SPECIFICATIONS FOR HIGHWAY BRIDGES

PART Ⅴ SEISMIC DESIGN
Interim English Translation Version

道路橋示方書　Ⅴ耐震設計編
（英訳暫定版）

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Japan Road Association, Working Group for English Translation of the Specifications for Highway Bridges

（社）日本道路協会　英文示方書 WG

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2019年９月

日本道路協会橋梁委員会英文示方書 WG

Precautions for interim English translation of the Specifications for Highway Bridges

The Specifications for Highway Bridges are set forth by the Japanese Ministry of Land, Infrastructure, Transport and Tourism and issued to road administrators. All provisions of Part I to Part V package a series of fundamental and mandated performance requirements and corresponding widely accepted standards deemed to satisfy the requirements. The mandatory parts include the performance matrix and the required achievement levels for reliability where the level of load combinations and the level of resistance reliability are stipulated. The Japan Road Association (JRA) has a Technical Committee for Bridges comprising specialists from the Ministry of Land, Infrastructure, Transport and Tourism and related agencies, different road administrators, and specialists from the academic and industry sectors to study the future codes of practice and to develop technical guidance books, handbooks, and references to supplement the Specifications for Highway Bridges. In particular, the Japan Road Association publishes commentaries on the Specifications for Highway Bridges as the *Specifications for Highway Bridges and Commentaries*.

With this background, the Japan Road Association publishes an English translation of the *Specifications for Highway Bridges and Commentaries* and plans to publish an English translation of the latest 2017 version of the *Specifications for Highway Bridges and Commentaries*. However, the publication will take longer to translate because the 2017 version was a comprehensive revision. For this reason, the mandatory parts issued by the Ministry of Land, Infrastructure, Transport and Tourism will be translated first and sequentially made public part by part on the Japan Road Association website for the purpose of reference.

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The Association would appreciate any suggestions on the English translation that will improve the quality of the English translation version.

September 2019

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V SEISMIC DESIGN

CHAPTER 1 GENERAL

1.1 Scope

This Part shall apply to design that will be performed to satisfy the bridge performance specified in Clause 1.8 of Part I (hereafter, referred to as "the seismic design of a bridge") under the situation in which the effects of earthquake are to be considered.

1.2 Definition

The terms used in this part are defined as below:

(1) Limit value
Value that is determined by considering an appropriate safety allowance for which a limit state of the bridge or its member or the like can be regarded as not exceeded

(2) Plastic behavior of a member
Transition in which the response of the member exceeds the elastic range

(3) Seismic horizontal strength
Horizontal strength that can be delivered by the member under repeated loading due to the effects of earthquake after it has developed plastic behavior

(4) Ductility
Capability of deformation of the member while retaining a horizontal force stably under repeated loading due to the effects of earthquake after it has developed plastic behavior

(13) Seismically Isolated bridges
Bridge for which the effects of both moderate elongation of the natural period of the bridge with the aid of bearings and reduction in the responses of members by energy absorption using bearings are considered in its seismic design; Bearings that bring about the effects are called seismic isolation bearings.

1.3 Investigation

In performing the seismic design of a bridge, not only shall the provisions specified on investigations in each Part be complied with, but at least some investigations among the investigations 1) to 3) also shall be performed according a plan for obtaining necessary information on matters necessary for the seismic design in order to carry out the design of the load carrying performance of superstructure, substructure, connection between superstructure and substructure, members and the like, and other necessary matters, and also materials required for the design, conditions for maintenance and management, and conditions for construction will appropriately be considered in design.

1) Investigation of conditions of the bridge site environment

2) Investigation of construction conditions

3) Investigation of maintenance and management conditions

1.4 Matters to Be Considered in Seismic Design on the Selection of Bridge Site and Type

In performing the seismic design of a bridge, a bridge site or a bridge type shall generally be selected appropriately so that the bridge will not be affected by the tsunamis, slope collapse and the like, and fault displacement that can be brought about by supposed earthquakes. If a bridge site or bridge type that will be affected by those events is unavoidably selected, necessary measures, such as use of a structure that at least does not easily suffer fatal damage, shall be taken in order to ensure consistency also with the regional disaster prevention plan and the like.

1.5 Accuracy of Design Calculation

(1) The accuracy of design calculation shall appropriately be determined according to the design conditions.

(2) Design calculation shall generally be made to three significant figures at the final stage.

1.6 Material Conditions Required by Design

(1) From the perspective of the relationship with conditions, such as environmental conditions in which materials are exposed, construction conditions, and maintenance and management conditions, materials to be used shall be those of which mechanical and chemical characteristics required by design are clearly shown, and for which quality needed can be ensured.

(2) The characteristics of materials to be used shall be represented by measurable physical

quantities.

(3) Materials to be used for steel members shall comply with the provisions of Part II, those to be used for concrete members shall comply with the provisions of Part III, and those to be used for substructures shall comply with the provisions of Part IV.

1.7 Construction Conditions Required by Design

(1) In performing the seismic design of a bridge, construction conditions required by design shall appropriately be considered.
(2) The provisions of this Part are specified on the premises that steel members satisfy the provisions of Chapter 20 of Part II, that concrete members satisfy those of Chapter 17 of Part III, and that members composing substructures satisfy those of Chapter 15 of Part IV. Accordingly, if those provisions are difficult to satisfy, construction conditions shall appropriately be set, and shall be considered in design.

1.8 Maintenance and Management Conditions Required by Design

(1) In performing the seismic design of a bridge, maintenance and management conditions required by design shall appropriately be considered.

(2) In considering the maintenance and management conditions required by design, at least the following 1) and 2) shall be considered.

1) Viewpoint of ease of the maintenance and management of members to be installed for seismic design, such as an unseating prevention structure

2) Viewpoint of the influence of members to be installed for seismic design, such as an unseating prevention structure, on ease of the maintenance and management of other members and the like

1.9 Matters to be Described in Design Documents

(1) Items necessary for execution of construction work and maintenance shall be described in design documents.

(2) In design drawings and the like, at least the following matters 1) to 6) in addition to the matters specified in Clause 1.9 of Part I shall generally be described.

1) Matters regarding materials to be used

2) Construction method and procedures required by design

3) Construction quality (construction accuracy and inspection criteria) required by design

4)

Matters regarding the maintenance and management considered in design; particularly, the members and portions from which plastic behavior under an earthquake is expected, and feasibility of supposed repair to those members and portions

5) Technical standards and the like that were applied in design

6) Matters regarding ground condition

CHAPTER 2　BASICS OF SEISMIC DESIGN OF A BRIDGE

2.1　General

(1) The seismic design of a bridge shall be performed in such a manner that the bridge performance specified in Clause 1.8 of Part I will be satisfied.

(2) In performing the seismic design of a bridge, it shall be classified as either of the two classes shown in Table-2.1.1 according to its importance in seismic design by considering its role in the society after an earthquake, its position in the regional disaster prevention plan. The two classes are the class of bridges of standard importance in seismic design and the class of bridges of particularly high importance in seismic design (hereafter, referred to as "Class A bridges" and "Class B bridges," respectively).

Table2.1.1 Classification of Importance of Bridges in Seismic Design

Classification of importance of bridges in seismic design	Definitions
Class A bridges	Bridges other than Class B bridges
Class B bridges	・ Bridges of National expressways, urban expressways, designated urban expressways, Honshu-Shikoku highways, and general national highways. ・Particularly important bridges among bridges in prefectural roads from the perspective of double sections, railway bridges, bridge over roadways, or positions in regional disaster prevention plans and usage of the relevant roads, and other factors ・Particularly important bridges among bridges in municipal roads from the perspective of double sections, railway bridges, bridge over roadways, or positions in regional disaster prevention plans and usage of the relevant roads, and other factors

(3) In performing the seismic design of a bridge, the following provisions 1) to 3) shall be satisfied.

1) When the load carrying performance of the bridge is to be represented by the load carrying

performance of a superstructure, a substructure, and the connection between the superstructure and the substructure, the superstructure, the substructure, and the connection between the superstructure and the substructure shall at least have the load carrying performance necessary for satisfying the load carrying performance of a bridge specified in Clause 2.3 of Part I.

2) When the load carrying performance of a superstructure, a substructure, and the connection between the superstructure and the substructure is to be represented by the load carrying performance of the members and the like that compose those structures, the members and the like shall at least have the load carrying performance necessary for satisfying the load carrying performance of a bridge specified in Clause 2.3 of Part I.

3) Other matters necessary for satisfying the performance of the bridge shall appropriately be set, and performance necessary for those matters shall be achieved.

(4) Structural analysis methods for estimating the effects of earthquake among the design methods specified in Clause 1.8.2 of Part I shall generally comply with Clause 2.6.

2.2 Basic Matters Regarding Load Carrying Performance

2.2.1 Situations to Be Considered in Verification of Load Carrying Performance

For the seismic design of a bridge, the design situation including the effects of earthquake specified in Clause 3.1 of Part I is considered in the dominance situation by variable action and dominance situation by accidental action specified in Clause 2.1 of Part I in performing verification of the load carrying performance of superstructures, substructures, and the connections between superstructures and substructures and in verification of members and the like.

2.2.2 States to Be Considered in Verification of Load Carrying Performance

(1) For the seismic design of a bridge, the states of superstructures, substructures, and the connections between superstructures and substructures that are to be considered to satisfy the state of the bridge specified in Clause 2.2 of Part I shall be set according to the classifications 1) to 3).

1) State in which the superstructure, substructure, or connection between a superstructure and a substructure has not developed degradation in load-supporting capacity and can be used without any extra caution as to load-carrying capacity

2) State in which the superstructure, substructure, or connection between a superstructure and a

substructure has developed degradation in load-supporting capacity but can be used with extra caution within a range supposed for load-carrying capacity in advance because the degree of the degradation is limited

3) State in which the superstructure, substructure, or connection between a superstructure and a substructure has not completely lost load-supporting capacity

(2) For verification of the load carrying performance of members and the like, the states of the members and the like that are to be considered to satisfy the state of the bridge specified in Clause 2.2 of Part I shall be set according to the classifications 1) to 3).

1) State in which the member or the like has not developed degradation in load-supporting capacity

2) State in which although the member or the like has developed degradation in load-supporting capacity, the degree of the degradation is limited and within a range supposed in advance

3) State in which the member or the like has not completely lost load-supporting capacity

2.2.3 Load Carrying Capacity

(1) In performing the seismic design of a bridge, superstructures, substructures, the connections between superstructures and substructures, and members and the like shall be designed to stay with required reliability during the design service life in a state that is set in Clause 2.2.2 to be considered in verification of load carrying performance corresponding the situation that is set in Clause 2.2.1 to be considered in verification of load carrying performance so that the load carrying performance of a bridge specified in Clause 2.3 of Part I will be satisfied.

(2) When Clauses 2.3 to 2.5 are complied with, the provision (1) is deemed to be satisfied.

2.3 Situations for which the Effects of Earthquake Are to Be Considered in Verification of Load Carrying Performance

(1) For the seismic design of a bridge, the situations specified in Clause 2.2.1 shall appropriately be set in performing verification of the load carrying performance of superstructures, substructures, the connections between superstructures and substructures, and members and the like, at least by using the characteristic values of actions, a combination of actions, a load combination factor, and load factors according to the Clause 3.2 of Part I.

(2) As the effects of earthquake (EQ) specified in Clause 8.19 of Part I, the effects of the following 1) to 5) shall generally be considered.

1) Inertia force caused by the weight of the structure and that of soil (hereafter, referred to as "inertia force")

2) Seismic earth pressure

3) Seismic hydrodynamic pressure

4) Ground vibration displacement

5) Effects of lateral spreading due to liquefaction (hereafter, referred to as "lateral spreading force of the ground")

(3) The characteristic values of the effects of earthquake specified in (2)1) to (2)5) shall appropriately be set separately for the dominance situation by variable action and the dominance situation by accidental action on the basis of the characteristic value of the earthquake ground motion to be considered as an earthquake ground motion acting on the bridge.

(4) In case the characteristic values of the earthquake ground motion acting on the bridge are set, for an earthquake ground motion acting on a ground surface that shall be appropriately defined as a surface above which the inertia force is taken into account, and below which the inertia force is not taken into account (hereafter, referred to as "the ground surface in seismic design").

(5) The characteristic values of the earthquake ground motion acting on the bridge shall be set as inputs to the ground surface in seismic design.

(6) The characteristic values of the earthquake ground motion acting on the bridge shall be set according to the provisions of Chapter 3.

(7) The effects of earthquake specified in (2)1) to (2)5) shall be considered according to the following provisions 1) to 5).

1) The inertia force shall be calculated according to the provisions of Clause 4.1.

2) The earth pressure during an earthquake shall be calculated according to the provisions of Clause 4.2.

3) The hydrodynamic pressure during an earthquake shall be calculated according to the provisions of Clause 4.3.

4) The effects of ground vibration displacement on the bridge shall appropriately be set according to structural conditions and ground conditions.

5) The lateral spreading force of the ground shall be calculated according to the provisions of Clause 4.4.

2.4 Limit States of Situations for which the Effects of Earthquake Are to Be Considered in Verification of Load Carrying Performance

2.4.1 General

(1) For verification of the load carrying performance of superstructures, substructures, the connections between superstructures and substructures(hereafter these are referred to as “each structure”), and members that compose each structure and the like in performing the seismic design of a bridge, the limits of the states that are specified in Clause 2.2.2 as states to be considered in verification of load carrying performance shall appropriately be set as the limit states of the each structure and the members that compose each structure and the like.

(2) For verification of the load carrying performance of a bridge in performing the seismic design of the bridge, when the Limit state 1, Limit state 2, and Limit state 3 of the bridge are to be represented by the limit states of each structure, the limit states of each structure shall generally and separately be set according to the provisions of Clauses 2.4.2 to 2.4.4 and then combined.

(3) For verification of the load carrying performance of each structure in performing the seismic design of a bridge, when the Limit state 1, Limit state 2, and Limit state 3 of them are to be represented by the limit states of the members and the like that compose them, the limit states of the members that compose each structure shall generally be set according to the provisions of Clauses 2.4.5 and then combined.

2.4.2 Limit States of Superstructures, Substructures, and Connections between Superstructures and Substructures Corresponding to Limit State 1 of a Bridge

In performing the seismic design of a bridge, when the Limit state 1 of the bridge specified in Clause 4.1 of Part I is to be represented by the limit states of each structure, the following 1) to 3) shall be complied with.

1) Superstructure

Limit state 1 of the superstructure specified in Clause 3.4.2 of Part II or Clause 3.4.2 of Part III

2) Substructure

Limit state 1 of the substructure specified in Clause 3.4.2 of Part IV

3) Connection between superstructure and substructure

Limit state 1 of the bearing support specified in Clause 10.1.4 of Part I when a bearing support is to be used

2.4.3 Limit States of Superstructures, Substructures, and Connections between Superstructures and Substructures Corresponding to Limit State 2 of a Bridge

In performing the seismic design of a bridge, when the Limit state 2 of the bridge specified in Clause 4.1 of Part I is to be represented by the limit states of each structure, the following 1) to 3) shall be complied with. However, when the limit state of a substructure is set to Limit state 2, the limit state of the connection between a superstructure and the substructure which combined with shall generally be set to Limit state 1; when the limit state of the connection between a superstructure and a substructure is set to Limit state 2, the limit state of the substructure which combined with shall generally be set to Limit state 1. 1) Superstructure Limit state 1 of the superstructure specified in Clause 3.4.2 of Part II or Clause 3.4.2 of Part III 2) Substructure Limit state 1 or Limit state 2 of the substructure specified in Clause 3.4.2 of Part IV 3) Connection between superstructure and substructure Limit state 1 or Limit state 2 of the bearing support specified in Clause 10.1.4 of Part I when a bearing support is to be used

2.4.4 Limit States of Superstructures, Substructures, and Connections between Superstructures and Substructures Corresponding to Limit State 3 of a Bridge

In performing the seismic design of a bridge, when the Limit state 3 of the bridge specified in Clause 4.1 of Part I is to be represented by the limit states of each structure, the following 1) to 3) shall be complied with. However, when the limit state of a substructure is set to Limit state 3, the limit state of the connection between a superstructure and the substructure which combined with shall generally be set to Limit state 1; when the limit state of the connection between a superstructure and a substructure is set to Limit state 3, the limit state of the substructure which combined with shall generally be set to Limit state 1. 1) Superstructure Limit state 1 or Limit state 3 of the superstructure specified in Clause 3.4.2 of Part II or Clause 3.4.2 of Part III 2) Substructure Limit state 1 or Limit state 3 of the substructure specified in Clause 3.4.2 of Part IV 3) Connection between superstructure and substructure

Limit state 1 or Limit state 3 of the bearing support specified in Clause 10.1.4 of Part I when a bearing support is to be used

2.4.5 General

(1) In performing the seismic design of a bridge, when the Limit state 1 of each structure that is specified in Clause 4.2 of Part I is to be represented by the limit states of members that compose each structure, the limit states of one of the members and the like that compose each structure shall generally be set to reached to the Limit state 1 of members and the like specified in Clause 2.4.6.

(2) In performing the seismic design of a bridge, when the Limit state 2 of each structure that is specified in Clause 4.2 of Part I is to be represented by the limit states of members that compose each structure, one of the members that compose the each structure shall generally reach the Limit state 2 of members specified in Clause 2.4.6 while no other member exceeds the Limit state 1. A member that will reach the Limit state 2 of members and the like shall generally be selected in such a manner that at least plastic behavior is expected from the member with the position and range of the plastic behavior allowing damage to easily be investigated and repaired.

(3) In performing the seismic design of a bridge, when the Limit state 3 of each structure that is specified in Clause 4.2 of Part I is to be represented by the limit states of members that compose each structure, one of the members that compose each structure shall generally reach the Limit state 3 of members specified in Clause 2.4.6 while no other member exceeds the Limit state 1.

2.4.6 Limit States of Members and the Like

(1) The Limit state 1 that is specified in Clause 4.3 of Part I for members and the like that compose each structure can be set comply with the provisions of Clause 3.4.3 of Part II, those of Clause 3.4.3 of Part III, and those of Clause 3.4.3 of Part IV.

(2) The Limit state 2 that is specified in Clause 4.3 of Part I for members and the like that compose each structure shall be the state of limit at which although the behavior of the member and the like loses reversibility, the load carrying ability can be ensured within expected scope.2)

(3) The Limit state 3 that is specified in Clause 4.3 of Part I for members and the like that compose each structure can be set comply with the provisions of Clause 3.4.3 of Part II, those of Clause 3.4.3 of Part III, and those of Clause 3.4.3 of Part IV.

(4) The limit sates of members and the like shall generally appropriately be associated with engineering indicators representing the states.

(5) When the effects of earthquake are to be considered, engineering indicators and limit states shall be associated with each other according to the provisions of Clause 3.4.1 of Part II, those of Clause 3.4.1 of Part III, and those of Clause 3.4.1 of Part IV, and, in setting characteristic values corresponding to the limit states, following 1) and 2) shall be satisfied.

1) Influences for the states of members and the like effecting repetitive action due to an earthquake shall be considered.

By using the method based of appropriate knowledge that strength of members and the like, nonlinear hysteresis properties, and failure modes which corresponding to each condition of members and the like can be considered.

(6) For each structure, and members and the like that compose each structure, when the characteristic values and limit values of engineering indicators are determined according to the provisions of Chapter 6, Chapter 8, and later chapters, the provision (4) and (5)may be regarded as satisfied.

2.5 Verification of Load Carrying Performance

(1) In performing the seismic design of a bridge, verification of the load carrying performance of each structure, and members that compose each structure and the like shall be performed by using an appropriate method to confirm that the load carrying performance specified in Clause 2.2.3 is satisfied.

(2) When verification of the load carrying performance of a bridge is to be represented by verification of the load carrying performance of members and the like according to the provisions of Clause 5 of Part I, the verification of the load carrying performance of the members and the like shall generally comply with the following provisions 1) and 2).

1) It shall be confirmed, by demonstrating that Inequalities (2.5.1) and (2.5.2) are satisfied for the combination of actions as specified in Clause 2.3(1), the Limit state 1 and Limit state 3 or Limit state 2 and Limit state 3 of the member or the like that are determined according to the load carrying performance of the member or the like as specified in Clause 2.4.6 are not exceeded, with required reliability in both cases.

$$\sum S_i(\gamma_{pi}\ \gamma_{qi}\ P_i) \leq \xi_1\ \phi_{RS}\ R_S \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2.5.1)$$

$$\sum S_i(\gamma_{pi}\ \gamma_{qi}\ P_i) \leq \xi_1\ \xi_2\ \phi_{RU}\ R_U \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2.5.2)$$

where

P_i: Characteristic value of action;

S_i: Effect of action, which is the response value of the member or the like calculated for the characteristic value of action;

R_S: Characteristic value related to the resistance of the member or the like and corresponding to the Limit state 1 or Limit state 2 of the member or the like;

R_U: Characteristic value related to the resistance of the member or the like and corresponding to the Limit state 3 of the member or the like;

γ_{pi}: Load combination factor;

γ_{qi}: Load factor;

ξ_1: Modifier for structural modeling uncertainties;

ξ_2: Modifier for the consequence of failure;

ϕ_{RS}: Resistance factor related to the resistance of the member or the like and corresponding to the Limit state 1 or Limit state 2 of the member or the like;

ϕ_{RU}: Resistance factor related to the resistance of the member or the like and corresponding to the Limit state 3 of the member or the like.

2) If an event to be used for representing a limit state of the member or the like is difficult to classify as either the Limit state 1 or Limit state 2 of the member or the like or the Limit state 3 of the member or the like, the relevant event shall be used for representing the Limit state 3 of the member or the like, and for the combination of actions as specified in Clause 2.3(1) it shall be confirmed, by demonstrating that Inequalities (2.5.2) is satisfied, that the Limit state 3 of the member or the like is not exceeded, with required reliability.

3) When the combination of actions (ix) specified in Clause 3.3(1) of Part I is considered, not the provisions 1) and 2) but the provisions of Clause 3.5 of Part II, those of Clause 3.5 of Part III, and those of Clause 3.5 of Part IV shall be complied with.

(3) The effect of action in Inequalities (2.5.1) and (2.5.2) shall be calculated according to the provisions of Clause 2.6 and those of Chapters 3, 4, and 5.

(4) The characteristic values of actions, load combination factors and load factors shall be set according to the provisions of Clause 2.3.

(5) The resistance factors and the characteristic values of resistance in Inequalities (2.5.1) and (2.5.2) shall be calculated according to the provisions of Chapter 6, Chapter 8, and later chapters.

(6) The modifier for structural modeling uncertainties in Inequalities (2.5.1) and (2.5.2) shall generally be set to 1.00 when the combination of actions (xi) specified in Clause 3.3(1) of Part I is considered.

(7) The modifier for the consequence of failure in Inequalities (2.5.1) and (2.5.2) shall be calculated according to the provisions of Chapter 6, Chapter 8, and later chapters.

(8) For the effects of ground vibration displacement on the member shall be complied with so that the limit states of the member or the like in a situation in which the effects of earthquake specified in 1) to 3) of Clause 2.3(2) are to be confirmed and shall be prevented from being exceeded, and structural measures shall be taken to allow the structure of the underground portion to have appropriate deformability.

(9) When in seismic designing of bridges members and the like that are expected plasticity are to be connected, the following provisions 1) to 3) shall be complied with for the connection among each structure.

1) The relationships between the limit states of superstructures, substructures, and the connections between superstructures and substructures and the limit states of the joints between each structure shall be clarified, and design shall be performed in such a manner that the entire system of those structures will deliver its required function.

2) Structures to be connected shall be designed to ensure the states required by the load-bearing capacity mechanism of each joint between each structure and the states required by the load-bearing capacity mechanism of each structure to be connected.

3) The joints between those structures shall be designed to be capable of reliably transferring the mutual sectional forces developed between the structures.

(10) In the seismic design of a bridge on the ground with a soil layer develop liquefaction, the effects of liquefaction on the bridge shall appropriately be considered. However, when the combination of actions (ix) specified in Clause 3.3 of Part I is considered, the effects of liquefaction on the bridge may be excluded from consideration.

(11) In case according to the provisions of Chapter 7, the effects on the bridge specified in the provision (10) may be regarded as appropriately considered.

(12) When the effects of liquefaction on the bridge are to be considered, the necessary bridge performance shall be achieved both in the case where liquefaction is assumed and in the case where liquefaction is not assumed.

(13) When the plastic behavior of the foundation is to be considered, necessary performance shall be achieved both in a case where the plastic behavior of the foundation is assumed and in a case where the plastic behavior of the foundation is not assumed.

2.6 Structural Analysis

For the calculation of response values, the purpose of verification, the vibration characteristics of the bridge and those of members and the like that compose the bridge, the resistance characteristics of the ground, and the like shall be considered, and an analysis theory and an analysis model that are capable of appropriately estimating the effects of earthquake shall be used in a range for which their applicability has been verified.

2.7 Other Necessary Matters

2.7.1 General

(1) In performing the seismic design of a bridge, examinations shall be performed on matters required for verification of load carrying performance of bridges and, the other, matters necessary for achieving the performance of the bridge in seismic design.

(2) As matters necessary for achieving (1), the following provisions 1) to 3) shall be satisfied.

1) When a bearing support is to be used in a connection between a superstructure and a substructure, an appropriate structural type shall be used so that the substructure can support the superstructure without losing stability in the event of destruction of the bearing support.

2) When a bearing support is to be used in a connection between a superstructure and a substructure, appropriate measures shall be taken so that the superstructure will not easily fall from the substructure in the event of destruction of the bearing support.

3) For Class B bridges, when a bearing support is to be used in a connection between a superstructure and a substructure, an examination shall be made on the necessity of measures that allow the function to be restored quickly in the event of its destruction. When the measures are necessary, the range in which they are feasible from the perspective of structural design shall be examined, and shall be applied to structural design on an as-needed basis.

(3) When measures are taken according to the provisions of Clause 13.3, the provision (2)2) may be regarded as satisfied.

2.7.2 Considerations for Structural Design

For the seismic design of a bridge, the range in which it is feasible from the perspective of structural design shall be examined at least from the viewpoints 1) to 5) with consideration given also to its economy and consistency with the regional disaster prevention plan and related road network plans, and shall be applied to structural design on an as-needed basis.

1) Viewpoint of the method for confirming that construction quality required by design

2) Viewpoint of structural complementarity or redundancy for coping with the possibilities of

damage to some of the members and connections between superstructures and substructures of the bridge, ground deformation, and the like; For this viewpoint, examinations shall be made at least on the following i) and ii).

i) Consideration for avoiding brittle failure of all members that are not expected plastic behavior

ii) Consideration for minimizing the effects of torsion on members

3) Viewpoint of minimizing locations in which inspections and repairs are difficult to make after an earthquake

4) Viewpoint of selecting members for which renewal and repair methods to be applied after an earthquake should be considered in advance, and of designing a bridge structure that allows the selection to reliably be made

5) Viewpoint of designing constructional details that do not allow local stress concentration, complex behavior, stagnant water, and the like to easily occur

CHAPTER 3 CHARACTERISTIC VALUES OF EARTHQUAKE GROUND MOTIONS THAT WILL ACT ON A BRIDGE

3.1 Setting of Characteristic Values of Earthquake Ground Motions

(1) In setting the situations for which the effects of earthquake are to be considered in verification of the load carrying performance as specified in Clause 2.3, the earthquake ground motion that often occurs during the design service life of the bridge (hereafter, referred to as "Level 1 Earthquake Ground Motion") and the earthquake ground motion that markedly rarely occurs during the design service life of the bridge but will have immense impacts on the bridge once occurring (hereafter, referred to as "Level 2 Earthquake Ground Motion") shall appropriately be set.

(2) In setting the characteristic values of earthquake ground motions, the following 1) to 3) shall be considered.

1) Characteristic of the earthquake ground motions, seismic response characteristic of the bridge, and effects of variations in those characteristics

2) Vibration characteristic of the ground and effects of variations in the characteristic

3) Intensity of earthquake ground motions according to the scales, occurrence locations, and the like of earthquakes that occur in the surrounding area of a bridge, and effects of variations in the intensity

(3) When the characteristic values of Level 1 and Level 2 Earthquake Ground Motions are set according to the provisions of Clauses 3.2 to 3.7, the provisions (1) and (2) may be regarded as satisfied.

3.2 Characteristic Value of Level 1 Earthquake Ground Motion

The characteristic value of Level 1 Earthquake Ground Motion shall be calculated from the acceleration response spectrum given by Equation (3.2.1) based on the ground type in seismic design that is classified according to the provisions of Clause 3.6 for the ground surface in seismic design specified in Clause 3.5.

$$S = c_Z S_0 \cdots\cdots \quad (3.2.1)$$

where

S : Acceleration response spectrum of Level 1 Earthquake Ground Motion (m/s^2) (The value shall be rounded off to two decimal places.)

c_Z : Modification factor for Level 1 Earthquake Ground Motions for zones specified in Clause 3.4

S_0 : Standard acceleration response spectrum of Level 1 Earthquake Ground Motion (m/s^2), which shall be the value of an acceleration response spectrum with a damping ratio of 0.05 that is specified in Table 3.2.1 as a value corresponding to the ground type and natural period T (s) specified in Clause 3.6

Table 3.2.1 Standard Acceleration Response Spectra S_0

Ground Type	S_o (m/s^2) with Natural Period T(s)		
Type I	$T < 0.10$ $S_o = 4.31T^{1/3}$ but $S_o \geqq 1.60$	$0.10 \leqq T \leqq 1.10$ $S_o = 2.00$	$1.10 < T$ $S_o = 2.20/T$
Type II	$T < 0.2$ $S_o = 427T^{1/3}$ but $S_o \geqq 200$	$0.20 \leqq T \leqq 1.30$ $S_o = 2.50$	$1.30 < T$ $S_o = 3.25/T$
Type III	$T < 0.34$ $S_o = 430T^{1/3}$ but $S_o \geqq 240$	$0.34 \leqq T \leqq 1.50$ $S_o = 3.00$	$1.50 < T$ $S_o = 4.50/T$

3.3 Characteristic Value of Level 2 Earthquake Ground Motion

(1) For the characteristic value of Level 2 Earthquake Ground Motion, the following two types of earthquake ground motions shall be considered: the earthquake ground motion corresponding to a large-scale interplate earthquake (hereafter, referred to as "Level 2 Earthquake Ground Motion (Type I)") and the earthquake ground motion corresponding to an inland near-field earthquake (hereafter, referred to as " Level 2 Earthquake Ground Motion (Type II) ").

(2) The characteristic values of Level 2 Earthquake Ground Motion (Type I) and Level 2 Earthquake Ground Motion (Type II) shall be calculated, respectively, from the acceleration response spectra given by Equations (3.3.1) and (3.3.2) based on the ground type in seismic design that is classified according to the provisions of Clause 3.6 for the ground surface in seismic design specified in Clause 3.5.

$$S_{\mathrm{I}} = c_{Iz} S_{I0} \cdots\cdots (3.3.1)$$

$$S_{\mathrm{II}} = c_{IIz} S_{II0} \cdots\cdots (3.3.2)$$

where

S_I : Level 2 Earthquake Ground Motion (Type I) acceleration response spectra (m/s^2) (The value shall be rounded off to two decimal places.)

S_{II} : Level 2 Earthquake Ground Motion (Type II) acceleration response spectra (m/s^2) (The value shall be rounded off to two decimal places.)

c_{Iz} :Modification factor for zones for Level 2 Earthquake Ground Motion (Type I) specified in Clause 3.4

c_{IIz} :Modification factor for zones for Level 2 Earthquake Ground Motion (Type II) specified in Clause 3.4

S_{I0} : Standard acceleration response spectrum of Level 2 Earthquake Ground Motion (Type I) (m/s^2), which is the value of an acceleration response spectrum with a damping ratio of 0.05 that is specified in Table 3.3.1 as a value corresponding to the ground type and natural period T (s) specified in Clause 3.6

S_{II0} : Standard acceleration response spectrum of Level 2 Earthquake Ground Motion (Type II) (m/s^2), which is the value of an acceleration response spectrum with a damping ratio of 0.05 that is specified in Table 3.3.2 as a value corresponding to the ground type and natural period T (s) specified in Clause 3.6

Table 3.3.1 Standard Acceleration Response Spectrum S_{I0} of Level 2 Earthquake Ground Motion (Type I)

Ground Type	S_{I0} (m/s^2) with Natural Period T(s)		
Type I	T<0.16 S_{I0}=25.79 $T^{1/3}$	0.16≦T≦0.6 S_{I0}=14.00	0.60 <T S_{I0}=8.40/T
Type II	T < 0.22 S_{I0}=21.53$T^{1/3}$	0.22≦T≦0.90 S_{I0}=13.00	0.90 < T S_{I0}=11.70/T
Type III	T < 0.34 S_{I0}=17.19$T^{1/3}$	0.34≦T≦1.40 S_{I0}=12.00	1.4 0< T S_{I0}=16.80/T

Table 3.3.2 Standard Acceleration Response Spectrum S_{II0} of Level 2 Earthquake Ground Motion (Type II)

Ground Type	S_{II0} (m/s^2) with Natural Period T(s)		
Type I	T < 0.30 S_{II0}=44.63$T^{2/3}$	0.30≦T≦0.70 S_{II0}=20.00	0.70 < T S_{II0}=11.04/$T^{5/3}$
Type II	T < 0.40 S_{II0}=32.24$T^{2/3}$	0.40≦T≦1.20 S_{II0}=17.50	1.20 < T S_{II0}=23.71/$T^{5/3}$
Type III	T < 0.50 S_{II0}=23.81$T^{2/3}$	0.50≦T≦1.50 S_{II0}=15.00	1.50 < T S_{II0}=29.48/$T^{5/3}$

3.4 Modification Factor for Zones

Modification factor for zones cz for Level 1 Earthquake Ground Motion, modification factor for zones cIz for Level 2 Earthquake Ground Motion (Type I), and modification factor for zones cIIz for Level 2 Earthquake Ground Motion (Type II) shall be determined in accordance with Table 3.4.1 for different zones. If the bridging point is on the boundary between zones, the larger modification factor shall be used.

Table 3.4.1 Zones and Modification Factors for Zones

Zone	Modification Factor			Definitions
	cz	*cIz*	*C IIz*	
A1	1.0	1.2	1.0	Chiba Pref. (covering only Tateyama City, Kisarazu City, Katsuura City, Kamogawa City, Kimitsu City, Futtsu City, Minamiboso City, Isumi-gun, Awa-gun) Kanagawa Pref. Yamanashi Pref. (covering only Fujiyoshida City, Tsuru City, Otsuki City, Uenohara City, Nishiyatsushiro-gun, Minamikoma-gun, and Minamitsuru-gun) Shizuoka Pref. Aichi Pref. (covering only Nagoya City, Toyohashi City, Handa City, Toyokawa City, Tsushima City, Kariya City, Nishio City, Gamagori City, Tokoname City, Inazawa City, Shinshiro City, Tokai City, Obu City, Chita City, Toyoake City, Tahara City, Aisai City, Kiyosu City, Yatomi City, Ama City, Ama-gun, Chita-gun, Nukata-gun, and Toei-cho town in Kitashitara-gun Mie Pref. (excluding Tsu City, Matsusaka City, Nabari City, Kameyama City, Inabe City, Iga City, Mie-gun, and Komono-cho) Wakayama Pref. (covering only Shingu City, Nishimuro-gun, and Higashimuro-gun) Tokushima Pref. (covering only Naka-gun and Kaifu-gun)
A2	1.0	1.0	1.0	Areas other than A1, B1, B2, and C
B1	0.85	1.2	0.85	Ehime Pref. (covering only Uwajima City, Kitauwa-gun,

					and Minamiuwa-gun) Kochi Pref. (excluding areas shown in B2) Miyazaki Pref. (covering only Nobeoka City, Hyuga City, Koyu-gun (excluding Nishimera-son and Kijo-cho), and Kadogawa-cho in Higashiusuki-gun)	
	B2	0.85	1.0	0.85	Hokkaido (covering only Sapporo City, Hakodate City, Otaru City, Muroran City, Kitami City, Yubari City, Iwamizawa City, Abashiri City, Tomakomai City, Bibai City, Ashibetsu City, Ebetsu City, Akabira City, Mikasa City, Chitose City, Takikawa City, Sunagawa City, Utashinai City, Fukagawa City, Furano City, Noboribetsu City, Eniwa City, Date City, Kitahiroshima City, Ishikari City, Hokuto City, Ishikari-gun, Matsumae-gun, Kamiiso-gun, Kameda-gun, Kayabe-gun, Futami-gun, Yamakoshi-gun, Hiyama-gun, Nishi-gun, Okushiri-gun, Setana-gun, Kudou-gun, Shimamaki-gun, Suttsu-gun, Isoya-gun, Abuta-gun, Iwanai-gun, Furuu-gun, Shakotan-gun, Furubira-gun, Yoichi-gun, Sorachi-gun, Yubari-gun, Kabato-gun, Uryu-gun, Kamikawa-gun (Kamikawa General Subprefectural Bureau) covering Higashikagura-cho, Kamikawa-cho, Higashikawa-cho, and Biei-cho, Yufutsu-gun, Abashiri-gun, Shari-gun, Tokoro-gun, Usu-gun, and Shiraoi-gun) Aomori Pref. (covering only Aomori City, Hirosaki City, Kuroishi City, Goshogawara City, Mutsu City, Tsugaru City, Hirakawa City, Higashitsugaru-gun, Nishitsugaru-gun, Nakatsugaru-gun, Minamitsugaru-gun, Kitatsugaru-gun, and Shimokita-gun) Akita Pref., Yamagata Pref. Fukushima Pref. (covering only Aizuwakamatsu City, Koriyama City, Shirakawa City, Sukagawa City, Kitakata City, Iwase-gun, Minamiaizu-gun, Yama-gun, Kawanuma-gun, Onuma-gun, and Nishishirakawa-gun) Niigata Pref.	

					Toyama Pref. (covering only Uozu City, Namerikawa City, Kurobe City, and Shimoniikawa-gun) Ishikawa Pref. (covering only Wajima City, Suzu City, and Housu-gun) Tottori Pref. (covering only Yonago City, Kurayoshi City, Sakaiminato City, Tohaku-gun, Saihaku-gun, and Hino-gun) Shimane Pref., Okayama Pref., Hiroshima Pref. Tokushima Pref. (covering only Mima City, Miyoshi City, Mima-gun, and Miyoshi-gun) Kagawa Pref. (covering only Takamatsu City, Marugame City, Sakaide City, Zentsuji City, Kanonji City, Mitoyo City, Shozu-gun, Kagawa-gun, Ayauta-gun, and Nakatado-gun) Ehime Pref. (excluding areas shown in B1) Kochi Pref. (covering only Nagaoka-gun, Tosa-gun, Agawa-gun (excluding the area of former Ino-cho in Ino-cho)) Kumamoto Pref. (excluding areas shown in C) Oita Pref. (excluding areas shown in C) Miyazaki Pref. (excluding areas shown in B1)	
	C	0.7	0.8	0.7	Hokkaido (covering only Asahikawa City, Rumoi City, Wakkanai City, Mombetsu City, Shibetsu City, Nayoro City, Kamikawa-gun (Kamikawa General Subprefectural Bureau) covering Takasu-cho, Toma-cho, Pippu-cho, Aibetsu-cho, Wassamu-cho, Kembuchi-cho, and Shimokawa-cho, Nakagawa-gun (Kamikawa General Subprefectural Bureau), Mashike-gun, Ruimoi-gun, Tomamae-gun, Teshio-gun, Soya-gun, Esashi-gun, Rebun-gun, Rishiri-gun, and Monbetsu-gun) Yamaguchi Pref., Fukuoka Pref., Saga Pref., Nagasaki Pref. Kumamoto Pref. (covering only Arao City, Minamata City, Tamana City, Yamaga City, Uto City, Kamiamakusa City, Amakusa City, Tamana-gun, Ashikita-gun, and	

					Amakusa-gun) Oita Pref. (covering only Nakatsu City, Bungotakada City, Kitsuki City, Usa City, Kunisaki City, Higashikunisaki-gun, and Hayami-gun) Kagoshima Pref. (excluding Amami City and Oshima-gun) Okinawa Pref.

3.5 Ground Surface in Seismic Design

The ground surface in seismic design shall be the top surface of the ground from which horizontal resistance during an earthquake is expected, and shall be set to one of the following 1) to 3) ground surfaces, whichever is the deepest.

1) Ground surface in design specified in Clause 8.4.2 of Part IV

2) Undersurface of the footing when the foundation is equipped with a footing

3) Undersurface of the soil layer when a soil layer from which subgrade reaction during an earthquake is not expected is present

However, when soil layers from which subgrade reaction during an earthquake is not expected are present as an alternate layer, the ground surface in seismic design shall be set to the top surface of the soil layer that is 3 m or more in thickness and present at the shallowest depth, and from which subgrade reaction is expected.

In this case, the soil layer from which subgrade reaction during an earthquake is not expected refers to a soil layer that exhibits zero for the coefficient of subgrade reaction, the upper limit of subgrade reaction, and the maximum unit skin friction (hereafter, they are referred to as "soil parameters in seismic design"), and shall fall under either of the following i) or ii).

i) A soil layer that is judged according to the provisions of Clause 7.2 to be one that will develop liquefaction that affects the bridge, and that have soil parameters in seismic design that are estimated to be zeros according to the provisions of Clause 7.3

ii) A cohesive soil layer that is present at a depth of 3 m or less from the ground surface, and that has an unconfined compressive strength estimated to be 20 kN/m^2 or smaller by an unconfined compression test or an in-situ test (hereafter, referred to as "very soft soil in seismic design")

3.6 Ground Type for Seismic Design

3.6.1 General

The ground types in seismic design shall be classified according to Table 3.6.1 by using TG, the fundamental natural period of the ground that is present in the range from the ground surface in seismic design specified in Clause 3.7 to the ground surface. When the ground surface in seismic design is set to the ground surface, its ground type in seismic design shall be classified as Type I.

Table 3.6.1 Ground Types in Seismic Design

Ground Type	Fundamental natural period of Ground, T_G (s)
Type I	$T_G < 0.2$
Type II	$0.2 \leqq T_G < 0.6$
Type III	$0.6 \leqq T_G$

3.6.2 Fundamental Natural Period of the Ground

(1) The fundamental natural period of the ground, TG, shall appropriately be calculated on the basis of subsurface investigation and the like.

(2) When the fundamental natural period of the ground, TG, is calculated from Equation (3.6.1), the provision (1) may be regarded as satisfied.

$$T_G = 4\sum_{i=1}^{n}\frac{H_i}{V_{si}} \quad \cdots\cdots \quad (3.6.1)$$

where

T_G : Fundamental natural period of the ground (s)

H_i : Thickness of the i-th soil layer (m)

V_{si} : Average shear 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 base ground surface for seismic design

(3) The mean shear wave velocity Vsi to be used for Equation (3.6.1) shall be determined by using an appropriate method to measure or estimate the shear wave velocity in the stratum at the bridge construction site.

(4) When the mean shear wave velocity Vsi is determined by direct measurement using an

appropriate method, such as elastic wave exploration and PS logging, or by estimation using Equation (3.6.2), the provision (3) may be regarded as satisfied.

For cohesive soil layers

$V_{si} = 100N_i^{1/3}$ $(1 \leq N_i \leq 25)$ · · · · · · · · · · · · · · · · (3.6.2)

For sandy soil layers

$V_{si} = 80N_i^{1/3}$ $(1 \leq N_i \leq 50)$

where

N_i: Mean N value of the *i*-th stratum determined by the standard penetration test

3.7 Ground Surface in Seismic Design

(1) The ground surface in seismic design shall be the top surface of the sufficiently strong ground that spreads uniformly across the bridge site, and that exists below the ground regarded as ground that develops vibration in the seismic design of the bridge.

(2) Highly stiff strata with a mean shear wave velocity of approximately 300 m/s or higher may be regarded as sufficiently strong ground specified in the provision (1).

CHAPTER 4 CHARACTERISTIC VALUES OF THE EFFECTS OF EARTHQUAKE

4.1 Inertia Force

4.1.1 General

(1) For inertia forces, structural systems that can be regarded as those that generate the same vibration due to the vibration characteristics of the bridge during an earthquake (hereafter, referred to as "design vibration unit") shall appropriately be set, and the intensity of the inertia forces shall appropriately be calculated for each design vibration unit while the direction in which the inertia forces acts are appropriately set.

(2) The intensity of horizontal inertia forces shall generally be calculated according to the provisions of 4.1.2 in dynamic analysis, and according to the provisions of 4.1.3 in static analysis.

(3) The directions in which horizontal inertia forces act shall be the direction in which the relevant member receives the greatest impact, and the direction perpendicular to that direction. The horizontal inertia forces shall independently be applied in each of those directions. The direction in which the relevant member receives the greatest impact, and the direction perpendicular to that direction shall generally be defined by the following provisions 1) to 4).

1) The directions of inertia forces acting on a pier shall be the direction in which bending moment is generated around the axis in which the moment of inertia on the pier is minimized, and the direction perpendicular to that direction.

2) The directions of inertia forces acting on an abutment shall be the direction in which the horizontal component of earth pressure acts, and the direction perpendicular to that direction.

3) The directions of inertia forces acting on the foundation shall be the directions of inertia forces acting on abutments or piers supported by the foundation.

4) The directions of inertia forces acting on a superstructure shall be the longitudinal and transverse directions to the bridge axis.

(4) For the following 1) or 2), not only shall the provision (3) be complied with, but vertical inertia forces shall also appropriately be considered.

1) Joint and joint between a bearing support and a superstructure or substructure

2) Pier that receives great eccentric moment due to a permanent action

(5) When the supporting point for supporting a superstructure on the top of a substructure is movable with respect to the directions in which inertia forces act, the provision (2) shall be excluded from consideration, and the following 1) and 2) instead of inertia forces acting on the superstructure shall be considered for the substructure.

1) Static frictional force on the bearing in a design situation in which Level 1 Earthquake Ground Motion is to be considered

2) The following forces in a design situation in which Level 2 Earthquake Ground Motion is to be considered: for piers, force that is determined by multiplying one-half of the dead-load reaction from the corresponding superstructure by the design horizontal seismic coefficient specified in Clause 4.1.6; for abutments, static frictional force on the corresponding bearing

4.1.2 Inertia Forces in Dynamic Analysis

(1) In dynamic analysis, the intensity of inertia forces shall be determined by multiplying the mass of the structure by its response acceleration on the basis of an accelerogram appropriately that is set for the dynamic analysis by considering the intensity, spectral characteristics, phase characteristics, and duration of Level 1 and Level 2 Earthquake Ground Motions, the damping ratio of the bridge, and the like.

(2) When the provisions (3) to (5) are complied with, the accelerogram for the provision (1) may be regarded as appropriately set.

(3) An accelerogram to be used for the dynamic analysis shall be obtained by making an amplitude adjustment to an existing typical strong earthquake motion record in such a manner so as to provide the accelerogram with the same characteristics as of the acceleration response spectra of Level 1 Earthquake Ground Emotions calculated from Equations (3.2.1) and Level 2 Earthquake Ground Motions calculated from Equations (3.3.1) and (3.3.2). If the damping ratio of the bridge substantially deviates from 0.05, the acceleration response spectra that are obtained by multiplying those calculated from Equations (3.2.1), (3.3.1), and (3.3.2) by the modification factor for damping ratio c_D calculated from Equation (4.1.1) shall be used as the acceleration response spectra Level 1 and Level 2 Earthquake Ground Motions.

$$c_D = \frac{1.5}{40h + 1} + 0.5 \quad \cdots\cdots\cdots\cdots\cdots\cdots (4.1.1)$$

h: Damping ratio

(4) In selecting a strong earthquake motion record to be subjected to amplitude adjustment, the following provisions 1) and 2) shall be considered. In a design situation in which Level 2 Earthquake Ground Motion is to be considered, at least three accelerograms that are subjected to amplitude adjustment and different in phase characteristics shall be used. In a design situation in which Level 1 Earthquake Ground Motion is to be considered, one accelerogram shall be used.

1) The acceleration response spectrum of the strong earthquake motion record to be subjected to amplitude adjustment shall have characteristics similar to those of the target acceleration response

spectrum.

2) When the plastic behavior of members is to be considered, the strong earthquake motion record shall have a substantial influence on the nonlinear response of the bridge from the following characteristics of i) and ii).

i) For Level 2 Earthquake Ground Motion (Type I), the strong earthquake motion record shall have the phase characteristics in which the repeat of earthquake ground motions with long duration has a substantial influence on the nonlinear response of the bridge.

ii) For Level 2 Earthquake Ground Motion (Type II), the strong earthquake motion record shall have the phase characteristics in which earthquake ground motions with short duration but with a large amplitude have a substantial influence on the nonlinear response of the bridge.

(5) In calculating inertia forces, the same accelerogram for Level 1 Earthquake Ground Motion and the same accelerogram for Level 2 Earthquake Ground Motion shall generally be used for each design vibration unit.

4.1.3 Inertia Forces in Static Analysis

In static analysis, the intensity of inertia forces shall be determined by calculating the natural period of design vibration unit specified in Clause 4.1.5 and the design horizontal seismic coefficient specified in Clause 4.1.6, and then, multiplying the mass of the structure by the design horizontal seismic coefficient.

4.1.4 Design Vibration Unit

(1) Design vibration units shall appropriately be set by dividing the bridge into structural systems that allow inertia forces to be calculated on the assumption that each of them generates the same vibration during an earthquake, with consideration given to the stiffness and heights of piers and abutments, the characteristics of the foundation and its surrounding ground, and the effects of the characteristics and support conditions of superstructures on the vibration characteristics of the bridge.

(2) When design vibration units are set according to the following provisions 1) to 3), the provision (3) may be deemed to be satisfied.

1) When connected superstructures are supported on the tops of multiple substructures by fixed or elastic support in a direction in which inertia forces act, the structural system consisting of the multiple substructures and the superstructure portion supported by the substructures shall be regarded as one design vibration unit with respect to the direction in which the inertia forces act.

2) When a superstructure is supported on the top of one substructure by fixed or elastic support in a direction in which inertia forces act, the structural system consisting of the substructure and the superstructure portion supported by the substructure shall be regarded as one design vibration unit with respect to the direction in which the inertia forces act.

3) When a superstructure is supported on the top of one substructure by movable support in a direction in which inertia forces act, the structural system consisting only of the substructure shall be regarded as one design vibration unit with respect to the direction in which the inertia forces act.

4.1.5 Natural Period of Design Vibration Unit

(1) The natural period of a design vibration unit shall appropriately be calculated by considering the effects of deformation of individual members and the like that compose the bridge.

(2) When the natural period of a design vibration unit is calculated by an eigenvalue analysis using a model appropriately created according to the provisions of Clause 2.6, the provision (1) may be deemed to be satisfied. In this case, if there exist a very soft soil layer in seismic design according to the provision of Clause 3.5 and a soil layer judged to be one that will develop liquefaction according to the provision of Clause 7.2, the natural period in seismic design shall be calculated without reducing the values of soil parameters. In static analysis, however, the natural periods of some design vibration units may be calculated according to the following provision 1) or 2).

1) When the design vibration unit consists of one substructure and a superstructure portion supported by the substructure or only of one substructure, its natural period shall be calculated from Equation (4.1.2).

$$T = 2.01\sqrt{\delta} \qquad \cdots\cdots \qquad (4.1.2)$$

where

T : Natural period of the design vibration unit (s)

δ : Displacement that occurs at the height of the inertia force acting on the superstructure, when a force equivalent to the sum of 80% of the weight of the substructure above the ground surface in seismic design and the total weight of the superstructure portion supported by the substructure is applied in the direction of the inertia force (m)

2) When the design vibration unit consists of multiple substructures and a superstructure portion

supported by them, its natural period shall be calculated from Equation (4.1.3).

$$T = 2.01\sqrt{\delta} \quad \cdots\cdots \quad (4.1.3)$$

$$\delta = \frac{\int w(s)\ u(s)^2\, ds}{\int w(s)\ u(s)\ ds} \quad \cdots\cdots \quad (4.1.4)$$

where

$w(s)$:Weight of the superstructure or the substructure at position s (kN/m)

$u(s)$: Displacement that occurs at position s in the direction in which the inertia force acts, when a horizontal force equivalent to the sum of the weight of the superstructure and the weight of the substructure above the ground surface in seismic design is applied in the direction of the inertia force (m)

Here, the symbol $\int$ denotes the integral throughout the entire portion of the design vibration unit.

4.1.6 Design Horizontal Seismic Coefficient

(1) The design horizontal seismic coefficient shall be set by appropriately considering damping characteristics due to the vibration characteristics of the structure in determining the characteristic value specified in Chapter 3 for the earthquake ground motion that will act on the bridge.

(2) When the provisions (3) to (6) are complied with, the provision (1) may be deemed to be satisfied.

(3) Design horizontal seismic coefficient for Level 1 Earthquake Ground Motion shall be calculated by Equation (4.1.5). If the value calculated by Equation (4.1.5) is smaller than 0.10, the design horizontal seismic coefficient for Level 1 Earthquake Ground Motion shall be set to 0.10.

$$k_h = c_Z k_{h0} \quad \cdots\cdots\cdots\cdots \quad (4.1.5)$$

where

k_h : Design horizontal seismic coefficient (The value shall be rounded off to two decimal places.)

k_{h0}: Standard value of the design horizontal seismic coefficient for Level 1 Earthquake Ground Motion, shown in Table 4.1.1 below.

c_z : Seismic zone factor of Level 1 Earthquale Ground Motion specified in Clause 3.4

Table 4.1.1 Standard Values of the Design Horizontal Seismic Coefficient for Level 1 Earthquake Ground Motion, k_{h0}

Ground Type	k_{h0} - Values in Terms of Natural Period T (s)		
Type I	$T < 0.10$ $k_{h0} = 0.431T^{1/3}$ but $k_{h0} \geqq 0.16$	$0.10 \leqq T \leqq 1.10$ $k_{h0} = 0.20$	$1.10 < T$ $k_{h0} = 0.213T^{-2/3}$
Type II	$T < 0.20$ $k_{h0} = 0.427T^{1/3}$ but $k_{h0} \geqq 0.20$	$0.20 \leqq T \leqq 1.30$ $k_{h0} = 0.25$	$1.30 < T$ $k_{h0} = 0.298T^{-2/3}$
Type III	$T < 0.34$ $k_{h0} = 0.430T^{1/3}$ but $k_{h0} \geqq 0.24$	$0.34 \leqq T \leqq 1.50$ $k_{h0} = 0.30$	$1.50 < T$ $k_{h0} = 0.393T^{-2/3}$

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

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 (4.1.6).

$$k_{Ih} = c_{Iz} k_{Ih0} \quad \cdots\cdots \quad (4.1.6)$$

where

k_{Ih} : Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I) (The value shall be rounded off to two decimal places.)

k_{Ih0} : Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type I), according to Table 4..1.2

c_{Iz} : Seismic zone factor of Level 2 Earthquake Ground Motion (Type I)specified in Clause 3.4

Table 4.1.2 Standard Values of the Design Horizontal Seismic Coefficient for Level 2 Earthquake Ground Motion (Type I), $k_{II\,h0}$

Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T < 0.16$ $k_{hc0} = 2.58T^{1/3}$	$0.16 \leqq T \leqq 0.60$ $k_{hc0} = 1.40$	$0.60 < T$ $k_{hc0} = 0.996T^{-2/3}$
Type II	$T < 0.22$	$0.22 \leqq T \leqq 0.90$	$0.90 < T$

	k_{hc0}=2.15$T^{1/3}$	k_{hc0}= 1.30	k_{hc0} =1.21$T^{-2/3}$
Type III	T< 0.34 k_{hc0} =1.72$T^{1/3}$	0.34≦T ≦1.40 k_{hc0}=1.20	1.40<T k_{hc0}=1.50$T^{-2/3}$

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 (4.1.7).

$$k_{II\,h} = c_{II\,z} k_{II\,h0} \quad \cdots\cdots (4.1.7)$$

where

$k_{II\,h}$: Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type II) (The value shall be rounded off to two decimal places.)

$k_{II\,h0}$: Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion (Type II), according to Table 4.1.3

$c_{\mathrm{II}\,z}$: Modification factor for zones of Level 2 Earthquake Ground Motion (Type II), specified in Clause 3.4

Table 4.1.3 Standard Values of Design Horizontal Seismic Coefficient for Level 2 Earthquake Ground Motion (Type II), khc0

Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	T < 0.30 k_{hc0} = 4.46$T^{2/3}$	0.30≦T ≦0.70 k_{hc0} = 2.00	0.70 < T k_{hc0} = 1.24$T^{-4/3}$
Type II	T < 0.40 k_{hc0} = 3.22$T^{2/3}$	0.40≦T ≦1.20 k_{hc0} = 1.75	1.20 < T k_{hc0} = 2.23$T^{-4/3}$
Type III	T < 0.50 k_{hc0} = 2.38$T^{2/3}$	0.50≦T ≦1.50 k_{hc0} = 1.50	1.50 < T k_{hc0} = 2.57$T^{--4/3}$

(5) The design horizontal seismic coefficient on the ground to be used for the calculation of the inertia force due to soil weight shall be calculated from Equations (4.1.8), (4.1.9), and (4.1.10)

$$k_{hg} = c_Z k_{hg0} \quad \cdots\cdots (4.1.8)$$

$$k_{\mathrm{I}hg} = c_{\mathrm{I}z} k_{\mathrm{I}hg0} \quad \cdots\cdots (4.1.9)$$

$$k_{\mathrm{II}hg} = c_{\mathrm{II}z} k_{\mathrm{II}hg0} \quad \cdots\cdots (4.1.10)$$

where

k_{hg}: Design horizontal seismic coefficient at base ground surface (The value shall be rounded off to two decimal places.)

k_{hg0}: Standard value of the design horizontal seismic coefficient at the base ground surface for Level 1 Earthquake Ground Motion; the value shall be 0.16, 0.20, and 0.24 for the ground type of Type I, Type II, and Type III specified in Clause 3.6, respectively.

$k_{\mathrm{I}hg}$: Design horizontal seismic coefficient at the base ground surface for Level 2 Earthquake Ground Motion (Type I) (The value shall be rounded off to two decimal places.)

$k_{\mathrm{I}hg0}$: Standard value of the design horizontal seismic coefficient at the base ground surface for Level 2 Earthquake Ground Motion (Type I); the value shall be 0.50, 0.45, and 0.40 for the ground type of Type I, Type II, and Type III specified in Clause 3.6, respectively.

$k_{\mathrm{II}hg}$: Design horizontal seismic coefficient at the base ground surface for Level 2 Earthquake Ground Motion (Type II) (The value shall be rounded off to two decimal places.)

$k_{\mathrm{II}hg0}$: Standard value of the design horizontal seismic coefficient at the base ground surface for Level 2 Earthquake Ground Motion (Type II); the value shall be 0.80, 0.70, and 0.60 for the ground type of Type I, Type II, and Type III specified in Clause 3.6, respectively.

(6) In calculating inertia forces, the same values of the design horizontal seismic coefficients calculated from Equations (4.1.5), (4.1.6), and (4.1.7) shall generally be applied to each design vibration unit. However, the inertia force due to soil weight shall be calculated by using the values of the design horizontal seismic coefficient calculated for the base ground surface from Equations (4.1.8), (4.1.9), and (4.1.10) according to the ground type at the height of the substructure.

4.2 Seismic Earth Pressure

(1) Earth pressure during an earthquake shall be set by appropriately considering the type of the structure, soil conditions, the intensity of stress developed in soil, uncertainty involved in the estimation of the dynamic characteristics of soil, and the like.

(2) The surface of action of earth pressure on an abutment shall be determined according to the provisions of Clause 8.7 of Part I.

(3) When the provision (4) is complied with, the provision (1) may be deemed to be satisfied.

(4) Earth pressure during an earthquake shall be a uniform load, and the characteristic value of its earth pressure intensity in the active state shall be calculated from Equation (4.2.1).

$$P_{EA} = rxK_{EA} + q'K_{EA} \qquad \cdots\cdots (4.2.1)$$

where

P_{EA} : Active seismic earth pressure (kN/m^2) at depth x

K_{EA}: Coefficient of active seismic earth pressure to be calculated by Equation (4.2.2)

$$\left.\begin{array}{l} \text{1) Between soil and concrete behind the abutment} \\ \quad \text{Gravelly soil: } K_{EA} = 0.21 + 0.90k_h \\ \quad \text{Sandy soil : } K_{EA} = 0.24 + 1.08k_h \\ \\ \text{2) Between soil and soil behind the abutment} \\ \quad \text{Gravelly soil: } K_{EA} = 0.22 + 0.81k_h \\ \quad \text{Sandy soil : } K_{EA} = 0.26 + 0.97k_h \end{array}\right\} \cdots\cdots\cdots\cdots (4.2.2)$$

k_h : Design horizontal seismic coefficient that is used for calculating earth pressure during an earthquake. For Level 1 Earthquake Ground Motion, the design horizontal seismic coefficient on the ground surface specified in Clause 4.1.6(5) shall be used, and for Level 2 Earthquake Ground Motion, the design horizontal seismic coefficient on abutments and the foundations of abutments specified in Clause 11.3 shall be used

r : Unit weight of soil (kN/m^3)

x : Depth of active seismic earth pressure (P_{EA}) to the wall surface : (m)

q' : Surcharge on the ground surface during an earthquake (kN/m^2)

For q', only surcharges that act during an earthquake without fail shall be considered, and live loads shall be excluded.

4.3 Seismic Hydrodynamic Pressure

(1) Hydrodynamic pressure during an earthquake shall appropriately be set by considering the water level, the shape and dimensions of the substructure, and the like.

(2) When the provision (3) is complied with, the provision (1) may be deemed to be satisfied.

(3) 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 Clause 4.1.1

1) Seismic hydrodynamic pressure acting on a wall structure with water on one side only:
Resultant force of seismic hydrodynamic pressure acting on a wall structure with water on one side only and height of the resultant force shall be calculated by Equations (4.3.1) and (4.3.2), respectively. (Refer to Figure 4.3.1).

$$p = \frac{7}{12} \times k_h w_0 b h^2 \quad \cdots\cdots \quad (4.3.1)$$

$$h_g = \frac{2}{5} \times h \quad \cdots\cdots \quad (4.3.2)$$

where

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

k_h: Design horizontal seismic coefficient for Level 1 Earthquake Ground Motion specified in Clause 4.1.6

w_0: Unit weight of water (kN/m^3)

h : Water depth (m)

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

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

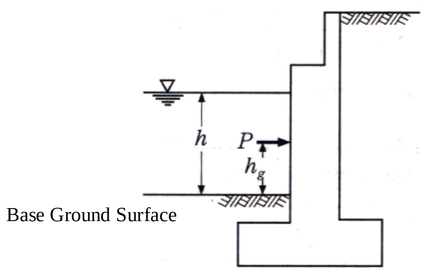

Figure 4.3.1 Seismic Hydrodynamic Pressure Acting on a Wall Structure

2) Seismic hydrodynamic pressure acting on a column structure completely surrounded by water:

Resultant 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 (4.3.3) and (4.3.4), respectively. (Refer to Figure 4.3.2).

$$\left.\begin{array}{l} \text{when } \dfrac{b}{h} \leqq 2.0 \\ P = \dfrac{3}{4} k_h w_0 A_0 h \dfrac{b}{a}\left(1 - \dfrac{b}{4h}\right) \\ \text{when } 2.0 < \dfrac{b}{h} \leqq 4.0 \\ P = \dfrac{3}{4} k_h w_0 A_0 h \dfrac{b}{a}\left(0.7 - \dfrac{b}{10h}\right) \\ \text{when } 4.0 < \dfrac{b}{h} \\ P = \dfrac{9}{40} k_h w_0 A_0 h \dfrac{b}{a} \end{array}\right\} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (4.3.3)$$

$$h_g = \frac{3}{7} h \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4.3.4)$$

where

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

k_h: Design horizontal seismic coefficient for Level 1 Earthquake Ground Motion specified in Clause 4.1.6

w_0: Unit weight of water (kN/m^3)

h : Water depth (m)

h_g: Height from the base 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 (m^2)

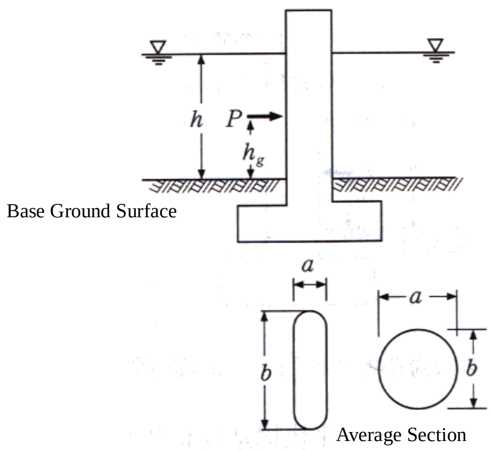

Figure 4.3.2 Seismic Hydrodynamic Pressure Acting on a Column Structure

4.4 Lateral Spreading Force of the Ground

4.4.1 General

(1) The lateral spreading force of the ground shall appropriately be set by considering ground conditions, topographical conditions, the installation location of the substructure, and the like.

(2) When the ground is judged to develop lateral spreading affecting bridges according to the provision of Clause 4.4.2 and the lateral spreading force of the ground that acts on the pier foundation is set according to the provisions of Clauses 4.4.3, the provision (1) may be deemed to be satisfied.

4.4.2 Judgment on whether the Ground Will Develop Lateral Spreading

The ground that falls under both the following 1) and 2) shall be judged to be ground that will develop lateral spreading that affects the bridge.

1) Ground that is in a coastal area and located 100 m or less away from the water front formed by a revetment with a difference of elevation of 5 m or more between the back ground elevation and the depth of the water bottom in the front
2) Ground with a 5-m or thicker soil layer that is judged according to the provisions of Clause 7.2 to be a soil layer expected to develop liquefaction, and that continuously extends in the horizontal direction from the water front

4.4.3 Calculation of Lateral Spreading Force of the Ground

The lateral spreading force of the ground to be applied to the pier foundation shall be calculated according to the following provisions 1) and 2) when, as illustrated in Figure 4.4.1, a soil layer that will not develop liquefaction (hereafter, referred to as "a non-liquefying layer") exists near the ground surface while a soil layer that will develop liquefaction (hereafter, referred to as "a liquefiable layer") exists below the non-liquefying layer.

1) Lateral spreading force for members that are located in a non-liquefying layer within the range where the effects of lateral spreading are to be consideredshall be calculated from Equation (4.4.1). For the ground that has no non-liquefying layer over a liquefiable layer and accordingly develops liquefaction to the ground surface, it is not necessary to consider Equation (4.4.1).

2) Lateral spreading force for members that are located in a liquefiable layer within the range where

the effects of lateral spreading are to be considered shall be calculated from Equation (4.4.2).

$$q_{NL} = c_s c_{NL} K_P \gamma_{NL} x \qquad (0 \leqq x \leqq H_{NL}) \cdots\cdots\cdots\cdots\cdots\cdots \quad (4.4.1)$$

$$q_L = c_s c_L \{ \gamma_{NL} H_{NL} + \gamma_L (x - H_{NL}) \} \qquad (H_{NL} < x \leqq H_{NL} + H_L) \cdots\cdots\cdots\cdots \quad (4.4.2)$$

where

q_{NL}: Lateral spreading force per unit area (kN/m^2) acting on a member in a non-liquefiable layer at the depth x

q_L : Lateral spreading force per unit area (kN/m^2) acting on a member in a liquefiable layer at the depth x

c_s: Modification factor on distance from the water front. c_s shall take a value shown in Table 4.4.1.

c_{NL}: Modification factor of the lateral spreading force in a non-liquefiable layer. c_{NL} shall be a value in Table 4.4.2, according to liquefaction index P_L (m^2) obtained from Equation (4.4.3).

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5\mathrm{x}) dx \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4.4.3)$$

Cc_L : Modification factor of the lateral spreading force in a liquefiable layer (can be taken as 0.3)

K_P: Passive earth pressure coefficient, according to the provisions of Clause 8.7 of Part I

γ_{NL}: Mean unit weight of a non-liquefiable layer (kN/m^3)

γ_L : Mean unit weight of a liquefiable layer (kN/m^3)

x : Depth from the ground surface (m)

H_{NL}: Thickness of non-liquefiable layer (m)

H_L : Thickness of liquefiable layer (m)

F_L : Liquefaction resistance factor calculated by Equation (7.2.1). If $F_L \geqq 1$, $F_L = 1$.

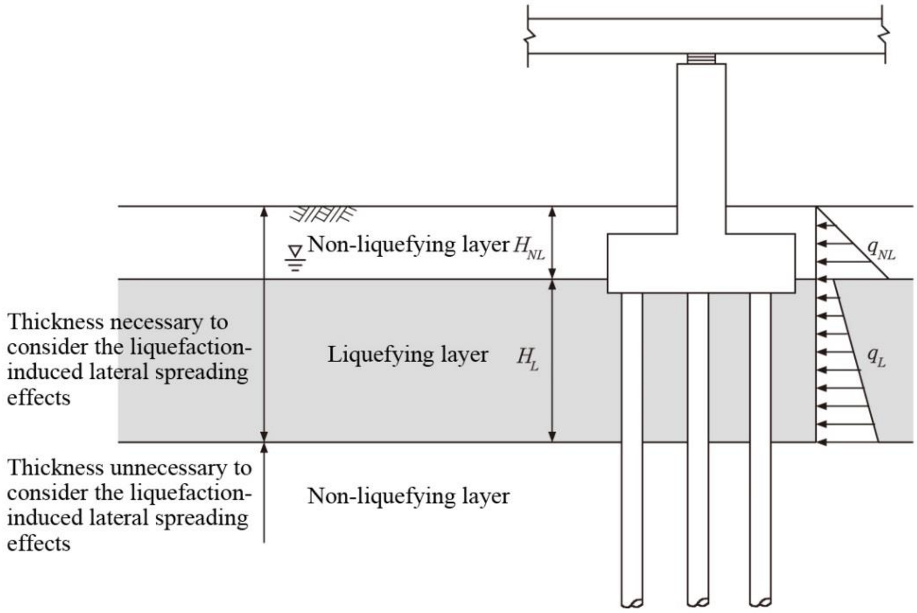

Figure 4.4.1 Model for Calculating Lateral Spreading Force

Table 4.4.1 Modification Factor c_s on Distance from Water Front

Distance From Water Front s (m)	Modification Factor c_s
$s \leqq 50$	1.0
$50 < s \leqq 100$	0.5
$100 < s$	0

Table 4.4.2 Modification Factor c_{NL} of Lateral Spreading Force in Non-Liquefiable Layer

Liquefaction Index P_L (m^2)	Modification Factor c_{NL}
$P_L \leqq 5$	0
$5 < P_L \leqq 20$	$(0.2\,P_L - 1)/3$
$20 < P_L$	1

CHAPTER 5 METHOD FOR STRUCTURAL ANALYSIS

5.1 General

(1) In performing the seismic design of a bridge, the dynamic analysis specified in Clause 5.2 shall generally be used to calculate the response values of the sectional force, stress, displacement, and the like brought about by inertia forces. However, the static analysis specified in Clause 5.3 may be used, in cases where the plastic behavior of members and the like is not to be considered and the following 1) applies, or in cases where the plastic behavior of members and the like is to be considered and one of the following provisions from 1) to 3) apply.

1) The primary natural vibration mode is dominant.

2) The members and portions exhibiting plastic behavior are clearly identified.

3) The adaption of Energy Conservation Principle has verified.

(2) Ground resistance shall generally be considered below the ground surface in seismic design specified in Clause 3.5.

5.2 Dynamic Analysis

(1) For dynamic analysis, time-history response analysis shall generally be used.

(2) For response value calculation in dynamic analysis, members shall be modeled in such a manner as to satisfy the following provisions 1) to 3).

1) Members shall appropriately be modeled according to their material characteristics, the resistance characteristics of the ground, and the like so that the behavior of the bridge during an earthquake can be analyzed on the basis of the structural properties of the bridge.

2) Members shall appropriately be modeled according to their mechanical properties and hysteresis properties.

3) The damping characteristics of the bridge shall appropriately be modeled by considering the vibration characteristics of the members and the like that compose the bridge.

(3) In dynamic analysis, response values shall be determined as the averages of the response values that are calculated by using accelerograms specified in Clause 4.1.2 in a design situation in which Level 2 Earthquake Ground Motion is to be considered.

5.3 Static Analysis

(1) For static analysis, step-by-step incremental load analysis and Energy Conservation Principle shall generally be adapted.

(2) For response value calculation in static analysis, members shall appropriately be modeled according to their material characteristics, the resistance characteristics of the ground, and the like so that the behavior of the bridge during an earthquake can be analyzed on the basis of the structural properties of the bridge.

CHAPTER 6 DESIGN OF MEMBERS AND THE LIKE IN A SITUATION IN WHICH THE EFFECTS OF EARTHQUAKE ARE TO BE CONSIDERED

6.1 Limit States of Members and the Like in a Situation in which the Effects of Earthquake Are to Be Considered

(1) In a situation in which the effects of earthquake are to be considered, the characteristic values or limit values for the Limit state 1 of members and the like shall be determined according to the following provisions 1) to 3).

1) For steel members, the provisions of Chapter 5, and Chapter 9 to Chapter19 of Part II shall be complied with.

2) For concrete members, the provisions of Chapter 5, Chapter 7, and later chapters of Part II shall be complied with. In a design situation in which Level 2 Earthquake Ground Motion is to be considered, the provisions of Clause 6.4 and those of Chapter 8, and later chapters shall be complied with for concrete members that will be subjected to prestress.

3) For members that compose substructures, the provisions of Chapter 5, and Chapter 7 to Chapter 14 of Part IV shall be complied with.

(2) In a situation in which the effects of earthquake are to be considered, when plastic behavior of members and the like is expected, the characteristic values or limit values for the Limit state 2 of members and the like shall be determined according to the following provisions from 1) to 3).

1) For reinforced concrete members, the provisions of Clause 6.2 and those of Chapter 8 and later chapters shall be complied with.

2) For steel members, the provisions of Clause 6.3 and those of Chapter 8 and later chapters shall be complied with.

3) For concrete members that are introduced prestress, the provisions of Clause 6.4 and those of Chapter 8 and later chapters shall be complied with.

(3) In a situation in which the effects of earthquake are to be considered, the characteristic values or limit values for the Limit state 3 of members and the like shall be determined according to the following provisions 1) and 2).

1) When plastic behavior is expected, the following provisions i) to iii) shall be complied with.

i) For reinforced concrete members, the provisions of Clause 6.2 and those of Chapter 8 and later chapters shall be complied with.

ii) For steel members, the provisions of Clause 6.3 and those of Chapter 8 and later chapters shall be complied with.

iii) For concrete members that are intoroduced prestress, the provisions of Clause 6.4 and those of Chapter 8 and later chapters shall be complied with.

2) When plastic behavior is not expected, the following provisions i) to iii) shall be complied with.

i) For steel members, the provisions of Chapter 5, and Chapter 9 to Chapter 19 of Part II shall be complied with.

ii) For concrete members, the provisions of Chapter 5, and Chapter 7 to Chapter 16 of Part III shall be complied with.

iii) For members that compose substructures, the provisions of Chapter 5, and Chapter 7 to Chapter 14 of Part IV shall be complied with.

6.2 Reinforced Concrete Members in which Plastic Behavior Is Expected

6.2.1 Members that Receive Bending Moment and Axial Force

(1) For reinforced concrete members for which plastic behavior is expected, when the reinforced concrete members receive bending moment and axial force and satisfy not only the structural details specified in Clause 6.2.5 but also the following provisions (2) to (4), Limit state 2 or Limit state 3 may be regarded as not exceeded.

(2) The response values that occur in members shall be no greater than their limit values corresponding to Limit state 2 or Limit state 3. The characteristic values and limit values of the displacement and curvature corresponding to Limit state 2 or Limit state 3 shall appropriately be set according to structural conditions to be applied.

(3) A model that allows the material characteristics of members to appropriately be analyzed shall be used so that the sectional force, stress, displacement, curvature, ductility factor, and the like that occur in the members in response to force acting on them can appropriately be calculated.

(4) For reinforced concrete members, when the characteristic values and limit values corresponding to Limit state 2 or Limit state 3 are set on the basis of the bending moment-curvature relationship that is set according to the provisions of Clause 6.2.2, the provision (3) may be regarded as satisfied.

6.2.2 Bending Moment-Curvature Relationship of Reinforced Concrete Members

The bending moment-curvature relationship of a reinforced concrete member shall be determined according to the following provisions 1) to 3). When the intensity of the yield bending moment and that of the bending moment equivalent to Limit state 2 or Limit state 3 become greater than or equal to that of the cracking bending moment and the intensity of the bending moment equivalent to Limit state 2 or Limit state 3 become greater than or equal to that of the yield bending moment, the relationship shall generally be the trilinear type shown in Figure 6.2.1. In this case, the yield bending moment shall be the bending moment that occurs when the outermost tensile reinforcement reaches its yield strength, and the cracking bending moment shall be the bending moment that occurs when the outermost concrete reaches its bending tensile strength.

1) Fiber strain is proportional to the distance from the neutral axis.

2) The stress-strain curve of concrete and the stress-strain curve of a reinforcement shall be determined according to the provisions of Clause 6.2.3.

3) In setting the characteristic value of the bending moment equivalent to Limit state 2 or Limit state 3, a value that is obtained in the state of a limit at which necessary strength cannot be assumed under the compression strain of the concrete or the tensile strain of the primary tensile reinforcement shall be regarded as the characteristic value of the strain equivalent to Limit state 2 or Limit state 3.

In setting the characteristic value of bending moment equivalent to Limit state 2 or Limit state 3, the strain of concrete or the strain of the axial tensile reinforcement that occurs when the necessary strength for resistance to compression is not expected for concrete, or when the necessary strength for tensile resistance is not expected for the axial tensile reinforcement shall be regarded as the Limit of the compression strain of the concrete or the limit of the tensile strain of the axial tensile reinforcement and shall appropriately be set as the characteristic value corresponding to Limit state 2 or Limit state 3 based on the provision of Clause 2.4.6(5).

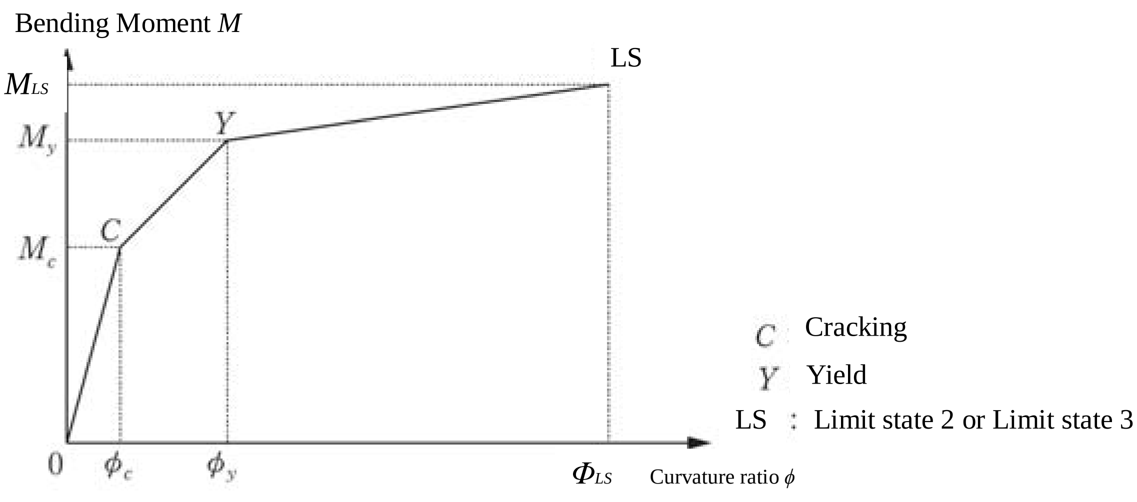

Figure 6.2.1 Bending Moment-Curvature Relationship of a Reinforced Concrete Member

6.2.3 Stress-Strain Curve of Concrete and Stress-Strain Curve of Reinforcements

(1) The stress-strain curve of concrete shall be determined by Equation (6.2.1) based on Figure 6.2.2 with the confining effect of lateral confining reinforcements.

$$\left.\begin{array}{ll}\sigma_c = E_c \varepsilon_c \left\{1 - \dfrac{1}{n}\left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right\} & (0 \leqq \varepsilon_c \leqq \varepsilon_{cc}) \\ \sigma_c = \sigma_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{ccl})\end{array}\right\} \cdots\cdots \quad (6.2.1)$$

$$n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - \sigma_{cc}} \cdots\cdots \quad (6.2.2)$$

$$\sigma_{cc} = \sigma_{ck} + 3.8 \alpha \rho_s \sigma_{sy} \cdots\cdots \quad (6.2.3)$$

$$\varepsilon_{cc} = 0.002 + 0.033 \beta \frac{\rho_s \sigma_{sy}}{\sigma_{ck}} \cdots\cdots \quad (6.2.4)$$

$$E_{des} = 11.2 \frac{\sigma_{ck}^{\ 2}}{\rho_s \sigma_{sy}} \cdots\cdots \quad (6.2.5)$$

$$\rho_s = \frac{4A_h}{sd} \leqq 0.018 \cdots\cdots \quad (6.2.6)$$

where

σ_c : Stress of concrete (N/mm^2)

σ_{cc} : Maximum compressive stress of concrete confined by a lateral confining reinforcement (N/mm^2)

σ_{ck} : Design strength of concrete (N/mm^2)

ε_c : Strain of concrete

ε_{cc} : Strain of concrete under the peak compressive stress

ε_{ccl} : Limit of compressive strain of concrete

E_c : Young's modulus of concrete (N/mm^2), which is shown in Table 4.2.3 of Part III

E_{des} : Descending gradient (N/mm^2)

ρ_s : Volume ratio of a lateral confining reinforcement ; the smallest value in concrete portion divided by a lateral confining reinforcement placed parallel to the direction of the inertia force used for seismic design

A_h : Sectional area of each lateral confining reinforcement (mm^2)

s : Spacing of lateral confining reinforcement (mm)

d : Effective length of a lateral confining reinforcement for consideration of the lateral confining effect of concrete (mm)

σ_{sy}: Yield stress of lateral confining reinforcement (N/mm^2) ; the upper limit shall be 345 N/mm^2.

α,β: Cross-section correction factors, which shall be α = 1.0 and β = 1.0 for circular sections and α = 0.2 and β = 0.4 for rectangular sections

n : A constant defined by Equation (6.2.2)

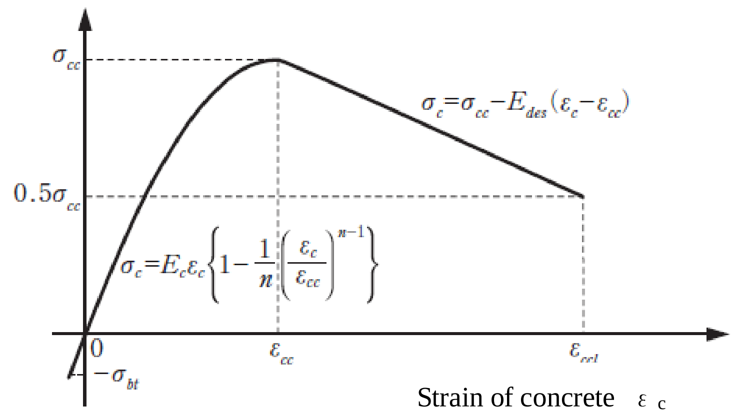

Figure 6.2.2 Stress-Strain Curve of Concrete

(2) The stress-strain curve of a longitudinal reinforcement shall be calculated from Equation (6.2.7) based on Figure 6.2.3.

$$\left.\begin{array}{ll} \sigma_s=-\sigma_{sy} & (\varepsilon_s<-\varepsilon_{sy}) \\ \sigma_s=E_s\varepsilon_s & (-\varepsilon_{sy}\leqq\varepsilon_s\leqq\varepsilon_{sy}) \\ \sigma_s=\sigma_{sy} & (\varepsilon_{sy}<\varepsilon_s\leqq\ \varepsilon_{st}) \end{array}\right\} \cdots\cdots\cdots\cdots \text{(6.2.7)}$$

where

σ_s: Unit stress of a longitudinal reinforcement (N/mm^2)

σ_{sy}: Yield stress of a longitudinal reinforcement (N/mm^2)

E_s: Young's modulus of a longitudinal reinforcement (N/mm^2)

ε_s: Strain of a longitudinal reinforcement

ε_{sy}: Yield strain of a longitudinal reinforcement, calculated from Equation (6.2.8)

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s} \quad \cdots\cdots \quad (6.2.8)$$

ε_{st}: Limit of tensile strain of a longitudinal reinforcement

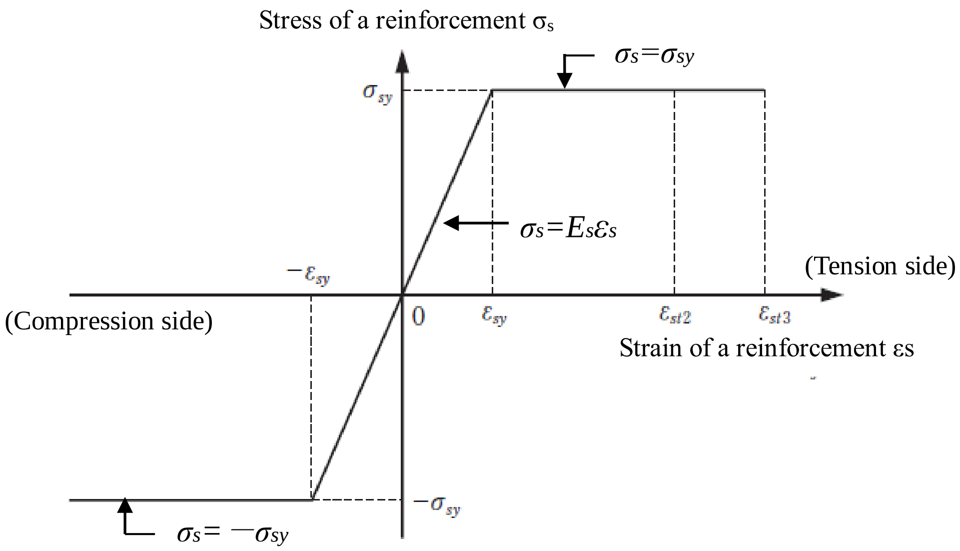

Figure 6.2.3 Stress-Strain Curve of a Longitudinal Reinforcement

6.2.4 Members that Receive Shearing Force

(1) For reinforced concrete members for which plastic behavior is expected, when the reinforced concrete members that receive shearing force satisfy the provisions (2) and (3), Limit state 2 and Limit state 3 may be regarded as not exceeded.

(2) The shearing force that occurs in members shall be no greater than the limit value of shearing force specified in Clause 5.8.2(3) of Part III. The characteristic value and limit value of the shearing force of reinforced concrete members shall appropriately be set according to structural conditions and the degree of plastic behavior.

(3) When the following provisions 1) to 3) are complied with in cases where the design reference strength of concrete is 30 N/mm^2 or lower, the characteristic value of shearing force may be regarded as appropriately set.

1) The modification factors to be used for calculation of the mean shear stress $\tau\gamma$ that can be supported by concrete shall be determined according to the following provisions i) to iv).

i) The modification factor c_e for the effective height d shall be determined according to Table 6.2.1.

Table 6.2.1 Modification Factor c_e for Effective Height d

Effective height (mm)	1,000 or smaller	3,000	5,000	10,000 or larger
c_e	1.0	0.7	0.6	0.5

ii) The modification factor c_{pt} for the reinforcement ratio p_t of reinforcements longitudinally arranged on the tensile side shall be determined according to Table 6.2.2.

Table 6.2.2 Modification Factor c_{pt} for Axial Tensile Reinforcement Ratio p_t

Axial tensile reinforcement ratio (%)	0.2	0.3	0.5	1.0 or more
c_{pt}	0.9	1.0	1.2	1.5

iii) The increase coefficient c_{dc} of shearing force supported by concrete according to the shear span ratio shall be 1.0.

iv) The modification factor c_c for the effect of cyclic loading shall be determined according to Table 6.2.3.

Table 6.2.3 Modification Factor c_c for the Effect of Cyclic Loading when Plastic Behavior Is to Be Considered

Level 2 Earthquake Ground Motion	Type I	Type II
c_c	0.6	0.8

2) The factor of reduction c_{ds} of shearing force supported by shear reinforcements according to the shear span ratio shall be 1.0.

3) For the characteristic value Sc of shearing force that can be supported by concrete, the effect of increase in shearing force supported by concrete due to axial compressive force shall be excluded from consideration.

6.2.5 Structural Details of Reinforced Concrete Members for Securing Ductility

(1) In order to prevent brittle failure and secure necessary ductility, lateral confining reinforcements of an appropriate type shall be arranged at appropriate intervals in reinforced concrete members so that the lateral confining reinforcements can reliably deliver the effect of preventing buckling of longitudinal reinforcements and the effect of confining concrete surrounded with the lateral confining reinforcements.

(2) When the following provisions 1) to 5) are complied with, the provision (1) may be regarded as satisfied.

1) A deformed bar shall be used for a hoop reinforcement of lateral confining reinforcements. The diameter of the deformed bar shall be 13 mm or more and less than the diameter of a longitudinal reinforcement. Hoop reinforcements shall be arranged at intervals of 300 mm or smaller.

2) Hoop reinforcements shall generally be arranged in such a manner that they enclose longitudinal reinforcements ,and the end of the hoop reinforcements shall be anchored into concrete surrounded with the hoop reinforcements by hooks specified in the following provision i), ii), or iii). Lap splices without hook shall generally not be used. When rectangular angle hooks are used, the hooks shall be anchored so that they will not fall off even when cover concrete spalls off. In addition, the joints of the hoop reinforcements shall be staggered in the axial direction. The bending shape and bending internal radius of hooks appropriate for each reinforcement type shall be determined according to the provisions of Clause 5.2.6 of Part III. The straight length of the hooks from the bending ends shall be longer than the values provided below:

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

ii) Acute angle hook: 10 times the diameter of the hoop reinforcement

iii) Rectangular angle hook: 12 times of the diameter of the hoop reinforcement

3) When hoop reinforcements are lapped at any place other than the corners of a rectangular cross section, hoop reinforcements shall have a lap length of at least 40 times the diameter of the hoop reinforcements. In addition, the hoop reinforcements shall generally be provided with hooks specified in the provision 2).

4) Of lateral confining reinforcements, cross ties shall meet the following requirements:

i) Reinforcements of the same material and diameter as those of hoop reinforcements shall generally be used.

ii) The arrangement interval at cross section shall generally be within 1 m.

iii) Reinforcements shall be arranged in all cross sections in which hoop reinforcements are arranged.

iv) Cross ties shall generally be anchored to internal concrete by hooking semi-circular or acute angle hooks specified in the provision 2) onto hoop reinforcements arranged in the circumferential direction of the cross sections . When longitudinal reinforcements are to be arranged into two or more tiers, these hooks shall be hooked onto hoop reinforcements arranged at the outermost location.

v) Cross ties shall generally penetrate through the cross section of the member by using one continuous reinforcing bar or two pairs of reinforcing bars with a joint within the cross section of the member. When a joint is to be set inside cross section of the member, an appropriate joint structure shall be selected so that joint strength equivalent to the strength of a cross tie can be secured.

5) For reinforced concrete members with a hollow cross section, a cross-sectional shape and arrangement of reinforcements shall appropriately be selected with consideration given to the characteristics of the hollow cross section so that they can reliably deliver ductility.

6.3 Steel Members for which Plastic Behavior Is Expected

6.3.1 Members that Receive Bending Moment and Axial Force

(1) For steel members for which plastic behavior is assumed, when the steel members that receive bending moment and axial force satisfy not only the structural details specified in Clause 6.3.4 but also the provisions (2) to (5), Limit state 2 or Limit state 3 may be regarded as not exceeded.

(2) The response that occur in members shall be no greater than their limit values corresponding to Limit state 2 or Limit state 3. The characteristic values and limit values of the displacement and curvature corresponding to Limit state 2 or Limit state 3 shall appropriately be set according to structural conditions to be applied.

(3) A model that allows the material characteristics of members to appropriately be evalated shall be used so that the sectional force, stress, displacement, curvature, ductility factor, and the like that occur in each of the members in response to force acting on them can appropriately be calculated.

(4) For steel members, when the characteristic values and limit values corresponding to Limit state 2 or Limit state 3 are set on the basis of the bending moment-curvature relationship that is set according to the provisions of Clause 6.3.2, the provision (3) may be regarded as satisfied.

(5) The characteristic values corresponding to the limit states of steel members shall generally be

determined on the basis of a loading test that is performed with consideration given to the effects of repeat by using a specimen with structural details equivalent to those of design target steel members. The characteristic values that are set as those corresponding to Limit state 2 on the basis of the loading test shall generally be the values that occur when the horizontal force reaches its maximum. As the characteristic values corresponding to Limit state 3, the characteristic values corresponding to Limit state 2 shall generally be used.

6.3.2 Bending Moment-Curvature Relationship of Steel Members

The bending moment-curvature relationship of a steel member shall be determined according to the following provisions 1) to 4).

1) Fiber strain is proportional to the distance from the neutral axis.
2) The stress-strain curve of steel concrete with which the steel member are to be filled shall generally be determined in accordance with the provisions of Clause 6.3.3.
3) In setting the characteristic value of the bending moment corresponding to Limit state 2 or Limit state 3, the limit of compression strain that occurs at the center of the plate thickness of the steel at the compressive extreme fiber when the steel member reaches its maximum strength shall appropriately be set as the characteristic value of the strain corresponding to Limit state 2 or Limit state 3 based on the provisions of Clause 2.4.6(5).
4) The skeleton curve of the bending moment – curvature relationship shall be determined by using either the bilinear type model shown in Figure 6.3.1 or the trilinear type model shown in Figure 6.3.2 according to the presence or absence of concrete fill and its cross-sectional shape. The stiffness changing point and skeleton curve shall be determined according to the presence or absence of concrete fill and its cross-sectional shape in accordance with the following provisions. In this case, the influence of axial force and that of eccentric moment shall be taken into consideration.
   i) For concrete-unfilled rectangular-section steel members, the bending moment and curvature (ϕ_{yc}, M_{yc}) that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of the yield strain specified in Clause 6.3.3 for the first time shall be regarded as the yield bending moment and the yield curvature (ϕ_y, M_y). A bilinear model shall be created by connecting this point to the point (ϕ_a, M_a) corresponding to the bending moment and curvature that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of strain corresponding to Limit state 2 or Limit state 3 for the first time.

ii) For concrete-unfilled circular-section steel members, a trilinear model shall be created by connecting the following three point: the point (ϕ_{yc}, M_{yc}) corresponding to the bending moment and curvature that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of the yield strain for the first time; the point (ϕ_{yt}, M_y) corresponding to the bending moment and curvature that occur when the tensile strain at the center of the plate thickness of the steel at the tensile extreme fiber reaches the yield strain for the first time; the point (ϕ_a, M_a) corresponding to the bending moment and curvature that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of strain corresponding to Limit state 2 or Limit state 3 for the first time.

iii) For concrete-filled rectangular- or circular-section steel members, either the bending moment and curvature (ϕ_{yc}, M_{yc}) that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of the yield strain for the first time or the bending moment and curvature (ϕ_{yt}, M_{yt}) that occur when the tensile strain at the center of the plate thickness of the steel at the tensile extreme fiber reaches the characteristic value of the yield strain for the first time shall be regarded as the yield bending moment and the yield curvature (ϕ_y, M_y), whichever are smaller. A bilinear model shall be created by connecting this point to the point (ϕ_a, M_a) corresponding to the bending moment and curvature that occur when the compression strain at the center of the plate thickness of the steel at the compressive extreme fiber reaches the characteristic value of strain corresponding to Limit state 2 or Limit state 3 for the first time.

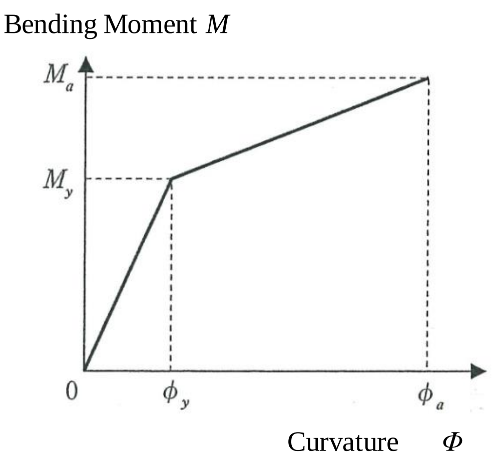

Figure 6.3.1 Bilinear Skeleton Curve of Steel Member

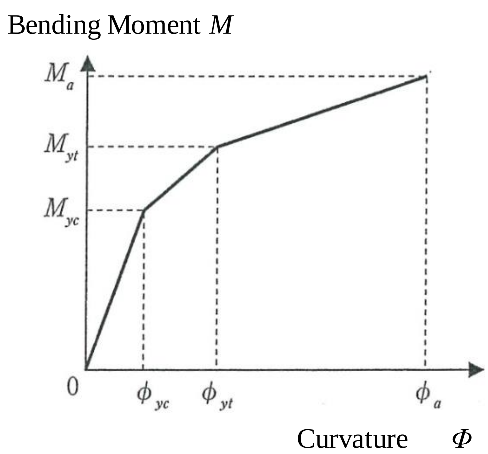

Figure 6.3.2 Trilinear Skeleton Curve of Steel Member

6.3.3 Stress-Strain Curve of Steel and That of Concrete to Be Filled into Steel Members

(1) The stress-strain curve of steel shall be determined from Equation (6.3.1) in accordance with Figure 6.3.3.

$$\left.\begin{array}{ll}\sigma_s=-\sigma_y+\dfrac{E_s}{100}(\varepsilon_s+\varepsilon_y) & (-\varepsilon_a \leqq \varepsilon_s < -\varepsilon_y) \\ \sigma_s=E_s\varepsilon_s & (-\varepsilon_y \leqq \varepsilon_s \leqq \varepsilon_y) \\ \sigma_s=\sigma_y+\dfrac{E_s}{100}(\varepsilon_s-\varepsilon_y) & (\varepsilon_s > \varepsilon_y)\end{array}\right\} \cdots\cdots (6.3.1)$$

where:

σ_s: Stress of steel (N/mm^2)

σ_y: Yield stress of steel (N/mm^2)

E_s: Young's modulus of steel (N/mm^2), given in Table 4.2.1 of Part II

ε_s: Strain of steel

ε_y: Yield strain of steel calculated from Equation (6.3.2)

$$\varepsilon_y=\frac{\sigma_y}{E_s} \cdots\cdots (6.3.2)$$

ε_a: Limit of compression strain of steel

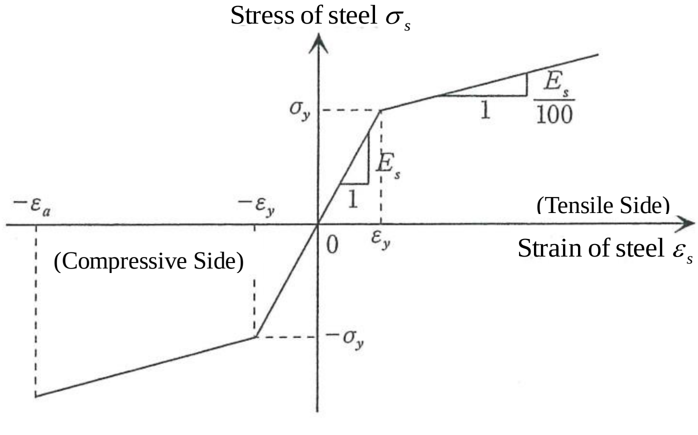

Figure 6.3.3 Stress-Strain Curve of Steel

(2) The stress-strain curve of concrete to be filled into steel members shall be calculated from

Equation (6.3.3) in accordance with Figure 6.3.4 on the assumption that concrete is not resistant to tensile force.

$$\left.\begin{array}{ll}\sigma_c = 0.85\sigma_{ck}\left\{\dfrac{\varepsilon_c}{0.002}\left(2-\dfrac{\varepsilon_c}{0.002}\right)\right\} & (0 \leqq \varepsilon_c \leqq 0.002) \\ \sigma_c = 0.85\sigma_{ck} & (\varepsilon_c > 0.002)\end{array}\right\} \cdots\cdots\cdots (6.3.3)$$

where:

σ_c: Stress of concrete (N/mm^2)

σ_{ck}: Design reference strength of concrete (N/mm^2)

ε_c: Strain of concrete

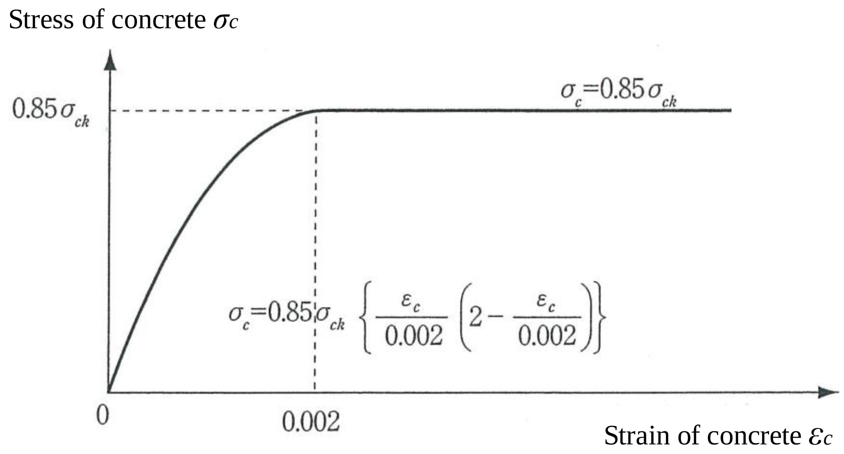

Figure 6.3.4 Stress-Strain Curve of Concrete to Be Filled into Steel Members

6.3.4 Structural Details of Steel Members for Securing Ductility

(1) In order to prevent brittle failure and secure ductility in steel members that are not filled with concrete, not only the provisions of Clauses 5.4.1, 5.4.2, 5.4.3, and 19.8 of Part II shall be satisfied, but the dimensions of the members also shall be set within a range in which the limit value of compressive stress against local buckling is its upper limit.

(2) In order to prevent brittle failure and secure ductility in steel members that are filled with concrete, the range of their inside to be filled with concrete shall be determined in such a manner

that buckling will not occur around the boundary between a cross section with concrete and a cross section with no concrete.

6.4 Concrete Members that Are introduced Prestress

(1) In a situation in which Level 2 Earthquake Ground Motion is to be considered, when the limit states of a concrete member that are introduced prestress are to be set not according to the provisions of Part III, characteristic values corresponding to the limit states and limit values at which the limit states can be regarded as not exceeded shall appropriately be set according to the provisions of Clause 2.4.6(4) and (5). Limit state 1, Limit state 2, and Limit state 3 may be regarded as not exceeded when the responses of the member do not exceed the limit values corresponding to Limit state 1, Limit state 2, and Limit state 3.

(2) A model that allows the material characteristics of members to appropriately be analyzed shall be used so that the sectional force, stress, displacement, curvature, ductility factor, and the like that occur in each of the members in response to force acting on them can appropriately be calculated.

6.5 Design of Joints

(1) For design of a joint, the relationship between the Limit state 1 or Limit state 2 and Limit state 3 of members that will be connected to be integrated and the Limit state 1 or Limit state 2 and Limit state 3 of the joint shall be clarified. The joint shall then be designed in such a manner that the members that will be connected to be integrated will deliver required performance.

(2) The joint shall be designed to be capable of transmitting the stress between members.

(3) Conditions required for the joint and each member to be connected shall be clarified so that the joint will deliver the required joint performance as specified in (2), and the joint shall be designed to satisfy the conditions.

(4) A loadcarrying mechanism that shares the stress occurring in the joint shall appropriately be set with consideration given also to the repeat of loading due to the effects of earthquake, and shall be designed to have a structure that reliably achieves the sharing.

(5) When concrete members are to be connected by using a steel or a reinforcement, a load carrying mechanism that share sectional force occurring in the joint shall appropriately be set so as to minimize the effects of the loss of bond and the detachment of a reinforcement or the like also for the repeat of loading due to the effects of earthquake under the considered effects of the loss

of bond and the detachment of a reinforcement or the like. In addition, the limit state and the characteristic value and the limit value for the limit state shall be set.

(6) When the load carrying mechanism of the joint is not different between situations in which the effects of earthquake are to be considered and situations other than the situations in which the effects of earthquake are to be considered, the provisions on joints specified in Chapter 9 of Part II, Chapter 7 of Part III, and Chapter 5 of Part IV shall be complied with.

CHAPTER 7 LIQUEFACTION OF THE GROUND

7.1 General

The effects of liquefaction on the bridge shall be considered according to the following provisions 1) and 2).

1) A judgment as to whether liquefaction that affects the bridge will occur shall be made according to the provisions of Clause 7.2.

2) For a soil layer that is judged according to the provisions of Clause 7.2 to be a layer in which liquefaction that affects the bridge will occur, the values of its soil parameters in seismic design shall be reduced according to the provisions of Clause 7.3 and considered in design.

7.2 Assessment of Soil Liquefaction Affecting Bridge

(1) A judgment as to liquefaction that affects the bridge shall be targeted on soil layers that fall under the provision (2), and made according to the provision (3).

(2) In alluvial soil layers that fall under all of the following conditions 1) to 3), liquefaction that affects the bridge may occur during an earthquake. Accordingly, a judgment as to liquefaction shall be made on the soil layers.

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 ground surface.
2) Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index I_P of 15 or less, even if FC is larger than 35%.
3) Soil layer having a particle size at 50% pass on the grading curve (D_{50}) of 10 mm or less and a particle size at 10% pass on the grading curve (D_{10}) of 1 mm or less.

(3) The factor of liquefaction resistance F_L, shall be calculated from Equation (7.2.1) separately for Level 1 Earthquake Ground Motion and Level 2 Earthquake Ground Motion. Soil layers with an F_L value of 1.0 or smaller shall be judged to be those in which liquefaction that affects the bridge will occur.

$$F_L = R / L \quad \cdots\cdots \quad (7.2.1)$$

where

F_L : Factor of liquefaction resistance

R : Dynamic shear strength ratio, which is calculated according to the provision (4)

L : Shearing stress ratio at earthquake, which is calculated according to the provision (5)

(4) Dynamic shear strength ratio R shall generally be calculated from Equation (7.2.2) for both Level 1 Earthquake Ground Motion and Level 2 Earthquake Ground Motion.

$$R = c_W R_L \quad \cdots\cdots \quad (7.2.2)$$

(For Level 1 Earthquake Ground Motion and Level 2 Earthquake Ground Motion (Type I))

c_W=1.0

(For Level 2 Earthquake Ground Motion (Type II))

$$c_W = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases} \quad \cdots\cdots \quad (7.2.3)$$

$$R_L = \begin{cases} 0.0882\sqrt{(0.85N_a + 2.1)/1.7} & (N_a < 14) \\ \end{cases} \quad \cdots\cdots \quad (7.2.4)$$

$$N_a = \begin{cases} c_{FC}(N_1 + 2.47) - 2.47 & (D_{50} < 2\text{mm}) \quad (14 \leqq N_a) \\ \{1 - 0.36\log_{10}(D_{50}/2)\}N_1 & (D_{50} \geqq 2\text{mm}) \end{cases} \quad \cdots\cdots \quad (7.2.5)$$

$$c_{FC} = \begin{cases} 1 & (0\% \leq FC < 10\%) \\ (FC + 20)/30 & (10\% \leq FC < 40\%) \\ (FC - 16)/12 & (40\% \leq FC) \end{cases} \quad \cdots\cdots \quad (7.2.7)$$

where

c_W : Modification factor on earthquake ground motion

R_L : Cyclic triaxial shear stress ratio

N: N value obtained from the standard penetration test

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2

N_a : Modified N value taking into account the effects of particle size

σ_{vb}': Effective overburden pressure at a depth from the ground surface when the standard penetration test is conducted (kN/m^2)

c_{FC}: Modification factors of N value on fine content

FC : Fine content (%) (percentage by mass of fine soil passing through the 75μm mesh)

D_{50}: 50% pass particle size (mm)

(5) Shearing stress ratio at earthquake L shall generally be calculated from Equation (7.2.8) for both Level 1 Earthquake Ground Motion and Level 2 Earthquake Ground Motion.

$$L = r_d \, k_{hgL} \sigma_v / \sigma'_v \quad \cdots\cdots \quad (7.2.8)$$

$$r_d = 1.0 - 0.015x \quad \cdots\cdots \quad (7.2.9)$$

$$k_{hgL} = c_z k_{hgL0} \quad \cdots\cdots \quad (7.2.10)$$

where

r_d : Reduction factor of seismic shear stress ratio in the depth direction

k_{hgL}: Design horizontal seismic coefficient at the base ground surface for liquefaction assessment (The value shall be rounded off to two decimal places.)

c_z: Modification factor for zones, which shall be c_z, the modification factor for Level 1 Earthquake Ground Motion specified in Clause 3.4 for Level 1 Earthquake Ground Motion, c_{Iz} specified in Clause 3.4 for Level 2 Earthquake Ground Motion (Type I), or c_{IIz} specified in Clause 3.4 for Level 2 Earthquake Ground Motion (Type II).

k_{hgL0}: Standard value of the design horizontal seismic coefficient on the base ground surface for liquefaction assessment, which shall be the value listed in Table 7.2.1.

σ_v: Total overburden pressure at a depth x (m) from the ground surface (kN/m^2)

σ_v': Effective overburden pressure at a depth x (m) from the ground surface (kN/m^2)

x : Depth from the ground surface (m)

Table 7.2.1 Standard Value k_{hgL0} of Design Horizontal Seismic Coefficient on the Base Ground Surface for Liquefaction Assessment

	Level 1 Earthquake Ground Motion	Level 2 Earthquake Ground Motion	
		Type I	Type II
Type I ground	0.12	0.50	0.80
Type II ground	0.15	0.45	0.70
Type III ground	0.18	0.40	0.60

7.3 Soil Layers Subject to Reduction in the Values of Soil Parameters in Seismic Design and How to Handle them

For soil layers that are judged according to the provisions of Clause 7.2 to be those in which liquefaction that affects the bridge will occur, their soil parameters in seismic design shall be reduced by multiplying these soil parameters by the factor of reduction D_E listed in Table 7.3.1 as corresponding to the value of the resistivity F_L against liquefaction that is calculated separately for

Level 1 Earthquake Ground Motion and Level 2 Earthquake Ground Motion.

Table 7.3.1 Factor of Reduction D_E for Soil Parameter in Seismic Design

Range of F_L	Depth from Ground Surface x (m)	Dynamic shear strenght ratio R	
		$R \leqq 0.3$	$0.3 < R$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/6
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	2/3	1
	$10 < x \leqq 20$	1	1

CHAPTER 8 REINFORCED CONCRETE PIERS

8.1 Scope

This chapter shall apply to the design of reinforced concrete single column piers and reinforced-concrete single-column rigid-frame piers as reinforced concrete piers for which plastic behavior is assumed.

8.2 General

In designing a reinforced concrete pier for which plastic behavior is assumed, the following provisions 1) to 5) shall be satisfied.

1) For reinforced concrete piers for which plastic behavior is assumed, seismic horizontal strength shall be calculated on the basis of the characteristic values and limit values of limit states that are appropriately being set with consideration given to failure modes. The failure modes shall generally be classified into the flexural-failure type, the shear failure after flexural yielding type, and the shear failure type.

2) When the provisions of Clause 8.3 are satisfied, seismic horizontal strength may be regarded as calculated according to failure modes that are appropriately classified.

3) The limit states of reinforced concrete piers corresponding to failure modes shall be determined according to the provisions of Clause 8.4.

4) When the eccentric moment due to the dead load of a superstructure and the like is acting, its effects shall be considered according to the provisions of Clause 8.8.

5) The joint with a footing shall comply with the provisions of Clause 8.11.

8.3 Judgment on Failure Modes of Reinforced Concrete Piers and their Seismic Horizontal Strength

The failure modes of reinforced concrete piers shall be judged according to the provision 1), and their seismic horizontal strength shall be calculated according to the provision 2).

1)i) The failure mode in the out-of-plane direction of a reinforced concrete single column pier or a reinforced-concrete single-column rigid-frame pier shall be judged by using Equation (8.3.1).

$$\left.\begin{array}{ll} P_u \leqq P_s & : \text{ Flexural (or bending) failure} \\ P_s < P_u \leqq P_{s0} & : \text{ Shear failure after flexural yielding} \\ & \end{array}\right\} \quad \cdots\cdots\cdots\cdots \quad (8.3.1)$$

$P_{s0} < P_u$: Shear failure

where

P_u: Ultimate horizontal strength (N) of the reinforced concrete pier specified in Clause 8.5

P_s: Limit value of shear force (N) for the reinforced concrete pier specified in Clause 8.6

$_{s0}$: Limit value of shear force (N) for the reinforced concrete pier which calculated according to the provisions of Clause 8.6 with 1.0 as the correction coefficient c_c related to the influence of the cyclic loading

ii) The failure mode in the in-plane direction of a reinforced-concrete single-column rigid-frame pier shall be judged by using Equation (8.3.2).

$$\left.\begin{array}{ll} S_i \leqq P_{si} & \text{: Flexural failure} \\ P_{si} < S_i \leqq P_{s0i} & \text{: Shear failure after flexural yielding} \\ P_{s0i} < S_i & \text{: Shear failure} \end{array}\right\} \cdots\cdots\cdots \quad (8.3.2)$$

where

S_i: Shear force (N) that occurs at the i-th plastic hinge when sectional force equivalent to the ultimate horizontal strength specified in Clause 8.7 occurs

P_{si}: Limit value of shear force (N) at the i-th plastic hinge calculated according to the provision of Clause 8.6

$_{s0i}$: Limit value of shear force (N) at the i-th plastic hinge which calculated according to the provisions of Clause 8.6 with 1.0 as the correction coefficient c_c related to the influence of the cyclic loading

2)i) The seismic horizontal strength in the out-of-plane direction of a reinforced concrete single column pier or a reinforced-concrete single-column rigid-frame pier shall be calculated from Equation (8.3.3) according to its failure mode.

$$\left.\begin{array}{l} P_a = P_u \text{ (Flexural failure) (where } P_c < P_u) \\ P_a = P_u \text{ (Shear failure after flexural yielding)} \\ P_a = P_{s0} \text{ (Shear failure)} \end{array}\right\} \cdots\cdots\cdots \quad (8.3.3)$$

where

P_a: Seismic horizontal strength (N) of the reinforced concrete pier

P_c: Horizontal strength at cracking (N) of the reinforced concrete pier, which shall be calculated from Equation (8.3.4)

$$P_c = \frac{Z_c}{h}\left(\sigma_{bt} + \frac{N}{A}\right) \quad \cdots\cdots \quad (8.3.4)$$

where

Z_c : Section modulus of a column with consideration of longitudinal reinforcement at the section of bottom of pier (mm^3)

σ_{bt} : Flexural tensile strength of concrete (N/mm^2) to be calculated by Equation (8.3.5)

$$\sigma_{bt} = 0.23 \ \sigma_{ck}^{2/3} \quad \cdots\cdots \quad (8.3.5)$$

N : Axial force acting on the section of bottom of pier (N)

A : Sectional area of a column, with consideration of longitudinal reinforcement at the section of bottom of pier (mm^2)

h : Height of superstructural inertial force from the bottom of pier. (mm)

σ_{ck} : Design strength of concrete (N/mm^2)

ii) The seismic horizontal strength in the in-plane direction of a reinforced-concrete single-column rigid-frame pier shall be calculated from Equation (8.3.6) according to its failure mode.

$$\left.\begin{array}{ll} P_a = P_u & \text{(Flexural failure type)} \\ P_a = P_u & \text{(Shear failure after flexural yielding type)} \\ P_a = P_i & \text{(Shear failure type)} \end{array}\right\} \quad \cdots\cdots \quad (8.3.6)$$

where

P_u: Ultimate horizontal strength (N) of the reinforced-concrete single-column rigid-frame pier specified in Clause 8.7

P_i: Limit value of shear force (N) at the *i*-th plastic hinge calculated according to the provisions of Clause 8.6

8.4 Limit States of Reinforced Concrete Piers

(1) For reinforced concrete piers in the failure mode of the flexural-failure type, when the following 1) and 2) in addition to the provisions of Clauses 8.9 to 8.11 are satisfied, the Limit state 1 of the reinforced concrete piers may be regarded as not exceeded.

1) Horizontal displacement that occurs in the reinforced concrete pier does not exceed the limit

value of horizontal displacement calculated from Equation (8.4.1).

$\delta_{yEd} = \xi_1 \cdot \phi_{RY} \cdot \delta_{yE}$ ・・・・・・・・・・・・・・・・・・・・ (8.4.1)

where

δ_{yEd}: Limit value of horizontal displacement (mm) corresponding to the Limit state 1 of the reinforced concrete pier

ξ_1: Modifier for structural modeling uncertainties, which shall be 1.00

ϕ_{RY}: Resistance factor, which shall be 1.00

δ_{yE}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 1 of the reinforced concrete pier; for reinforced concrete single column piers, it shall be calculated according to the provisions of Clause 8.5; for reinforced-concrete single-column rigid-frame piers, it shall be calculated according to the provisions of Clause 8.7.

2) Shear force that occurs in the reinforced concrete pier does not exceed the limit value of shear force specified in Clause 8.6.

(2) For reinforced concrete piers in the failure mode of the flexural-failure type, when the following 1) to 3) in addition to the provisions of Clauses 8.9 to 8.11 are satisfied, the Limit state 2 of the reinforced concrete piers may be regarded as not exceeded.

1) Horizontal displacement that occurs in the reinforced concrete pier does not exceed the limit value of horizontal displacement calculated from Equation (8.4.2).

$\delta_{ls2d} = \xi_1 \cdot \phi_s \cdot \delta_{ls2}$ ・・・・・・・・・・・・・・・・・・・・・・ (8.4.2)

where

δ_{ls2d}: Limit value of horizontal displacement (mm) corresponding to the Limit state 2 of the reinforced concrete pier for which plastic behavior is assumed

δ_{ls2}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 2 of the reinforced concrete pier for which plastic behavior is assumed; for reinforced concrete single column piers, it shall be calculated according to the provisions of Clause 8.5; for reinforced-concrete single-column rigid-frame piers, it shall be calculated according to the provisions of Clause 8.7.

ξ_1: Modifier for structural modeling uncertainties, which shall be 1.00

ϕ_s: Resistance factor, which shall be 0.65

2) Shear force that occurs in the reinforced concrete pier does not exceed the limit value of shear force specified in Clause 8.6.

3) The residual displacement δ_R occurring in the reinforced concrete pier that is calculated from Equation (8.4.3) does not exceed the limit value of residual displacement. This limit value of residual displacement shall appropriately be set according to the function that is required for the bridge after an earthquake. When a separate examination is not to be performed, it shall generally be set to 1/100 of the height of inertia force acing on the superstructure with respect to the bottom end

of the pier.

$$\delta_R = c_R(\mu_r - 1)(1 - r)\delta_{yE} \cdots\cdots\cdots\cdots \quad (8.4.3)$$

c_R : Modification factor for residual displacement, which shall be 0.6

r : Ratio of secondary stiffness post-yield to yield stiffness of reinforced concrete pier; it shall be 0.0

μ_r : Maximum response ductility factor of the reinforced concrete pier, which shall be a value obtained by dividing the maximum response displacement of the reinforced concrete pier by δ_{yE}; in static analysis, the maximum response ductility factor shall be calculated from Equation (8.4.4).

$$\mu_r = \frac{1}{2}\left\{\left(\frac{c_{2z}\,k_{hc0}\,W}{P_a}\right)^2 + 1\right\} \cdots\cdots\cdots\cdots \quad (8.4.4)$$

k_{hc0}: Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion, it shall be K_{Ih0} or K_{IIh0} specified in Clause 4.1.6 according to type of earthquake motion.

c_{2z}: Seismic zone factor for Level 2 Earthquake Ground Motion. Either c_{Iz} or c_{IIz} shall be used as per Clause 3.4 in accordance with the type of earthquake ground motion.

W: Equivalent weight (N), which shall be calculated from Equation (8.4.5)

$$W = W_U + c_p W_p \cdots\cdots\cdots\cdots \quad (8.4.5)$$

c_p : Coefficient of equivalent weight, which shall be 0.5

W_U : Weight of the superstructure portion (N) supported by the relevant reinforced concrete pier

W_P : Weight of the reinforced concrete pier (N)

(3) For reinforced concrete piers in the failure mode of the flexural-failure type, when the following 1) and 2) in addition to the provisions of Clauses 8.9 to 8.11 are satisfied, Limit state 3 may be regarded as not exceeded.

1) Horizontal displacement that occurs in the reinforced concrete pier does not exceed the limit value of horizontal displacement calculated from Equation (8.4.6).

$$\delta_{ls3d} = \xi_1 \cdot \xi_2 \cdot \phi_s \cdot \delta_{ls3} \cdots\cdots\cdots\cdots \quad (8.4.6)$$

where

δ_{ls3d}: Limit value of horizontal displacement (mm) corresponding to the Limit state 3 of the reinforced concrete pier for which plastic behavior is assumed

δ_{ls3}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 3 of the reinforced concrete pier for which plastic behavior is asumed; for reinforced concrete single column piers, it shall be calculated according to the provisions of Clause 8.5; for reinforced-concrete single-column rigid-frame piers, it shall be calculated according to the provisions of Clause 8.7.

ξ_1: Modifier for structural modeling uncertainties, which shall be 1.00

ξ_2: Modifier for the consequence of failure, which shall be 1.00

ϕ_s: Resistance factor, which shall be 0.65

2): Shear force that occurs in the reinforced concrete pier does not exceed the limit value of shear force specified in Clause 8.6.

(4) For reinforced concrete piers in the failure mode of the shear-failure-after-flexural-yielding type or the shear-failure type, when the following 1) and 2) are satisfied, the Limit state 1 of the reinforced concrete pier may be regarded as not exceeded.

1) Displacement that occurs in the reinforced concrete pier does not exceed the limit value of horizontal displacement calculated from Equation (8.4.1).

2) Shear force that occurs in the reinforced concrete pier does not exceed the limit value of shear force specified in Clause 8.6.

(5) For reinforced concrete piers in the failure mode of the shear-failure-after-flexural-yielding type or the shear-failure type, when (4)1) and (4)2) are satisfied, the Limit state 3 of the reinforced concrete pier may be regarded as not exceeded.

8.5 Horizontal Strength and Horizontal Displacement Corresponding to the Limit States of Reinforced Concrete Single Column Piers

For reinforced concrete single column piers, the characteristic value of horizontal displacement δ_{yE} corresponding to Limit state 1, that of horizontal displacement δ_{ls2} corresponding to Limit state 2, that of horizontal displacement δ_{ls3} corresponding to Limit state 3, yield strength Py corresponding to Limit state 1 (hereafter, referred to as "yield horizontal strength"), and ultimate horizontal strength Pu shall be calculated according to the following provisions 1) to 6). This calculation shall apply to the reinforced concrete single column piers with solid cross sections that satisfy the following: a longitudinal reinforcement ratio is up to 2.5%; a lateral confining reinforcement ratio is up to 1.8%; axial compressive stress at the bottom of the column is up to 3 N/mm^2; the types of the longitudinal reinforcements are SD345, SD390, and SD490; the type of the lateral confining reinforcements is SD345; the design reference strength of concrete is 21 to 30 N/mm^2.

1) Fiber strain is proportional to the distance from the neutral axis.

2) Skeleton curve between horizontal force and horizontal displacement relationship shall be expressed by an ideal elasto-plasticity model shown in Figure 8.5.1.

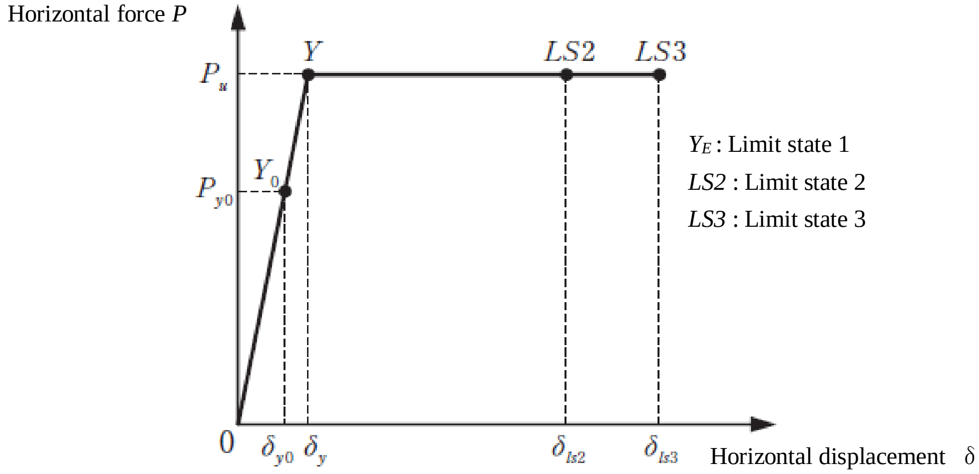

Figure 8.5.1 Model of Horizontal Force and Horizontal Displacement Relationship of a Single–Column Type Reinforced Concrete Pier

3) The stress-strain curve of concrete and that of reinforcements shall be determined according to the provisions of Clause 6.2.3. The limit compression strain of concrete ε_{ccl} shown in Figure 8.5.2 shall be calculated from Equation (8.5.1). The tensile strain ε_{st2} and ε_{st3} of longitudinal reinforcements corresponding to the Limit state 2 and Limit state 3 of reinforced concrete single column piers for which plastic behavior is assumed shall be based on Figure 8.5.3 and calculated from Equations (8.5.2) and (8.5.3), respectively.

$$\varepsilon_{ccl}=\varepsilon_{cc}+\frac{0.5\sigma_{cc}}{E_{des}} \quad \cdots\cdots\cdots\cdots (8.5.1)$$

$$\varepsilon_{st2}=0.025 \cdot L_p^{0.15}\ \phi^{-0.15}\ \beta_s^{0.2}\ \beta_{co}^{0.22} \quad \cdots\cdots\cdots\cdots (8.5.2)$$

$$\varepsilon_{st3}=0.035 \cdot L_p^{0.15}\ \phi^{-0.15}\ \beta_s^{0.2}\ \beta_{co}^{0.22} \quad \cdots\cdots\cdots (8.5.3)$$

Where

ε_{cc}: Strain when reaching the maximum compressive stress calculated from Equation (6.2.4)

ε_{ccl}: Limit compression strain of concrete confined by lateral confining reinforcements

σ_{cc}: Maximum compressive stress of concrete confined by lateral confining reinforcements (N/mm^2) calculated from Equation (6.2.3)

E_{des}: Descent gradient (N/mm^2) calculated from Equation (6.2.5)

ε_{st2}: Tensile strain of longitudinal reinforcements corresponding to Limit state 2

ε_{st3}: Tensile strain of longitudinal reinforcements corresponding to Limit state 3

ϕ: Diameter of longitudinal reinforcements (mm) that is used to calculate the tensile strain of the longitudinal reinforcements corresponding to each limit state from Equation (8.5.2) or (8.5.3)

$_p$: Length of a plastic hinge (mm) calculated from Equation (8.5.4)

$$\left.\begin{array}{l} L_p = 9.5\,\sigma_{sy}^{1/6}\,\beta_n^{-1/3}\,\phi' \\ \text{in which}\quad L_p \leqq 0.15h \end{array}\right\} \cdots\cdots\cdots \quad (8.5.4)$$

σ_{sy}: Yield stress of a longitudinal reinforcement (N/mm^2)

β_n: Spring constant representing resistance against a buckling of a longitudinal reinforcement (N/mm^2), to be calculated from Equation (8.5.5) regardless of the sectional configuration

$$\beta_n = \beta_s + \beta_{co} \cdots\cdots\cdots \quad (8.5.5)$$

β_s: Spring constant (N/mm^2), indicating the resistance of a lateral confining reinforcement; calculated from Equation (8.5.6)

$$\beta_s = \frac{384\,E_0\,I_h}{n_s\,d'^3\,s} \cdots\cdots\cdots \quad (8.5.6)$$

E_0: Young's modulus of a lateral confining reinforcement (N/mm^2)

I_h: Moment of inertia of a lateral confining reinforcement (mm^4)

d': Effective length of a lateral confining reinforcement (mm), used to calculate the plastic hinge length. The effective length shall be selected from the greatest value of the concrete divided by the lateral confining reinforcement placed parallel to the direction of application of the inertia force used for seismic design. For circular sections, it shall be a value of 0.8 times the diameter of concrete surrounded by lateral confining reinforcements arranged in the outermost position.

n_s: Number of compression-side longitudinal reinforcements arranged in the concrete portion in which the effective length d' of lateral confining reinforcements that is used for calculation of the plastic hinge length is the largest; when reinforcements are arranged into multiple tiers, it shall be the total number of the reinforcements.

s: Interval of a lateral confining reinforcement (mm)

β_{co}: Spring constant (N/mm^2), indicating the resistance of cover concrete; calculated from Equation (8.5.7)

$$\beta_{co} = 0.01c_0 \cdots\cdots\cdots \quad (8.5.7)$$

c_0: Distance from the outermost surface of a longitudinal reinforcement to be placed in the outermost concrete that has the lateral confining reinforcement with the largest effective length d' to be calculated for the plastic hinge length to the surface of the concrete (mm)

ϕ': Diameter of an longitudinal reinforcement to be placed in concrete that has the lateral confining reinforcement with the largest effective length *d'* to be calculated for the plastic hinge length (mm); when a longitudinal reinforcement with a diameter of 40 mm or more is used, 40 mm shall be used for ϕ'.

h: Distance from the pier lower end to the height of inertia force acting to the superstructure (mm)

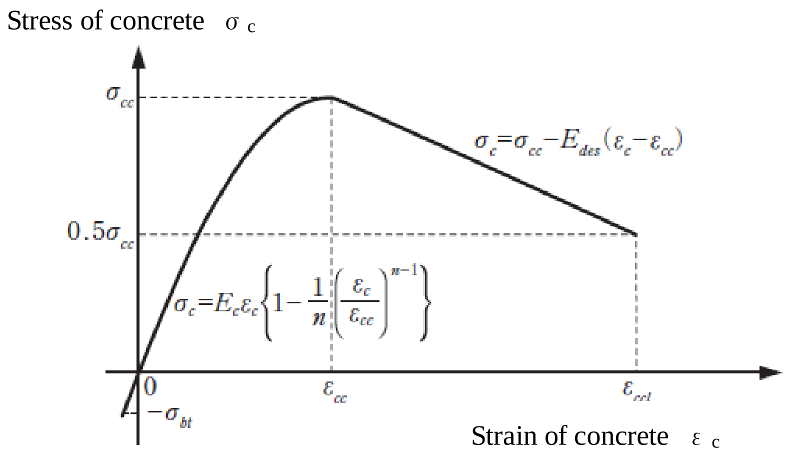

Figure 8.5.2 Stress-Strain Curve of Concrete

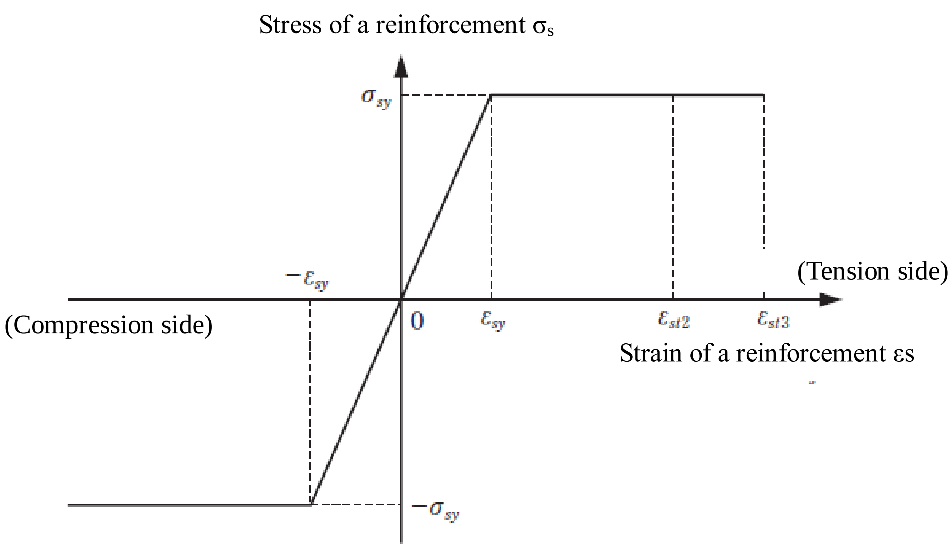

Figure 8.5.3 Stress-Strain Curve of Longitudinal Reinforcement

4) The Limit state 1 shall correspond to the elastic limit point of the ideal elasto-plasticity type skeleton curve shown in Figure 8.5.1. Yield horizontal strength P_y and the characteristic value of horizontal displacement δ_{yE} corresponding to Limit state 1 shall be calculated from Equations (8.5.8)

and (8.5.9), respectively.

$$P_y = \frac{M_{ls2}}{h} \quad \text{(8.5.8)}$$

$$\delta_y = \frac{M_{ls2}}{M_{y0}} \delta_{y0} \quad \text{(8.5.9)}$$

where

P_y: Yield horizontal strength (N) of the reinforced concrete single column pier

δ_{yE}: Characteristic value of horizontal displacement (mm) corresponding to Limit state 1

M_{ls2}: Bending moment (N•mm) in the cross section at the bottom of the pier corresponding to Limit state 2

δ_{y0}: Horizontal displacement (mm) that occurs when the tensile strain of longitudinal reinforcements reaches the yield strain at the position of the outermost longitudinal tensile reinforcements (hereafter, referred to as "initial yield displacement"); it shall be calculated on the basis of the curvature distribution obtained when the initial yield horizontal strength P_{y0} calculated from Equation (8.5.10) is applied to the height of inertia force acting on the superstructure.

$$P_{y0} = \frac{M_{y0}}{h} \quad \text{(8.5.10)}$$

where

P_{y0}: Horizontal strength (N) of the reinforced concrete single column pier when the outermost longitudinal tensile reinforcements yield

M_{y0}: Bending moment at the section of the bottom of pier at the time when a longitudinal tensile reinforcement at the outermost edge yields (N•mm)

5) Limit state 2 shall be the state that takes place when at the position of the outermost longitudinal tensile reinforcements, the tensile strain of longitudinal reinforcements reaches the tensile strain corresponding to the Limit state 2 of the reinforced concrete pier, or when at the position of the outermost longitudinal compressive reinforcements, the compression strain of concrete reaches the limit compression strain, whichever occurs earlier. The horizontal strength that occurs when this state takes place shall be defined as the ultimate horizontal strength P_u and calculated from Equation (8.5.11). The characteristic value of horizontal displacement δ_{ls2} corresponding to Limit state 2 shall be calculated from Equation (8.5.12).

$$P_u = \frac{M_{ls2}}{h} \quad \cdots \quad (8.5.11)$$

$$\delta_{ls2}=k_2 \cdot (\delta_{yE}+(\phi_{ls2}- \phi_y)L_p(h-L_p/2)) \quad \cdots \quad (8.5.12)$$

where

P_u: Ultimate horizontal strength (N) of the reinforced concrete single column pier

δ_{ls2}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 2 of the reinforced concrete single column pier

k_2: Modification factor, which shall be 1.3.

ϕ_{ls2}: Curvature (1/mm) that occurs when Limit state 2 is reached in the cross section at the bottom of the pier

ϕ_y: Curvature (1/mm) that occurs when Limit state 1 is reached in the cross section at the bottom of the pier; it shall be calculated from Equation (8.5.13).

$$\phi_y = \left(\frac{M_{ls2}}{M_{y0}}\right)\phi_{y0} \quad \cdots \quad (8.5.13)$$

ϕ_{y0}: Curvature at the time when a longitudinal tensile reinforcement at the outermost edge of the section of the bottom of pier yields (1/mm)

6) Limit state 3 shall be the state that takes place when at the position of the outermost longitudinal tensile reinforcements, the tensile strain of longitudinal reinforcements reaches the tensile strain corresponding to the Limit state 3 of the reinforced concrete pier, or when at the position of the outermost longitudinal compressive reinforcements, the compression strain of concrete reaches the limit compression strain, whichever occurs earlier. The characteristic value of horizontal displacement δ_{ls3} corresponding to Limit state 3 shall be calculated from Equation (8.5.14).

$$\delta_{ls3} = \delta_y + (\phi_{ls3} - \phi_y)L_p(h - L_p/2) \quad \cdots \quad (8.5.14)$$

where

δ_{ls3}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 3 of the reinforced concrete single column pier

ϕ_{ls3}: Curvature (1/mm) that occurs when Limit state 3 is reached in the cross section at the bottom of the pier

k_3: Modification factor, which shall be 1.3.

8.6 Limit Values of Shear Force of Reinforced Concrete Piers

The limit values of the shear force of reinforced concrete piers shall be calculated according to the provisions of Clause 6.2.4.

8.7 Earthquake resistance capacity and Horizontal Displacement Corresponding to the Limit States of Reinforced-Concrete Single-Story Rigid-Frame Piers

(1) The characteristic values of horizontal displacement corresponding to limit states in the out-of-pane direction of a reinforced-concrete single-story rigid-frame pier shall be calculated separately for each column member according to the provisions of Clause 8.5.

(2) The characteristic values of horizontal displacement corresponding to limit states in the in-plane direction of a reinforced-concrete single-story rigid-frame pier shall be calculated according to the provision 1). When a plastic hinge is to be considered for a beam member, the provision 2) shall be satisfied.

1) The characteristic value of horizontal displacement δ_{yE} corresponding to Limit state 1 and ultimate horizontal strength P_u of the reinforced-concrete single-story rigid-frame pier and the characteristic value of its horizontal displacement corresponding to Limit state 2 or Limit state 3 shall be calculated according to the provisions of Clause 8.5 and the following provisions i) and ii).

i) An analysis shall be performed by using an analysis model that allows changes in axial force acting on each column member and formation of plastic hinges in multiple locations to be considered.

ii) The Limit state 2 of a reinforced-concrete single-story rigid-frame pier judged to be a flexural-failure type pier shall be a state that takes place when all the plastic hinges formed in multiple locations reach the tensile strain of longitudinal reinforcements corresponding to the Limit state 2 specified in Clause 8.5 3), or when the compression strain of concrete reaches the limit compression strain. The Limit state 3 shall be a state that takes place when all the plastic hinges formed in multiple locations reach the tensile strain of longitudinal reinforcements corresponding to the Limit state 3 specified in Clause 8.5 3), or when the compression strain of concrete reaches the limit compression strain.

2) When a plastic hinge is to be considered for a beam member, the shear force that occurs in the relevant portion shall satisfy Inequality (8.7.1).

$$V_b / P_{si} \leqq 1 \quad \cdots\cdots \quad (8.7.1)$$

where

V_b: Shear force (N) that occurs in the beam member in a dominance situation by variable action

P_{si}: The limite value of shear force at the i-th plastic hinge reaches calculated according to the provision of Clause 8.6

8.8 Seismic Horizontal Strength and Limit States of Reinforced Concrete Single Column Piers under the Action of Eccentric Moment due to the Dead Load of a Superstructure and the Like

(1) The characteristic values of horizontal displacement corresponding to the seismic horizontal strength and limit states of a reinforced concrete single column pier under the action of eccentric moment due to the dead load of a superstructure and the like shall be calculated not only with consideration given to the provisions (2) to (5) but also according to the provisions of Clauses 8.3 and 8.4.

(2) The ultimate horizontal strength of a reinforced concrete single column pier that is used for judgment on its failure mode shall be calculated from Equation (8.8.1).

$$P_{uE} = P_u - \frac{M_0}{h} \quad \cdots\cdots \quad (8.8.1)$$

$$M_0 = D\, e \quad \cdots\cdots \quad (8.8.2)$$

where

P_{uE}: Ultimate horizontal strength (N) of the reinforced concrete single column pier under the action of eccentric moment due to the dead load of a superstructure and the like

M_0: Eccentric moment caused by dead load of the superstructural or others (N・mm)

P_u: Ultimate horizontal strength (N) of the reinforced concrete single column pier, which shall be determined according to the provisions of Clause 8.5 5)

h: Distance from the bottom of pier to the height of inertia force acting the superstructure

D: Dead weight of superstructural or others (N)

e: Eccentric distance from the centroid of the pier section to the center of gravity of the superstructure or others (mm)

(3) When the failure mode is the flexural-failure type or shear failure after flexural yielding type, seismic horizontal strength shall be calculated from Equation (8.8.3).

$$P_{aE} = P_{uE} \quad \cdots\cdots (8.8.3)$$

where

P_{aE}: Seismic horizontal strength (N) of the reinforced concrete single column pier under the action of eccentric moment due to the dead load of a superstructure and the like

(4) The maximum response ductility factor μ that is used for calculation of the residual displacement δ_R shall be a value obtained by dividing the maximum response displacement of the reinforced concrete pier by a value obtained by subtracting the initial displacement δ_{0E} occurring at the height of inertia force acting on the superstructure under the action of eccentric moment due to the dead load of the superstructure and the like from δ_{yE}.

(5) When the failure mode is the shear-failure type, the limit value of the shear force of the reinforced concrete single column pier under the action of eccentric moment due to the dead load of a superstructure and the like shall be calculated without considering the effects of the eccentric moment.

8.9 Structural Details of Reinforced Concrete Piers

8.9.1 General

(1) For reinforced concrete piers, the arrangement of reinforcements in regions for which plastic behavior during an earthquake is to be considered shall satisfy the following provisions 1) and 2) in order to reliably ensure ductility. For a reinforced concrete single column pier, when a plastic hinge is to be formed in the bottom of the pier, the region for which plastic behavior is to be considered shall be that starting at the bottom of the pier and having a length equivalent to 0.4 times the distance h from the bottom of the pier to the height of inertia force acting on the superstructure. For a reinforced-concrete single-story rigid-frame pier, the region for which plastic behavior is to be considered shall be that starting at the end of the beam and having a length equivalent to 0.4 times the following distance h: for column members, it shall be one-half of the height of the beam axis line from the bottom of the pier; for the beam, it shall be one-half of the distance between the centers of column members that support the beam at both ends.

1) Longitudinal reinforcements shall be arranged to ensure the seismic horizontal strength specified

in Clause 8.3.

2) Lateral confining reinforcements shall be arranged at intervals and in a manner that assure the effects of controlling buckling of longitudinal reinforcements, and confining concrete enclosed by lateral confining reinforcements.

(2) When the provisions of Clause 8.9.2 are complied with, the provision (1) may be regarded as satisfied.

(3) When consideration for structural design is to be made to the reinforced concrete pier according to the provision of Clause 2.7.2 2)i), the pier shall be designed to be of the failure mode of the flexural-failure type specified in Clause 8.3 and shall satisfy at least the provision (1).

(4) The seismic horizontal strength of the reinforced concrete pier shall satisfy Inequality (8.9.1).

$$P_a \geqq 0.4c_{2z}W \quad \cdots\cdots\cdots (8.9.1)$$

Where,

P_a: Seismic horizontal strength (N) of the reinforced concrete pier; it shall be calculated from Equation (8.3.3) for the out-of-plane direction of reinforced concrete single column piers or reinforced-concrete single-story rigid-frame piers; for the in-plane direction of reinforced-concrete single-story rigid-frame piers, it shall be calculated from Equation (8.3.6).

c_{2z}: Seismic zone factor for Level 2 Earthquake Ground Motion. Either $c_{\mathrm{I}z}$ or $c_{\mathrm{II}z}$ shall be used as per Clause 3.4 in accordance with the type of earthquake ground motion.

W: Equivalent weight (N), which shall be calculated from Equation (8.4.5); when the failure mode of the reinforced concrete pier is the shear-failure type, the value of c_p specified in Clause 8.4(2)3) shall be set to 1.0.

8.9.2 Structural Details for Ensuring Ductility

(1) In reinforced concrete piers, longitudinal reinforcements shall satisfy the structural details specified in Chapter 5 of Part IV and be arranged in such a manner that the longitudinal reinforcements will reliably function in case that covering concrete spalls off from regions for which plastic behavior is to be considered.

(2) Lateral confining reinforcements shall be arranged not only according to the provisions of Clause 6.2.5 but also in such a manner that hoop reinforcements among the lateral confining reinforcements will satisfy the following provisions 1) and 2).

1) The interval of arrangement of hoop reinforcements in regions for which plastic behavior is to be considered shall be smaller than or equal to a value that is selected from Table 8.9.1 according to the

diameter of the hoop reinforcements, and smaller than or equal to 0.2 times the sectional width. The sectional width shall be the length of the shorter side for rectangular sections and shall be the diameter for circular sections.

2) In regions other than regions for which plastic behavior is to be considered, the upper limit of the interval of arrangement of hoop reinforcements may be set to 300 mm. If the interval of arrangement of hoop reinforcements is to be changed in the height direction, the change of the interval shall be gradual.

Table 8.9.1 Upper Limit of the Interval of Hoop Reinforcements (mm)

Diameter of a hoop reinforcements φ_h (mm)	$13 \leqq \varphi_h < 20$	$20 \leqq \varphi_h < 25$	$25 \leqq \varphi_h < 30$	$30 \geqq \varphi_h$
Upper limit of the interval of hoop reinforcements (mm)	150	200	250	300

(3) In principle, longitudinal reinforcements shall not be cut off so that the seismic horizontal strength and ductility of a reinforced concrete pier will be exerted without fail. For tall piers higher than 30 m, longitudinal reinforcements may be cut off. In this case, however, cut-off shall be performed according to the provisions of Clause 8.10.

(4) Reinforcements shall be arranged so that a plastic hinge will not be formed in a joint region between a column member and a beam member of a rigid-frame pier.

5) When 1) to 3) below are satisfied, Clause 6.2.5(2)5) may be regarded as satisfied.

1) Member sections in the plastic zone or in a range under its influence shall be designed to be solid cross sections.

2) When a zone that reliably stays in the elastic range within a member is designed to be a hollow cross section, a taper shall be provided for the transition portion from a solid cross section to a hollow cross section taking stress transmission into consideration. The dimensions of the taper shall generally be as follows: the ratio of its width at the base to its axial width is 1:3; the width at the base is 0.5 times the thickness of the width of the hollow cross section.

3) For rectangular sections, a haunch shall be provided for corners within hollow cross sections, and reinforcing bars shall be installed to surround its joint regions. The dimensions of the haunch shall generally be as follows: the ratio of its width to its axial width is 1:1; the width is 0.5 times the thickness of the width of the hollow cross section.

8.10 Cut-off of Longitudinal Reinforcements of Reinforced Concrete Piers

(1) Longitudinal reinforcements shall not be cut off in regions for which plastic behavior is to be considered.

(2) Cut-off positions shall be determined in such a manner that plastic behavior will not occur earlier in regions other than those for which plastic behavior is assumed.

(3) For reinforced concrete single column piers, when cut-off positions are set according to Equation (8.10.1), the provision (2) may be regarded as satisfied. A cut-off position refers to the position of an end of a longitudinal reinforcement that will be anchored at the middle position.

$$h_i = h\left(1 - \frac{M_{yi}}{2M_{yB}}\right) + D \quad \cdots\cdots \quad (8.10.1)$$

where

h_i : Height from the column bottom to the i-th cut-off position of longitudinal reinforcement (mm)

h : Height from the column bottom to the height of inertia force acting to the superstructure(mm)

M_{yi}: Yield bending moment of the section on i-th cut-off portion of the longitudinal reinforcement from the column bottom (N・mm)

M_{yB}: Yield bending moment at the column bottom section (N・mm)

D : Embedment length (mm) that is set to ensure a required difference between the bending moment occurring in the cut-off cross section and the yield bending moment; it shall be set to the dimensions of the cross section of the pier: for rectangular sections, it shall be the length of the shorter side; for circular sections, it shall be the diameter.

(3) The reduction rate of longitudinal reinforcements shall generally be one-third or lower for each cut-off position. If cut-off is performed at heights different between the longitudinal and transverse directions to the bridge axis, the reduction rate shall be determined separately for each cut-off plane.

(4) In regions that have a length equivalent to the length of the shorter side or 1.5 times the diameter of the cross section of the pier both above and below each cut-off position, the interval of arrangement of hoop reinforcements shall be 150 mm or smaller. The interval of arrangement of hoop reinforcements shall be taken according to Clause 8.9.2(2)2) and shall not rapidly be changed.

8.11 Design of Joints between Reinforced Concrete Piers and Footings

Joints between reinforced concrete piers and footings shall be designed according to the provisions of Clause 7.5 of Part IV

CHAPTER 9 STEEL COLUMNS

9.1 Scope

This chapter shall apply to the seismic design of steel columns in which plastic behavior is assumed.

9.2 General

In designing a steel column for which plastic behavior is to be assumed, the following provisions 1) to 4) shall be satisfied.

1) Limit state 2 and Limit state 3 for steel pier shall be determined according to the provisions of Clause 9.3 and 9.4 respectively.

2) When a steel column is under the action of eccentric moment due to the dead load of the superstructure and the like, its effects shall appropriately be considered.

4) Joints with foundations shall be designed according to the provisions of Clause 9.6.

9.3 Limit State 2 and Limit State 3 of Steel Columns

(1) When a steel column, in which plastic behavior is assumed, satisfies not only the provisions of Clauses 9.5 and 9.6 but also the following 1) and 2), Limit state 2 may be regarded as not exceeded.

1) Horizontal displacement that occurs in the steel column does not exceed the limit value of horizontal displacement calculated from Equation (9.3.1).

$$\delta_{ls2d} = \xi_1 \cdot \phi_s \cdot \delta_{ls2} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (9.3.1)$$

where

δ_{ls2d}: Limit value of horizontal displacement (mm) corresponding to the Limit state 2 of the steel column for which plastic behavior is assumed

δ_{ls2}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 2 of the steel column for which plastic behavior is assumed; it shall be calculated according to the provisions of Clause 9.4.(4)

ξ_1: Modifier for structural modeling uncertainties, which shall be 1.00.

ϕ_s: Resistance factor, which shall be 0.75.

2) The residual displacement δ_R occurring in the steel column, that is calculated from Equation

(9.3.2), does not exceed the limit value of residual displacement. This limit value of residual displacement shall appropriately be set according to the function required for the bridge after an earthquake. It shall generally be set as 1/100 of the height of inertia force acing on the superstructure with respect to the bottom end of the pier, when a separate examination is not to be performed,.

$\delta_R = c_R (\mu_r - 1)(1 - r)\, \delta_y$ ・・・・・・・・・・・・・・・・・・・・・(9.3.2)

where

c_R: Modification factor for residual displacement, which shall be a value shown in Table 9.3.1

μ_r: Maximum response ductility factor of the steel column, which shall be a value obtained by dividing the maximum response displacement of the steel column by the yield displacement δ_y

r: Ratio of the secondary post-yield stiffness to the yield stiffness of the steel column; it shall be a value shown in Table 9.3.1.

δ_y: Yield displacement (mm) of the steel column, which is calculated according to the provision of Clause 9.4(6)

Table 9.3.1 Ratio *r* of the Secondary Post-Yield Stiffness to the Yield Stiffness and Modification factor C_R for residual displacement, used in Calculation of the Residual Displacement of a Steel Column,

Type of steel columns	r	c_R
Steel columns without concrete filling	0.2	0.45
Steel columns with concrete filling	0.1	0.45

(2) When a steel column, for which plastic behavior is assumed, satisfies not only the provisions of Clauses 9.5 and 9.6 but also the provision (1)1), Limit state 3 may be regarded as not exceeded.

9.4 Horizontal Strength and Horizontal Displacement Corresponding to the Limit States of Steel Columns

(1) The bending moment-curvature relationship of a steel column shall be determined according to the provisions of Clause 6.3.2.

(2) The stress-strain curve of steel column and concrete filled in the steel column shall be determined according to the provisions of Clause 6.3.3.

(3) The limit compressive strain ε_a of steels, which is shown in Figure 9.4.1, shall be calculated as follows according to the concrete filling and their cross-sectional shape, when the provisions for structural details of Clause 9.5 are satisfied.

For hollow steel columns, the calculation shall be made according to the provision 1) for rectangular sections, and according to the provision 2) for circular sections. For concrete-filled steel columns, the calculation shall be made according to the provision 3) for rectangular sections, and according to the provision 4) for circular sections. The limit strain ε_a calculated according to the provisions 1), 2), 3), or 4) shall be applied to steel columns that are formed from structural steels specified in Clause 9.1 of Part I excluding SM570, SMA570W, SBHS400, SBHS400W, SBHS500, and SBHS500W. In addition, if the limit strain is to be applied to a pier outside the applicable scopes of the following provisions 1) to 4), it shall appropriately be set according to the provisions of Clause 2.4.6.

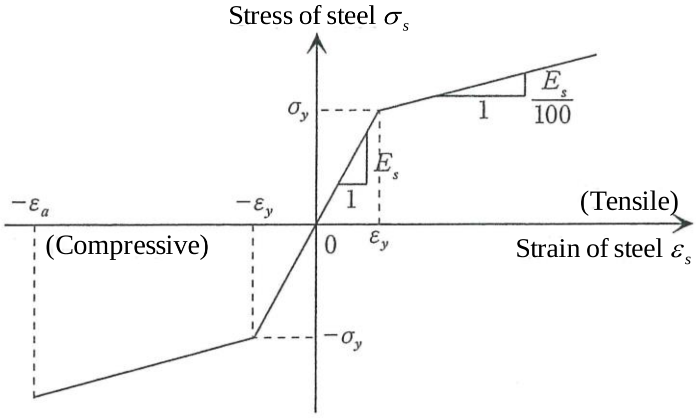

Figure 9.4.1 Stress-Strain Curve of Steel

1) The limit strain ε_a of a hollow steel column with a rectangular section shall be calculated from Equation (9.4.1). Equation (9.4.1) shall be applicable to the following hollow steel columns with a rectangular section: the steel columns shall be equipped with longitudinal stiffeners and a diaphragm, and the R_F, R_R, and γ_l/γ_l^* of their flange shall be almost equal to the R_F, R_R, and γ_l/γ_l^* of their web. Equation (9.4.1) shall be applicable to steel columns that satisfy the following conditions: $0.5 \leqq b_W/b_F \leqq 2.0$; $0.3 \leqq R_F \leqq 0.5$; $0.3 \leqq R_R \leqq 0.5$; $\gamma_l/\gamma_l^* \geqq 1.0$; $0.2 \leqq \bar{\lambda} \leqq 0.5$; $2.5 \leqq l'/b' \leqq 9.0$; $0 \leqq N/N_y \leqq 0.5$.

$$\varepsilon_a = \left\{ \frac{(1.58 - N/N_y)^{3.16} \times (1.68 - R_R)^{2.48} \times (0.65 - R_F)^{0.41} \times (23.87 - l'/b')^{2.9} \times (\alpha')^{0.3}}{2500 \times (N/N_y + 1.0) \times (b_W/b_F)^{0.17}} + 0.5 \right\} \varepsilon_y$$

………………………………………………………………… (9.4.1)

b_W: Overall width (mm) of the stiffened plate (web) specified in Clause 5.4.3 of Part II

b_F: Overall width (mm) of the stiffened plate (flange) specified in Clause 5.4.3 of Part II

$$b' = \frac{b_W + b_F}{2} \quad \text{(9.4.2)}$$

$$l' = \frac{l}{2} \quad \text{(9.4.3)}$$

l: Effective buckling length (mm) specified in Clause 17.3 of Part II

N: Axial force (N) acting on steel column

N_y: Axial force (N) when the full face of the steel cross section yields; this value is obtained by multiplying the sectional area of the steel cross section by yield strength σ_y of steel

R_R: Width-thickness ratio parameter of the stiffened plate (flange) for the steel cross section for which plastic behavior is assumed, which shall be calculated from Equation (9.4.4)

$$R_R = \frac{b_F}{t_F}\sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\mu^2)}{\pi^2 k_R}} \quad \text{(9.4.4)}$$

t_F: Plate thickness (mm) of stiffened plate (flange) specified in Clause 5.4.3 of Part II

μ: Poisson's ratio of steel

k_R: Buckling coefficient ($= 4n^2$)

E: Young's modulus of steel (N/mm^2), which is shown in Table 4.2.1 of Part II

n: Number of panels separated by longitudinal stiffeners specified in Clause 5.4.3 of Part II

R_F: Width-thickness ratio parameter of the stiffened plate (flange) for the steel cross section for which plastic behavior is assumed, which shall be calculated from Equation (9.4.5)

$$R_F = \frac{b_F}{t_F}\sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\mu^2)}{\pi^2 k_F}} \quad \text{(9.4.5)}$$

k_F: Buckling coefficient, which shall be calculated from Equation (9.4.6)

$$\left.\begin{aligned} k_F &= \frac{(1+\alpha^2)^2 + n\gamma_l}{\alpha^2(1+n\delta_l)} \quad (\alpha \leqq \alpha_0) \\ k_F &= \frac{2\left(1+\sqrt{1+n\gamma_l}\right)}{1+n\delta_l} \quad (\alpha > \alpha_0) \end{aligned}\right\} \quad \text{(9.4.6)}$$

α: Aspect ratio of the stiffened plate (flange) specified in Clause 5.4.3 of Part II

α_0: Critical aspect ratio specified in Clause 5.4.3 of Part II

δ_l: Cross-section ratio of one longitudinal stiffener specified in Clause 5.4.3 of Part II

γ_l: Stiffness ratio of longitudinal stiffeners specified in Clause 5.4.3 of Part II

γ_l^*: Stiffness ratio of longitudinal stiffeners, which shall be calculated from Equation (9.4.7)

$$\left.\begin{aligned} \gamma_l^* &= 4\alpha^2 n(1+n\delta_l) - \frac{(1+\alpha^2)^2}{n} \quad (\alpha \leqq \sqrt[4]{1+n\gamma_l}) \\ \gamma_l^* &= \frac{1}{n}\left[\left\{2n^2(1+n\delta_l)-1\right\}^2 - 1\right] \quad (\alpha > \sqrt[4]{1+n\gamma_l}) \end{aligned}\right\} \cdots\cdots (9.4.7)$$

$$\alpha' = a/b' \cdots\cdots (9.4.8)$$

a: Interval of lateral stiffeners (mm) specified in Clause 5.4.3 of Part II

$\bar{\lambda}$: Slenderness ratio parameter, which shall be calculated from Equation (9.4.9)

$$\bar{\lambda} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{l}{r} \cdots\cdots (9.4.9)$$

r: Radius (mm) of rotation of steel cross section with respect to an axis parallel with flange

2) The limit strain ε_a of a hollow steel column with a circular section shall be calculated from Equation (9.4.10). Equation (9.4.10) shall be applicable to steel columns that satisfy the following conditions: $0.03 \leqq R_t \leqq 0.08$; $0.2 \leqq \bar{\lambda} \leqq 0.4$; $0 \leqq N/N_y \leqq 0.2$.

$$\varepsilon_a = (20 - 140R_t)\,\varepsilon_y \cdots\cdots (9.4.10)$$

where:

R_t: Radius-thickness ratio parameter of the steel cross section for which plastic behavior is to be considered, which shall be calculated from Equation (9.4.11)

$$R_t = \frac{R}{t}\frac{\sigma_y}{E}\sqrt{3(1-\mu^2)} \cdots\cdots (9.4.11)$$

R: Radius (mm) at the center of plate thickness

t: Wall thickness (mm) of steel pipe

3) The limit strain ε_a of a concrete-filled steel column with a rectangular section shall be calculated from Equation (9.4.12). In this case, Equation (9.4.12) shall be applicable to calculation for a concrete-filled rectangular-section steel column that has a longitudinal stiffener and a diaphragm with the R_F, R_R, and $\gamma_l/\gamma_{l\bullet req}$ of its flange being respectively almost equal to the R_F, R_R, and $\gamma_l/\gamma_{l\bullet req}$ of its web. The applicable range shall be as follows: $0.5 \leqq b_W/b_F \leqq 2.0$; $0.3 \leqq R_F \leqq 0.5$; $0.3 \leqq R_R \leqq 0.5$; $\gamma_l/\gamma_{l\bullet req} \geqq 1.0$; $0.2 \leqq \bar{\lambda} \leqq 0.5$; $2.5 \leqq l'/b' \leqq 9.0$; $0 \leqq N/N_y \leqq 0.5$; the design reference strength of the filled concrete is no more than 24N/ mm^2

For concrete-filled steel columns with a rectangular section under action of axial force satisfying $0 \leqq N/Ny \leqq 0.2$, however, the applicable range of the limit strain ε_a calculated from Equation (9.4.12) may be as follows $0.5 \leqq b_W/b_F \leqq 2.0$; $0.3 \leqq R_F \leqq 0.7$; $0.3 \leqq R_R \leqq 0.7$; $\gamma_l/\gamma_{l\bullet req} \geqq 1.0$; $0.2 \leqq \bar{\lambda} \leqq 0.5$; $2.5 \leqq l'/b' \leqq 9.0$.

$$\varepsilon_a = 7\varepsilon_y \qquad \cdots\cdots (9.4.12)$$

where:

$\gamma_{l\bullet req}$: Required stiffness ratio of longitudinal stiffeners specified in Clause 5.4.3 of Part II

4) The limit strain ε_a of a concrete-filled steel column with a circular section shall be calculated from Equation (9.4.13). Equation (9.4.13) shall be applicable to concrete-filled steel columns with a circular section equipped with a diaphragm, when the following conditions are satisfied: $0.03 \leqq R_t \leqq 0.12$; $0.2 \leqq \bar{\lambda} \leqq 0.4$; $0 \leqq N/N_y \leqq 0.2$.

$$\varepsilon_a = 5\varepsilon_y \qquad \cdots\cdots (9.4.13)$$

(4) The characteristic value δ_{ls2} of horizontal displacement corresponding to the Limit state 2 of the steel column shall be calculated from Equation (9.4.14).

$$\delta_{ls2} = k \cdot \delta_a \qquad \cdots\cdots (9.4.14)$$

where:

δ_{ls2}: Characteristic value of horizontal displacement (mm) corresponding to the Limit state 2 of the steel column

δ_a: Horizontal displacement (mm) corresponding to the maximum horizontal force of the steel column in the bending moment-curvature relationship that is set for the steel column according to the provision (1) to (3)

k: Modification factor, which shall be 1.3

(5) The horizontal strength Pu of the steel column, which is applied for the design of joint with foundation and the design of column foundation, shall be calculated from Equation (9.4.15). In this case, the effects of eccentric moment due to the dead load of a superstructure and the like shall not be considered.

$$P_a = \frac{M_{a0}}{h} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{(9.4.15)}$$

where:

P_u: Horizontal strength (N) of the steel column

M_{a0}: Bending moment (N･mm) that occurs in the cross section at the bottom of the steel column when the steel column reaches the limit strain ε_a that is calculated without considering the effects of eccentric moment due to dead load of a superstructure or the like

h: Distance(mm) from the bottom of the steel column to the height of inertia force acting on the superstructure

(6) The yield displacement δ_y of the steel column shall be calculated as the horizontal displacement that occurs at (ϕ_y, M_y) when it is a concrete-filled steel column or a hollow steel column with a rectangular section, and as the horizontal displacement that occurs at (ϕ_{yt}, M_{yt}) when it is a hollow steel column with a circular section.

9.5 Structural Details of Steel Columns

9.5.1 General

(1) A steel column shall be designed to have a structure for preventing brittle failure and reliably ensuring deformability.

(2) When the provisions of Clause 9.5.2 are complied with, the provision (1) may be regarded as satisfied.

(3) The horizontal strength of steel columns shall satisfy Inequality (9.5.1).

$$P_u \geqq 0.4 c_{2z} W \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{(9.5.1)}$$

where:

P_u: Seismic horizontal strength (N) of the steel column

c_{2z}: Modification factor for zones for Level 2 Earthquake Ground Motion, for which cIz or cIIz specified in Clause 3.4 shall be used according to the type of the earthquake ground motion

W: Equivalent weight (N) which shall be calculated from Equation (8.4.5)

9.5.2 Structural Details for Ensuring Deformability

1) Hollow steel columns shall comply with the provision of Clause 6.3.4(1)
Concrete-filled steel columns shall comply with the provision of Clause 6.3.4(2).

9.6 Design of Joints between Steel Columns and Foundation

(1) A joint that connects a steel column to a foundation shall be designed to have a structure that will reliably transmit the sectional force at each of the portions of the steel column and the foundation other than the joint in case that they reach Limit state 3.
(2) When plastic behavior is to be considered for the steel column, its joint shall be designed to be capable of transmitting sectional force equivalent to the horizontal strength of the steel column calculated from Equation (9.4.15).
(3) A load-bearing capacity mechanism for sharing stress occurring in the joint and the limit states of joint shall appropriately be set according to the structural type of the joint.

CHAPTER 10　PIER FOUNDATIONS

10.1　Scope

This chapter shall apply to the seismic design of pier foundations in design situations in which Level 2 Earthquake Ground Motion is to be considered.

10.2　General

In designing a pier foundation, the following provisions 1) to 6) shall be satisfied.

1) The response values of the pier foundation shall be calculated by considering the forces acting on the pier foundation as specified in Clause 10.3. When it is judged according to the provisions of Clause 7.2 that liquefaction that affects the bridge will occur, response values shall be calculated both under the condition that the effects of liquefaction will occur and under the condition that the effects of liquefaction will not occur. When it is judged according to the provisions of Clause 4.4.2 that lateral spreading that affects the bridge will occur, response values shall be calculated by considering only its effect.

2) In cases where Clauses 10.3(1)1) and 2) are to be considered, when the plastic behavior of the pier foundation is assumed, the response ductility factor and response displacement of the pier foundation shall be calculated according to the provisions of Clause 10.4. Sectional force that occurs in each member shall be the value that occurs when the response ductility factor and response displacement which calculated as above occur.

) In cases where Clause 10.3(1)3) is to be considered, the horizontal displacement of the top of the pier foundation shall be calculated. Sectional force that occurs in each member shall be the value that occurs when the horizontal displacement which calculated as above occurs.

4) In cases where Clauses 10.3(1)1) and 2) are to be considered, the characteristic values or limit values corresponding to the limit states of a pile foundation, a caisson foundation, a steel pipe sheet pile foundation, a diaphragm wall foundation, and a caisson type pile foundation shall be determined according to the provisions of Clauses 10.9, 11.9, 12.10, 13.9, and 14.8 of Part IV, respectively.

5) In cases where Clause 10.3(1)3) is to be considered, when the horizontal displacement at the top of the foundation does not exceeded two times the one when the foundation yields, the pier foundation may be regarded as not exceeding Limit state 2. In calculating the characteristic values of resistance corresponding to limit states in this case, the horizontal resistance of the soil layer within the range in which lateral spreading force of the ground needs to be considered shall not be taken into account.

6) The partial effect of the ground vibration displacement specified in the provision of 2.3 shall be taken into necessary consideration with proper regard for structural condition, ground condition and the like. The necessary consideration is deemed to be made if a flexible structure such as a pile foundation has enough ductility against the ground vibration displacement, at least around the boundary of soil layers, where the depth direction distribution of the ground vibration displacement suddenly changes.

10.3 Forces Acting on a Pier Foundation

(1) Forces acting on a pier foundation shall be the following 1) to 3).

1) Inertia force of the structure specified in Clause 4.1

2) Inertia force equivalent to the design horizontal seismic coefficient on the ground surface specified in Clause 4.1.6 for the structural portion from the ground surface in seismic design specified in Clause 3.5 to the ground surface of the earth

3) Lateral spreading force of the ground specified in Clause 4.4

(2) When the plastic behavior of the pier is assumed, the inertia force of the structure specified in the provision (1)1) shall be determined by considering the design horizontal seismic coefficient calculated from Equation (10.3.1).

$k_{hp} = c_{dF}\, k_{hN}$ ・・・・・・・・・・・・・・・・・・・・・・ (10.3.1)

where

k_{hp}: Design horizontal seismic coefficient of the pier foundation (the value shall be rounded off to two decimal places.)

c_{dF}: Modification factor for calculation of the design horizontal seismic coefficient of the pier foundation; the value shall be 1.10.

k_{hN}: Design horizontal seismic coefficient that is obtained by converting the sectional force occurring at the bottom of the pier during an earthquake; it shall be calculated from Equation (10.3.2) when the pier develops plastic behavior.

$k_{hN} = P_u / W$ ・・・・・・・・・・・・・・・・・・・・・・・ (10.3.2)

P_u: Horizontal strength (N) of the pier supported by the pier foundation; it shall be the ultimate horizontal strength calculated according to the provisions of Clause 8.5 5) for reinforced concrete piers; it shall be the horizontal strength calculated according to the provisions of Clause 9.4(5) for steel columns.

W: Equivalent weight (N), which shall be calculated from Equation (8.4.5); for reinforced concrete piers in the failure mode of the shear failure type, the coefficient of equivalent weight *cp* specified in Clause 8.4(2)3) shall be set to 1.0.

10.4 Calculation of Response Ductility Factor and Response displacement of a Pier Foundation when its Plastic Behavior Is Assumed

(1) When the plastic behavior of a pier foundation is assumed, its response ductility factor and response displacement shall be calculated from Equations (10.4.1) and (10.4.2) by using its design horizontal seismic coefficient specified in the provision (2).

$$\mu_{Fr} = \frac{1}{r}\left\{-(1-r) + \sqrt{1-r+r(k_{hF}/k_{hyF})^2}\right\} \quad (r \neq 0) \cdots\cdots(10.4.1)$$

$$\mu_{Fr} = \frac{1}{2}\left\{1 + (k_{hF}/k_{hyF})^2\right\} \quad (r = 0) \cdots\cdots(10.4.2)$$

where:

μ_{Fr}: Response ductility factor of the pier foundation

δ_{Fr}: Response displacement of a superstructure at the acting point of its inertia force, caused by the foundation deformation (m)

$_{Fy}$: Yield displacement of the pier foundation, which is determined separately for each foundation type according to the provisions of Clause 10.9, 11.9, 12.10 and 13.9 of Part IV (m).

: Ratio of secondary stiffness to yielding stiffness of the pier foundation.

k_{hyF}: Horizontal seismic coefficient at yielding of the foundation (the value shall be rounded off to two decimal places.)

k_{hcF}: Design horizontal seismic coefficient of the pier foundation for which plastic behavior is expected; it shall be specified in the provision (2) (the value shall be rounded off to two decimal places.)

(2) The design horizontal seismic coefficient that will be used for calculating the response ductility factor and response displacement of the pier foundation shall be calculated from Equation (10.4.3).

$$k_{hF} = c_D c_{2z} k_{h0} \cdots\cdots(10.4.3)$$

where:

k

c_D: Modification factor for damping ratio, which shall appropriately be set by considering the effects of dissipation of vibration energy to surrounding soil and those of the nonlinearity of the foundation body and ground resistance. It shall generally be set to 1.00 for spread foundations and 2/3 for caisson foundations, pile foundations, steel-pipe sheet pile foundations, and diaphragm wall foundations.

c_{2z}:	Modification factor for zones for Level 2 Earthquake Ground Motion; either c_{Iz} or c_{IIz} that is specified in Clause 3.4 is used in accordance with the type of earthquake ground motion.
k_{h0}:	Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion; either k_{Ih0} or k_{IIh0} that is specified in Clause 4.1.6 is used in accordance with the type of the earthquake ground motion specified in Clause 4.1.6

CHAPTER 11 ABUTMENTS AND FOUNDATIONS OF ABUTMENTS

11.1 Scope

This chapter shall apply to the seismic design of abutments and the foundations of abutments in design situations in which Level 2 Earthquake Ground Motion is to be considered. When abutments and the foundations of abutments have the same vibration characteristics as those of piers, however, the seismic design of the abutments and the foundations of abutments shall comply with the provisions of Chapters 8 and 10.

11.2 General

In designing an abutment and the foundation of the abutment, the following provisions 1) to 5) shall be satisfied.

1) The response values of the abutment and the foundation of the abutment shall be calculated by considering the forces acting on the abutment and the foundation of the abutment as specified in Clause 11.3.

2) When the plastic behavior of the foundation of the abutment is assumed, the response ductility factor of the foundation of the abutment shall be calculated according to the provision of Clause 11.4. Sectional force that occurs in each member shall be the value that occurs when this calculated value of the response ductility factor as above occurs.

3) The characteristic values or limit values corresponding to the limit states of a pile foundation, a caisson foundation, a steel pipe sheet pile foundation, a diaphragm wall foundation, and a caisson type pile foundation shall be determined according to the provisions of Clauses 10.9, 11.9, 12.10, 13.9, and 14.8 of Part IV, respectively.

34) Except as coming under the following i) or ii), when the abutment exceeds neither Limit state 1 nor Limit state 3 in the design situation in which Level 1 Earthquake Ground Motion is to be considered, it may be regarded as an abutment that will exceed neither Limit state 2 nor Limit state 3 in the design situation in which Level 2 Earthquake Ground Motion is to be considered.

i) When the abutment is on the ground that contains a soil layer judged according to the provisions of Clause 7.2 to be a layer in which liquefaction that affects the bridge will occur.

ii) When the load support condition of the abutment against Level 2 Earthquake Ground Motion is different from the one against Level 1 Earthquake Ground Motion.

5) The partial effect of the ground vibration displacement specified in the provision of 2.3 shall be taken into necessary consideration with proper regard for structural condition, ground condition and

the like. The necessary consideration is deemed to be made if a flexible structure such as a pile foundation has enough ductility against the ground vibration displacement, at least around the boundary of soil layers, where the depth direction distribution of the ground vibration displacement suddenly changes.

2)

11.3 Forces Acting on an Abutment and the Foundation of the Abutment

(1) Forces acting on an abutment and the foundation of the abutment shall be the following 1) and 2).

1) Earth pressure during an earthquake specified in Clause 4.2

2) Inertia force of the structure and the ground on footing specified in Clause 4.1

(2) The design horizontal seismic coefficient that will be used for calculating the acting forces listed in the provision (1) other than inertia force from superstructures shall be calculated from Equation (11.3.1) on the basis of the design horizontal seismic coefficient of the ground surface specified in the provision of Clause 4.1.6(5).

$$k_{hA} = c_A c_{2z} k_{hg0} \quad \cdots\cdots\cdots\cdots \quad (11.3.1)$$

where:

k_{hA}: Design horizontal seismic coefficient that will be used for designing the abutment and the foundation of the abutment (the value shall be rounded off to two decimal places.)

c_A: Modification factor for the design horizontal seismic coefficient of the abutment and the foundation of the abutment, which shall generally be 1.00

c_{2z}: Modification factor for zones for Level 2 Earthquake Ground Motion; either $c_{\mathrm{I}z}$ or $c_{\mathrm{II}z}$ that is specified in Clause 3.4 is used in accordance with the type of earthquake ground motion.

k_{hg0}: Standard value of the design horizontal seismic coefficient on the ground surface for Level 2 Earthquake Ground Motion; either $k_{\mathrm{I}hg0}$ or $k_{\mathrm{II}hg0}$ that is specified in Clause 4.1.6(5) is used in accordance with the type of the earthquake ground motion.

11.4 Calculation of the Response Ductility Factor of the Foundation of an Abutment when its Plastic Behavior Is to Be Assumed

(1) When the plastic behavior of the foundation of an abutment is assumed, the response ductility factor of the foundation of the abutment shall be calculated from Equation (11.4.1) by appropriately considering the nonlinear behavior of the foundation, the effects of earth pressure and the like.

$$\mu_{Ar} = \delta_{Ar} / \delta_{Ay} \quad \cdots\cdots (11.4.1)$$

$$\delta_{Ar} = \mu'_{Ar} \delta'_{Ay} + \delta_0 \quad \cdots\cdots (11.4.2)$$

$$\delta_{Ay} = \delta'_{Ay} + \delta_0 \quad \cdots\cdots (11.4.3)$$

$$\left.\begin{aligned} \mu'_{Ar} &= \frac{1}{r}\left\{-(1-r)+\sqrt{1-r+r(k_{hA}/k_{hyA})^2}\right\} \quad (r \neq 0) \\ \mu'_{Ar} &= \frac{1}{2}\left\{1+(k_{hA}/k_{hyA})^2\right\} \quad (r=0) \end{aligned}\right\} \cdots\cdots (11.4.4)$$

where:

μ_{Ar}: Response ductility factor of the abutment foundation

δ_{Ar}: Horizontal displacement at the acting point of suprerstructural inertia force, which is caused by deformation of the abutment foundation (m)

δ_{Ay}: Yield displacement of the abutment foundaiton, which is determined separately for each foundation type according to the provisions of Clause 10.9.2, 11.9.2, 12.10.2, 13.9.2, or 14.8.2 of Part IV (m)

μ'_{Ar}: Response ductility factor of the abutment foundation assuming that $k_h = 0$ and $\delta_A = \delta_0$ are taken as the origin

δ'_{Ay}: Yield displacement of the abutment foundation assuming that $k_h = 0$ and $\delta_A = \delta_0$ are taken as the origin (m)

δ_A: Horizontal displacement at the acting point of superstructural inertia force (m)

δ_0: Horizontal displacement at the acting point of suprerstructural inertia force due to the active earth pressure during earthquake which shall be calculated by assuming $k_{hA} = 0$ (m)

r: Ratio of the secondary stiffness to yielding stiffness of the abutment foundation.

k_{hyA}: Horizontal seismic coefficient at yielding of the abutment foundation (the value shall be rounded off to two decimal places.)

k_{hA}: Design horizontal seismic coefficient of the abutment and the abutment foundation, which is calculated from Equation (11.3.1)

CHAPTER 12 SUPERSTRUCTURES

12.1 Scope

This chapter shall apply to the seismic design of superstructures in design situations in which Level 2 Earthquake Ground Motion is to be considered.

12.2 General

(1) In cases where a superstructure satisfies the structural details specified in Clause 12.5, it may be regarded as a superstructure that does not exceed the Limit state 1 of the superstructure, when all the members and the like that compose the superstructure do not exceed the Limit state 1 of the members and the like specified in Clause 6.1; it may be regarded as a superstructure that does not exceed the Limit state 3 of the superstructure, when all the members and the like that compose the superstructure do not exceed the Limit state 3 of the members and the like specified in Clause 6.1.

(2) When a prestressed concrete boxed girder that receives bending moment and axial force satisfies the provisions of Clause 12.3, it may be regarded as a girder that does not exceed Limit state 1.

3) In cases where plastic behavior of a prestressed concrete boxed girder that receives bending moment and axial force satisfies the provisions of Clause 12.4 is expected, it may be regarded as a girder that does not exceed Limit state 3.

12.3 Limit State 1 of Prestressed Concrete Boxed Girders

(1) When a prestressed concrete boxed girder that receives bending moment and axial force satisfies not only the provisions of Chapter 5.2 to Chapter 5.4 of Part III but also the provisions of (2), it may be regarded as a girder that does not exceed Limit state 1.

(2) The limit at which although members and the like have suffered damaged, their behavior has reversibility shall be defined the Limit State 1. The characteristic values and limit values corresponding to this state shall be appropriately be set according to the provisions of Clause 6.4, and the corresponding response values shall not exceed the limit values.

12.4 Limit State 3 of Prestressed Concrete Boxed Girders

(1) When a prestressed concrete boxed girder that receives bending moment and axial force satisfies not only the provisions of Chapter 5.2 to Chapter 5.4 of Part III but also the provisions of (2), it may be regarded as a girder that does not exceed Limit state 3.

(2) The limit at which although members and the like lose reversibility of behavior, the load-carrying capacity is not completely lost shall be defined as Limit state 3. The characteristic values and limit values corresponding to this state shall appropriately be set according to the provisions of Clause 6.4, and the corresponding response values shall not exceed the limit values.

12.5 Structural Details

12.5.1 Structural Details of Superstructures

(1) The structural details of steel superstructures shall comply with the provisions of Chapters 5 and 9 to 19 of Part II.

(2) The structural details of concrete superstructures shall comply with the provisions of Chapter 5 and Chapter 7 to 16 of Part III.

(3) When measures are taken according to Clause 2.7.2 2) i) to prevent brittle failure from easily occurring in members, the structural details shall be appropriately set to ensure the ductility. The reinforced members and steel members shall satisfy at least the provisions of Clauses 6.2.5 and 6.3.4.

12.5.2 Structural Details of the Joints between Bearing Supports and Superstructures

(1) For the joints between bearing supports and steel superstructures, stiffeners shall be provided in the portions of the steel superstructures immediately above the bearing ends and the like to prevent partial deformation that may be developed due to concentrated loads on those portions, and cross beams, diaphragms, or the like shall be provided to reinforce the joints so as to prevent the bridge from developing out-of-plane deformation due to seismic force that acts in the transverse direction to the bridge axis.

(2) The joints between bearing supports and concrete superstructures shall comply with the provisions of Clause 10.5 of Part III.

CHAPTER 13 CONNECTIONS BETWEEN SUPERSTRUCTURES AND SUBSTRUCTURES

13.1 Bearing Supports

13.1.1 Forces Acting on a Bearing Support

(1) The forces acting on a bearing support shall be set in consideration of the structural type of the bridge, the types of bearings, the load distribution among the bearings, and the like.

(2) When provisions (3) and (4) are complied with, provision (1) may be regarded as satisfied.

(3) The force due to the effects of earthquakes among the horizontal forces that act on a bearing support shall be the inertia force of the superstructure specified in Clause 4.1. When plastic behavior is to be expected for a reinforced concrete pier or the foundation in statically analysis, it shall be the horizontal force that occurs at the height of the inertia force acting on the superstructure when response displacement of pier and foundation to be expertly plastic behabor reaches its maximum.

(4) The force due to the effects of earthquakes among the vertical forces that act on a bearing support shall be determined according to provisions 1) and 2) below.

1) the values calculated from Equations (13.1.1) and (13.1.2). Both the normal direction of vertical forces and that of their reaction forces shall be the downward direction.

$$R_{B\cancel{i}max} = R_{D\cancel{i}} + \sqrt{R^2_{HEQ\cancel{i}} + R^2_{VEQ\cancel{i}}} \quad \cdots\cdots\cdots \quad (13.1.2)$$

$$R_{B\cancel{i}min} = R_{D\cancel{i}} + \sqrt{R^2_{HEQ\cancel{i}} + R^2_{VEQ\cancel{i}}} \quad \cdots\cdots\cdots \quad (13.1.3)$$

Where

R_{Bmax}: Maximum value (kN) of vertical force that occurs in the bearing supports

R_{Bmin}: Minimum value (kN) of vertical force that occurs in the i-th bearing supports

R_D: Reaction force (kN) that occurs in the bearing supports due to the dead load of the superstructure

R_{HEQ}: Vertical reaction force (kN) that occurs in the bearing supports under the action of the horizontal force calculated according to provision (3)

R_{VEQ}: Vertical reaction force (kN) that occurs in the bearing supports in response to the design vertical seismic coefficient. It shall be calculated from Equation (13.1.3)

$R_{VEQ} = \pm k_v R_D$ ·· (13.1.3)

k_V: Design vertical seismic coefficient, which shall be a value obtained by multiplying the design horizontal seismic coefficient on the ground surface specified in Clause 4.1.5 by a factor shown in Table 13.1.1.

Table 13.1.1 Multiplying Coefficients for the design horizontal coefficient

	Level 1 Earthquake Ground Motion	Level 2 Earthquake Ground Motion	
		Type I	Type II
Multiplying Coefficients	0.5	0.5	0.67

2)

2) The vertical force shall be $-0.3R_D$ in the design situation in which Level 2 Earthquake Ground Motion is to be considered. However, this does not apply to cases with positive R_{Bmin} when a bearing support for which bearing support performance is ensured after earthquakes without confining vertical displacement is to be used.

13.1.2 Characteristic Values and Limit Values Corresponding to the Limit States of Bearing Supports

(1) The limit states of a bearing support shall be determined according to Clause 10.1.4 of Part I.

(2) When the limit states of members and the like that compose a bearing support are to beset , the limit values which is considered not to excess the limit states shall be set properly according to the provisions of Clause 2.4.6 (3) In setting provisions (1) and (2), ranges 1) and 2) below shall be considered.

1) Range in which the mechanical properties specified in provisions i) and ii) have been demonstrated by experiment

i) States in which functional loss occurs in the load-transmission function, the following deformation function, and the like that are required for bearings shall be identified, and safety against those states shall be able to be ensured.

ii) Stable behavior shall be exhibited without degradation in strength under repeated action due to an earthquake.

2) Range in which methods for analyzing the mechanical properties of bearings, such as their load-displacement relationship and damping characteristics, have clearly been established

(4) For range (3)1), at least experimental conditions 1) and 2) below shall be considered.

1) Resistant force mechanism appropriate to vertical and horizontal forces that act on the bearing support

2) Temperature and other environmental conditions under which bearings are supposed to be used

13.1.3 Verification of the Load Carrying Performance of Bearing Supports

When the responses of each portion of a bearing support under the action of the forces calculated according to Clause 13.1.1 satisfy 1) to 3) below, the bearing support may be regarded as a bearing support with the reliability required to not exceed its limit states.

1) Limit state 1 of the bearing support
The limit value of resistance of members and the like corresponding to Limit state 1 of the bearing support that is set according to the provisions of Clause 13.1.2 is not exceeded.

2) Limit state 2 of the bearing support
The limit value of resistance of members and the like corresponding to Limit state 2 of the bearing support that is set according to the provisions of Clause 13.1.2 is not exceeded.

3) Limit state 3 of the bearing support
The limit value of resistance of members and the like corresponding to Limit state 3 of the bearing support that is set according to the provisions of Clause 13.1.2 is not exceeded.

13.1.4 Attaching portion between Superstructures/Substructures

1) An attaching portion between a superstructure andsubstructure shall be designed to have a structure that will reliably achieve a load-bearing capacity mechanism that is appropriately planned to distribute according to the provisions of Clause 6.5.with consideration also given to repeated loading due to the action of earthquakes.

(

13.2 Expansion Gaps and Expansion Joints

13.2.1 Expansion Gaps

(1) For adjacent superstructures and the gap parts between superstructures and abutments/ columns, expansion gaps necessary for preventing collision shall generally be provided in design situations in which the effects of earthquakes are to be considered.

(2) When an expansion gap at the end of a superstructure is set to a value greater than or equal to

the value calculated from Equation (13.2.1), the expansion gap necessary for preventing collision between adjacent superstructures and the gap parts between superstructures and abutments/ columns may be regarded as having been provided.

$$S_{BR} = \begin{cases} c_B\, u_s + L_A & \text{(between superstructures adjacent in the longitudinal direction to the bridge axis)} \\ u_s + L_A & \text{(between a superstructure and an abutment, or a superstructure and a truncated portion of a pier head)} \end{cases} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (13.2.1)$$

Where

S_{BR} : Required joint gap width between the ends of adjacent superstructures (mm)

u_s : Maximum relative displacement (mm) that occurs between the superstructure and the substructure at the position for which the expansion gap width is calculated in the design situation in which Level 2 Earthquake Ground Motion is to be considered. In cases such as when two connected superstructures are supported with one pier when the expansion gap width is calculated by using the modification factor for natural period difference c_B, this value shall be set to the maximum relative displacement between whichever superstructure and substructure has the longer natural period in design vibration unit including each superstructure

L_A : Margine of joint gap width (mm)

c_B : Modification factor for natural period difference of the expansion gap width. It shall be set to a value that is selected from Table 13.2.1 according to the natural period difference ΔT of the two connected superstructures adjacent in the longitudinal direction to the bridge axis.

Table 13.2.1 Joint Gap Width Modification Factor for Natural Period Difference between Adjacent Girders c_B

Ratio of Natural Period Difference of adjacent girders $\Delta T/T_1$	c_B
$0 \leqq \Delta T/T_1 < 0.1$	1
$0.1 \leqq \Delta T/T_1 < 0.8$	$\sqrt{2}$
$0.8 \leqq \Delta T/T_1 \leqq 1.0$	1

Notes: $\Delta T = T_1 - T_2$, where T_1 and T_2 refer to the natural periods in design vibration unit including each superstructure of the two connected girders adjacent in the longitudinal direction to the bridge axis. However, T_1 is assumed equal to or greater than T_2.

13.2.2 Expansion Joints

(1) In design situations in which the effects of earthquakes are to be considered under the situation dominated by variable action, the expansion-contraction length of expansion joints shall be ensured longer than the value calculated from Equation (13.2.2) However, it shall be less than the value specified in Clause 10.3.3 of Part I,

$$\left.\begin{aligned} L_{ER} &= c_B \delta_R + L_A \quad \text{(between superstructures adjacent in the longitudinal direction to the bridge axis)} \\ L_{ER} &= \delta_R + L_A \quad \text{(between a superstructure and an abutment)} \end{aligned}\right\} \quad \text{.....(13.2.2)}$$

where

L_{ER} : Design expansion-contraction length (mm) of the expansion joint in the design situation in which the effects of earthquakes are to be considered

L_A : Allowance of expansion-contraction length (mm)

δ_R : Maximum relative displacement (mm) between the superstructure and the substructure that occurs in the position of the expansion joint in the design situation in which the effects of earthquakes are to be considered under the situation dominated by variable action

c_B : Modification factor for natural period difference of the joint gap width. It shall be set to a value that is selected from Table 13.2.1 according to the natural period difference ΔT of the two connected superstructures adjacent in the longitudinal direction to the bridge axis.

(2) An expansion joint and the joint between the expansion joint and a superstructure/substructure shall be designed to be capable of reliably transmitting acting forces to the superstructure and the substructure in the design situation in which the effects of earthquakes are to be considered under the situation dominated by variable action.

13.3 Unseating Prevention Systems

13.3.1 General

(1) An unseating prevention system shall consist of systems acting independently in directions 1) to 3) below and these directions shall be considered in design.

1) Longitudinal direction to the bridge axis
2) Transverse direction to the bridge axis
3) Rotation direction within the horizontal plane (hereafter, referred to as "rotation direction")

(2) When the provisions of Clause 13.3.2, those of Clause 13.3.3, and those of Clause 13.3.4 are complied with in the longitudinal direction to the bridge axis, in the transverse direction to the bridge axis, and in the rotation direction, respectively, appropriate measures to prevent the superstructure from easily falling may be regarded as implemented.

(3) When the provisions of Clause 13.3.9 are complied with, appropriate measures to prevent the superstructure from easily falling may be regarded as implemented, but not according to provision (2).

13.3.2 Measures to Prevent Superstructures from Easily Falling in the Longitudinal Direction to the Bridge Axis

(1) Measures to prevent superstructures from easily falling in the longitudinal direction to the bridge axis shall be implemented by both ensuring the seat length specified in provision (2) and providing an unseating prevention system specified in provision (3).

(2) The seat length in the longitudinal direction to the bridge axis shall be ensured in such a manner that provisions 1) to 3) below will be satisfied.

1) The required seat length shall be ensured at the edges of supporting points for connected superstructures. In case supporting points on substructures are not present within the area, shown in Figure 13.3.1, that is located on holizontal projection plane of the bridge face on the superstructures, the required seat length shall also be ensured at those supporting points.

2) The required seat length shall be ensured from the ends of the connected superstructures in the longitudinal direction to the bridge axis.

3) The required seat length shall be calculated according to the provision of Clause 13.3.5(1).

(3) An unseating prevention system shall have the structure specified in Clause 13.3.6 and be installed according to provisions 1), 2),and 3) below.

1) An unseating prevention system shall be installed at a supporting point for the connected superstructures at which the required seat length has been ensured.

2) An unseating prevention structure shall be installed in such a manner that it will function in a range where the deviation of the superstructure from the substructure supporting the superstructure will not exceed the seat length in the longitudinal direction to the bridge axis.

3) When an unseating prevention structure is installed in such a manner that it will function in a range no greater than 0.75 times the seat length in the longitudinal direction to the bridge axis, provision 2) may be regarded as satisfied.

(4) When a bridge equipped with connected superstructures that are supported at both ends in the longitudinal direction to the bridge axis by abutments satisfies provisions 1) to 3) below, it may be regarded as a bridge that is provided with the same function as that of an unseating prevention system, in coperation between parapet and abutment back fill, but not according to provision (3).

1) The abutments shall be equipped with parapets specified in Clause 7.4.4 of Part IV and designed to resist the pressure of abutment back fill. ただし，橋脚と同様の振動特性を有する橋台は除く.

2) If the superstructures are displaced in the longitudinal direction to the bridge axis at one end, they shall be constrained with the abutment parapet located at the other end.

3) When the bridge is in the state specified in provision 2), the ends of the superstructures shall stay on substructures.

#1 Longitudinal direction to the bridge axis

#2 Bridge face

#3 (Plan view)

#4 Range in which the bridge face of the superstructure is projected

#5 Bridge face

#6 Supporting point

#7 (Cross-sectional view)

#8

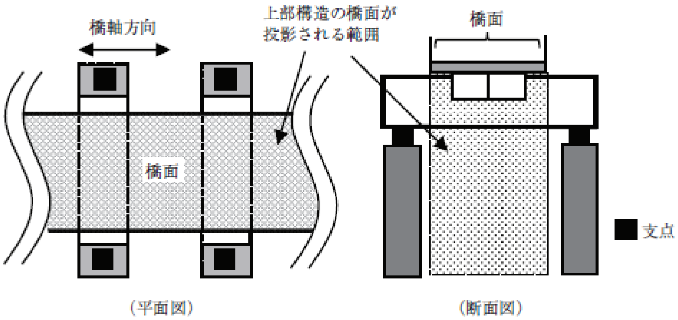

Figure 13.3.1 Structure in which the supporting point of the substructure is not present on the horizontal projection plane of the bridge face of the superstructure

13.3.3 Measures to Prevent Superstructures from Easily Falling in the Transverse Direction to the Bridge Axis

(1) Measures to prevent superstructures from easily falling in the transverse direction to the bridge axis shall be implemented by ensuring the seat length specified in provision (2).

(2) The seat length in the transverse direction to the bridge axis shall be ensured in such a manner that provisions 1) to 3) below will be satisfied.

1) The required seat length shall be ensured at all supporting points for connected superstructures.

2) The required seat length shall be ensured in the transverse direction to the bridge axis.

3) The required seat length shall be a length calculated according to the provision of Clause 13.3.5(1) that allows superstructures to stay stably on substructures when the superstructures move by the distance equal to this length in the transverse direction to the bridge axis relative to the substructures. When the required seat length calculated according to the provision of Clause 13.3.5(1) is different between the two ends of the connected superstructures, the longer length shall be used for the required seat length.

13.3.4 Measures to Prevent Superstructures from Easily Falling in the Rotation Direction

(1) Measures to prevent superstructures from easily falling in the rotation direction shall be implemented when the supposed rotation behavior of connected superstructures within a horizontal plane is not constrained by superstructures adjacent to them, the gap parts of columns, or abutment parapets.

(2) Measures to prevent superstructures from easily falling in the rotation direction shall be implemented by both ensuring the seat length specified in provision (2) and providing the lateral displacement restraining structure specified in provision (3).

(3) The seat length in the rotation direction shall be ensured in such a manner that provisions 1) to 3) below will be satisfied.

1) The required seat length shall be ensured at the edges of supporting points for connected superstructures.

2) The required seat length shall be ensured from the ends of the connected superstructures in a direction perpendicular to the ends of the superstructures.

3) The required seat length shall be calculated according to the provision of Clause 13.3.5(2).

(4) As the lateral displacement restraining structure, a structure specified in Clause 13.3.7 shall be installed according to provisions 1) and 2) below.

1) (英訳なし) Lateral displacement restraining structure shall be installed in a position in

which it constrains the behavior of the bearing line in the rotatable direction.

2) The lateral displacement retaining structure shall be installed in such a manner that it will function in a range where the deviation of the superstructure from the substructure supporting the superstructure will not exceed the seat length in the rotation direction.

13.3.5 Required girder seat length

(1) The required seat length shall be the length calculated from Equation (13.3.1), if provided that this value is smaller than that calculated from Equation (13.3.2), that shall be the length calculated from Equation (13.3.2).

$$S_{ER} = u_R + u_G \geq S_{EM} \quad \cdots\cdots \quad (13.3.1)$$

$$S_{EM} = 0.7+0.005\, l \quad \cdots\cdots \quad (13.3.2)$$

$$u_G = \varepsilon_G L \quad \cdots\cdots \quad (13.3.3)$$

where

S_{ER} : Required girder seat length (m)

u_R : Maximum response deformation (m) that occurs at the bearing support in the design situation in which Level 2 Earthquake Ground Motion is to be considered. When lateral spreding is to be considered, it shall include the maximum response deformation that will occur in response to lateral spreding. . However, in cases where it is judged that the lateral spreading force of the ground that affects the bridge as specified in Chapter 4.4 will occur, when horizontal displacement at the top of the foundation acting lateral movement force exceeds the horizontal displacement that will occur if the foundation yields, an additional 0.5 m shall be added to the value.

u_G : Relative displacement (m) of the ground that occurs due to ground strain during earthquakes

S_{EM} : Minimum value of the required seat length (m)

ε_G : Seismic ground strain, which shall be 0.00250, 0.00375, and 0.00500 for Type I, II, and III in the classification of the ground type, respectively; when connected superstructures are supported by substructures constructed on ground of different ground types, the value corresponding to the ground type that develops the largest ground stain in the design situation in which the effects of earthquakes are to be considered.

L : Distance (m) between substructures that will be used to calculate the required seat length

l : Span length (m); when two superstructures are supported on one pier with different span lengths on each sides, the larger span length shall be applied.

(2) The required seat length in the rotation direction shall be the length calculated calculated from Equation (13.3.4). If the series of lengths calculated for the two ends of a superstructure are different from each other, the larger length shall be applied.

$$S_{E\theta R} \geq 2L_\theta \sin(\alpha_E/2) \cos(\alpha_E/2 - \theta) \quad \cdots\cdots \quad (13.3.4)$$

where

$S_{E\theta R}$: Required girder seat length of a bridge that is classified as a bridge specified in Clause by 13.3.4 (1) (m)

L_θ : Length of a continuous superstructure (m)

θ : Angle for considering rotation conditions (degree)

α_E : Marginal unseating rotation angle (degree). α_E can generally be taken as 2.5 degrees.

13.3.6 Unseating Prevention Structure

(1) Horizontal force that acts on unseating prevention structures shall be calculated from Equation (13.3.5).

1) For unseating prevention structures achieved by constraint between a superstructure and a substructure

$H_F = P_{LG}$

But only under $H_F \leq 1.5\, R_d$ ・・・(13.3.5)

2) For unseating prevention structures achieved by the mutual connection between two connected superstructures

$H_F = 1.5\, R_d$

where

H_F : Horizontal force (kN) that acts on the unseating prevention structure

P_{LG} : Horizontal maximum strength (kN) in the longitudinal direction to the bridge axis

R_d : Vertical reaction force (kN) that occurs due to the dead load of the superstructure in the supporting point of the substructure on which the required seat length is ensured. When an unseating prevention structure that is achieved by mutual connection between two connected girders is to be used, the larger vertical reaction force shall be applied.

(2) An unseating prevention structure shall be designed in such a manner that it will stay in the elastic range under the action of the horizontal force specified in provision (1) and can produce

required force within the range that does not exceed the seat length.

13.3.7 Lateral Displacement Confining Structure

(1) Horizontal force that acts on lateral displacement restraining structures shall be calculated from Equation (13.3.6).

$$\left.\begin{array}{l} Hs = P_{TR} \\ \text{But only} \quad Hs \leq 3k_h R_d \end{array}\right\} \cdots\cdots \quad (13.3.6)$$

where

H_s : Horizontal force (kN) that acts on the lateral displacement restraining structure

P_{TR} : Horizontal maximum strength (kN) in the transverse direction to the bridge axis

k_h : Design horizontal seismic coefficient corresponding to Level 1 Earthquake Ground Motion; this seismic coefficient is determined in according to the provisions of Clause 4.1.6.

R_d : Vertical reaction force (kN) that occurs due to the dead load of the superstructure in the supporting point of the substructure on which the required seat length is ensured

(2) A lateral displacement restraining structure shall be designed in such a manner that it will stay in the elastic range under the action of the horizontal force specified in provision (1) and can produce required force within the range that does not exceed the seat length.

13.3.8 Considerations in Structural Design for Unseating Prevention Structures and Lateral Displacement Restraining Structures

Unseating prevention structures and lateral displacement restraining structures shall be designed and arranged by considering provisions 1) to 4) below.

1) Unseating prevention structures and lateral displacement restraining structures shall be designed to have a structure that is capable of reducing the impact force acting on them to the extent possible.
2) The systems in the longitudinal and transverse directions to the bridge axis and the system in the rotation direction shall be designed in such a manner that each of the systems will work independently and cooperate to prevent superstructures from easily falling even if relative displacement occurs between the superstructures and substructures in directions other than those considered in design.
3) Unseating prevention structures, lateral displacement restraining structures, and their

peripheral structures shall be designed and arranged in such a manner as to minimize portions for which inspections and repairs for dealing with the effects of age deterioration of those structures are difficult to make.

4) Unseating prevention structures and lateral displacement restraining structures shall be designed and arranged prevending factors in degradation, such as dust and stagnant water locating on the connecting point between superstructure and substructure, and superstructure and substructure

13.3.9 Exceptions to Installation of Unseating Prevention Structures and Lateral Displacement Restraining Structures

(1) For bridges with three or more spans, when the bridges are equipped with connected superstructures, and when substructures are present within the area that is projected from the bridge face on the superstructures, and If any of the following conditions applies 1) or 2) below, the required girder seat length shall be ensured only according to the provisions of Clauses 13.3.2 to 13.3.4. However, the required girder seat length in the rotation direction is calculated according to the provisions of Clauses 13.3.5(1).

1) When the connections between superstructures and substructures are rigidly connected at two or more substructures

2) When rigid connection, elastic support or fixed support, or a combination of these approaches in the longitudinal direction to the bridge axis, is used in four or more substructures except substructures equipped with only one bearing on one bearing line. However, cases in which half or more of the inertia force that occurs due to the weight of connected superstructures in a design situation in which Level 2 Earthquake Ground Motion in the longitudinal direction to the bridge axis is to be considered is distributed on one bearing line are not restricted.

(2) For bridges with three or more spans in which connected superstructures that do not fall under the conditions specified in provision (1) are supported by four or more substructures, or for rigid-frame bridges, when the provision of Clause 13.3.4(1) appliesthe following 1) to 3) below. However, the scope of this provision shall exclude substructures equipped with only one bearing on one bearing line.

1) Provisions of Clause 13.3.2 are complied with in the longitudinal direction to the bridge axis.

2) Provisions of Clause 13.3.3 are complied with in the transverse direction to the bridge axis.

3) Provisions of Clause 13.3.4(3)1), 2), and Clause 13.3.5(1) are complied with in the rotation

direction.

CHAPTER 14 SEISMICALLY ISOLATED BRIDGES

14.1 Scope

This chapter shall apply to the seismic design of seismically isolated bridges.

14.2 General

(1) When Limit state 2 of a seismically isolated bridge is to be represented by the limit states of its superstructures, substructures, and seismic isolation bearings, 1) to 3) below shall be complied with.

1) Superstructures
   Limit state 1 of superstructures that is specified in Clause 3.4.2 of Part II or Clause 3.4.2 of Part III
2) Substructures
   i)Limit state 1 of substructures that is specified in Clause 3.4.2 of Part IV
   ii) A state of a limit in which the plastic behavior of substructures does not affect the certainty of energy absorption by seismic isolation bearings in a range where although Limit state 1 of substructures is exceed, Limit state 2 is not exceeded
3) Seismic isolation bearings
   Limit state 2 of bearing supports that is specified in Clause 10.1.4 of Part I

(2) If any of the following conditions applies 1) to 5) below, a seismically isolated bridge shall generally not be used.

1) When the ground around the foundation has a soil layer for which the values of soil parameters in seismic design specified in Clause 3.5.3) are zeros
2) When responses that occur in members other than seismic isolation bearings are not easily reduced by elongation of the natural period of the bridge, such as a bridge that originally has a long natural period due to the large flexibility of its substructures
3) When elongation of the natural period of the bridge may cause resonance between the ground and the bridge because the ground around the foundation is soft
4) When tensile force occurs in rubber bearing bodies in the situation dominated by permanent action
5) When foundation is designed expecting to have plasticity.

(3) For seismically isolated bridges, expansion gaps that can cover displacement in design shall be

provided at the ends of superstructures. When fixed support is selected for bearing conditions in the transverse direction to the bridge axis on the assumption of energy absorption by seismic isolation bearings in the longitudinal direction to the bridge axis, consideration shall be made to prevent members constraining deformation in the transverse direction to the bridge axis from constraining the deformation of the seismic isolation bearings in the longitudinal direction to the bridge axis.

(4) In cases where reduction in inertia force by energy absorption is not assumed for a structure for distributing horizontal forces during an earthquake, when seismic isolation bearings are to be used in the structure, the effects of energy absorption by the seismic isolation bearings shall not be considered.

14.3 Limit State 1 of Substructures of Seismically Isolated Bridges

When provisions 1) and 2) are complied with, Limit state of substructures according to the provisions of Clauses 14.2(1)2)ii) may be regarded as not exceeded in a seismically isolated bridge.

1) For reinforced concrete piers, the horizontal displacement shall not exceed the limit value of horizontal displacement calculated from Equation (14.3.1).

$$\delta_{ls2di} = \delta_{ls2d} / \alpha_m \quad \cdots\cdots\cdots\cdots (14.3.1)$$

However, if δ_{ls2d} / α_m is less than or equal to δ_{yEd}, $\delta_{ls2di} = \delta_{yEd}$ shall hold true.

The parameters in this equation have the following meanings.

δ_{ls2di}: Limit value of horizontal displacement (mm) corresponding to Limit state of the reinforced concrete piers of the seismically isolated bridge

δ_{ls2d}: Limit value of horizontal displacement (mm) corresponding to Limit state 2 of the reinforced concrete piers when plastic behavior is expected for them. It shall be calculated from Equation (8.4.2).

α_m: Coefficient that is used for calculating the limit value of horizontal displacement corresponding to Limit state of the reinforced concrete piers possible to energy absorption by seismic isolatiobn bearing. It shall be set to 2.0.

δ_{yEd}: Limit value of horizontal displacement (mm) corresponding to Limit state 1 of the reinforced concrete piers. It shall be calculated from Equation (8.4.1).

2) The horizontal displacement shall not exceed the limit state 1 of foundation according to the provisions of Part IV Clauses 9 to 14 do not exceed.