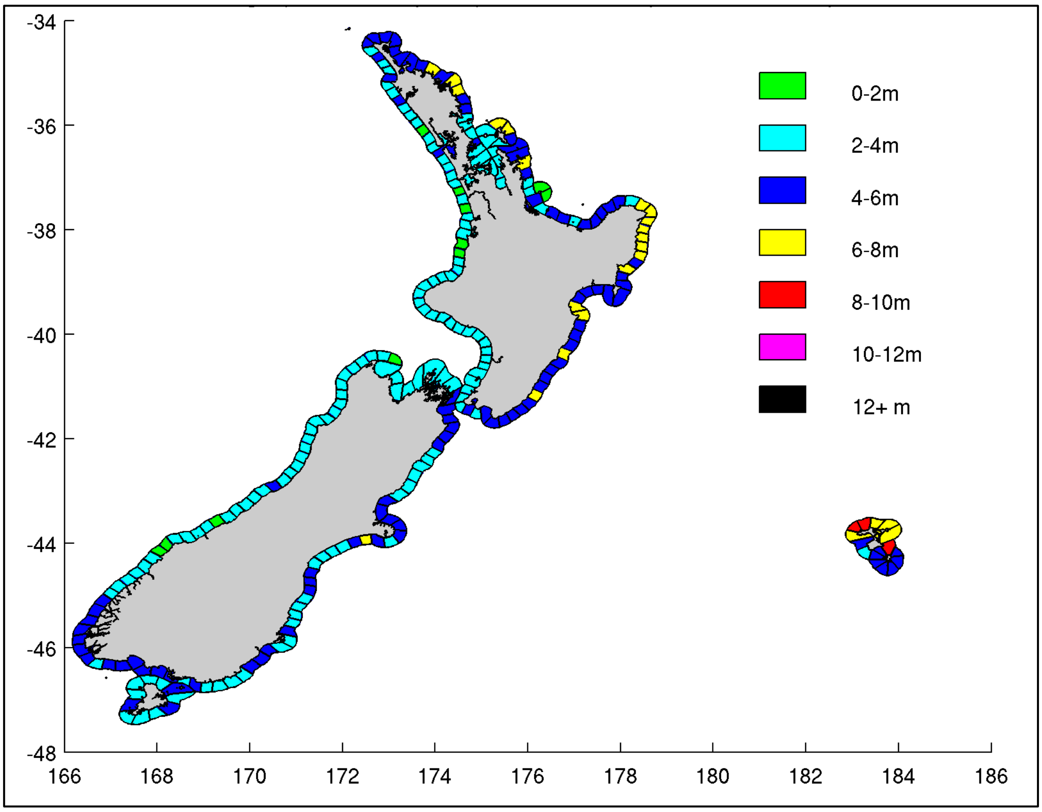


Figure 5.9(e): Tsunami height (maximum amplitude) in metres at 50th percentile (100 year return period)

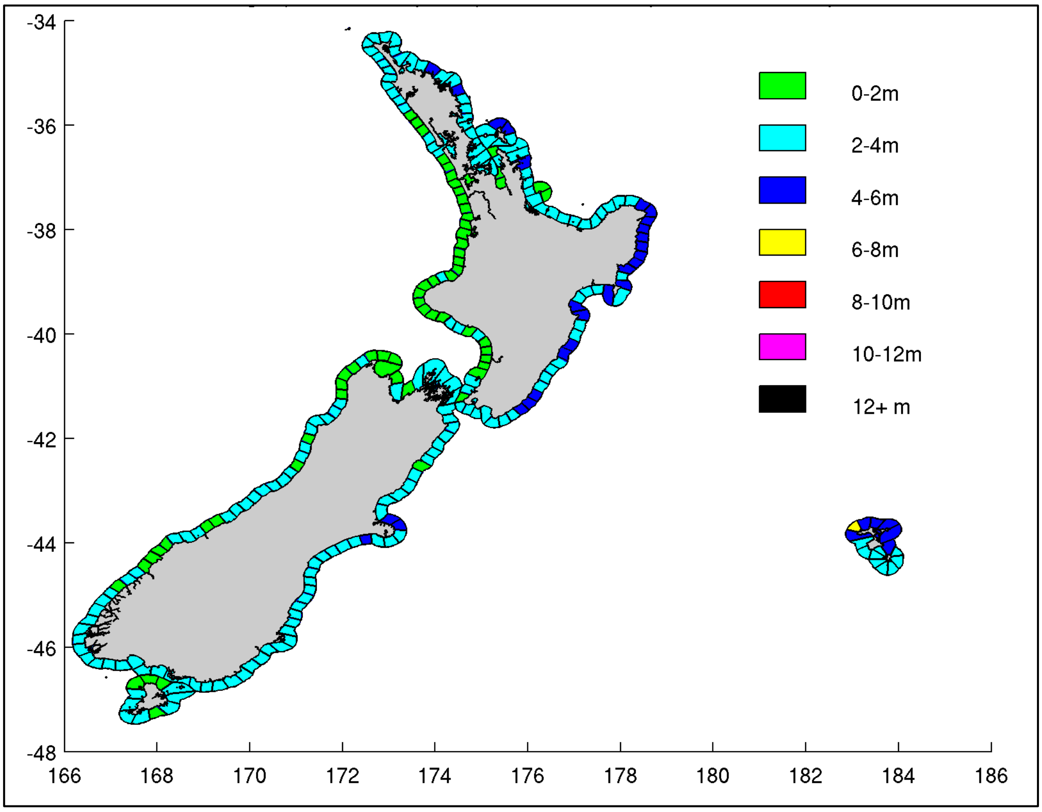
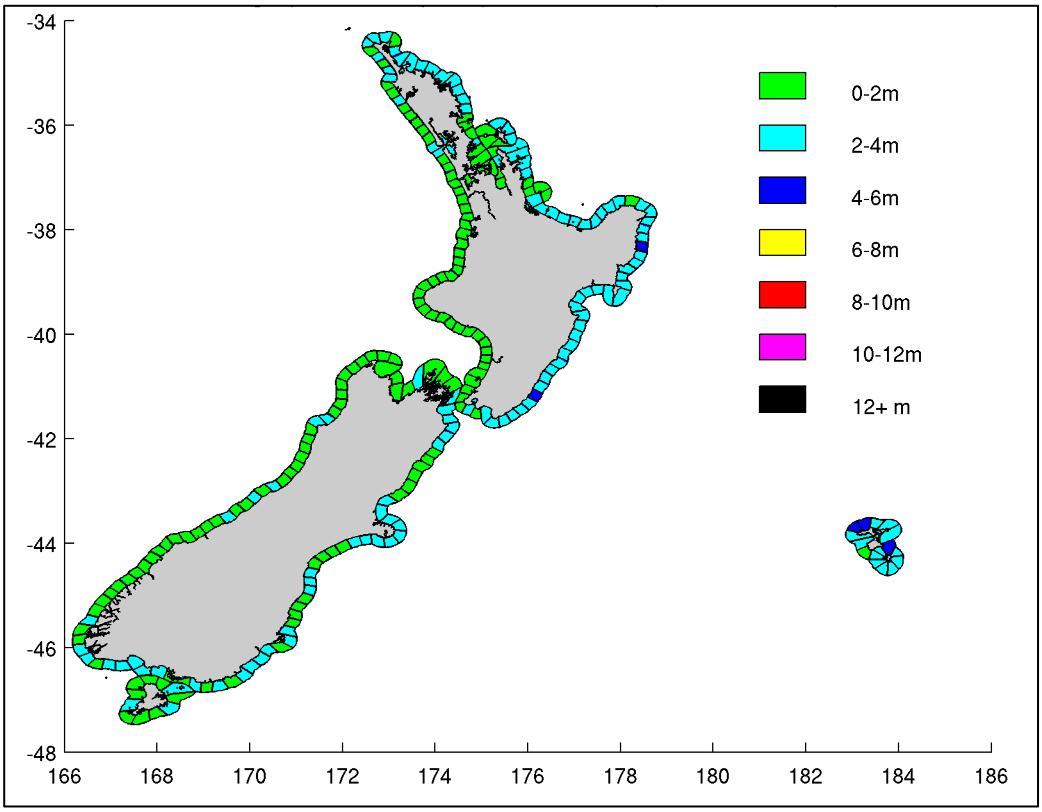


Figure 5.9(f): Tsunami height (maximum amplitude) in metres at 50th percentile (50 year return period)



5.9 Low-damage design

5.9.1 Introduction	Low-damage design is a generic term to describe structures which are highly resilient to large earthquake actions. These structures are characterised by their ability to rapidly return to service after a DCLS seismic event and experience only limited damage after a CALS event. This section of the *Bridge manual* will be further developed through future amendments
5.9.2 Application of low-damage design	It is desirable that all new bridges are low-damage structures. Low-damage design shall therefore be considered for all bridges categorised as importance level 3, 3+ and 4 structures. Low-damage options shall be presented in the structure options report*. Where low-damage design is not feasible, the reasons why it is not feasible shall be discussed in the report.
5.9.3 Forms of low-damage design	Recognised forms of low-damage design are: • integral or semi-integral structures that are 'locked-in' to the ground longitudinally, as defined in 5.6.8, and transversely are either 'locked-in' (with a specifically designed resisting system) or respond as specified in one of the categories below • structures that respond elastically up to CALS event

* Where the bridge is procured by the design and construct delivery model, or similar, the first opportunity the designer has to report on the low-damage design may be in subsequent reports, in which case the low-damage design option(s) shall be discussed in those subsequent reports.

5.9.3 continued

- structures that respond elastically up to DCLS events and within the material strain limits specified in 5.3.5 for limited ductile structures at CALS
- base isolated structures designed in accordance with 5.6.14
- rocking systems with supplementary damping and self-centring capability designed using the principles outlined in *PRESSS design handbook*(34)
- single span structures with fully integral abutments comprising shallow pad foundations that are designed to slide on their foundations. Sliding movement shall not exceed the values given in table 5.10.

Table 5.10: Maximum displacement of foundations designed to slide

Earthquake limit state	Maximum residual displacement
SLS ($R_u/4$)	5mm
DCLS (R_u)	300mm
CALS ($1.5R_u$)	450mm

5.9.4 Performance requirements

Key performance requirements of low-damage design are:

- the structure is essentially undamaged in a DCLS event; or
- damage in DCLS and CALS events is limited to pre-defined areas with other parts of the structure capacity protected
- damage shall only occur in readily accessible areas that can be rapidly repaired using a pre-defined method
- self-centring to at least an extent that does not compromise vertical load carrying capacity immediately following the CALS event.

5.9.5 Conflicting requirements

Where design requirements for low-damage design conflict with other sections of the *Bridge manual*, the design rules set out in documents cited in 5.9.3 shall take precedence. Conflicting requirements shall be identified in the structure design statement and shall be subject to approval from the road controlling authority.

5.10 References

(1) Standards New Zealand NZS 1170.5:2004 *Structural design actions*. Part 5 Earthquake actions – New Zealand. (Incorporating Amendment No. 1: 2016)

(2) Standards New Zealand NZS 3101.1&2:2006 *Concrete structures standard*. (Incorporating Amendment No. 3: 2017)

(3) Standards New Zealand NZS 3404 Parts 1 and 2:1997 *Steel structures standard*.

(4) Standards Australia and Standards New Zealand jointly AS/NZS 1170.0:2002 *Structural design actions*. Part 0 General principles. (Incorporating Amendment No. 5: 2011)

(5) Priestley MJN, Seible F and Calvi GM (1996) *Seismic design and retrofit of bridges*. John Wiley & Sons Inc., New York, NY, USA.

(6) Priestley MJN, Calvi GM and Kowalski MJ (2007) *Displacement-based seismic design of structures*. IUSS Press, Pavia, Italy.

(7) Standards New Zealand NZS 1170.5 Supplement 1:2004 *Structural design actions*. Part 5 Earthquake actions – New Zealand Commentary. (Incorporating Amendment No. 1: 2016)

(8) New Zealand Society for Earthquake Engineering (2019) *Guideline for the design of seismic isolation systems for buildings*. Wellington. (Draft for trial use.)

(9) Stirling M et al (2000) *Probabilistic seismic hazard assessment of New Zealand: New active fault data, seismicity data, attenuation relationships and methods*. Client Report 2000/53 prepared for the Earthquake Commission Research Foundation, Institute of Geological and Nuclear Science, Lower Hutt.

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(16) Priestley MJN, Park R and Heng NK (1979) *Influence of foundation on the seismic response of bridge piers*. Bulletin of the New Zealand Society for Earthquake Engineering, vol. 12, no. 1.

(17) Wood JH (2012) *Performance of linkage bars for restraint of bridge spans in earthquakes*. Report to New Zealand Transport Agency, Wellington.

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(19) Newmark NM (1967) *Problems in wave propagation in soil and rocks*. Proceedings of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, University of New Mexico Press.

(20) American Association of State Highway and Transportation Officials (2011) *Guide specifications for LRFD seismic bridge design*. 2nd edition. Washington DC, USA.

(21) Society of Structural Engineers *Gen-Col*. Last accessed 13 January 2022. <www.sesoc.org.nz/software/>. (Available to members only.)

(22) Standards Australia and Standards New Zealand jointly AS/NZS 5100.6:2017 *Bridge design.* Part 6 Steel and composite construction.

(23) Standards Australia and Standards New Zealand jointly AS/NZS 1554.3: 2014 *Structural steel welding*. Part 3 Welding of reinforcing steel. (Incorporating Amendment No. 1: 2017)

(24) Ministry of Business, Innovation & Employment (2006) Practice Advisory 8: *Don't be undone – anchor your spiral*. Wellington.

(25) Wood JH (2007) *Earthquake design of rectangular underground structures*. Bulletin of the New Zealand Society for Earthquake Engineering, vol. 40, no.1.

(26) Not used.

(27) Standards New Zealand NZS 3404.1:2009 *Steel structures standard*. Part 1 Materials, fabrication, and construction. Withdrawn.

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