DESIGN SPECIFICATIONS FOR HIGHWAY BRIDGES

PART V SEISMIC DESIGN

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Chapter 1  General

1.1 Scope

This Part shall apply to the seismic design of highway bridges.

1.2 Definition of Terms

The terms used in this part are defined as below:

(1) Seismic performance
    Performance of bridges subjected to the effects of earthquakes.

(2) Limit state
    Limit state of a whole bridge and each structural member capable of meeting the requirements of seismic performance.

(3) Liquefaction
    Phenomenon of destruction of a soil layer when a saturated sandy soil loses its shear strength due to a sudden rise of pore water pressure caused by earthquake ground motion.

(4) Liquefaction-induced ground flow
    Phenomenon of the ground moving laterally due to liquefaction.

(5) Ground type for seismic design
    Type of ground for engineering purpose, classified depending on its dynamic characteristics during an earthquake.

(6) Ground surface to be considered in seismic design
    Ground surface assumed for seismic design.

(7) Bedrock for seismic design
    Upper surface of a fully hard ground layer that exists over a wide area in the
construction site, and is normally situated below a surface soil layer shaking with a ground motion different from the bedrock motion during an earthquake.

(8) Seismic coefficient method
A verification method of seismic performances in which seismic forces are statically applied to a structure and ground, with use of the seismic coefficients taking into account the seismic actions caused by an earthquake.

(9) Ductility design method
A verification method of seismic performances in which seismic forces are statically applied to a structure and ground, considering the lateral capacity, ductility, and energy absorption in the nonlinear range of the structure.

(10) Static verification method
Method verifying the seismic performances based on static analysis.

(11) Dynamic verification method
Method verifying the seismic performances based on dynamic analysis.

(12) Design vibration unit
A structural system that can be regarded as a single vibration unit during an earthquake.

(13) Plasticity
A phenomenon of a structural member in which the member deforms beyond its elastic limit when subjected to the seismic forces.

(14) Seismic lateral strength
Lateral strength of a structural member when subjected to repeated seismic forces.

(15) Ductility
Performance of a structural member capable of sufficiently deforming in the plastic ranges, while keeping its lateral strength unchanged when subjected to repeated seismic forces.
(16) Plastic hinge
A specific portion of an Reinforced Concrete structural member capable of performing ductile behavior when subjected to repeated alternate deformations. In the calculation of ultimate horizontal displacement, a length of an estimated plastic hinge in the axial direction of the member is defined as the plastic hinge length, and sections in the plastic hinge length is defined as the plastic hinge zone.

(17) Horizontal inertia force distributed structure
Structure supporting a superstructure with a plural number of substructures, for the purpose of distributing the seismic inertia forces to these substructures. The structure includes a bridge with elastic supports such as rubber bearings and isolation bearings, and a bridge with plural fixed supports.

(18) Seismically-Isolated bridges
Bridge with isolation bearings intended to make natural period of the bridge longer and to increase the damping characteristics to decrease the inertia forces during an earthquake.

(19) Unseating prevention system
Structural system having a seat length, an unseating prevention structure, an excessive displacement stopper, and a structure for protecting the superstructure from subsidence, in order to prevent a superstructure from unseating due to an earthquake.
2.1 Fundamentals of Seismic Design

(1) In the seismic design of a bridge, the seismic performance required depending on the levels of design earthquake ground motion and the importance of the bridge shall be ensured.

(2) In the seismic design, topographical, geological and soil conditions, site conditions, etc. shall be taken into account. A structural type with high seismic performance shall be selected. Necessary seismic performance shall be secured in the design of individual structural members of the bridge and the entire bridge system.

2.2 Principles of Seismic Design

(1) Two levels of design earthquake ground motions shall be considered in the seismic design of a bridge. The first level corresponds to an earthquake with high probability of occurrence during the bridge service life (called “Level 1 Earthquake Ground Motion” hereafter), and the second level corresponds to an earthquake with less probability of occurrence during the bridge service life but strong enough to cause critical damage (called “Level 2 Earthquake Ground Motion”). For the Level 2 Earthquake Ground Motion, two types of earthquake ground motion shall be taken into account, namely, Type I of an interplate earthquake with a large magnitude and Type II of an inland near-field type earthquake.

(2) Depending on the importance factors such as road class, bridge functions and structural characteristics, bridges shall be classified into two groups: bridges of standard importance (Class A), and bridges of high importance (Class B). The definitions of the importance classification are specified in Section 2.3.

(3) Seismic performances of bridges shall have the following three levels, in view of the seismic behavior of bridges:

1) Seismic Performance Level 1
   Performance level of a bridge keeping its sound functions during an earthquake.

2) Seismic Performance Level 2
   Performance level of a bridge sustaining limited damages during an earthquake and capable of recovery within a short period.
3) Seismic Performance Level 3

Performance level of a bridge sustaining no critical damage during an earthquake.

4) Depending on the levels of design earthquake ground motions and the importance of bridges, the seismic design of bridges shall conform to the following.

1) Both Class A and Class B bridges shall be designed so that the Seismic Performance Level 1 is ensured when subjected to the Level 1 Earthquake Ground Motion.

2) Class A bridges shall be designed so that the Seismic Performance Level 3 is ensured when subjected to the Level 2 Earthquake Ground Motion, while Class B bridges should be designed so that the Seismic Performance Level 2 is ensured when subjected to the Level 2 Earthquake Ground Motion.

5) Bridges shall be designed so that unseating of superstructures can be prevented, even though structural failures may occur due to structural behavior or ground failure unexpected in the seismic design.

### 2.3 Classification of Importance of Bridges

The importance for both Class A bridges and Class B bridges shall be classified as defined in Table-2.3.1.

<table>
<thead>
<tr>
<th>Class</th>
<th>Definitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A bridges</td>
<td>Bridges other than Class B bridges</td>
</tr>
</tbody>
</table>
| Class B bridges        | • Bridges of National expressways, urban expressways, designated city expressways, Honshu-Shikoku highways, and general national highways.  
                        | • Double-deck bridges and overbridges of prefectural highways and municipal roads, and other bridges, highway viaducts, etc., especially important in view of regional disaster prevention plans, traffic strategy, etc. |
Chapter 3  Loads to be considered in Seismic Design

3.1 Loads to be considered in Seismic Design and their Combinations

(1) The following loads shall be taken into account in the seismic design:

1) Primary loads
   The primary loads defined in Section 2.1 of the Common Provisions, excluding the live load and the impact, shall be considered:
   a. Dead load \((D)\)
   b. Prestress force \((PS)\)
   c. Effect of creep of concrete \((CR)\)
   d. Effect of drying shrinkage of concrete \((SH)\)
   e. Earth pressure \((E)\)
   f. Hydraulic pressure \((HP)\)
   g. Buoyancy or uplift \((U)\)

2) Secondary loads
   Effects of earthquake \((EQ)\)

(2) Combinations of loads shall be as follows:
   Primary loads + Effects of earthquake \((EQ)\), specified in (1)

(3) Loads and their combinations shall be determined in such manners that they cause the most adverse stresses, displacements and effects.

3.2 Effects of Earthquake

The following seismic forces shall be taken into account to determine the effects of earthquake.

(1) Inertia force due to the dead weight of the structure (called “inertia force” hereafter)
(2) Earth pressure during an earthquake
(3) Hydrodynamic pressure during an earthquake
(4) Effects of liquefaction and liquefaction-induced ground flow
(5) Ground displacement during an earthquake
Chapter 4  Design Earthquake Ground Motions

4.1 General

Level 1 and Level 2 Earthquake Ground Motions shall be determined in accordance with the provisions in Section 4.2 and 4.3, respectively. However, design earthquake ground motions at a bridge site can be determined in consideration of the information on earthquake histories around the bridge site, active faults, earthquakes occurring in the plate-boundaries near the site, geological structures, geotechnical conditions, and existing strong motion records, if this procedure is more appropriate.

4.2 Level 1 Earthquake Ground Motion

(1) Level 1 Earthquake Ground Motion shall be determined in accordance with the acceleration response spectrum specified in (2).

(2) The acceleration response spectrum shall be provided, in principle, at the ground surface to be considered in seismic design prescribed in Section 4.6, and shall be calculated by Equation (4.2.1).

\[ S = c_z c_D S_0 \]  \hspace{1cm} (4.2.1)

where

\[ S \]  :  Acceleration response spectra for Level 1 Earthquake Ground Motion (rounded to an integral by neglecting decimals)

\[ c_z \]  :  Modification factor for zones specified in Section 4.4

\[ c_D \]  :  Modification factor for damping ratio. It shall be calculated by Equation (4.2.2) in accordance with the damping ratio \( h \).

\[ c_D = \frac{1.5}{40h + 1} + 0.5 \]  \hspace{1cm} (4.2.2)

\[ S_0 \]  :  Standard acceleration response spectra (gal) for Level 1 Earthquake Ground Motion. It shall be a value in Table 4.2.1 in accordance with the ground type specified in Section 4.5 and the fundamental period \( T \).
Table 4.2.1 Standard Acceleration Response Spectra $S_0$

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>$S_0$ (gal) with Fundamental Period $T$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>$T &lt; 0.1$ $S_0 = 431T^{1/3}$ but $S_0 \geq 160$</td>
</tr>
<tr>
<td>Type II</td>
<td>$T &lt; 0.2$ $S_0 = 427T^{1/3}$ but $S_0 \geq 200$</td>
</tr>
<tr>
<td>Type III</td>
<td>$T &lt; 0.34$ $S_0 = 430T^{1/3}$ but $S_0 \geq 240$</td>
</tr>
</tbody>
</table>

4.3 Level 2 Earthquake Ground Motion

(1) Level 2 Earthquake Ground Motion shall be determined in accordance with the acceleration response spectrum specified in (2).

(2) The acceleration response spectrum shall be provided, in principle, at the ground surface to be considered in seismic design prescribed in Section 4.6, and shall be calculated by Equations (4.3.1) and (4.3.2) according to the type of seismic ground motion specified in Section 2.2, respectively.

\[
S_I = c_z c_D S_{I0} \quad \text{...(4.3.1)}
\]

\[
S_{II} = c_z c_D S_{II0} \quad \text{...(4.3.2)}
\]

where

- $S_I$ : Type I acceleration response spectra (rounded off to 1 gal)
- $S_{II}$ : Type II acceleration response spectra (rounded off to 1 gal)
- $c_z$ : Modification factor for zones specified in Section 4.4
- $c_D$ : Modification factor for damping ratio. It shall be calculated using Equation (4.2.2) in accordance with the damping ratio $h$.
- $S_{I0}$ : Standard acceleration response spectra (gal) Type I. It shall be a value from
Table 4.3.1 in accordance with the ground type specified in Section 4.5 and the fundamental period T.

\( S_{I0} \) : Standard acceleration response spectra (gal) Type II. It shall be a value from Table 4.3.2 in accordance with the ground type specified in Section 4.5 and the fundamental period T.

Table 4.3.1 Standard Acceleration Response Spectra Type I, \( S_{I0} \)

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>( S_{I0} ) (gal) with Fundamental Period ( T(s) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>( T \leq 1.4 ) ( S_{I0}=700 ) ( T &gt; 1.4 ) ( S_{I0}=980/T )</td>
</tr>
<tr>
<td>Type II</td>
<td>( T &lt; 0.18 ) ( S_{I0}=1,505T^{1/3} ) ( S_{I0} \geq 700 ) ( 0.18 \leq T \leq 1.6 ) ( S_{I0}=850 ) ( T &gt; 1.6 ) ( S_{I0}=1,306T )</td>
</tr>
<tr>
<td>Type III</td>
<td>( T &lt; 0.29 ) ( S_{I0}=1,511T^{1/3} ) ( S_{I0} \geq 700 ) ( 0.29 \leq T \leq 2.0 ) ( S_{I0}=1,000 ) ( T &gt; 2.0 ) ( S_{I0}=2,000/T )</td>
</tr>
</tbody>
</table>

Table 4.3.2 Standard Acceleration Response Spectra Type II, \( S_{I10} \)

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>( S_{I10} ) (gal) with Fundamental Period ( T(s) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>( T &lt; 0.3 ) ( S_{I10}=4,463T^{2/3} ) ( S_{I10}=2,000 ) ( 0.3 \leq T \leq 0.7 ) ( S_{I10}=1,104/T^{5/3} )</td>
</tr>
<tr>
<td>Type II</td>
<td>( T &lt; 0.4 ) ( S_{I10}=3,224T^{2/3} ) ( S_{I10}=1,750 ) ( 0.4 \leq T \leq 1.2 ) ( S_{I10}=2,371/T^{5/3} )</td>
</tr>
<tr>
<td>Type III</td>
<td>( T &lt; 0.5 ) ( S_{I10}=2,381T^{2/3} ) ( S_{I10}=1,500 ) ( 0.5 \leq T \leq 1.5 ) ( S_{I10}=2,948/T^{5/3} )</td>
</tr>
</tbody>
</table>
4.4 Modification Factor for Zones

Modification factor for zones shall be determined in accordance with Table 4.4.1 for different zones. When the bridge site is just located on the border between different zones, the higher factors shall be adopted.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Modification Factor $c_z$</th>
<th>Definitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0</td>
<td>Regions other than the followings</td>
</tr>
<tr>
<td>B</td>
<td>0.85</td>
<td>Regions specified in item (II) of the Table in Section 1 ($Z$-values), “$Z$-values, methods for calculating $R_t$ and $A_i$ and standard for a specific administrative organ to designate areas with remarkably unstable ground “ (Ministry of Construction Notification)</td>
</tr>
<tr>
<td>C</td>
<td>0.7</td>
<td>Regions specified in items (III) and (IV) of the Table in Section 1 ($Z$-values), “$Z$-values, methods for calculating $R_t$ and $A_i$ and standard for a specific administrative organ to designate areas with remarkably unstable ground “ (Ministry of Construction Notification)</td>
</tr>
</tbody>
</table>
4.5 Ground Type for Seismic Design

Grounds types in seismic design shall be classified, in principle, into those types defined in Table 4.5.1, in accordance with the ground characteristic value $T_G$ calculated from Equation (4.5.1). When the ground surface lies on the same level as the surface of a bedrock in seismic design, the ground type shall be Type I.

$$T_G = 4 \sum_{i=1}^{n} \frac{H_i}{V_{si}}$$  

(4.5.1)

where

- $T_G$: Characteristic value of ground (s)
- $H_i$: Thickness of the $i$-th soil layer (m)
- $V_{si}$: Average shear elastic wave velocity of the $i$-th soil layer (m/s)
- $i$: Number of the $i$-th soil layer from the ground surface when the ground is classified into $n$ layers from the ground surface to the surface of a bedrock in seismic design

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Characteristic Value of Ground, $T_G$(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>$T_G &lt; 0.2$</td>
</tr>
<tr>
<td>Type II</td>
<td>$0.2 \leq T_G &lt; 0.6$</td>
</tr>
<tr>
<td>Type III</td>
<td>$0.6 \leq T_G$</td>
</tr>
</tbody>
</table>

4.6 Ground Surface to be considered in Seismic Design

Ground surface to be considered in seismic design shall generally be the ground surface in the normal design. However, for sites of unstable soil layers whose seismic subgrade reactions cannot be anticipated, the ground surface to be considered in seismic design shall be appropriately assumed by taking the effects into account.
Chapter 5  Verification of Seismic Performance

5.1 General

(1) In verifying seismic performance, the limit state of each structural member shall be appropriately determined in accordance with the limit states of the bridge specified in Sections 5.2 to 5.4.

(2) The verification shall ensure that the state of each structural member due to the design earthquake ground motions does not exceed its limit state defined in (1). The verification method is specified in Section 5.5.

(3) Based on the provisions in Section 5.6, the verification shall be performed such that the unseating of the superstructure can be prevented, even though the structural failures may occur due to structural behavior or ground failure unexpected in the seismic design.

5.2 Limit States of Bridges for Seismic Performance Level 1

Limit states of bridges for Seismic Performance Level 1 shall be properly established so that the mechanical properties of the bridges are maintained within the elastic ranges. When the limit state of each structural member is defined in terms of the stress, working stresses generated by an earthquake should not exceed its allowable value.

5.3 Limit States of Bridges for Seismic Performance Level 2

(1) Limit states of bridges for Seismic Performance Level 2 shall be properly established so that only the structural member in which the generations of plastic hinges are allowed deforms plastically within a range of easy functional recovery.

(2) The structural member in which the generations of plastic hinges are allowed shall be selected so that reliable energy absorption and recovery in a short period are ensured.

(3) These structural members shall be properly combined in accordance with the structural character of the bridge. Then, the limit state of each member shall be appropriately determined in consideration of the combination.
5.4 Limit States of Bridges for Seismic Performance Level 3

(1) Limit states of bridges for Seismic Performance Level 3 shall be properly established so that only the structural member in which the generations of plastic hinges are allowed deforms plastically within a range of the ductility limit of the member.

(2) The structural member in which the generations of plastic hinges are allowed shall be selected so that reliable energy absorption is ensured.

(3) These structural members shall be properly combined in accordance with the structural character of the bridge. Then, the limit state of each member shall be appropriately determined in consideration of the combination.

5.5 Basic Principles to Verify Seismic Performance

(1) The seismic performance shall be verified by a proper method in accordance with factors such as design earthquake ground motions, structural type and limit states of the bridge.

(2) For bridges without complicated seismic behavior, the above provision (1) is deemed to have met if the seismic performances are verified in accordance with the static methods specified in Chapter 6. On the other hand, for bridges with complicated seismic behavior, the seismic performances shall be verified in accordance with the dynamic methods specified in Chapter 7 in order to meet provision (1).

5.6 Unseating Prevention Measures of Bridge Superstructure

(1) Adequate measures against unseating of superstructures shall be taken when the superstructure separates structurally from the substructure, and with large relative displacements.

(2) Provision (1) is deemed to be satisfactory if the unseating prevention systems specified in Chapter 16 are installed.
Chapter 6  Verification Methods of Seismic Performance Based on Static Analysis

6.1 General

(1) Verification of seismic performance of a bridge by static method shall be based on the seismic coefficient method.

(2) In verifying the seismic performance for Level 1 Earthquake Ground Motion using the static method, loads shall be calculated according to the provisions specified in Section 6.2. The verification of Seismic Performance Level 1 shall be carried out by the seismic coefficient method described in Section 6.3, where the dynamic structural characteristics in the elastic range are taken into account.

(3) In verifying the seismic performance for Level 2 Earthquake Ground Motion using the static method, loads shall be calculated according to the provisions specified in Section 6.2. The verification of Seismic Performance Level 2 or Level 3 shall be carried out by the ductility design method described in Section 6.4.

6.2 Calculation Methods of Loads in Static Analysis

6.2.1 General

(1) In verifying the seismic performance of a bridge using the static method, careful consideration shall be given to the effects of earthquake listed in Section 3.2, such as inertia force, seismic earth pressure, hydrodynamic pressure, liquefaction and liquefaction-induced ground flow of ground.

(2) In calculation of inertia force, seismic earth pressure and hydrodynamic pressure, Sections 6.2.2, 6.2.4 and 6.2.5 shall be referred to, respectively. Consideration of the influence of liquefaction and liquefaction-induced ground flow shall be based on the provisions specified in Section 8.1.

(3) For structures below the ground surface described in Section 4.6, inertia force,
seismic earth pressure or hydrodynamic pressure need not be applied to the structures.

6.2.2 Inertia Force

(1) Inertia force shall be calculated in terms of the natural period of each design vibration unit specified in Section 6.2.3. For Level 1 and Level 2 Earthquake Ground Motion, Sections 6.2.3 and 6.4.2 shall be referred to, respectively.

(2) Inertia forces shall be generally considered in two horizontal directions perpendicular to each other. It can be assumed that the inertia forces in the two orthogonal directions, i.e. the longitudinal and transverse directions to the bridge axis, act individually. However, in the design of substructure in which the horizontal component of the earth pressure acts in a different direction from the bridge axis, the inertia force can be assumed to act in the directions of the horizontal component of earth pressure and its perpendicular direction.

(3) In the design of supports, vertical inertia forces shall be taken into account, in addition to the inertia forces in the two horizontal directions mentioned in the above (2).

(4) The inertia force of a superstructure shall be assumed to act at the height of the center of gravity of the superstructure. However, in case of a straight bridge, the acting height of the inertia force can be considered to be at the bottom of its support in the case of the longitudinal direction to the bridge axis.

6.2.3 Calculation Method of Natural Period

(1) Natural periods shall be appropriately calculated with consideration of the effects of deformations of structural members and foundations. In case of unstable ground during an earthquake, natural periods shall be obtained without reducing geotechnical parameters specified in Section 8.2.4.

(2) When a design vibration unit consists of one substructure and its supporting superstructural part, natural periods can be calculated by Equation (6.2.1)
\[ T = 2.01\sqrt{\delta} \]  \hspace{1cm} \text{(6.2.1)}

where

- \( T \) : Natural period of the design vibration unit (s)
- \( \delta \) : Lateral displacement of the superstructure at the height of the superstructural inertia force when 80\% of the weight of the substructure above the design ground surface and the entire weight of the superstructure jointly act in the directions of their inertia forces (m)

(3) When a design vibration unit consists of multiple substructures and their supporting superstructural part, natural periods shall be obtained by Equation (6.2.2).

\[ T = 2.01\sqrt{\delta} \]  \hspace{1cm} \text{(6.2.2)}

\[
\delta = \frac{\int w(s) u(s)^2 \, ds}{\int w(s) u(s) \, ds} \]  \hspace{1cm} \text{(6.2.3)}

where

- \( T \) : Natural period of the design vibration unit (s)
- \( w(s) \) : Weight of the superstructure or the substructure at position s (kN/m)
- \( u(s) \) : Lateral displacement of each structure at position s in the direction of the inertia force when a lateral force equal to the sum of the weight of the superstructure and that of the substructure above the design ground surface act in the direction of the inertia force (m)

Here, the symbol \( \int \) denotes the integral throughout the entire portion of the design vibration unit.
6.2.4 Seismic Earth Pressure

(1) Seismic earth pressure shall be determined with careful consideration of factors such as structural type, soil conditions, level of earthquake ground motions and dynamic behavior of the ground.

(2) Seismic earth pressure shall be assumed as a distributed load. The strength of an active earth pressure shall be calculated by Equation (6.2.4)

\[ P_{EA} = r \times K_{EA} + q'K_{EA} \]  \hspace{2cm} (6.2.4)

where

- \( P_{EA} \): Active earth pressure strength (kN/m²) during an earthquake at depth \( x \) (m)
- \( K_{EA} \): Coefficient of seismic active earth pressure to be calculated by Equation (6.2.5)

1) Between soil and concrete
   - Sand or gravel: \( K_{EA} = 0.21 + 0.90k_h \)
   - Sandy soil: \( K_{EA} = 0.24 + 1.08k_h \) \hspace{2cm} (6.2.5)

2) Between soil and soil
   - Sand or gravel: \( K_{EA} = 0.22 + 0.81k_h \)
   - Sandy soil: \( K_{EA} = 0.26 + 0.97k_h \)

\( k_h \): Design horizontal seismic coefficient used in seismic earth pressure calculation
\( r \): Unit weight of soil (kN/m³)
\( q' \): Surcharge on the ground surface during an earthquake (kN/m²)

Here, \( q' \) shall be the surcharge actually acting during an earthquake, excluding vehicular live load.
6.2.5 Seismic Hydrodynamic Pressure

(1) Hydrodynamic pressure shall be appropriately determined with consideration of factors such as water level, shape and size of the section of a substructure and level of design seismic ground motion.

(2) Seismic hydrodynamic pressure acting on a substructure subjected to Level 1 Earthquake Ground Motion can be calculated by the following equations. The seismic hydrodynamic pressure can be assumed to act in the same direction of the inertia force of the superstructure specified in Section 6.3.2.

1) Seismic hydrodynamic pressure acting on a wall structure with water on one side only:

Resulting force of seismic hydrodynamic pressure acting on a wall structure with water on one side only and height of the resulting force shall be calculated by Equations (6.2.6) and (6.2.7), respectively. (Refer to Fig. 6.2.1).

\[ P = \frac{7}{12} k_h w_0 b h^2 \] \hspace{1cm} (6.2.6)

\[ h_g = \frac{2}{5} h \] \hspace{1cm} (6.2.7)

where

- \( P \) : Resultant force of seismic hydrodynamic pressure acting on a structure (kN)
- \( k_h \) : Design lateral seismic coefficient for Level 1 Earthquake Ground Motion specified in Section 6.3.3
- \( w_0 \) : Unit weight of water (kN/m³)
- \( h \) : Water depth (m)
- \( h_g \) : Height from the ground surface to the resultant force of hydrodynamic pressure (m)
- \( b \) : Structural width in the direction perpendicular to the direction of hydrodynamic pressure (m)

[Figure 6.2.1: Seismic Hydrodynamic Pressure Acting on a Wall Structure]
2) Seismic hydrodynamic pressure acting on a column structure completely surrounded by water:

Resulting force of seismic hydrodynamic pressure acting on a column structure completely surrounded by water and the height of the resultant force shall be calculated by Equations (6.2.8) and (6.2.9), respectively. (Refer to Fig. 6.2.2).

\[
P = \begin{cases} 
\frac{3}{4} k_h w_0 A_0 h \frac{b}{a} \left( 1 - \frac{b}{4h} \right) & \text{when } \frac{b}{h} \leq 2.0 \\
\frac{9}{40} k_h w_0 A_0 h \frac{b}{a} & \text{when } \frac{b}{h} > 2.0
\end{cases}
\]

\[P = \begin{cases} 
\frac{3}{4} k_h w_0 A_0 h \frac{b}{a} \left( 0.7 - \frac{b}{10h} \right) & \text{when } 2.0 < \frac{b}{h} \leq 4.0 \\
\frac{9}{40} k_h w_0 A_0 h \frac{b}{a} & \text{when } 4.0 < \frac{b}{h}
\end{cases}
\]

\[h_g = \frac{3}{7} h\]

(6.2.8) \hspace{1cm} (6.2.9)

where

\[P\]: Resultant force of seismic hydrodynamic pressure acting on a structure (kN)

\[k_h\]: Design lateral seismic coefficient for Level 1 Earthquake Ground Motion specified in Section 6.3.3

\[w_0\]: Unit weight of water (kN/m³)

\[h\]: Water depth (m)

\[h_g\]: Height from the ground surface to the resultant force of hydrodynamic pressure (m)

\[b\]: Structural width in the direction perpendicular to the direction of hydrodynamic pressure (m)

\[a\]: Structural width in the direction of hydrodynamic pressure (m)

\[A_0\]: Sectional area of the substructure (m²)
6.3 Verification of Seismic Performance Level 1 for Earthquake Ground Motion

6.3.1 General

In verifying Seismic Performance Level 1 the seismic coefficient method considering dynamic structural characteristics in elastic range shall be used according to the provisions specified in Section 6.3.4. In this process, the sectional force and deformation in each member shall be calculated, when subjected to the loads specified in Section 6.2 and the inertia force specified in Section 6.3.2.

6.3.2 Calculation Method of Inertia Force

Inertia force shall be defined as the lateral force equal to the product of the weight of a structure and the design horizontal seismic coefficient specified in Section 6.3.3, and be considered acting on the structure in the direction of the inertia force of a design vibration unit. However, in the case of a movable bearing support between the superstructure and the substructure, static frictional force of the bearing support shall be applied to the substructure, rather than the inertia force of the superstructure.
6.3.3 Design Horizontal Seismic Coefficient

(1) Design horizontal seismic coefficient for Level 1 Earthquake Ground Motion shall be calculated by Equation (6.3.1). When the $k_h$-value obtained by this Equation is less than 0.1, $k_h$ shall be taken as 0.1

$$k_h = c_z k_{h0}$$  

(6.3.1)

where

- $k_h$: Design horizontal seismic coefficient (rounded to two decimals)
- $k_{h0}$: Standard value of the design horizontal seismic coefficient for Level 1 Earthquake Ground Motion, shown in Table 6.3.1 below.
- $c_z$: Modification factor for zones specified in Section 4.4

In verifying seismic performance for Level 1 Earthquake Ground Motion, inertia force caused by the soil weight and seismic earth pressure shall be calculated using design horizontal seismic coefficient at ground level obtained by Equation (6.3.2)

$$k_{hg} = c_z k_{hg0}$$  

(6.3.2)

where

- $k_{hg}$: Design horizontal seismic coefficient at ground level (rounded to two decimals)
- $k_{hg0}$: Standard value of the design horizontal seismic coefficient at ground level for Level 1 Earthquake Ground Motion. Values of 0.16, 0.2 and 0.24 shall be used for ground types I, II and III, respectively.
Table 6.3.1 Standard Values of the Design Horizontal Seismic Coefficient for Level 1 Earthquake Ground Motion, $k_{h0}$

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>$k_{h0}$ - Values in Terms of Natural Period $T$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>$T &lt; 0.1$, $k_{h0} = 0.431 T^{1/3}$ if $k_{h0} \geq 0.16$ or $0.1 \leq T \leq 1.1$, $k_{h0} = 0.2$ or $1.1 &lt; T$, $k_{h0} = 0.213 T^{2/3}$</td>
</tr>
<tr>
<td>Type II</td>
<td>$T &lt; 0.2$, $k_{h0} = 0.427 T^{1/3}$ if $k_{h0} \geq 0.20$ or $0.2 \leq T \leq 1.3$, $k_{h0} = 0.25$ or $1.3 &lt; T$, $k_{h0} = 0.298 T^{2/3}$</td>
</tr>
<tr>
<td>Type III</td>
<td>$T &lt; 0.34$, $k_{h0} = 0.430 T^{1/3}$ if $k_{h0} \geq 0.24$ or $0.34 \leq T \leq 1.5$, $k_{h0} = 0.3$ or $1.5 &lt; T$, $k_{h0} = 0.393 T^{2/3}$</td>
</tr>
</tbody>
</table>

(2) As a principle, the design horizontal seismic coefficient to be used shall be identical within the same design vibration unit.

6.3.4 Verification of Seismic Performance Level 1

Reinforced Concrete Columns or Reinforced Concrete abutments shall be verified according to Section 5.1, and foundations shall be in accordance with Sections 5.1 and 9.1 of Part IV Substructures. Steel piers and steel superstructures shall follow Part II Steel Structures. Concrete superstructures shall be based on the design loads specified in Chapter 4 of Part III Concrete Structures. Seismic isolated bridges and bearing supports shall be verified on the basis of the provisions of Chapter 9 and Section 15.1 of Part V, respectively.

6.4 Seismic Performance Verification Level 2 Earthquake Ground Motion

6.4.1 General

(1) In verifying Seismic Performance Level 2 and Level 3, the ductility design method shall be employed according to the provisions in Section 6.4.5. Sectional force and deformation in each member subjected to loads specified in Section 6.2 and inertia forces in Section 6.4.2 shall be calculated.
(2) In this process, seismic performance of each design vibration unit with one substructure and its supporting superstructural part shall generally be verified using the ductility design method, after obtaining the inertia force by the provision of section 6.4.2.

6.4.2 Calculation Method of Inertia Force

Inertia force defined as the lateral force equal to the product of the structural weight and the design horizontal seismic coefficient specified in Section 6.4.3, shall act in the direction of the inertia force of a design vibration unit. However, where the bearing support between the superstructure and the substructure is movable in the direction of the inertia force, inertia force of the superstructure shall be determined by the product of one half of the reaction from the superstructural weight and the design horizontal seismic coefficient specified in Section 6.4.3.
6.4.3 Design Horizontal Seismic Coefficient

(1) Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion shall be calculated according to the following:

1) Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I)

Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I) shall be calculated by Equation (6.4.1). When the product of the standard value of the design horizontal seismic coefficient \( k_{hc0} \) and the modification factor for zones \( cz \) is less than 0.3, design horizontal seismic coefficient shall be obtained by multiplying the force reduction factor \( c_s \) by 0.3. In addition, when the design horizontal seismic coefficient is less than 0.4 times the modification factor for zones \( cz \), the design horizontal seismic coefficient shall be equal to 0.4 times \( cz \).

\[
k_{hc} = c_s c_z k_{hc0} \quad \text{...............................................................(6.4.1)}
\]

where

\( k_{hc} \) : Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I) (rounded to two decimals)
\( k_{hc0} \) : Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I), according to Table 6.4.1
\( c_s \) : Force reduction factor specified in Section 6.4.4.
\( c_z \) : Modification factor for zones specified in Section 4.4

In assessing liquefaction potential of a sandy soil layer in the seismic performance verification for Level 2 Earthquake Ground Motion (Type I), the design horizontal seismic coefficient at the ground level calculated by Equation (6.4.2) shall be used. The design horizontal seismic coefficient used for calculation of inertia force caused by the soil weight and the seismic earth pressure shall be obtained with reference to Section 13.2.

\[
k_{hg} = c_z k_{hc0} \quad \text{...............................................................(6.4.2)}
\]

where
$k_{hg}$ : Design horizontal seismic coefficient at the ground level for Level 2 Earthquake Ground Motion (Type I) (rounded to two decimals).

$k_{hg0}$ : Standard value of the design horizontal seismic coefficient at the ground level for Level 2 Earthquake Ground Motion (Type I) (rounded to two decimals). For ground Types I, II and III, the values shall be 0.3, 0.35 and 0.40, respectively.

**Table 6.4.1 Standard Values of the Design Horizontal Seismic Coefficient for Level 2 Earthquake Ground Motion (Type I), $k_{hc0}$**

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Values of $k_{hc0}$ in Terms of Natural Period $T$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>$T \leq 1.4$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 0.7$</td>
</tr>
<tr>
<td></td>
<td>$1.4 &lt; T$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 0.876T^{2/3}$</td>
</tr>
<tr>
<td>Type II</td>
<td>$T &lt; 0.18$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 1.51T^{1/3}$ but $k_{hc0} \geq 0.7$</td>
</tr>
<tr>
<td></td>
<td>$0.18 \leq T \leq 1.6$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 0.85$</td>
</tr>
<tr>
<td></td>
<td>$1.6 &lt; T$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 1.16T^{2/3}$</td>
</tr>
<tr>
<td>Type III</td>
<td>$T &lt; 0.29$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 1.51T^{1/3}$ but $k_{hc0} \geq 0.7$</td>
</tr>
<tr>
<td></td>
<td>$0.29 \leq T \leq 2.0$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 1.0$</td>
</tr>
<tr>
<td></td>
<td>$2.0 &lt; T$</td>
</tr>
<tr>
<td></td>
<td>$k_{hc0} = 1.59T^{2/3}$</td>
</tr>
</tbody>
</table>

2) Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type II)

Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type II) shall be calculated by Equation (6.4.3). When the product of the standard value ($k_{hc0}$) of the design horizontal seismic coefficient and the modification factor for zones ($c_z$) is less than 0.6, the design horizontal seismic coefficient shall be 0.6 times the force reduction factor ($c_s$). In addition, when the design horizontal seismic coefficient is less than 0.4 times the modification factor for zones, the design horizontal seismic coefficient shall be 0.4 times the modification factor for zones.

$$k_{hc} = c_s c_z k_{hc0} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cd-
\( k_{h0} \): Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type II), according to Table 6.4.2.

\( c_s \): Force reduction factor specified in Section 6.4.4.

\( c_z \): Modification factor for zones, specified in Section 4.4.

In assessing liquefaction potential of a sandy soil layer in seismic performance verification for Level 2 Earthquake Ground Motion (Type II), the design horizontal seismic coefficient at the ground level calculated by Equation (6.4.4) shall be used. The design horizontal seismic coefficient used for calculation of inertia force caused by the soil weight and seismic earth pressure shall be obtained from Section 13.2.

\[
k_{hh} = c_s k_{h0} \tag{6.4.4}
\]

where

\( k_{hh} \): Design horizontal seismic coefficient at the ground level for Level 2 Earthquake Ground Motion (Type II) (rounded to two decimals).

\( k_{h0} \): Standard value of the design horizontal seismic coefficient at the ground level for Level 2 Earthquake Ground Motion (Type II) (rounded to two decimals). For ground Types I, II and III, the values shall be 0.80, 0.70 and 0.60, respectively.

### Table 6.4.2 Standard Values of Design Horizontal Seismic Coefficient for Level 2 Earthquake Ground Motion (Type II), \( k_{h0} \)

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Values of ( k_{h0} ) in Terms of Natural Period ( T ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>( T &lt; 0.3 ) ( k_{h0} = 4.46T^{2/3} ) ( k_{h0} = 2.0 ) ( 0.7 &lt; T ) ( k_{h0} = 1.24T^{-4/3} )</td>
</tr>
<tr>
<td>Type II</td>
<td>( T &lt; 0.4 ) ( k_{h0} = 3.22T^{2/3} ) ( k_{h0} = 1.75 ) ( 1.2 &lt; T ) ( k_{h0} = 2.23T^{-4/3} )</td>
</tr>
<tr>
<td>Type III</td>
<td>( T &lt; 0.5 ) ( k_{h0} = 2.38T^{2/3} ) ( k_{h0} = 1.50 ) ( 1.5 &lt; T ) ( k_{h0} = 2.57T^{-4/3} )</td>
</tr>
</tbody>
</table>

The highest value of design horizontal seismic coefficient shall generally be used in each design vibration unit.
6.4.4 Force Reduction Factor

(1) Force reduction factor \( c_s \) shall be appropriately determined with considerations of mechanical properties of structural members, including their plastic characters.

(2) For a structural system that can be modeled as a one degree-of-freedom system having an elasto-plastic force-displacement relation, force reduction factor shall be calculated by Equation (6.4.5)

\[
c_s = \frac{1}{\sqrt{2\mu_a - 1}}
\]  

(6.4.5)

where

\( \mu_a \): Allowable ductility ratio for the structural system having an elasto-plastic force-displacement relation. \( \mu_a \) can be obtained by Equation (10.2.3) for the case of an Reinforced Concrete Columns

6.4.5 Verification of Seismic Performance Levels 2 and 3

Reinforced Concrete Columns shall be verified in accordance with Section 6.4.6. Verification of pier foundations and abutment foundations shall follow Sections 6.4.7 and 6.4.8, respectively. For verification of superstructures, reference shall be made to Section 6.4.9. Bearing supports shall be verified according to Section 6.4.10.
6.4.6 Performance Verification for Reinforced Concrete Columns

(1) Verification of Seismic Performance Level 2

Reinforced Concrete single column piers and Reinforced Concrete single-story rigid-frame piers shall be verified according to Equations (6.4.6) and (6.4.7).

\[ k_{hc}W \leq Pa \]  \hspace{1cm} (6.4.6)
\[ \delta R \leq \delta Ra \]  \hspace{1cm} (6.4.7)

where

- \( k_{hc} \) : Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion specified in Section 6.4.3
- \( W \) : Equivalent weight (N) in the ductility design method obtained by Equation (6.4.8)
  \[ W = W_U + c_pW_p \]  \hspace{1cm} (6.4.8)
- \( c_p \) : Equivalent weight (N) coefficient determined from Table 6.4.3
- \( W_U \) : Weight of the superstructural part supported by the pier concerned (N)
- \( W_P \) : Weight of the pier (N)
- \( P_a \) : Lateral strength of a Reinforced Concrete Columns, calculated from Section 10.2 (N)
- \( \delta R \) : Residual displacement (mm) of a pier, obtained by Equation (6.4.9)
  \[ \delta R = c_R (\mu_r - 1) \left( 1 - r \right) \delta_y \]  \hspace{1cm} (6.4.9)
- \( c_R \) : Modification factor on residual displacement, a factor of 0.6 shall be taken for Reinforced Concrete Columns.
- \( r \) : Ratio of the secondary post-yielding stiffness to the yielding stiffness of a pier, a ratio of 0 shall be taken for Reinforced Concrete Columns
- \( \delta_y \) : Yield displacement (mm) of piers, calculated by Section 10.3 for Reinforced Concrete Columns
- \( \mu_r \) : Maximum response ductility ratio of piers, obtained by Equation (6.4.10)
  \[ \mu_r = \frac{1}{2} \left( \left( \frac{c_z k_{hco}W}{P_a} \right)^2 + 1 \right) \]  \hspace{1cm} (6.4.10)
- \( k_{hco} \) : Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion, specified in Section 6.4.3
- \( c_z \) : Modification factor for zones specified in Section 4.4
- \( \delta Ra \) : Allowable residual displacement (mm) of piers. \( \delta R \) shall be 1/100 times the
height from the bottom of the pier to the height of inertia force of the superstructure.

Table 6.4.3  Equivalent Weight Calculation Coefficient, \( c_p \)

<table>
<thead>
<tr>
<th>Bending Failure, or Shear Failure after Flexural Yielding</th>
<th>Shear Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>

(2) Verification of Seismic Performance Level 3

Reinforced Concrete single column piers and Reinforced Concrete single-story rigid-frame piers, shall be verified in accordance with Equation (6.4.6)

6.4.7 Performance Verification for Pier Foundations

(1) Pier foundations shall be verified according to their structural types. Responses of foundations subjected to loads described in (2) shall satisfy the conditions specified in (3).

(2) When responses of the piers reach the plastic ranges, the dead loads plus the inertia forces obtained by the design horizontal seismic coefficient in Equation (6.4.11), shall be considered as design loads. When responses of the piers remain within the elastic ranges, the sectional force at the pier bases shall be considered as design loads.

\[
k_{hp} = c_{df} P_u / W
\]

(6.4.11)

where

- \( k_{hp} \): Design horizontal seismic coefficient used in the verification of pier foundations based on the ductility design method (rounded to two decimals)
- \( c_{df} \): Modification factor used in calculation of the design lateral coefficient for pier foundations based on the ductility design method (rounded to two decimals)
- \( P_u \): Lateral strength (kN) of the pier supported by the foundation. For Reinforced Concrete Columns, \( P_u \) shall be calculated by referring to Section 10.3. However, in case of shear failure according to the provisions of Section 10.2,
the shear strength based on Section 10.5 shall be used. For steel piers, the maximum lateral force obtained from inelastic hysteretic models specified in Section 11.2 shall be adopted.

\( W \): Equivalent weight (kN) used in the ductility design method, obtained by Equation (6.4.8)

As for underground structural portions above the ground surface described in Section 4.6 and the substructural portions like pile footings below the ground surface that may greatly affect the seismic pile behavior, the inertia forces obtained by multiplying the design horizontal seismic coefficient specified in Section 6.4.3 and their weights, shall be considered in the design.

(3) When the loads specified in (2) above are considered, pier foundations shall generally be verified to ensure that yielding of the foundation specified in Section 12.3 will never generate.

However, when piers have their lateral strength sufficiently higher than the seismic force due to the design horizontal seismic coefficient, or when the effects of liquefaction are large, plastic behavior of pier foundations may be allowed. Under these circumstances, response ductility ratios and response displacements of the pier foundations calculated in accordance with Section 12.4 shall not exceed the allowable ductility ratios and the allowable displacements of the pier foundations, respectively, as specified in Section 12.5.
6.4.8 Performance Verification for Abutment Foundations

Abutment foundations resting on the ground where liquefaction described in Section 8.3.2 may affect the safety of the bridge, they shall be verified in accordance with the provisions provided in Chapter 13.

6.4.9 Performance Verification for Superstructures

Superstructures that may sustain major effects of earthquake shall be verified in a manner that the response values of the superstructural members do not exceed their allowable values. The allowable values for superstructural members can be found in Chapter 14, depending on types and properties of structural members.

6.4.10 Performance Verification for Bearing Supports

Bearing supports shall be verified by the provision of Chapter 15, depending on their structural characteristics. Response values of the supports shall not exceed the allowable values.
Chapter 7 Verification Methods of Seismic Performance Based on Dynamic Analysis

7.1 General

(1) Seismic performance by dynamic method shall be verified according to the provisions in Section 7.4. Sectional force, displacement and other responses of each structural member when subjected to earthquake ground motions described in Section 7.2, shall be calculated through dynamic analysis.

(2) In the dynamic analysis an appropriate model shall be established depending on the purpose of the analysis and the level of design earthquake ground motion, as specified in 7.3, and a suitable analysis method shall be selected.

7.2 Design Earthquake Ground Motions for Dynamic Analysis

The design earthquake ground motion in dynamic analysis shall be obtained as follows:

(1) In case of the response spectrum method:
   Acceleration response spectra obtained from Equation (4.2.1) for Level 1 Earthquake Ground Motion, and Equations (4.3.1) or (4.3.2) for Level 2 Earthquake Ground Motion shall be applied to the dynamic analysis.

(2) In case of the time-history response analysis method:
   Amplitudes of acceleration records obtained during past typical strong earthquakes shall be adjusted so that they have properties close, as far as possible, to those of the acceleration response spectra obtained by Equation (4.2.1) for Level 1 Earthquake Ground Motion and Equations (4.3.1) or (4.3.2) for Level 2 Earthquake Ground Motion, respectively. Acceleration records adjusted in such a manner shall be adopted for the analysis.
7.3 Analytical Models and Procedures

7.3.1 Analytical Models and Procedures

In a dynamic analysis, an appropriate analysis model and analysis method shall be employed, depending on the analysis purpose and the level of the design earthquake ground motion as shown below:

1. In verification of Seismic Performance Level 1 for Level 1 Earthquake Ground Motion, an analytical model capable of expressing dynamic properties of a bridge within the elastic ranges and a suitable analysis method shall be selected.
2. In verification of Seismic Performance Level 2 and Level 3 for Level 2 Earthquake Ground Motion, an analytical model and an analysis method capable of expressing inelastic effects of members in the plastic ranges shall be suitably adopted, when necessary.

7.3.2 Modeling of Structural Members

1. Structural members shall be properly modeled in consideration of their inelastic hysteretic properties.
2. Effects of the ground deformation (or subgrade reaction) can be modeled in terms of springs.
3. Bearing supports shall be properly modeled in accordance with their structural properties, inelastic hysteretic properties and damping characteristics.
4. In the case where a member having nonlinear behavior is modeled as an equivalent linear member by using the equivalent linear model approach, values of equivalent stiffness and equivalent damping ratio shall be appropriately determined.

7.4 Verification of Seismic Performance

1. Verification of Seismic Performance Level 1

1. Cross-sectional stresses in Reinforced Concrete Columns and Reinforced Concrete abutments obtained from a dynamic analysis shall not exceed the allowable stresses specified in Chapter 4 of Part IV Substructures.
2) Steel piers and steel superstructures shall be verified by comparing sectional forces obtained from a dynamic analysis with the provisions given in Part II Steel Structures.

3) In verification of pier foundations, sectional forces at the pier base obtained from a dynamic analysis shall be taken as the design seismic forces acting on the pier foundations. In the verification, reference shall be made to Sections 5.1 and 9.1 of Part IV Substructures.

4) In verification of concrete superstructures, cross-sectional stresses obtained from a dynamic analysis shall not exceed the allowable stresses specified in Chapter 3 of Part III Concrete Structures.

5) Bearing supports shall be verified so that the response values obtained from a dynamic analysis do not exceed the allowable values specified in Section 15.3.

(2) Verification of Seismic Performance Level 2

1) In verification of Reinforced Concrete Columns, response ductility ratios obtained from a dynamic analysis shall not exceed the allowable ductility ratios specified in Section 10.2. At the same time, residual displacements calculated from Equation (6.4.9) with the use of maximum response displacement at the height of inertial force of the superstructure, shall not exceed the allowable residual displacements specified in Section 6.4.6.

2) Steel piers shall be verified in accordance with the provisions of Chapter 11.

3) Pier foundations shall be verified in accordance with Section 6.4.7.

4) For steel and concrete superstructures, verification shall be carried out to ensure that their response values obtained from a dynamic analysis do not exceed the ultimate strength and the allowable deformation within the plastic range, as defined in Sections 14.2.1 and 14.3.1, respectively.

5) Bearing supports shall be verified to ensure that response values obtained from a dynamic analysis are not exceeding the allowable values specified in Section 15.3.

(3) Verification of Seismic Performance Level 3

1) In verification of Reinforced Concrete Columns, response ductility ratios obtained from a dynamic analysis shall not exceed the allowable ductility ratios provided in Section 10.2.

2) Steel piers shall be verified in accordance with Chapter 11.
3) Pier foundations shall be verified in accordance with Section 6.4.7.

4) For steel and concrete superstructures, verification shall be carried out to ensure that their response values obtained from a dynamic analysis do not exceed the strength and the allowable deformation within the plastic range, as defined in Sections 14.2.1 and 14.3.1, respectively.

5) Bearing supports shall be verified to ensure that response values obtained from a dynamic analysis are not exceeding the allowable values specified in Section 15.3.

(4) In verification of the seismic performance of a seismically-isolated bridge, reference shall be made to Chapter 9.

(4) In verification of seismic performance of a bridge with the use of a dynamic method, special attention shall be paid to guaranteeing the seismic resistance of the bridge as a whole system.
Chapter 8 Effects of Seismically Unstable Ground

8.1 General

(1) In verification of seismic performance of a bridge on a ground changing into unstable state during an earthquake, the effects of the unstable ground shall be taken into account. In the above, an unstable ground is defined as an extremely soft soil layer in seismic design, or a sandy layer susceptible to liquefaction and affect the bridge due to the liquefaction and the liquefaction-induced ground flow.

(2) In verifying seismic performance of a bridge with conditions shown in the above (1), a case assuming a stable ground shall also be considered, in order to ensure the seismic performance of the bridge for both the stable and unstable grounds.

8.2 Geotechnical Parameters of Extremely Soft Layer and Sandy Layer Prone to Liquefaction

8.2.1 General

For an extremely soft soil layer in seismic design specified in Section 8.2.2 or a sandy layer in Section 8.2.3 which may affect the bridge due to liquefaction, geotechnical parameters used in the seismic design shall be reduced in accordance with the provisions in Section 8.2.4.

8.2.2 Assessment of Extremely Soft Soil Layer in Seismic Design

For a clayey layer or a silt layer located up to three meters below the ground surface, and having compressive strength of 20kN/m² or less obtained from an unconfined compression test or an in-situ test, the layer shall be regarded as an extremely soft layer in the seismic design.

8.2.3 Assessment of Soil Liquefaction

(1) Sandy layer requiring liquefaction assessment

For an alluvial sandy layer having all of the following three conditions, liquefaction...
assessment shall be conducted in accordance with the provisions specified in (2) below, because liquefaction may affect the bridge during an earthquake.

1) Saturated soil layer having ground water level higher than 10 m below the ground surface and located at a depth less than 20 m below the present ground surface.
2) Soil layer containing a fine content (FC) of 35% or less or soil layer having a higher FC and plasticity index IP less than 15.
3) Soil layer with a mean grain diameter (D_{50}) less than 10 mm and a grain diameter at 10% pass (on the accumulation curve) (D_{10}) less than 1 mm.

(2) Assessment of liquefaction

For a soil layer requiring liquefaction assessment according to the provisions specified in 1) above, liquefaction resistance factor $F_L$ shall be calculated by Equation (8.2.1). If the result turns out to be less than 1.0, the layer shall be regarded as having liquefaction potential.

$$F_L = \frac{R}{L}$$  \hspace{1cm} (8.2.1)

$$R = c_w R_L$$  \hspace{1cm} (8.2.2)

$$L = r_d k_{tg} \sigma_v / \sigma_v'$$  \hspace{1cm} (8.2.3)

$$R_d = 1.0 - 0.015x$$  \hspace{1cm} (8.2.4)

$$\sigma_v = \gamma_{t1} h_w + \gamma_{t2} (x - h_w)$$  \hspace{1cm} (8.2.5)

$$\sigma_v' = \gamma_{t1} h_w + \gamma_{t2} (x - h_w)$$  \hspace{1cm} (8.2.6)

(For Type I Earthquake Ground Motion)

$$c_w = 1.0$$  \hspace{1cm} (8.2.7)

(For Type II Earthquake Ground Motion)

$$c_w = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases}$$  \hspace{1cm} (8.2.8)

where

- $F_L$: Liquefaction resistance factor
- $R$: Dynamic shear strength ratio
- $L$: Seismic shear stress ratio
- $c_w$: Modification factor on earthquake ground motion
- $R_L$: Cyclic triaxial shear stress ratio to be obtained by properties (3) below
- $r_d$: Reduction factor of seismic shear stress ratio in terms of depth
Design horizontal seismic coefficient at the ground surface for Level 2 Earthquake Ground Motion specified in Section 6.4.3

$\sigma_v$: Total overburden pressure (kN/m²)

$\sigma_v^\prime$: Effective overburden pressure (kN/m²)

$x$: Depth from the ground surface (m)

$\gamma_t$: Unit weight of soil above the ground water level (kN/m³)

$\gamma_t^\prime$: Effective unit weight of soil below the ground water level (kN/m³)

$h_w$: Depth of the ground water level (m)

\[ \sigma = \gamma_t x \]

\[ \sigma^\prime = \gamma_t^\prime (x - h_w) \]

(3) Cyclic triaxial shear stress ratio

Cyclic triaxial shear stress ratio $R_L$ shall be calculated by Equation (8.2.9).

\[
R_L = \begin{cases} 
0.0882 \sqrt{\frac{N_a}{1.7}} & (N_a < 14) \\
0.0882 \sqrt{\frac{N_a}{1.7}} + 1.6 \times 10^{-6} (N_a - 14)^{4.5} & (14 \leq N_a)
\end{cases}
\]  \hspace{1cm} (8.2.9)

where

<For Sandy Soil>

\[ N_a = c_1 N_i + c_2 \]

\[ N_i = \frac{170 N}{(\sigma_v^\prime + 70)} \]

\[ c_1 = \begin{cases} 
1 & (0\% \leq FC < 10\%) \\
\frac{(FC + 40)}{50} & (10\% \leq FC < 60\%) \\
\frac{FC}{20} - 1 & (60\% \leq FC)
\end{cases} \]

\[ c_2 = \begin{cases} 
0 & (0\% \leq FC < 10\%) \\
\frac{(FC - 10)}{18} & (10\% \leq FC)
\end{cases} \]

<For Gravelly Soil>

\[ N_a = \{1 - 0.36 \log_{10}(D_{50}/2)\} N_i \]

$RL$: Cyclic triaxial shear stress ratio

$N_i$: N value obtained from the standard penetration test

$N_a$: Equivalent N value corresponding to effective overburden pressure of 100 kN/m²

$Na$: Modified N value taking into account the effects of grain size

$c_1$, $c_2$: Modification factors of N value on fine content

$FC$: Fine content (%) (percentage by mass of fine soil passing through the 75 $\mu$m
8.2.4 Reduction of Geotechnical Parameters

(1) When a soil layer is considered to be an extremely soft layer according to the provisions specified in Section 8.2.2, its geotechnical parameters (shear modulus and strength) shall be assumed to be zero in the seismic design.

(2) For a sandy layer causing liquefaction and affecting a bridge according to the provisions of Section 8.2.3, geotechnical parameters in the seismic design shall be reduced in accordance with the liquefaction resistance factor $F_L$.

Geotechnical parameters of a sandy layer causing liquefaction and affecting a bridge shall be equal to the product of geotechnical parameters obtained without liquefaction and coefficient $D_E$ in Table 8.2.1. In the case of $D_E = 0$, geotechnical parameters (shear modulus and strength) shall be taken as 0 in the seismic design.

<table>
<thead>
<tr>
<th>Table 8.2.1 Reduction Factor $D_E$ for Geotechnical Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_L \leq 1/3$</td>
</tr>
<tr>
<td>$10 &lt; x \leq 20$</td>
</tr>
<tr>
<td>$1/3 &lt; F_L \leq 2/3$</td>
</tr>
<tr>
<td>$10 &lt; x \leq 20$</td>
</tr>
<tr>
<td>$2/3 &lt; F_L \leq 1$</td>
</tr>
<tr>
<td>$10 &lt; x \leq 20$</td>
</tr>
</tbody>
</table>

(3) Weight of a soil layer with reduced or zero geotechnical parameter in the seismic design
8.3 Verification of Seismic Performance of Foundations for Liquefaction-induced Ground Flow

8.3.1 General

(1) Ground with possible lateral movement

A ground with both of the following two conditions shall be treated as a ground with possible lateral movement affecting a bridge.

1) Ground within a distance less than 100 m from a water front in a shore area formed by a revetment with an elevation difference of 5 m or more between the water bottom and the ground surface behind.

2) Ground with a sandy layer thicker than 5 m that is assessed as a liquefiable layer according to the provisions in Section 8.2.3 and is distributed somewhat widely in the area of the water front.

(2) Verification of seismic performance of a bridge for liquefaction-induced ground flow

A pier foundation situated on a ground specified in (1) above shall be verified against possible liquefaction-induced ground flow. In the verification, lateral movement force specified in Section 8.3.2 shall act on the pier foundation, and horizontal displacement at the top of foundation shall not exceed two times the horizontal displacement at yielding of the foundation described in Section 12.3. However, the lateral movement force and the inertia force need not be considered simultaneously.
8.3.2 Calculation of Lateral Force on Foundations by Liquefaction-induced Ground Flow Force

When considering the effects of liquefaction-induced ground flow, lateral movement forces acting on pier foundations shall be obtained as below:

In the case where lateral movement occurs under the conditions shown in Fig. 8.3.1, lateral movement forces per unit area calculated by Equations (8.3.1) and (8.3.2) shall be applied to structural members in the non-liquefying layer and those in the liquefying layer, respectively, both of which are located above the depth in consideration of the effects of lateral movement. In this case, the horizontal resistance of the soil layer within the thickness necessary to consider the effects of lateral movement cannot be expected.

\[ q_{NL} = c_s c_{NL} K_p \gamma_{NL} x \quad (0 \leq x \leq H_{NL}) \]  
\[ q_{NL} = c_s c_{NL} \left( \gamma_{NL} H_{NL} + \gamma_L (x - H_{NL}) \right) \quad (H_{NL} < x \leq H_{NL} + H_L) \]

where
- \( q_{NL} \): Lateral movement force per unit area (kN/m²) acting on a structural member in a non-liquefying layer at the depth \( x \) (m)
- \( q_L \): Lateral movement force per unit area (kN/m²) acting on a structural member in a liquefying layer at the depth \( x \) (m)
- \( c_s \): Modification factor on distance from the water front. \( C_S \) shall take a value shown in Table 8.3.1.
- \( c_{NL} \): Modification factor of the lateral movement force in a non-liquefying layer. \( C_{NL} \) shall be a value in Table 8.3.2, according to liquefaction index \( P_L \) (m²) obtained from Equation (8.3.3).

\[ P_L = \int_0^{20} (1 - F_L) (0.5x - 10) \, dx \]  

where
- \( P_L \): Liquefaction index (m²)
- \( F_L \): Liquefaction factor
- \( K_p \): Passive earth pressure coefficient (in normal condition)
- \( \gamma_{NL} \): Mean unit weight of a non-liquefying layer (kN/m³)
- \( \gamma_L \): Mean unit weight of a liquefying layer (kN/m³)
- \( x \): Depth from the ground surface (m)
- \( H_{NL} \): Thickness of non-liquefying layer (m)
$H_L$: Thickness of liquefying layer (m)

$F_L$: Liquefaction resistance factor calculated by Equation (8.2.1). If $F_L \geq 1$, $F_L = 1$.

**Fig. 8.3.1 Model for Calculating Lateral Movement Force**

<table>
<thead>
<tr>
<th>Table 8.3.1</th>
<th>Modification Factor $C_s$ on Distance from Water Front</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance From Water Front $s$ (m)</td>
<td>Modification Factor $c_s$</td>
</tr>
<tr>
<td>$s \leq 50$</td>
<td>1.0</td>
</tr>
<tr>
<td>$50 &lt; s \leq 100$</td>
<td>0.5</td>
</tr>
<tr>
<td>$100 &lt; s$</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 8.3.2</th>
<th>Modification Factor $C_{NL}$ of Lateral Movement Force in Non-Liquefying Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquefaction Index $P_L$ (m$^2$)</td>
<td>Modification Factor $c_{NL}$</td>
</tr>
<tr>
<td>$P_L \leq 5$</td>
<td>0</td>
</tr>
<tr>
<td>$5 &lt; P_L \leq 20$</td>
<td>$(0.2 P_L - 1)/3$</td>
</tr>
<tr>
<td>$20 &lt; P_L$</td>
<td>1</td>
</tr>
</tbody>
</table>
Chapter 9 Verification of Seismic Performance of Seismically Isolated (Menshin) Bridges

9.1 General

(1) Introduction of seismic isolation design shall be considered in view of the effects of increase in natural period and energy absorption capacity of a bridge for both normal and seismic conditions. In particular, for a bridge meeting the following conditions, seismic isolation shall not normally be adopted.

1) A bridge on a soil layer for which the seismic geotechnical parameter determined from Section 8.2.4 is zero.
2) A bridge with a fairly flexible substructure and long fundamental natural period.
3) A bridge on a soft soil layer of long natural period that may cause resonance with that of the bridge if seismic isolation is introduced.
4) A bridge with uplift at bearing supports.

(2) When seismic isolation is adopted, the bridge should be designed with emphasis on enhancement of damping properties using a high energy absorption system, and on dispersion of seismic forces. Excessive increase in the natural period of the bridge shall be avoided.

(3) Natural period of a seismically-isolated bridge shall be determined in such a manner that energy is absorbed by the isolation bearings and that an increase in seismic displacement of the superstructure does not cause any adverse effects on the bridge functions.

(4) Isolation bearings having good performance by simple mechanisms shall be selected and used within a range of accurate mechanical behavior. Isolation bearings shall be firmly fixed to the superstructure and substructure by anchor bolts and be capable of replacement.

(5) When seismic isolation is adopted, gap allowances shall be kept between main structures such as abutments, piers and girders so that the displacement assumed in the design of an isolation system is possible.
When an isolation bearing is used as a horizontal force distributing structure during an earthquake, rather than a seismic force reduction structure through energy absorption, design structural response based on the damping ratio of the bridge shall not be reduced.

9.2 Verification of Seismic Performance of Seismically-Isolated Bridges

(1) Verification of seismic performance of a seismically-isolated bridge shall be based on the provisions specified in Section 5.5, together with proper consideration of the bridge properties. In calculating seismic response of the isolation bridge, the isolation bearing can be modeled in accordance with the provisions provided in Section 9.3.

(2) Allowable ductility ratio of Reinforced Concrete Columns in a seismically-isolated bridge shall be obtained by Equation (9.2.1)

\[
\mu_m = 1 + \frac{\delta_u - \delta_y}{\alpha_m \delta_y} \tag{9.2.1}
\]

where

- \( \mu_m \): Allowable ductility ratio of an Reinforced Concrete Columns in seismically-isolated bridge
- \( \alpha_m \): Safety factor used for the calculation of the Reinforced Concrete Columns. \( \alpha_m \) shall be calculated by Equation (9.2.2)

\[
\alpha_m = 2 \alpha \tag{9.2.2}
\]

where

- \( \alpha \): Safety factor used for calculation of the allowable ductility ratio of the Reinforced Concrete Columns, and is specified in Table 10.2.1
- \( \delta_y, \delta_u \): Yield displacement and ultimate displacement of the Reinforced Concrete Columns, respectively, and shall be calculated by referring to Section 10.3

(3) Isolation bearings shall be verified in accordance with the provisions in Chapter 15. In addition, only isolation bearings having the fundamental functions specified in Section 9.4 shall be basically selected.
Design of superstructural end of a seismically-isolated bridge shall be based on Section 14.4.

9.3 Analytical Model of Isolation Bearings

9.3.1 General

An isolation bearing can be modeled as a nonlinear member having inelastic hysteretic property or an equivalent linear member having equivalent stiffness and damping ratio. In the case of modeling an isolation bearing as a nonlinear member, reference shall be made to provisions in Section 9.3.2. In the case of modeling an isolation bearing as an equivalent linear member, design displacement and equivalent stiffness and damping ratio of the bearing shall be based on Section 9.3.3.

9.3.2 Inelastic Hysteretic Model of Isolation Bearings

In modeling an isolation bearing as a nonlinear member, a bilinear model illustrated in Fig. 9.3.1 shall be employed. In this case the initial stiffness and secondary stiffness shall be appropriately determined according to its properties.

![Inelastic Hysteretic Model of Isolation Bearing](image)

- $K_1$: Initial stiffness (kN/m)
- $K_2$: Secondary stiffness (kN/m)
- $u_y$: Horizontal yield displacement (m)
- $Q_y$: Horizontal yield force at the time of yielding (kN)

**Fig. 9.3.1 Inelastic Hysteretic Model of Isolation Bearing**
9.3.3 Equivalent Linear Model of Isolation Bearings

In modeling an isolation bearing as an equivalent linear member, design displacement, and equivalent stiffness and damping ratio shall be obtained in the following manner:

(1) Design displacement of an isolation bearing

To calculate equivalent stiffness and damping ratio of an isolation bearing, design displacement \( u_B \) and the effective design displacement \( u_{Be} \) can be obtained by Equations (9.3.1) and (9.3.2), respectively.

\[
\begin{align*}
    u_B &= \frac{k_B W_u}{K_B} & \text{(in case of verification for Level 1 Earthquake Ground Motion)} \\
    u_{Be} &= \frac{c_B P_u}{K_B} & \text{(in case of verification for Level 2 Earthquake Ground Motion)}
\end{align*}
\]

where
- \( u_B \): Design displacement of an isolation bearing (m)
- \( u_{Be} \): Effective design displacement of an isolation bearing (m)
- \( k_B \): Design horizontal seismic coefficient for Level 1 Earthquake Ground Motion, specified in Section 6.3.3
- \( P_u \): Horizontal force equal to the lateral strength of a pier when considering plastic behavior of the pier, and horizontal force corresponding to the maximum response displacement of a foundation within its plastic behavior (kN)
- \( c_B \): Dynamic modification factor used for the calculation of design displacement of isolation bearing, and can be taken as 1.2.
- \( K_B \): Equivalent stiffness of isolation bearing used for the calculation of \( u_B \) (kN/m)
- \( W_u \): Weight of superstructure, whose horizontal force is borne by the isolation bearing (kN)
- \( c_B \): Modification factor representing the dynamic properties of the inertia force, and shall be taken as 0.7.

(2) Equivalent stiffness and damping ratio of an isolation bearing

Equivalent stiffness and damping ratio of an isolation bearing can be calculated by Equations (9.3.3) and (9.3.4), respectively.
\[ K_B = \frac{F(u_{re}) - F(-u_{re})}{2u_{re}} \]  
(9.3.3)

\[ h_B = \frac{W}{2\pi W} \]  
(9.3.4)

where

\( K_B \) : Equivalent stiffness of an isolation bearing (kN/m)

\( h_B \) : Equivalent damping ratio of an isolation bearing

\( F(u) \): Horizontal force necessary to produce the horizontal displacement \( u \) of an isolation bearing (kN)

\( W \) : Elastic energy of an isolation bearing (kN·m), and equal to the triangular area in Fig. 9.3.2.

\( \Delta W \) : Total energy absorbed by isolation bearing, and equal to the area inside the hysteretic curve of horizontal displacement and force, shown in Fig. 9.3.2. (kN·m)

Fig. 9.3.2  Equivalent Stiffness and Damping Ratio of an Isolation Bearing
9.4 Basic Performance Requirement for Isolation Bearings

(1) Variations of the equivalent stiffness of an isolation bearing shall be within $\pm 10\%$ of the value obtained by Section 9.3.3. On the other hand, the equivalent damping ratio of an isolation bearing shall be greater than the value calculated from Section 9.3.3.

(2) An isolation bearing shall be stable when subjected to repeated load corresponding to the design displacement $u_B$ calculated in the verification for Level 2 Earthquake Ground Motion, as specified in Section 9.3.3.

(3) An isolation bearing shall possess positive tangential stiffness within the range of the design displacement $u_B$ calculated in the verification for Level 2 Earthquake Ground Motion, as specified in Section 9.3.3.

(4) An isolation bearing shall generally prevent an occurrence of residual displacement affecting post-earthquake bridge functions.

(5) Equivalent stiffness and the equivalent damping ratio of an isolation bearing shall be stable for environmental conditions including repeated live loads and temperature changes.

9.5 Other Structures for Reducing Effects of Earthquake

When reduction of seismic response of a bridge is expected by using structures or devices other than seismic isolation, careful investigation shall be carried out on dynamic characters and seismic behavior of the bridge, and unseating of the bridge shall be fully ensured.
Chapter 10 Lateral Strength and Ductility Capacity of Reinforced Concrete Columns

10.1 General

(1) In verification of Seismic Performance Level 2 and Level 3 of an Reinforced Concrete Columns, lateral strength and ductility capacity of the pier shall be calculated in accordance with Section 10.2. However, lateral strength and ductility capacity of a single story Reinforced Concrete rigid-frame pier or an Reinforced Concrete Columns subjected to eccentric moment due to dead load of the superstructure shall be calculated in accordance with Sections 10.8 and 10.9, respectively.

(2) When plastic behavior of an Reinforced Concrete Columns is expected in the seismic design, structural details shall conform to Sections 10.6 and 10.7, in order to ensure its plastic deformation performance.
10.2 Evaluation of Failure Mode, Lateral Strength and Ductility Capacity

(1) Failure mode of an Reinforced Concrete Columns shall be evaluated by Equation (10.2.1).

\[
P_u \leq P_s : \text{Flexural (or bending) failure} \\
P_s < P_u \leq P_{s0} : \text{Shear failure after flexural yielding} \\
P_{s0} < P_u : \text{Shear failure}
\]

where

- \(P_u\): Lateral strength of an Reinforced Concrete Columns, as specified in Section 10.3 (N)
- \(P_s\): Shear strength of an Reinforced Concrete Columns, as specified in Section 10.5 (N)
- \(P_{s0}\): Shear strength of an Reinforced Concrete Columns calculated by assuming that the modification factor on the effects of repeated alternative loads specified in Section 10.5 (N), is equal to 1.0.

(2) Lateral strength \(P_a\) of an Reinforced Concrete Columns shall be calculated by Equation (10.2.2), depending on the failure mode.

\[
P_a = \begin{cases} 
P_0 \text{ (Flexural failure) (where } P_c < P_a) \\ 
P_0 \text{ (Shear failure after flexural yielding)} \\ 
P_{s0} \text{ (Shear failure)} 
\end{cases}
\]

where

- \(P_0\): Lateral strength of an Reinforced Concrete Columns (N)
- \(P_c\): Lateral strength of an Reinforced Concrete Columns at cracking, specified in Section 10.3 (N)

(3) Ductility capacity \(\mu_a\) of an Reinforced Concrete Columns shall be calculated depending on the failure mode as follows:

1) Ductility capacity for flexural failure shall be calculated by Equation (10.2.3):
\[ \mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y} \]  

(10.2.3)

where

- \( \mu_a \): Ductility capacity of the Reinforced Concrete Columns
- \( \delta_u \): Ultimate displacement of the Reinforced Concrete Columns, specified in Section 10.3 (mm)
- \( \delta_y \): Yield displacement of the Reinforced Concrete Columns, specified in Section 10.3 (mm)
- \( \alpha \): Safety factor shown in Table 10.2.1

<table>
<thead>
<tr>
<th>Seismic Performance to be Verified</th>
<th>Safety Factor ( \alpha ) in Calculation of Ductility Capacity for Type I Earthquake Ground Motion</th>
<th>Safety Factor ( \alpha ) in Calculation of Ductility Capacity for Type II Earthquake Ground Motion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Performance Level 2</td>
<td>3.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Seismic Performance Level 3</td>
<td>2.4</td>
<td>1.2</td>
</tr>
</tbody>
</table>

2) For a pier of shear failure after flexural yielding or shear failures, the Ductility Capacity shall be taken as 1.0.
10.3 Calculation of Lateral Strength and Displacement

Lateral strength $P_c$ at cracking, yielding lateral strength $P_y$, the lateral strength $P_u$, yield displacement $\delta_y$, and ultimate displacement $\delta_u$ of a single-column Reinforced Concrete Columns shall be calculated for both Type I and Type II Earthquake Ground Motions defined in Section 2.2, with the following conditions:

1. Fiber strain is proportional to the distance from the neutral axis.
2. Skeleton curve between horizontal force and horizontal displacement shall be expressed by an ideal elasto-plastic model shown in Fig. 10.3.1.
3. Stress-strain curve and ultimate strain of concrete shall conform to Section 10.4.
4. Stress-strain curve of a reinforcing bar shall conform to Fig. 10.3.2.
5. Lateral strength $P_c$ at cracking shall be calculated by Equation (10.3.1)

$$P_c = \frac{W}{h}(\sigma_{bt} + \frac{N}{A}) \quad \text{.......................................................... (10.3.1)}$$

where
- $W$ : Section modulus of a pier with consideration of axial reinforcement at the pier bottom section (mm$^3$)
- $\sigma_{bt}$ : Flexural tensile strength of concrete (N/mm$^2$) to be calculated by Equation (10.3.2)

$$\sigma_{bt} = 0.23 \sigma_{ck}^{\frac{1}{3}} \quad \text{.......................................................... (10.3.2)}$$

- $N$ : Axial force acting on the pier bottom section (N)
- $A$ : Sectional area of a pier, with consideration of axial reinforcement at the pier bottom section (mm$^2$)
- $h$ : Height from the pier bottom to the height of superstructural inertial force (mm)
- $\sigma_{ck}$ : Design strength of concrete (N/mm$^2$)

6. Yield limit state denotes the elastic limit state in the skeleton curve of an ideal elasto-plastic model. Yield lateral strength and yield displacement shall be calculated by Equations (10.3.3) and (10.3.4), respectively.

$$P_y = \frac{M_u}{h} \quad \text{.......................................................... (10.3.3)}$$
\[ \delta_y = \frac{M_u}{M_{y0}} \delta_{y0} \]  

(10.3.4)

where

- \( \delta_{y0} \): Horizontal displacement at the time of yielding of axial tensile reinforcing bars at the outermost edge of the pier bottom section (called “initial yield displacement” hereafter) (mm)
- \( M_u \): Ultimate bending moment at the pier bottom section (N·mm)
- \( M_{y0} \): Bending moment at the time of yielding of axial tensile reinforcing bars at the outermost edge of the pier bottom section (N·mm)
- \( H \): Distance from the pier bottom to the height of superstructural inertial force (mm)

(7) Ultimate limit state denotes the state when concrete strain at the location of axial compressive reinforcement reaches the ultimate strain. With consideration of plastic hinges occurring at the damaged sections, the lateral strength and the ultimate displacement shall be calculated by Equations (10.3.5) and (10.3.6), respectively.

\[ P_u = \frac{M_u}{h} \]  

(10.3.5)

\[ \delta_u = \delta_y + (\phi_u - \phi_y) L_p (h - L_p/2) \]  

(10.3.6)

where

- \( L_p \): Plastic hinge length (mm) calculated by Equation (10.3.7)

\[ L_p = 0.2h - 0.1D \]  

(10.3.7)

in which \( 0.1D \leq L_p \leq 0.5D \)

- \( D \): Sectional depth (mm) (D shall be the diameter of a circular section, or the length of a rectangular section in the analytical direction)
- \( \phi_y \): Yield curvature at the pier bottom section (1/mm)
- \( \phi_u \): Ultimate curvature at the pier bottom section (1/mm)
Fig. 10.3.1 Model of Horizontal Force and Horizontal Displacement Relationship of a Single–Column Reinforced Concrete Columns

\[ \delta_{y0}, \delta_y, \delta_u \text{ Horizontal displacement } \delta \]

- \( Y_0 \): Initial yield limit state
- \( Y \): Yield limit state
- \( U \): Ultimate limit state

Stress in a reinforcing bar \( \sigma_s \)

\[ \sigma_s = \sigma_{sy} \]

\[ \sigma_s = E_s \varepsilon_s \]

Strain in a reinforcing bar \( \varepsilon_s \)

where

- \( \sigma_{sy} \): Yield point of the reinforcing bar (N/mm\(^2\))
- \( \sigma_s \): Stress in the reinforcing bar (N/mm\(^2\))
- \( E_s \): Young's modulus of the reinforcing bar (N/mm\(^2\))
- \( \varepsilon_s \): Strain in the reinforcing bar

Fig. 10.3.2 Skeleton Curve between Stress and Strain of a Reinforcing Bar
10.4 Stress–Strain Curve of Concrete

The stress–strain curve of concrete shall be determined by Equation (10.4.1) based on Fig.10.4.1.

\[
\sigma_c = \begin{cases} 
E_c \varepsilon_c \left(1 - \frac{1}{n} \left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right) & (0 \leq \varepsilon_c \leq \varepsilon_{cc}) \\
\sigma_{cc} - E_{des} (\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leq \varepsilon_{cu}) 
\end{cases} \tag{10.4.1}
\]

\[
n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - \sigma_{cc}} \tag{10.4.2}
\]

\[
\sigma_{cc} = \sigma_{ck} + 3.8 \alpha \rho_s \sigma_{sy} \tag{10.4.3}
\]

\[
\varepsilon_{cc} = 0.002 + 0.033 \beta \frac{\rho_s \sigma_{sy}}{\sigma_{ck}} \tag{10.4.4}
\]

\[
E_{des} = 11.2 \frac{\sigma_{ck}^2}{\rho_s \sigma_{sy}} \tag{10.4.5}
\]

\[
\varepsilon_{cu} = \begin{cases} 
\varepsilon_{cc} & \text{(For Type I Earthquake Ground Motion)} \\
\varepsilon_{cc} + 0.2 \sigma_{cc} \frac{E_{des}}{0.018} & \text{(For Type II Earthquake Ground Motion)} 
\end{cases} \tag{10.4.6}
\]

\[
\rho_s = \frac{4 A_h}{sd} \leq 0.018 \tag{10.4.7}
\]

where

- \(\sigma_c\) : Stress of concrete (N/mm²)
- \(\sigma_{cc}\) : Strength of concrete restrained by lateral confining reinforcement (N/mm²)
- \(\sigma_{ck}\) : Design strength of concrete (N/mm²)
- \(\varepsilon_c\) : Strain of concrete
- \(\varepsilon_{cc}\) : Strain of concrete under the maximum compressive stress
- \(\varepsilon_{cu}\) : Ultimate strain of concrete restrained by lateral confining reinforcement
- \(E_c\) : Young’s modulus of concrete (N/mm²) shown in Table 3.3.3 of Part I Common Provisions
- \(E_{des}\) : Descending gradient (N/mm²)
- \(\rho_s\) : Volume ratio of lateral confining reinforcement
- \(A_h\) : Sectional area of each lateral confining reinforcement (mm²)
- \(s\) : Spacings of lateral confining reinforcement (mm)
\[ d \] : Effective length (mm) of lateral confining reinforcement. It shall be the largest length of core concrete divided and restrained by lateral hoop ties or cross ties.

\( \sigma_{sy} \) : Yield point of lateral confining reinforcement (N/mm²)

\( \alpha, \beta \) : Modification factor on section. \( \alpha = 1.0, \beta = 1.0 \) for a circular section, and \( \alpha = 0.2, \beta = 0.4 \) for rectangular, hollow circular and hollow rectangular sections.

\( n \) : A constant defined by Equation (10.4.2)

Fig. 10.4.1 Stress-Strain Curve of Concrete

10.5 Shear Strength

Shear strength shall be calculated by Equation (10.5.1).

\[
P_s = S_c + S_s
\]

\[
S_c = c_c c_{pt} \tau_c b d
\]

\[
S_s = \frac{A_s \sigma_{sy} d (\sin \theta + \cos \theta)}{1.15 a}
\]

where

\( P_s \) : Shear strength (N)

\( S_c \) : Shear strength resisted by concrete (N)

\( \tau_c \) : Average shear stress that can be borne by concrete (N/mm²). Values in Table 10.5.1 shall be used.

\( c_c \) : Modification factor on the effects of alternating cyclic loading. \( c_c \) shall be taken as 0.6 for Type I Earthquake Ground Motion and 0.8 for Type II.
ce : Modification factor in relation to the effective height (d) of a pier section. Values in Table 10.5.2 shall be used.

cpt : Modification factor in relation to the axial tensile reinforcement ratio pt. Values in Table 10.5.3 shall be used.

b : Width of a pier section perpendicular to the direction in calculating shear strength (mm)

d : Effective height of a pier section parallel to the direction in calculating shear strength (mm)

pt : Axial tensile reinforcement ratio. It is the value obtained by dividing the total sectional areas of the main reinforcement on the tension side of the neutral axis by bd (%).

Sc : Shear strength borne by hoop ties (N)

Aw : Sectional area of hoop ties arranged with an interval of α and an angle of θ (mm²)

σ_{sy} : Yield point of hoop ties (N/mm²)

θ : Angle formed between hoop ties and the vertical axis (degree)

α : Spacings of hoop ties (mm)

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<thead>
<tr>
<th>Table 10.5.1 Average Shear Stress of Concrete τ_c (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Compressive Strength of Concrete σ_ck (N/mm²)</td>
</tr>
<tr>
<td>----------------------------------------------------------</td>
</tr>
<tr>
<td>Average Shear Stress of Concrete τ_c (N/mm²)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 10.5.2 Modification Factor ce in Relation to Effective Height d of a Pier Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Height (mm)</td>
</tr>
<tr>
<td>ce</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 10.5.3 Modification Factor cpt in Relation to Axial Tensile Reinforcement Ratio pt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Reinforcement Ratio (%)</td>
</tr>
<tr>
<td>cpt</td>
</tr>
</tbody>
</table>
10.6 Structural Details for Improving Ductility Performance

(1) Lapping of axial reinforcement

In order to ensure the lateral strength of a pier specified in Section 10.2, lappings of axial reinforcements shall not generally be placed within the plastic zone, when generation of plastic deformation of the pier is expected in the verification of its seismic performance.

(2) Arrangement of lateral hoop ties and cross ties

1) To prevent buckling of axial reinforcement and for the restraint of internal concrete, hoop ties and cross ties shall be arranged in an appropriate manner having suitable spacings, when generation of plastic deformation of the pier is expected in the verification of its seismic performance.

2) Hoop ties shall be deformed bars of at least 13mm diameter, and the spacings shall not generally be greater than 150mm in the plastic zone. However, in the case of adopting different spacings in the middle section of the pier, the spacings shall not be altered abruptly.

3) Hoop ties shall generally be arranged so as to enclose the axial reinforcement, and the ends of hoop ties with the hooks described below shall normally be fixed into the concrete inside a pier. Lap joints without hooks shall not be generally adopted. When right angle hooks are used, measures shall be taken to prevent the hooks from becoming ineffective, even if the cover concrete spalls. In addition, lappings of hoop ties shall be placed in a zigzag pattern along the pier height. Straight length of a hoop tie between two bending ends at the hooks shall be longer than the values provided below. Internal radius of bending of a hook shall be at least 2.5 times the hoop tie diameter. Here, internal radius of bending refers to the inside radius of a bent reinforcing bar. Provisions for the straight length of a hoop tie are as follows:

i) Semi-circular hook: 8 times the diameter of a hoop tie or 120mm whichever is the greater.

ii) Acute angle hook: 10 times the diameter of the hoop tie
iii) Right angle hook: 12 times of the diameter of the hoop tie

4) When hoop ties are lapped at any place other than the corners of a rectangular section, hoop ties shall generally have a lap length of at least 40 times the diameter of the hoop ties. In addition, the hoop ties shall generally conform to the provisions in 3) above.

5) Cross ties shall generally be arranged inside the pier section. In order to enhance the restraint of inside concrete, the following requirements shall be satisfied in arranging cross ties:

   i) Cross ties shall generally be of the same material and the same diameters as the hoop ties.
   ii) Cross ties shall generally be arranged in perimetric directions of a pier section.
   iii) Spacings of cross ties within a pier section shall not be greater than one meter.
   iv) Cross ties shall be arranged in all sections with hoop ties arranged.
   v) Cross ties shall generally be hooked up to the hoop ties arranged in the perimetric directions of the section. In case of double axial reinforcements, the cross ties shall be hooked up to the outside hoop ties.
   vi) Ends of cross ties being hooked up to the hoop ties shall be generally fixed into the concrete inside a pier, according to the provisions on semi-circular or acute angle hooks specified in 3).
   vii) A cross tie shall generally go through a pier section, with use of a continuous reinforcing bar or a pair of reinforcing bars with a joint within the pier section. In case of a joint within the pier section, an appropriate joint structure shall be selected so as to possess a joint strength equivalent to that of the cross tie.

10.7 Cut-off of Longitudinal Reinforcement at Mid-Height

(1) In order to increase ductility and ultimate strength of a pier, cut-off of the longitudinal reinforcement at mid-height of the pier shall not generally allowed.

(2) In case that the cut-off of the longitudinal reinforcement at mid-height is unavoidable, the cut-off position shall be carefully selected so as to prevent the occurrence of plastic behavior at that position. Generally, the position can be determined by Equation (10.7.1).
\[ h_i - h \left( 1 - \frac{M_{yi}}{2M_{yB}} \right) + D \]  \hspace{1cm} \text{(10.7.1)}

where

- \( h_i \): Height from the pier bottom to the \( i \)-th cut-off position of axial reinforcement (mm)
- \( h \): Height from the pier bottom to the height of the superstructural inertial force (mm)
- \( M_{yi} \): Yield bending moment of the section from the pier bottom to \( i \)-th cut-off portion of the axial reinforcement (N \cdot mm)
- \( M_{yB} \): Yield bending moment at the pier bottom section (N \cdot mm)
- \( D \): Smaller dimension of the pier, in the longitudinal and transverse directions to the bridge axis. In the case of a circular section, D shall be taken as the diameter of the section.

(3) When the cut-off of the longitudinal reinforcement at mid-height of a pier is allowed, the following structural details shall be considered:

1) Cut-off of the longitudinal reinforcement at mid-height of the pier shall not be located within the area of four times the length of a plastic hinge.

2) A reduction rate in the amount of axial reinforcement at a cut-off position shall not exceed 1/3. Notwithstanding this, in the case reduction is carried out at different heights in both the longitudinal and transverse directions, the reduction rate shall be determined individually at each section.

3) Within a section with height of 1.5 times of either the smaller dimension of a rectangular pier or the diameter of a circular pier measuring towards both above and below the cut-off position, the spacings of the hoop ties shall not exceed 150 mm. Furthermore, as mentioned in Section 10.6 (2), the spacings of the hoop ties shall not be altered abruptly.
10.8 Lateral Strength and Ductility Capacity of Reinforced Concrete Two Column Bents

Lateral strength and ductility capacity of an Reinforced Concretesingle-layer rigid-frame pier for both out-of-plane and in-plane directions shall be calculated as follows:

(1) Inertia force resisted by each column member in the out-of-plane direction of an Reinforced Concrete two column bents shall first be calculated. Then, lateral strength and ductility capacity of the pier shall be obtained from the provisions of Sections 10.2 and 10.5, by assuming that each column member is regarded as a single-column Reinforced Concrete Columns.

(2) Lateral strength and ductility capacity in the in-plane direction of an Reinforced Concrete two column bents shall be obtained by referring to the following 2), depending on the evaluation result on failure mode described in 1) below. In addition, in the case when plastic hinges in a beam member is allowed to be formed, the requirements as specified in 3) below shall be satisfied, in order to avoid brittle failure of the beam under the primary loads specified in Section 2.1 of Part I Common Provisions.

1) Failure mode of an Reinforced Concrete two column bents shall be evaluated from Equation (10.8.1) using shear force $S_i$, shear strengths $P_{si}$ and $P_{s0i}$, generated at each plastic hinge when an inertia force equivalent to the lateral strength is applied to the pier.

\[
\begin{align*}
S_i & \leq P_{si} : \text{Flexural failure} \\
P_{si} < S_i & \leq P_{s0i} : \text{Shear failure after flexural yielding} \\
P_{s0i} < S_i & : \text{Shear failure}
\end{align*}
\]

\[\text{Equation (10.8.1)}\]

where

$S_i$ : Shear force generated at i-th plastic hinge when an inertia force equivalent to the lateral strength is applied (N)

$P_{si}$ : Shear strength at i-th plastic hinge obtained from Section 10.5 (N)

$P_{s0i}$ : Shear strength at i-th plastic hinge calculated by using a modification factor of
1.0 on the effects of alternative cyclic forces, specified in Section 10.5 (N)

2) Lateral strength and ductility capacity of an Reinforced Concrete two column bents shall depend on the failure mode of the pier, and can be obtained as follows:

i) Lateral strength $P_a$ of an Reinforced Concrete two column bents shall be given by Equation (10.8.2).

$$
P_a = \begin{cases} 
  P_u & \text{(Flexural failure)} \\
  P_u & \text{(Shear failure after flexural yielding)} \\
  P_i & \text{(Shear failure)} 
\end{cases}$$

(10.8.2)

where

$P_u$: Lateral strength of an Reinforced Concrete two column bents (N)

$P_i$: Horizontal force when shear force generated at a plastic hinge exceeds the shear strength (N)

ii) Ductility capacity $\mu_a$ of an Reinforced Concrete two column bents shall be given by Equation (10.8.3).

$$
\mu_a = \begin{cases} 
  \frac{1}{\alpha} & \text{(Flexural failure)} \\
  1.0 & \text{(Shear failure after flexural yielding, and shear failure)} 
\end{cases}
$$

(10.8.3)

where

$\mu_a$: Ductility capacity of an Reinforced Concrete two column bents

$\delta_y$: Yield displacement of an Reinforced Concrete two column bents (mm)

$\delta_u$: Ultimate displacement of an Reinforced Concrete two column bents (mm)

$\alpha$: Safety factor shown in Table 10.2.1

iii) Yield displacement $\delta_y$, lateral strength $P_u$ and ultimate displacement $\delta_u$ of an Reinforced Concrete two column bents shall be calculated under the following conditions, besides the provisions specified in Section 10.3:

1) An analytical model used in the analysis shall be capable of considering change in axial forces acting on each column member and formation of plastic hinges at
plural sections.

2) Ultimate limit state of a Reinforced Concrete two column bents judged as flexural failure shall be an earlier one of the following two: a state when all plastic hinges formed in plural sections reach the ultimate limit state specified in Section 10.3 (7), and a state when curvature formed in a section of a plastic hinge reaches 2 times the ultimate curvature of the section.

3) When a plastic hinge is expected to occur in a beam, the sectional force generated in the beam shall be verified by Equation (10.8.4).

\[
\frac{V_b}{P_{si}} \leq 1 \quad \text{……………………………………………………………………………….. (10.8.4)}
\]

where

\(V_b\): Shear force acting on the beam when subjected to the primary loads (excluding the impact) as defined in Section 2.1 of Part I Common Provisions (N)

\(P_{si}\): Shear strength at i-th plastic hinge, calculated from the provisions in Section 10.5 (N)

(3) At the upper and lower column ends and both beam ends of an Reinforced Concrete two column bents where plastic hinges generate, structural details for improving ductility of an Reinforced Concrete Columns provided in Section 10.6 shall be satisfied within a section of 4 times the plastic hinge length given in Equation (10.3.7).

10.9 Effect of Eccentric Loading of Superstructure

(1) Lateral strength and ductility capacity of an Reinforced Concrete Columns subjected to an eccentric bending moment caused by superstructural dead load shall be obtained in the direction of the eccentric moment, from the provisions in (3) depending on failure mode judged by the requirements in (2) below. In the direction opposite to that of the eccentric moment, lateral strength and ductility capacity can be obtained by neglecting the influences of the eccentric moment. Furthermore, the yield stiffness of an Reinforced Concrete Columns for obtaining the natural period shall be calculated with provisions shown in (4) below.
(2) Failure mode of an Reinforced Concrete Columns subjected to an eccentric bending moment due to structural dead load shall be evaluated according to the criteria in Section 10.2 (1). However, lateral strength of an Reinforced Concrete Columns in judging failure mode shall be calculated by Equation (10.9.1).

\[ P_{aE} = P_u - \frac{M_0}{h} \]  \hspace{1cm} (10.9.1)

\[ M_0 = De \]  \hspace{1cm} (10.9.2)

where

- \( P_{aE} \): Lateral strength of the Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load (N)
- \( M_0 \): Eccentric bending moment caused by the superstructural dead load (N \cdot mm)
- \( P_u \): Lateral strength of the Reinforced Concrete Columns as per Section 10.3 (N)
- \( h \): Distance from the bottom of pier to superstructural inertia force (mm)
- \( D \): Superstructural dead weight (N)
- \( e \): Eccentric distance from the centroid of the pier section to the center of gravity of the superstructure (mm)

(3) For a pier with flexural failure, lateral strength and ductility capacity shall be calculated by Equations (10.9.3) and (10.9.4), respectively.

\[ P_{aE} = P_{uE} \]  \hspace{1cm} (10.9.3)

\[ \mu_E = 1 + \frac{\gamma_{aE} - \delta_{yE}}{\delta_{0E} - \delta_{yE}} \]  \hspace{1cm} (10.9.4)

where

- \( P_{aE} \): Lateral strength of an Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load (N)
- \( \mu_E \): Ductility capacity of an Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load
- \( \alpha \): Safety factor specified in Section 10.2
- \( \gamma_{aE} \): Ultimate displacement of an Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load (mm)
- \( \delta_{yE} \): Yield displacement of an Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load (mm)
- \( \delta_{0E} \): Initial displacement at the height of the superstructural inertia force, caused by the eccentric bending moment of the superstructural dead load (mm)
In the case of a shear failure after flexural yielding, the lateral strength shall be obtained from Equation (10.9.3), and the ductility capacity shall be taken as 1.0. Furthermore, the influences of the eccentric bending moment can be neglected in the case of a shear failure.

(4) Yield stiffness $K_{yE}$ of an Reinforced Concrete Columns used in calculating the natural period shall be obtained by Equation (10.9.5).

$$k_{yE} = \frac{P_{yE}}{\delta_{yE} - \delta_{0E}}$$  \hspace{1cm} (10.9.5)

where

$K_{yE}$: Yield stiffness of an Reinforced Concrete Columns subjected to eccentric bending moment caused by the superstructural dead load (N/mm)
Chapter 11 Verification of Seismic Performance of Steel Columns

11.1 General

(1) Allowable values such as allowable displacements of a steel pier for Seismic Performance Level 2 or Level 3, and inelastic hysteretic models in a dynamic analysis shall be determined in accordance with the provisions in Section 11.2.

(2) A steel pier shall be designed in such a manner that a ductile failure can be prevented, and the required ductility can be ensured. A steel pier designed in accordance with the structural details specified in Section 11.3 can be regarded as a satisfactory one.

(3) An anchorage block between a steel pier and an Reinforced Concrete foundation shall be designed for the Level 2 Earthquake Ground Motion without causing the plastic deformation at the anchorage. An anchorage designed in accordance with the provisions specified in Section 11.4 can be regarded as satisfactory.

11.2 Verification of Seismic Performance by Dynamic Analysis

(1) The Seismic Performance Level 2 or Level 3 shall be verified in accordance with the following provisions.

1) In verification of Seismic Performance Level 2, the maximum response of a steel pier obtained from a dynamic analysis shall not exceed its allowable value. In addition, the residual displacement calculated by substituting the maximum response displacement at the height of the point of the superstructural inertia force into Equation (6.4.9), shall not exceed the allowable residual displacement. Ratio ($r$) of the secondary post-yielding stiffness to the yielding stiffness used in calculation of the residual displacement in Equation (6.4.9), and residual displacement modification factor ($C_R$) shall be taken as the values defined in Table 11.2.1.
Table 11.2.1 Ratio ($r$) of the Secondary Post-Yielding Stiffness to the Yielding Stiffness Used in Calculation of the Residual Displacement of a Steel Pier, and Residual Displacement Modification Factor ($c_R$)

<table>
<thead>
<tr>
<th>Type of steel piers</th>
<th>$r$</th>
<th>$c_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel piers without concrete filling</td>
<td>0.2</td>
<td>0.45</td>
</tr>
<tr>
<td>Steel piers with concrete filling</td>
<td>0.05</td>
<td>0.35</td>
</tr>
</tbody>
</table>

2) In verification of Seismic Performance Level 3, the maximum response of a steel pier obtained from a dynamic analysis shall not exceed its allowable value.

(2) Allowable values such as allowable displacements of a steel pier for verifying the Seismic Performance Level 2 or Level 3 shall generally be determined in accordance with the results of repeating load test for specimens having structural details similar to those of the real pier. Allowable residual displacement shall not exceed one hundredth of the height from the pier base to the height of the superstructural inertia force.

(3) An inelastic hysteretic model of a steel pier in a dynamic analysis shall represent allowable values such as allowable displacements obtained from test results specified in (2), and elastoplastic behavior of the steel pier.

11.3 Structural Details

(1) Low-strength concrete is recommended for filling concrete inside steel piers. Concrete shall be filled up to a height capable of preventing the buckling of a steel section just above the concrete filling.

(2) Structural details of hollow steel piers shall follow the provisions in Sections 4.2 and 14.3 of Part II Steel Bridges, and shall avoid brittle failure, and enhance their ductile properties.

11.4 Design for Anchorage Block

Anchorage block of a steel pier shall generally be designed with strength not less than the lateral strength of the steel pier. When concrete is filled into the foundation, the strength of the steel pier shall be calculated by taking the effects of the filled concrete into account.
Chapter 12 Verification of Seismic Performance of Pier Foundations

12.1 General

(1) In the verification of Seismic Performance Level 2 or Level 3 for a pier foundation without plastic behavior, sectional forces, subgrade reactions, and displacements shall be calculated in accordance with Section 12.2. Yielding of the foundation shall be based on the provisions specified in Section 12.3.

(2) In the verification of Seismic Performance Level 2 or Level 3 for a pier foundation with plastic behavior, response ductility factors and response displacements shall be calculated in accordance with Section 12.4. Ductility capacities and allowable displacements shall be determined in accordance with Section 12.5.

(3) Sectional forces in a pier foundation member shall be verified in accordance with Section 12.6.

12.2 Calculation of Sectional Force, Ground Reaction Force, and Displacement of Pier Foundations

Sectional forces, subgrade reactions, and displacements of the pier foundation subjected to loads specified in Subsection 6.4.7(2), shall be calculated taking into account ground reaction forces, non-linear properties and uplift of the foundation from the supporting ground, as well as the foundation type.

12.3 Yielding of Pier Foundations

Yielding of a foundation shall be defined as a state when a rapid increase in horizontal displacement at the height of superstructural inertial force begins, as a result of yielding of foundation members, yielding of the ground, or uplift of the foundation from the ground.
12.4 Calculation of Foundation Response in Case of Plastic Hinges Generating in Pier Foundations

(1) Response ductility factor and response displacement of a pier foundation causing pier foundation yielding shall be obtained from Equations (12.4.1) and (12.4.2) with the use of the design horizontal seismic coefficient specified in (2):

\[
\mu_{Fr} = \frac{1}{r} \left\{ -1 + \sqrt{1 - r + r(k_{hc,F} + k_{hy,F})^2} \right\} \quad (r \neq 0)
\]

\[
\mu_{Fr} = \frac{1}{2} (1 + (k_{hc,F}/k_{hy,F})^2) \quad (r = 0)
\]

\[
\delta_{Fr} = \mu_{Fr} \delta_{Fy}
\]

(12.4.1)

(12.4.2)

Where

\( \mu_{Fr} \) : Response ductility factor of the pier foundation

\( \delta_{Fr} \) : Response displacement of a superstructure at the height of its inertia force, caused by the foundation deformation (m)

\( \delta_{Fy} \) : Horizontal displacement at the height of the superstructural inertia force, caused by the foundation yielding (m)

\( r \) : Ratio of secondary stiffness to yielding stiffness of the pier foundation

\( k_{hy,F} \) : Horizontal seismic coefficient when subjected to pier foundation yielding (rounded to two decimals).

\( k_{hc,F} \) : Design horizontal seismic coefficient of the pier foundation, used in the verification of the ductility design method specified in (2).

(2) Design horizontal seismic coefficient in calculating ductility factors and response displacements of a pier foundation shall be determined from Equation (12.4.3).

\[
k_{hc,F} = c_D c_z k_{hc,0}
\]

(12.4.3)

where

\( k_{hc,F} \) : Design horizontal seismic coefficient when considering plastic behavior of the pier foundation (rounded to two decimals)

\( c_D \) : Modification factor for damping ratio

\( c_z \) : Modification factor for zones specified in Section 4.4

\( k_{hc,0} \) : Standard value of design horizontal seismic coefficient for Level 2 Earthquake Ground Motion specified in Subsection 6.4.3
12.5 Ductility and Displacement Capacity of Pier Foundations

(1) Ductility capacity of a pier foundation shall be determined by considering that the function of a bridge can easily be recovered, even though some failures may occur in the pier foundation.

(2) Allowable displacements of a pier foundation shall be determined by considering that the function of a bridge can easily be recovered, even though some failures may occur in the pier foundation.

12.6 Design of Members of Pier Foundations

A sectional force in a pier foundation member calculated in accordance with Section 12.2 shall not exceed its ultimate strength.
Chapter 13 Verification of Seismic Performance of Abutment Foundations at Site Prone to Soil Liquefaction

13.1 General

(1) In the verification of Seismic Performance Level 2 or Level 3 for an abutment foundation on ground likely to liquefy and affect seismic response of the bridge, the horizontal design seismic coefficient and the response ductility factor shall be determined in accordance with the provisions in Sections 13.2 and 13.3, respectively. Also, allowable response ductility factor of the abutment foundation shall be determined in accordance with the provisions in Section 13.4.

(2) The sectional forces in members of an abutment foundation shall be verified in accordance with the provisions in Section 13.5.

13.2 Horizontal Seismic Coefficient for Performance Verification of Abutment Foundations

An abutment section located above the design ground surface, an overburden soil resting on a footing, and a footing of a pile foundation normally take a large part of the net weight of the overall foundation. Therefore, inertia forces of such structural and soil parts and design horizontal seismic coefficients in calculation of the seismic earth pressures shall be properly determined in accordance with the design horizontal seismic coefficient at the ground surface defined in Subsection 6.4.3.

13.3 Calculation of Response Ductility Factor of Abutment Foundations

Response ductility factors of an abutment foundation shall be obtained by appropriately taking into account the nonlinear behavior of the foundation and the effects of earth pressure.

13.4 Ductility Capacity of Abutment Foundations

Ductility capacitys of an abutment foundation shall be determined so that the bridge function can easily be recovered, even if some failures may occur in the foundation.
13.5 Design of Members of Abutment Foundation

Members of abutment foundations shall be verified so that sectional forces in any members are less than their ultimate strength.
Chapter 14 Verification of Seismic Performance of Superstructure

14.1 General

(1) In verifying Seismic Performance Level 2 or Level 3 for steel or concrete superstructures with plastic behavior, the ultimate strength and the allowable displacement shall be determined in accordance with the provisions in Sections 14.2 and 14.3, respectively.

(2) Verification of Seismic Performance Level 2 or Level 3 for steel or concrete superstructures without plastic behavior shall be performed in accordance with the verification methods specified in Chapter 4 of Part II Steel Structures, or those in Chapter 4 of Part III Concrete Structures.

(3) The ends of the superstructures such as the gap between adjacent decks and expansion joints, shall be verified in accordance with the provisions in Section 14.4.

14.2 Steel Superstructure

14.2.1 Strength and Displacement Capacity

Ultimate strength and allowable displacement of a steel superstructure in the plastic range shall be determined by either experimental results or appropriate analytical results. These values shall be obtained in terms of limit states of the superstructure specified in Sections 5.3 and 5.4.

14.2.2 Structural Details

The ends of the superstructures just above a bearing support where local deformation is likely to occur due to concentrated loads, shall be reinforced by placing reinforcing steel members to prevent local deformation. In addition, cross beams and diaphragms shall be set up at these portions to avoid out-of-plane girder deformation possibly caused by seismic forces in the perpendicular direction to the bridge axis.
14.3 Reinforced Concrete Superstructure

14.3.1 Strength and Displacement Capacity

Ultimate strength and allowable displacement of a concrete superstructure in the plastic range shall be determined by either experimental results or appropriate analytical results. These values shall be obtained in terms of limit states of the superstructure specified in Sections 5.3 and 5.4.

14.3.2 Structural Details

(4) Reinforcements in a concrete superstructure shall be placed by properly selecting splice structures, and configuration and arrangement of lateral bars, in consideration of the plastic range estimated in Reinforced Concrete members.

(5) Bearing supports of a concrete superstructure subjected to horizontal forces transmitted from bearing shoes and unseating prevention structures shall be designed in accordance with the provisions in Section 18.2 of Part III Concrete Structures.

14.4 Ends of Superstructure

14.4.1 Gap between Two Adjacent Girders

(6) In the design of the ends of the superstructure, necessary gap between the ends of two adjacent girders shall be taken for preventing a collision between two adjacent superstructures, a superstructure and an abutment, or a superstructure and the truncated portion of a pier head, when subjected to Level 1 and Level 2 Earthquake Ground Motion. In particular, for seismically-isolated bridges, sufficient spacing of adjacent girders shall be provided at the ends of superstructures, in order to ensure the expected effects of seismic isolation. However, in design of a normal bridge other than seismically-isolated bridges, the gap between the ends of two adjacent girders can be determined by considering that the collision does not happen for Level 1 Earthquake Ground Motion, if a verification confirms that the collision will not affect the sound seismic performance of the bridge when subjected to Level 2 Earthquake Ground Motion.
(7) The gap between the ends of two adjacent girders set up for avoiding the collision between two adjacent superstructures, a superstructure and an abutment, or a superstructure and a truncated portion of a pier head, shall not be less than the value obtained from Equation (14.4.1). However, for a complicated bridge in which the seismic behavior should be obtained by a dynamic method specified in Chapter 7, a relative displacement derived from the dynamic analysis shall be taken as $u_s$ in Equation (14.4.1).

$$S_B = \begin{cases} 
  u_s + L_A & \text{(between a superstructure and an abutment, or a superstructure and a truncated portion of a pier head)} \\
  c_B u_s + L_A & \text{(between two adjacent girders)} \end{cases} \quad (14.4.1)$$

where

- $S_B$: Length of spacing of adjacent decks at the ends of superstructures shown in Fig. 14.4.1 (mm)
- $u_s$: Maximum relative displacement between a superstructure and a substructure, generating at a position of calculation of the gap between adjacent decks when subjected to the Level 2 Earthquake Ground Motion (mm)
- $L_A$: Allowance of adjacent girders (mm)
- $c_B$: Gap modification factor on natural period difference. The values in Table 14.4.1 are based on natural period difference $\Delta T$ of the two adjacent girders.

### Table 14.4.1 Gap Modification Factor on Natural Period Difference of Adjacent Girders

<table>
<thead>
<tr>
<th>Ratio of Natural Period Difference to Longer Natural Period $\Delta T / T_1$</th>
<th>$c_B$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0 \leq \Delta T / T_1 &lt; 0.1$</td>
<td>1</td>
</tr>
<tr>
<td>$0.1 \leq \Delta T / T_1 &lt; 0.8$</td>
<td>$\sqrt{2}$</td>
</tr>
<tr>
<td>$0.8 \leq \Delta T / T_1 \leq 1.0$</td>
<td>1</td>
</tr>
</tbody>
</table>

Notes: Here, $\Delta T = T_1 - T_2$, and $T_1$ and $T_2$ represent the natural periods of two adjacent girders, respectively. However, $T_1$ is assumed equal to or greater than $T_2$. 

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14.4.2 Expansion Joints

(1) An expansion length of an expansion joint shall generally be greater than its design expansion length for Earthquake Ground Motion Level 1, as specified in (2). However, this provision may not be applicable if the lateral strength of the expansion joint necessary for Level 1 Earthquake Ground Motion is ensured, or a joint protector specified in Subsection 14.4.3 is installed to protect the expansion joint.

(2) Design expansion length during an earthquake shall generally be calculated by Equation (14.4.2). However, if the design expansion length specified in Subsection 4.2.2 of Part I Common Provisions is larger than the calculated one, the design length during an earthquake shall not be less than the value from the Common Provisions.

\[
L_E = \begin{cases} 
\delta R + L_A \quad \text{(between a superstructure and an abutment)} \\
C_B \delta R + L_A \quad \text{(between two adjacent girders)} 
\end{cases} 
\quad \ldots(14.4.2)
\]

where

- \(L_E\): Design expansion length during an earthquake (mm)
- \(L_A\): Expansion length allowance (mm)
- \(\delta R\): Relative displacement between a superstructure and a substructure, occurring at the expansion location when subjected to Level 1 Earthquake Ground Motion (mm)
\[ c_B \] : Gap modification factor on natural period difference. \( c_B \) takes the values from Table 14.4.1, depending on the natural period difference \( \Delta T \) of the two adjacent girders.

### 14.4.3 Joint Protectors

1. A joint protector capable of protecting the expansion joints against Level 1 Earthquake Ground Motion shall be designed. Also, it shall be set up in such a way that it does not interfere with the functions of the bearing supports and the expansion joints.

2. Design seismic forces in designing a joint protector shall generally be calculated from Equation (14.4.3). The stresses occurring in the joint protector due to the design seismic force shall not be greater than 1.5 times the allowable value.

\[
H_J = k_h R_d
\]

(14.4.3)

where
- \( H_J \) : Design seismic force of the joint protector (kN)
- \( k_h \) : Design horizontal seismic coefficient of Level 1 Earthquake Ground Motion specified in Subsection 6.3.3
- \( R_d \) : Dead load reaction (kN)

3. The gap between adjacent decks of a joint protector in the longitudinal direction to the bridge axis shall not be less than its design value specified in Subsection 4.2.2 of Part I Common Provisions and also not exceeding the allowable expansion length of the expansion joint.
Chapter 15 Verification of Seismic Performance of Bearing Support System

15.1 General

(1) Bearing support system capable of having functions required by Section 4.1 of the Part I Common Provisions shall be fundamentally designed for horizontal and vertical forces due to Level 1 and Level 2 Earthquake Ground Motion (referred as “Type B bearing supports” hereafter). In particular, the bearing support system of seismically-isolated bridges and bridges with horizontal force distributed structures shall conform to the above provision. However, in the case superstructures which do not mainly vibrate due to the restraints of abutments, or in the case Type B bearing supports cannot be adopted, a bearing support system together with excessive displacement stoppers may be designed in the following manner: Functions of the bearing support system shall be ensured for horizontal and vertical forces due to Level 1 Earthquake Ground Motion, and the bearing support system and the excessive displacement stoppers specified in Section 15.5 shall jointly resist the horizontal forces due to Level 2 Earthquake Ground Motion (referred as “Type A bearing supports” hereafter).

(2) Both Type A and Type B bearing supports shall be verified with the provisions shown in Section 15.3, and the design seismic forces specified in Section 15.2.

(3) In order to fully ensure functions of bearing support system, the bearing support system shall be designed with consideration of the structural details of the supports specified in Section 15.4, as well as superstructural details in Subsections 14.2.2 and 14.3.2.

15.2 Design Seismic Force for Performance Verification of Bearing Support System

(1) Design horizontal seismic forces of Type B bearing supports for Level 2 Earthquake Ground Motion shall be equal to the lateral strength of piers considering plastic behavior, or equal to the horizontal forces at the maximum response displacements of foundations considering plastic behavior.
(2) When Type A bearing supports are used, the design horizontal seismic force for Level 1 Earthquake Ground Motion shall correspond to the inertia force calculated by using the design horizontal seismic coefficients specified in Section 6.3.3.

(3) Design vertical seismic forces for both Type A and Type B bearing supports shall be obtained from Equations (15.2.1) and (15.2.2). However, when the $R_U$ value obtained from Equation (15.2.2) does not exceed $-0.3 R_D$ for Type B bearing supports, the $R_U$ value shall be taken as $-0.3 R_D$. Here, both downward seismic forces and downward reaction forces used in verifying the bearing support system shall be positive.

\[
R_L = R_D + \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \quad \text{---------------------------------------- (15.2.1)}
\]

\[
R_U = R_D - \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \quad \text{---------------------------------------- (15.2.2)}
\]

where

$R_L$ : Downward design vertical seismic force used in verification of bearing support system (kN)

$R_U$ : Upward design vertical seismic force used in verification of bearing support system (kN)

$R_D$ : Reaction force in the bearing support caused by the superstructural dead load (kN).

$R_{HEQ}$ : Vertical reaction force generated in the bearing support system when the design horizontal seismic force prescribed in the above (1) and (2) is applied on the bearing support in its axial direction (kN).

$R_{VEQ}$ : Vertical seismic force (kN) generated by the design vertical seismic coefficient $k_v$ which is obtained from the following Equation (15.2.3):

\[
R_{VEQ} = \pm k_v R_D \quad \text{---------------------------------------- (15.2.3)}
\]

$k_v$ : Design vertical seismic coefficient. When Type B bearing supports are used, $k_v$ shall be the product of the design horizontal seismic coefficient on the ground surface provided in Section 6.4.3 and the coefficient defined in Table 15.2.1. For Type A bearing supports, $k_v$ shall be the product of the design
horizontal seismic coefficient specified in Section 6.3.3 and the multiplying coefficients defined in Table 15.2.1.

Table 15.2.1 Multiplying Coefficients (k, can be obtained by multiplying the Coefficients by the Design Horizontal Seismic Coefficients)

<table>
<thead>
<tr>
<th>Multiplying Coefficients</th>
<th>Level 1 Earthquake Ground Motion (Type A Bearing Supports)</th>
<th>Level 2 Earthquake Ground Motion (Type B Bearing Supports)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level 1 Earthquake</td>
<td>Level 2 Earthquake</td>
</tr>
<tr>
<td></td>
<td>Ground Motion (Type A Bearing Supports)</td>
<td>Ground Motion (Type B Bearing Supports)</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.67</td>
</tr>
</tbody>
</table>

15.3 Performance Verification of Bearing Support System

(1) Verification of Type B bearing supports

Type B bearing supports subjected to the design horizontal seismic forces specified in Section 15.2 (1) and also the design vertical seismic forces in Section 15.2 (3) simultaneously, shall be verified in such a manner that the sectional forces occurring in the bearing support and the attached members do not exceed the ultimate strength. Here, the ultimate strength can be taken as 1.7 times the allowable stress.

Furthermore, Type B rubber bearing support or seismic isolation bearings shall be verified in such a way that in addition to the above provisions, the shear strain occurring in the bearing support does not exceed the allowable value, and also the safety of the bearing support against buckling shall be verified. Here, the allowable shear strain shall be properly determined according to the characteristics of rubber bearings or base isolation bearings actually used, in order to protect them from shear failure. Stiffness allowance of the bearing support used shall be within ±10% of the design value.

(2) Verification of Type A bearing supports

Type A bearing supports subjected to the design horizontal seismic forces specified in Section 15.2 (2) and also the design vertical seismic forces in Section 15.2 (3)
simultaneously, shall be verified in such a manner that the sectional forces occurring in the bearing support and the attached members shall not exceed the ultimate strength. Here, the ultimate strength can be taken as 1.5 times the allowable stress.

Furthermore, Type A rubber bearing supports shall be verified in such a way that in addition to the above the shear strain occurring in the bearing support does not exceed the allowable value, and also the safety of the bearing support is ensured. Here, the allowable shear strain shall be properly determined according to the characteristics of the rubber bearing actually used, in order to protect them from shear failure.

15.4 Structural Details of Bearing Support System

(1) Type B bearing support shall be positively fixed to the superstructure and substructure, and therefore be structurally functional.

(2) Sections of the superstructure or substructure connecting with a bearing support shall be sufficiently strengthened, and therefore be capable of withstanding seismic forces.

(3) In order to protect a bearing support from brittle failure, highly ductile materials shall be used in these parts, and stress concentration shall be minimized.

15.5 Excessive Displacement Stopper

(1) Excessive displacement stoppers shall be verified so that the sectional forces due to design seismic forces obtained from Equation (15.5.1) do not exceed the ultimate strength. Here, the ultimate strength can be taken as 1.5 times the allowable stress.

\[ H_s = 3 k_h R_D \]  

(15.5.1)

where

- \( H_s \) : Design seismic force used in the design of the excessive displacement stopper (kN)
- \( k_h \) : Design horizontal seismic coefficient of Level 1 Earthquake Ground Motion,
(2) Allowance length of an excessive displacement stopper shall be greater than its design value. The design value shall approximately be equal to the deformation limit of the bearing support in the longitudinal direction to the bridge axis. The excessive displacement stopper may have the functions of a joint protector at the same time, and in this case the design allowance length can be taken as the movement of the bearing support specified in Section 4.1.3 of Part I Common Provisions. However, the allowance length of the excessive displacement stopper shall not be greater than the allowable expansion length of the expansion joint.

(3) Excessive displacement stoppers shall not interfere with the functions of the bearing support, such as translational and rotational movements of the support.

(4) Excessive displacement stoppers shall allow easy maintenance and inspection of the bearing support.

(5) The location of excessive displacement stoppers shall not affect the functions of an unseating prevention structure specified in Chapter 16.
Chapter 16 Unseating Prevention System

16.1 General

(1) Unseating prevention systems consist of a seating length of the girder at the support, unseating prevention structure, excessive displacement stopper, and structure for protecting superstructure from subsidence. These components shall be appropriately selected in accordance with the bridge type, type of bearing supports, ground conditions, and others.

(2) As for unseating prevention systems in the longitudinal direction to the bridge axis, the seating length specified in Section 16.2 and unseating prevention structure specified in Section 16.3 shall be provided at the girder-end supports and at the movement joints. The unseating prevention structure can be omitted for a bridge in which longitudinal displacement is unlikely to occur in the bridge axis due to its structural characteristics, except those bridges shown in 2) of Section 16.5 (1), or those located on seismically unstable ground, as specified in Chapter 8.

(3) As for the unseating prevention systems in the transverse direction to the bridge axis, excessive displacement stoppers shall be installed at girder-end supports, and movement joints of a bridge specified in Section 16.5 (1), and at intermediate supports of a continuous girder of a bridge specified in Section 16.5 (2).

(4) When a tall bearing support is used for Class B bridges specified in Section 2.3, special consideration, including the introduction of a structure for protecting the superstructure from subsidence as specified in Section 16.4, shall be given.

16.2 Seat Length

(1) The seating length of a girder at its support shall not be less than the value obtained from Equation (16.2.1). In the case where the length is shorter than that obtained from Equation (16.2.2), the design seating length shall not be less than the value from the latter equation. When the direction of soil pressure acting on the substructure differs from the longitudinal direction to the bridge axis, as in cases of
a skew bridge or a curved bridge, the seat length shall be measured in the direction perpendicular to the front line of the bearing support.

\[ S_E = u_s + u_G \geq S_{EM} \]  \hspace{2cm} (16.2.1)

\[ S_{EM} = 0.7 + 0.005 \lambda \]  \hspace{2cm} (16.2.2)

\[ u_G = \varepsilon_G L \]  \hspace{2cm} (16.2.3)

where

- \( S_E \): Seating length of a girder at the support (m). \( S_E \) is the girder length from the girder end to the edge of the substructure crown, or the girder length on the movement joint, as shown in Fig. 16.2.1.

- \( u_s \): Maximum relative displacement between the superstructure and the substructure crown due to Level 2 Earthquake Ground Motion (m). In calculating \( u_s \), the effects of the unseating prevention structure and the excessive displacement stopper shall not be considered. When soil liquefaction and liquefaction-induced ground flow as specified in Chapter 8 may affect displacement of the bridges, such effects shall be considered.

- \( u_G \): Relative displacement of the ground caused by seismic ground strain (m)

- \( S_{EM} \): Minimum seating length (m)

- \( \varepsilon_G \): Seismic ground strain, \( \varepsilon_G \) can be assumed as 0.0025, 0.00375 and 0.005 for Ground Types I, II and III, respectively.

- \( L \): Distance between two substructures for determining the seating length (m).

- \( I \): Span length (m). When two superstructures with different span lengths are supported on one bridge pier, the longer one shall be taken as \( I \).

(2) In case of verifying seismic performance of a bridge with complicated dynamic structural behavior by a dynamic analysis as specified in Chapter 7, the maximum
relative displacement obtained from the dynamic analysis shall be taken as $u_0$ in Equation (16.2.1).

(2) For a skew bridge with superstructural shape meeting Equation (16.5.1), the seating length shall satisfy the provisions in (1) and be calculated by Equation (16.2.4). For an asymmetric skew bridge in which the two front lines of the bearing supports at both ends of the superstructure are oblique, $S_{E,\theta}$ shall be calculated with use of a smaller skew angle.

$$S_{E,\theta} \geq (L_\theta / 2) (\sin \theta - \sin (\theta - \alpha_E))$$  \hspace{1cm} (16.2.4)

where

- $S_{E,\theta}$: Seating length for the skew bridge (m)
- $L_\theta$: Length of a continuous superstructure (m)
- $\theta$: Skew angle (degree)
- $\alpha_E$: Limit of unseating rotation angle (degree), $\alpha_E$ can generally be taken as 5 degrees.

(4) For a curved bridge with superstructural shape meeting Equation (16.5.2), the seating length shall satisfy the provisions in (1) and be calculated by Equation (16.2.5).

$$S_{E,\phi} \geq \delta_E \left( \frac{\sin \phi}{\cos(\phi/2)} \right) + 0.3$$  \hspace{1cm} (16.2.5)

$$\delta_E = 0.005 \phi + 0.7$$  \hspace{1cm} (16.2.6)

where

- $S_{E,\phi}$: Seating length for the curved bridge (m)
- $\delta_E$: Displacement of the superstructure toward the outside direction of the curve (m)
- $\phi$: Fan-shaped angle by the two ends of a continuous girder of a curved bridge (degrees)

### 16.3 Unseating Prevention Structure

(1) Ultimate strength of an unseating prevention structure shall not be less than the design seismic force determined by Equation (16.3.1). Here, the ultimate strength may be taken as 1.5 times the allowable stress. In addition, the design allowance
length of the unseating prevention structure shall be taken as large as possible, but
within the value given by Equation (16.3.2).

\[
H_F = 1.5 R_d \hspace{10cm} (16.3.1)
\]

\[
S_F = c_F S_E \hspace{10cm} (16.3.2)
\]

where

- \( H_F \): Design seismic force of the unseating prevention structure (kN)
- \( R_d \): Dead load reaction (kN). In case of a structure connecting two adjacent
girders, the larger reaction shall be taken.
- \( S_F \): Maximum design allowance length of unseating prevention structure (m)
- \( S_E \): Seating length specified in Section 16.2 (m).
- \( c_F \): Design displacement coefficient of unseating prevention structure. The
  standard value can be taken as 0.75.

(2) The unseating prevention structure shall not disturb the functions of bearing
supports, such as translational and rotational movements of the supports.

(3) The unseating prevention structure shall be capable of moving in the transverse
direction to the bridge axis and also alleviating the seismic impacts.

(4) A structure for fixing an unseating prevention structure shall be capable of
transferring its design seismic force to both superstructure and substructure.

(5) The unseating prevention structure shall allow for easy maintenance and inspection
of the bearing supports.

16.4 Structure for Protecting Superstructure from Subsidence

Structure for protecting a superstructure from settling of an approach bank shall be
capable of keeping the superstructure at an appropriate height, even if the bearing
supports sustain seismic damages.
16.5 Excessive Displacement Stopper

(1) For the following bridges excessive displacement stoppers working in the perpendicular direction to the bridge axis, shall be installed in the girder-end supports, in addition to the unseating prevention system working in the longitudinal direction to the bridge axis.

1) Skew bridges with a small skew angle meeting Equation (16.5.1)
\[ \sin \frac{2\theta}{2} > \frac{b}{L} \]  \hspace{1cm} (16.5.1)

where
- \( L \): Length of a continuous superstructure (m)
- \( B \): Whole width of the superstructure (m)
- \( \theta \): Skew angle (degree)

2) Curved bridges satisfying Equation (16.5.2)
\[ \frac{115}{\phi} \frac{1 - \cos \phi}{1 + \cos \phi} > \frac{b}{L} \]  \hspace{1cm} (16.5.2)

where
- \( L \): Length of a continuous superstructure (m)
- \( B \): Whole width of the superstructure (m)
- \( \phi \): Fan-shaped angle (degree)

3) Bridges with narrow substructure crown in the bridge axis direction
4) Bridges with a small number of bearing supports on a substructure
5) Bridges with piers likely to move in the perpendicular direction to the bridge axis as a result of liquefaction-induced ground flow specified in Section 8.3.

(2) Bridges specified in 3), 4) and 5) of the above (1) shall have excessive displacement stoppers installed at intermediate supports.

(3) Excessive displacement stoppers in the perpendicular direction to the bridge axis shall be designed in accordance with the provisions in Section 15.5.