Summary of the Earthquake Resistant Design Standards for Railway Structures

by

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ABSTRACT

The devastating damage including the large-scale cave-in of many railway structures occurred in the Hyogoken-Nambu Earthquake of January 17, 1995. By drawing a lesson from this earthquake, a new seismic design code for railway structure has been drawn up. The main procedure of this new code can briefly be described, which is to (1) decide the earthquake resistance performance of structures based on their importance, (2) calculate the responses on the surface of deposit layer by inputting the design earthquake motion in the base layer, and (3) calculate the responses of the structures by inputting the ground responses calculated in the step (2) and check the earthquake resistance performance. For the design earthquake motions, the type of earthquake motion caused by an inland earthquake is added in addition to the normal types. The earthquake resistance performance is defined by the damage of members and stability of foundations, which includes three ranks depending on the retrofitting degree after earthquakes. Moreover, dynamic analysis methods are decided in this new code as main tools for the response calculation and safety check of structures.

1. INTRODUCTION

The Hyogoken-Nambu Earthquake occurred on January 17, 1995, and devastated railways, roads and many other important structures. This earthquake lead to re-examination of the existing seismic design standard at that time. Eventually, the Proposals on Seismic Standards for Civil Engineering Structures was prepared by the Japan Society of Civil Engineers and made public in two releases, in May 1995 and January 1996.

For the railway industry, the Committee for Railway Facility Seismic Structure Review set up by the Ministry of Transport immediately after the devastating earthquake, headed by Prof. Yoshiji Matsumoto, Science University of Tokyo, discussed the validity of the current seismic standard for railway facilities and published the results in the Basic Concept on New Seismic Design Standard, in July 1996. Based on this understanding, the Subcommittee for Seismic Standard Review, headed by Prof.

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Masanori Hamada, Waseda University, proposed the Design Standard for Railway Structures and Its Explanatory of Earthquake-Resistant Design (herein Earthquake Resistant Design Standard) after two years of study and discussion. This standard is intended for bridges, overpasses, foundation structures, earth pressure resisting structures, open cut tunnels, and fills. Its application to fills, however, is only valid when the fill requires seismic design due to strategic importance, susceptible location to earthquake damage, or recovery difficulty. This paper introduces this standard as mainly applied to bridges.

2. BASIC PRINCIPLE OF THE NEW EARTHQUAKE RESISTANT DESIGN STANDARD

A lesson must be learned from the Hyogoken-Nambu Earthquake in developing a new earthquake resistant design standards code. Damages of this disastrous earthquake to structures are omitted here because they are described many literatures. The following causes were inferred from the survey and analysis of such damages[1]:

(1) Many of the structures were known to satisfy the bearing capacity level required by the design standard at that time to be resistant to a design lateral seismic coefficient of 0.2. In reality, actual acceleration of the earthquake far surpassed such a design level, which is why those structures were damaged.

(2) Viaducts of the Shinkansen that suffered serious damage, including the fall of a bridge, were originally designed to be more resistant to bending force than shear force. This imbalance aggravated the level of damage done to these structures. This was partly due to the fact that allowable stress against shear was set larger in the then existing standard than in that of today's standard.

(3) There was a great gap in the magnitude of damage done to adjoining viaducts, with some totally collapsed and others only cracking in columns. This was mainly due to the difference in strength of the ground geology.

These facts helped identify necessary factors to consider in development of new seismic standards: specifically, inland type tremors for seismic design, failure mode for evaluation of safety of members and dynamic characteristics of subsurface geology.

Based on the findings above, one can easily assume that seismic motions to be allowed for in earthquake design will considerably increase once inland type earthquakes are taken into consideration. Also, considering that such earthquakes have a long recurrence period of over a few centuries, one can easily realize that in the seismic design of a structure, deformation performance (earthquake resistance performance) should be evaluated for members of the structure and its foundation so that the structure will allow damage but never collapse.

Earthquake design of a railway structure should therefore be carried out according to the following procedure. First, from the viewpoint of damage control, the degree of damage to a structure (earthquake resistance performance) should be identified. The responses on the subsurface layer of the ground should be calculated by using earthquake motions set for the foundation bed. The responses of a structure should be calculated by inputting the design earthquake motions into the structure. And finally, the earthquake resistance performance of the structure should be checked.

Two types of earthquake motion were considered in the development of our seismic design. One is L1 earthquake motion, which has a recurrence probability of a few times during the service life of the structure. The other is L2, which is a large earthquake motion such as from a large-scale plate border earthquake or an inland type earthquake that may occur during the life of the structure, even though the probability is low.

A structure should then be checked to see if it is resistant to such motions. During the process of developing this part of the standard, three kinds of performance were established based on the presumed level of repair or reinforcement that may be required following a major earthquake, considering the damage of members and stability of the foundation. Every structure built should satisfy these performances. Which performance the structure should be endowed with basically depends on the importance of the structure.

It is then decided that responses of a structure necessary to check these earthquake resistance performances be calculated by dynamic analysis. Static analysis may also be used, depending on the type of the structure.

The procedure of seismic design for a bridge and a viaduct based on the approach above is shown in Figure 1.

As indicated in the figure, there are two types of design methods. One is the non-linear spectrum method that easily calculates responses of a structure by selecting the type of ground based on geological survey and using the strength demand spectrum calculated by the proper earthquake motion determined for the ground selected. The other is the time history dynamic analysis that dynamically analyzes the detailed time history of the ground and the structure standing on it.

As a general rule, the non-linear spectrum method may be used. The other detailed analysis method may work better if a structure, as described later, is unable to describe its behavior in a system with only a single degree of freedom.

Major elements of earthquake resistant design, or setting of design earthquake motions, displacement of the structures, calculation methods for stress and other parameters, and safety checking methods for structures, are explained in the following pages.

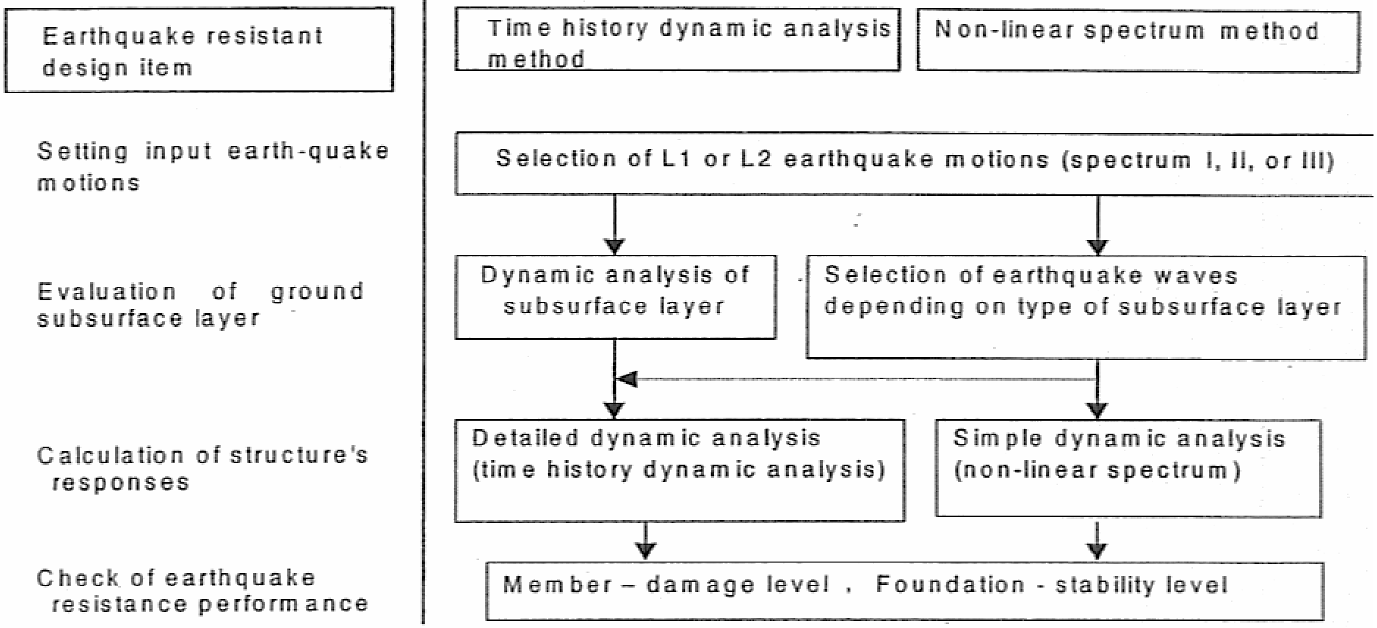

Fig. 1 Procedure of earthquake resistant design

3. SETTING OF DESIGN EARTHQUAKE MOTION

3.1 Setting of earthquake motions for the foundation bed

As earlier discussed, either L1 or L2 earthquake motion is selected in calculation of our proposed earthquake resistant design method.

L1 earthquake motion is conventionally used in combination with elastic design techniques and set either as static load (seismic coefficient method) or earthquake motion for dynamic analysis. Earthquake resistant design method using L1 has been developed based on vast experience. Thus, the current system can be used as is.

For L2 earthquake motion, the level of design earthquake motion typically considered was maximum 1G in elastic response acceleration for a structure standing on the typical type of ground. It is now important, after we experienced the large Hyogoken-Nambu Earthquake, to use values based on near field earthquake motions.

In this respect, the desired approach is to comprehensively consider geological information on active faults, geodetic information on crustal movement, and seismic information on earthquake activities, identify the positions of potentially dangerous active faults by area and predict the magnitude of each possible earthquake before determining earthquake motions. Prediction of recurrence period, scale, or characteristics of seismic motion of earthquakes caused by inland active faults is yet to ensure precision sufficient to provide a basis for earthquake resistant design. Therefore, for the time being we decided to establish standard earthquake motions in the area close to the active fault based on the record of strong motions and their analytical results of the past regional earthquakes, measured at the vicinity of their epicenters, including the devastating Hyogoken-Nambu Earthquake.

L1 and L2 earthquake motions will be set for the foundation bed for reasons described in Section 2. Their characteristics should then be determined in acceleration response spectra.
In doing this, the importance is put on setting of the foundation bed layer. It is ideal to assume a ground with the greatest possible shear elastic wave velocity. However, the subsurface layer of any good quality ground in which the foundation is typically placed may be used so that one can avoid extra problems in daily design work.
L1 earthquake motion in our proposed design method was then determined based on acceleration response spectra of good ground conventionally used in the allowable stress design method and the analysis of earthquake danger level with a recurrence period of 50 years. The maximum value of L1 is 250 gal (damping constant of 5%).

To determine acceleration response spectra of L2 earthquake motion, the following considerations were taken:

1) Acceleration response spectra (Spectra I) associated with marine earthquakes (magnitude class 8 with an epicenter distance of 30 - 40 km) considered in conventional design .

2) Acceleration response spectra (Spectra II), which are set for inland active fault earthquakes by statistical analysis based on past earthquake observation records. If the destruction mechanism of a fault can be identified, Spectra II may be substituted by acceleration response spectra calculated by analysis based on it (Spectra III).

Spectra II is usually obtained from the response spectra of waveforms generated by past earthquake motions and observed at good ground. In this case, it should not be an envelope of each response spectrum, but 90% probability of not being exceeded should be considered in setting of Spectra II. It should be noted that these spectra should be given as waveforms just on the fault. If there is an active fault near the construction site and the location of the fault is identified, its value may be used as one corrected by the distance from the fault to the structure.
The maximum value of Spectra II developed by this approach is 1700 gal (damping constant of 5%).
Design earthquake motions are generally set based on the result of the fault survey. In practice, fault survey will be reviewed by checking available literature such as the Active Faults in Japan and other related literature.

3.2 Setting of Design Earthquake Motions on the Ground Surface

When calculating responses of a structure to earthquake motions, it may be done by dynamic analysis by modeling the subsurface layer and the structure altogether using the earlier-mentioned earthquake motions of the foundation bed. As a general rule, however, the foundation structure will be replaced by supporting springs to form a multiple mass point model for the superstructure, and earthquake motions on the ground surface will be used as input earthquake motions. In this case, one needs to know the earthquake motions on the surface ground, which can be calculated from dynamic analysis of the ground. In reality, there are difficulties in this calculation process such as setting of relationships between strain of the ground and shear elastic coefficient or damping constant. Considering such difficulties in actual calculation, design ground surface earthquake motions were determined by the ground type for simplification of calculation. As earlier mentioned, the nature of the subsurface layer needs to be identified with the best possible precision, which requires more divisions than just the three which are typically used in conventional designs. Thus, design surface ground earthquake motions have been determined for each of those ground types, in our case eight. The characteristic of each surface ground earthquake motion is also given in the form of acceleration response spectra. The types of ground used in this approach will be divided depending on natural frequency calculated based on initial shear elastic wave velocity of the subsurface layer.

In summary, our standard uses eight types of ground, while design ground surface earthquake motion has been set for each Spectra I and Spectra II of both L1 and L2 earthquake motions.

4. Earthquake Resistance Performance of Structures

4.1 Earthquake resistance performance of structures

In our proposed standard, when earthquake resistant design is developed, earthquake resistance performance of a structure should first be set and then the structure should be checked for that performance. Although there is no clearly explained definition of performance design in general, our definition is to clarify the ideal condition of a structure and attempt to provide a design that can achieve that ideal.

For seismic design, the definition of performance design may be interpreted this way as follows. The conditions of damage a structure may suffer from design earthquake motions is clearly determined, and if it is right or not, it should be checked. In other words, it is a damage control design method. This concept is already assimilated in the Concrete Design Standard Specification issued by the Japan Society of Civil Engineers, which occurred in the context of diversification of standards, advancement of analysis techniques, or growing demand. The major advantage of this method is ease of understanding because it clearly explains damage conditions. In practice, any method will do if it can prove it fully serves the purpose, so any design method that fits the situation may be used, which helps improve further development of technology. For the purpose of seismic design of railway structures, the above mentioned method of determining performance with respect to possible damage to the structure and checking the structure for performance has been chosen.

In seismic design of a railway structure, the following three performances are established for consideration:

1) Earthquake resistance performance I (PI): capability of maintaining the original functions without any repair and no major displacement occurring after being hit by an earthquake

2) Earthquake resistance performance II (PII): capability of making quick recovery of the original functions with repairs after being hit by an earthquake

3) Earthquake resistance performance III (PIII): capability of keeping the entire structure system in place without collapse after being hit by an earthquake

These performances are about ease of recovery of the structure after being hit by an earthquake. Therefore, the relationship between earthquake motions and earthquake resistance performance has been established as follows: PI should be satisfied by a structure against L1 earthquake motions, PII by a structure of greater importance against L2 and P III by other structures against L2.

Earthquake resistance performance describes the seismic resistance of a structure in terms of the level of damage to its building members and the level of stability of its foundation structure. This means compliance of a structure with the intended earthquake resistance performance requires appropriate setting of the level of damage of component members and the level of stability of its foundation. For the damage level to members, the damage level to each member should be set considering the seismic resistant role of each such member in the context of the entire structure composed of various members. For the stability level of the foundation structure, as it has a big impact on deformation of the structure, it should be determined considering supporting force or deformation involved.

Fig. 2 shows the relationships among earthquake resistance performance required for bridges and viaducts the damage level of members, and the stability level of the foundation structure.

Earthquake resistance performance of a structure

PI: capability of maintaining the original functions without any repair and with no major displacement after being hit by an earthquake
PII: capability of making quick recovery of the original functions with repairs after being hit by an earthquake
PIII: capability of keeping the entire structure system in place without collapse after being hit by an earthquake

Damage of members

Damage level 1: No damage
Damage level 2: damage that may require repair depending on the situation
Damage level 3: damage requiring repair
Damage level 4: damage requiring repair and, depending on the situation, replacement of members

Stability of foundation

Stability level 1: no damage
Stability level 2: damage requiring repair depending on the situation
Stability level 3: damage requiring repair and, depending on situation, correction of the structure

Fig. 2 Relationships among earthquake resistance performance for bridges and viaducts, the damage level of members, and the stability level of the foundation

4.2 Principle of the damage level of members and stability level of the foundation

It is considered appropriate to determine the damage level of members by considering the characteristics of the members, damage, and repair methods and in terms of a relationship with displacement on an envelope of the load-displacement curve. Take reinforced concrete member for example. It maybe set as follows:

In case failure mode precedes bending under action of general axial compressive force, the envelope of the load-displacement curve of a member in question is drawn as in Fig. 3. Certain physical phenomena, as shown in the figure, occur at changing points of the envelope. Considering this, each damage level may be determined as follows: displacement up to Point B for damage level 1, up to Point C for damage level 2, up to Point D for damage level 3, and after Point D for damage level 4. Once the relationship between damage level and displacement is established, displacement may be a good check item if displacement of a member is directly calculated by analysis. Examples of repair methods corresponding to these damage levels are shown in Table 1.

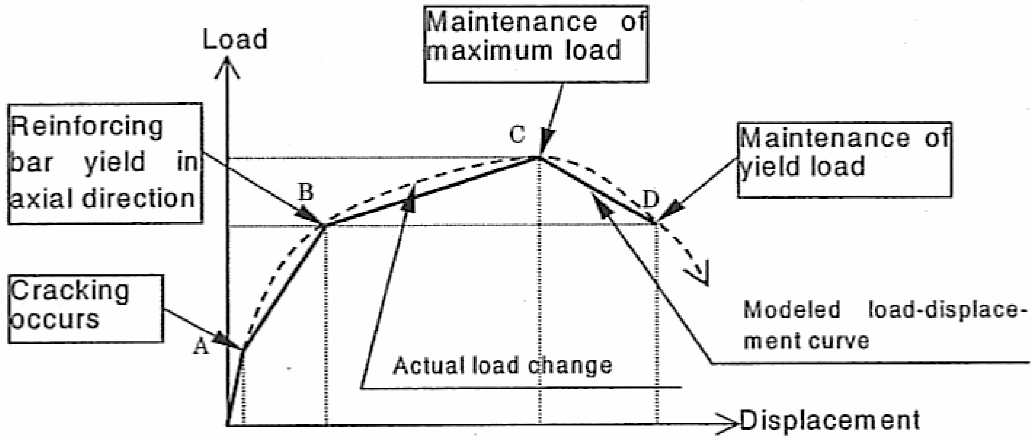

Fig. 3 Envelope of load-displacement curve for reinforced concrete members (under low axial force)

Table 1 Examples of repair methods by damage level for reinforced concrete and steel reinforced concrete members

	Damage level	Example of repair method
Damage level 1	No damage	No repair (may need to consider durability where necessary)
Damage level 2	Damage that may require repair depending on the situation	Filling of cracks and repair of sections wherever necessary
Damage level 3	Damage requiring repair	Filling of cracks and repair of sections. Hoops may require adjustment if necessary.
Damage level 4	Damage requiring repair and, depending on situation, change of members	• Filling of cracks, repair of sections, or correction of hoops • If rebar in the axial direction or steel is badly buckled, replacement of the damaged members may be necessary.

Determination of the stability level of the foundation requires identification of damage to the foundation in terms of bearing power and damage of its members. Damage to members should be determined as earlier mentioned. Damage level concerning stability should be determined by considering the effect of displacement of the foundation on usage of the structure and the level of bearing power of the foundation after the earthquake.

As indexes for that purpose, residual displacement and response plasticity rate of the foundation should be used. The latter is defined as the ratio of the foundation's response displacement at the time of the quake to yield displacement as calculated by the load-displacement curve of the foundation. Fig. 4 illustrates a simplified model of the load-displacement curve of the foundation.

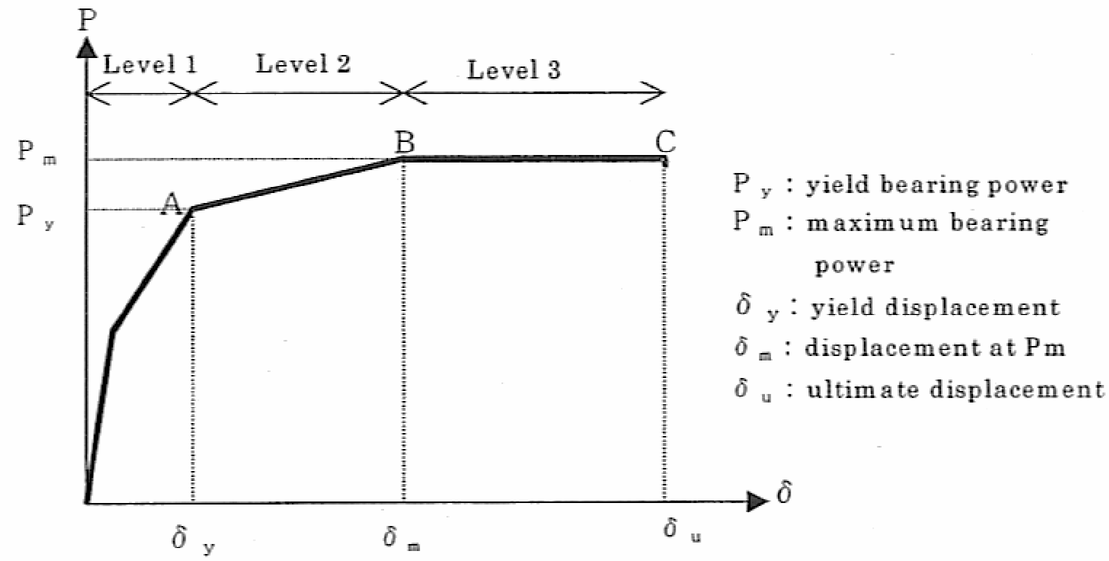

Fig. 4 Load-displacement curve of the foundation structure

Since the yield point in the calculation of displacement occurs by yield of the bearing ground or yield of members, these two, therefore, were put together to determine stability levels. Using those indexes, stability levels may be set as follows:

1) Stability level: In principle, load acting on the foundation should be less than its yield bearing capacity and no major displacement occur. Sectional force of members composing the foundation should exceed yield bearing capacity.

2) Stability level 2: Either bearing ground, members or both should have plasticity but yet maintain sufficient bearing power. No displacement detrimental to the maintenance of the structure's functions nor residual displacement should occur after the earthquake.

3) Stability level 3: Sufficient bearing power should be maintained to protect the structure from collapse by damage of the bearing ground or members.

Repair methods that may be applied to these stability levels are shown in Table 2.

Table2 Examples of repair methods by stability level

	Damage level	Suggested repair method
Stability level 1	• No damage • Action load less than yield bearing power	No repair
Stability level 2	Damage requiring repair depending on situation	Filling may be necessary at voids at the footing and around the foundation depending on situation.
Stability level 3	Damage requiring repair or correction of structure depending on the situation	• Ground improvement • Reinforcement of the foundation by expanding the footing diameter or driving more piles

4.3 Seismic performance of pile foundation

The seismic performance of pile foundation is confirmed by the stability level of pile foundation. The stability level of pile foundation is determined by considering the strength and deformation properties of soil and pile members. Table 3 shows the concept of the state of pile foundation corresponding to the seismic performance. Generally, the seismic performance Ⅰ corresponds to the stability level 1; the seismic performance Ⅱ to the stability level 2; and the seismic performance Ⅲ to the stability level 3.

Table 3 State of pile foundation corresponding to the seismic performance

Seismic performance	Stability level of pile foundation	State of pile foundation
Seismic performance Ⅰ	Level 1	Pile foundations do not yield.
Seismic performance Ⅱ	Level 2	Although pile foundations yield, they maintain a sufficient bearing capacity.
Seismic performance Ⅲ	Level 3	Although pile foundations reach the limit state, super structures do not collapse.

4.4 Yield point of a pile foundation

Yield point of a pile foundation is established according to the load-displacement curve of an overall structure, where the displacement increases rapidly mainly because of the subgrade reaction reaching the upper limit values or the stiffness of pile members decreasing due to the strength yielding. However, the yield point where the displacement rapidly increases in the load-displacement curve varies for different types of foundations. This makes it difficult to judge the yield point from a) the degree to which the subgrade reaction exceeds the upper limit values and b) the number of members damaged over the total number of members.

In order to investigate the causes of yield point, some common prototype pile foundations were chosen for trial designing. As a result, it was confirmed that the yield point appears when a) the subgrade reaction yields at the outermost edge of the indentation in side of pile group and b) half of the total number of pile members yields [2].

Therefore, the yield point of pile foundation with a common shape can be determined as the point when it reaches one of the states shown in Table 4. If a pile foundation has too many piles, it is difficult to determine the yield point by the criterion in Table 4. In this case, the yield point can be determined by taking into account the causes which intensify the displacement rapidly in the load-displacement curve.

Table 4 Yield point definition for pile foundation

Subgrade in the indentation–in side of pile group	When the vertical resistance of pile head in the outermost edge reach the upper limit value of design vertical capacity
Subgrade in the pulling-out side of pile group	When the vertical resistance of the head of a half (ignoring fractions) of total piles reach the upper limit of design pull-out resistance
Pile members	When the strength of a half (ignoring fractions) of the total piles yield

4.5 The limited values of the damage level of members and stability level of foundation

Damaged parts of a rigid frame viaduct, assumed based on the concept explained above, are illustrated in Fig. 5. Relationships between earthquake resistance performance of those damaged parts and damage levels of members or stability levels of the foundation are shown in Table 5 for limiting value guidelines.

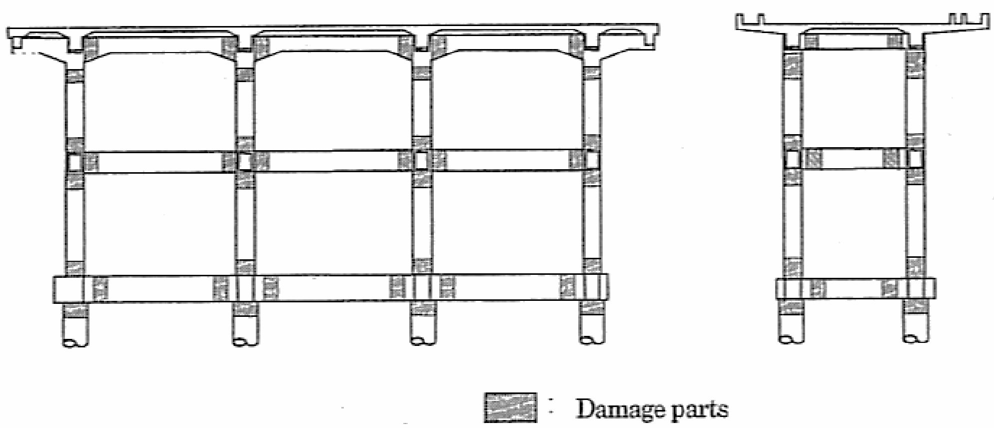

Fig. 5 damaged parts of a rigid frame viaduct

Table 5 Limiting value guidelines for earthquake resistance performance of a rigid frame viaduct, damage level of its members, and stability level of its foundation

Structure		PI	PII	PIII
Damage level of members	Superstructure girder and underground beam	1	2	3
	Other beams	1	3	4
	Columns	1	3	3
Stability level of foundation		1	2	3

4.6 Concept of degree of importance

Determination of the degree of importance of a railway structure requires consideration of various factors, including possible influences of the damage level of the structure on human life, neighborhood, society, operation speed of trains and frequency of service, and the degree of difficulty of recovery in case of damage. Based on this concept, greater importance has been given to the following structures:

1) Structures of the Shinkansen bullet lines and those of passenger railway lines in major big cities
2) Structures whose recovery is considered very difficult if the damage was done to cut tunnels, etc.

5. Evaluation of Surface Layer and Calculation of Displacement and Stress of Structures

L2 earthquake motions are given on the design foundation bed as earlier explained. Then, seismic design of a bridge should require response calculation of the subsurface layer using the earthquake motions of the foundation bed and evaluation of its earthquake resistance performance by inputting the said earthquake motions into the structure. In this case, since the earthquake motions are so large, both the ground layer and the structure are expected to show non-linear behavior. Therefore, evaluation of non-linearity of the ground layer and the structure is essential in seismic design.

5.1 Evaluation of subsurface layer

Characteristics of the subsurface layer must be carefully studied because the impact on earthquake resistance of the structure to be built is very great. Major methods to review the layer that requires sufficient care in earthquake resistant design of a structure are as follows:

(1) Irregular-shaped ground layer

According to past record, irregular shaped layers are known to cause serious earthquake damage due to resultant expansion of earthquake motions. An ideal method to evaluate such layers would be the finite element method. But

considering it is not a general design level, the incremental ratio of earthquake motions was calculated by ground response analysis and the results were already tabulated so that anybody can carry out appropriate calculation without difficulty.

(2) Liquefied ground layer

Liquefaction is a very serious factor to consider in seismic design. If any financially feasible measure is available, such as ground improvement that can stop liquefaction, it should be implemented. If not, the entire structure, including the superstructure, should be taken care of by comprehensive measures to prevent collapse or other disastrous damage against excessive response the structure may incur due to liquefaction or excessive displacement that the foundation may suffer due to ground subsidence or lateral flow.

In our method, judgment on whether the ground causes liquefaction or not when a major earthquake occurs is made using a conventional method that uses liquefaction resistivity. For the ratio of reinforcement against liquefaction required for that method, accumulated damage theory should be applied to adjust for the irregularity of the earthquake motions. The relationship between the ratio of reinforcement against liquefaction and the repetition time was corrected by the size of relative density, thereby totally improving calculation precision.

The Hyogoken-Nambu Earthquake caused lateral flow of the ground due to liquefaction that brought about serious damage. Learning from this lesson, it was decided to consider possible impacts of lateral flow into our design method. To evaluate such impacts, displacement of the ground that may cause lateral flow is assumed, and the assumed displacement is caused to act on the structure via coefficient of subgrade reaction, which follows the procedure same as that of the conventional response displacement methods. (Fig.9)

(3) Soft ground layer

Soft ground amplifies earthquake motions in its fragile subsurface, causes great displacement and consequently may damage the pile foundation. This must therefore be considered for foundations in ground that is likely to cause large displacements. Displacement in this case may be calculated from earthquake response analysis of the subsurface layer. However, to simplify actual design procedure, natural frequencies of the ground were tabulated by the type of earthquake motion so that displacement can be easily and quickly calculated.

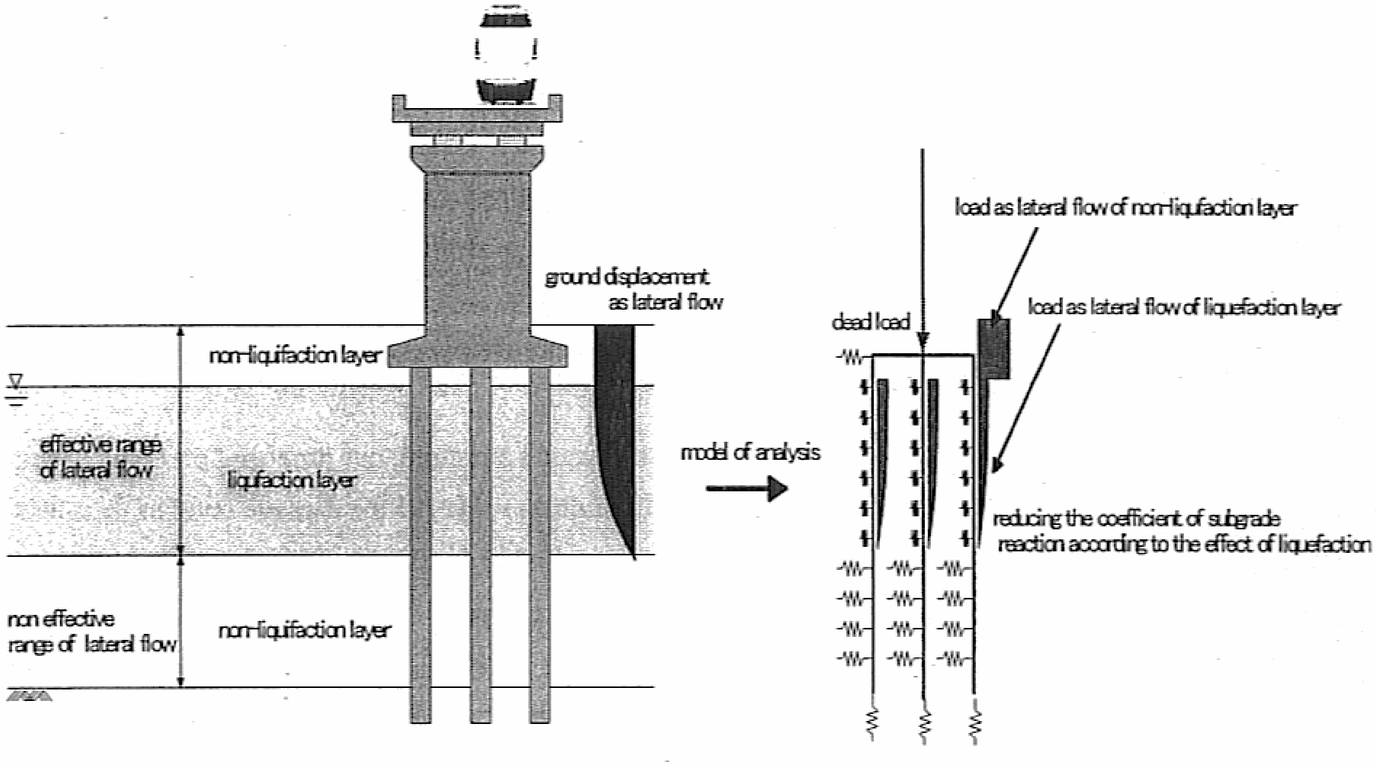

Fig.6 Calculation model of response of structures for lateral flow

5.2 Calculation of responses of structures

Dynamic analysis should be the main method for seismic design of bridges. For a general structure, hysteresis properties of the structure may be determined in advance and responses can be calculated by using non-linear response spectra (strength demand spectra) obtained by dynamic analysis. In our standard, this method is referred to as the non-linear spectrum method. Strength demand spectra are the relationships between the yield seismic coefficient of a structure and the natural frequency of that structure expressed per response ductility factor. If yield seismic coefficient of a structure and its natural frequency (frequency for which the gradient of a line linking the origin and the yield point on the load-displacement curve of the structure is obtained as rigidity of the structure) are known, response plasticity rate of the structure when it is hit by an earthquake (response displacement) will easily be calculated.

For such structures as multiple-spanned bridges, structures with long frequencies, or new types of bridges whose behavior cannot be expressed in a system of only a single degree of freedom, dynamic analysis using a model of a system with multiple degrees of freedom should be used.

Foundation structures should also be designed using responses (mainly effects of inertial force of the superstructure). In case the foundation rests in soft ground layer which causes post-earthquake displacement beyond negligible levels, such effect should be considered. This method was already employed as the response displacement method in conventional design approaches. In our standard, the combinations of effects of inertial force of the superstructure and the degree of effect of ground displacement should be caused to change according to the relationship between the natural frequency of the structure and that of the ground in order to design structures in a cost-effective way.

6. Safety Checking (Earthquake Resistance Performance) of Structures

In checking earthquake resistance performance of a structure, our proposed procedure specifies that responses calculated as in Section 5 satisfy limiting values of the damage level of members and those of the stability level of the foundation, both mentioned in Section 4. This process is based on advance understanding of the relationship between displacement of the structure and the damage levels, which can be known by static non-linear analysis (push-over analysis). This means earthquake resistance performance of a structure should be checked by comparing deformation of the structure calculated by this static non-linear analysis with that by dynamic analysis. This analysis, however, generally requires treatment of the superstructure and the foundation as a single body. Since the accuracy of soil properties is usually lower than that of structures and smaller of soil is proposed in the standard ,the safety of the structures may be overestimated under seismic load. To avoid this, the verification is also carried out when the strength of soil doubled.

6.1 Check of damage levels of members

In checking damage levels of reinforced concrete and steel reinforced concrete members, the first step is judging the type of failure. In other words, bending failure mode should be used when shear force as members reach bending bearing capacity, is smaller than design shear bearing capacity and shear failure mode be used when it is larger.

For cases where bending failure mode is used, it should be confirmed that deformation of members does not exceed the limiting value of the damage level that corresponds with the earthquake resistance performance. For the use of shear failure mode, exceeding of shear bearing capacity beyond response is a required prerequisite.

6.2 Limiting value of stability level of the foundation

For stability level of the foundation, the following must be checked:

1) Response plasticity rate of the foundation

2) Damage level of foundation members

The method specified in 6.1 should be used for (2) above. For (1), the prerequisite is that responses of the structure do not exceed limiting values of plasticity rate that correspond with earthquake resistance performance calculated by static non-linear analysis.

7. Conclusion

This paper outlines our proposed earthquake resistance design standard. A good seismic design requires techniques to calculate deformation or stress and to check that the calculations can explain past earthquake damage with good precision. Development of our method discussed here was also partly based on damage analysis of the Hyogoken-Nambu Earthquake. Since damage of this disastrous earthquake are still being vigorously analyzed by various research institutes, more new findings are expected to come. It is therefore our responsibility to assimilate any such new findings for improvement of our design approach.

As non-linearity of structures and grounds need to be considered, our design method has become very complicated. Although complexity is not always justified, those complexities explained herein are all assimilated into our method as necessary steps to express the degrees of damage to the structure. This, we believe, enables us to show that a structure will suffer the degree of damage corresponding to its earthquake resistance performance when a major earthquake hits, rather than simply saying that a structure should remain unscathed.

Calculation, however, is generally done by computers, to which all design parameters and values on the ground, structural members, etc., are manually input. Care must therefore be taken in this manual work. Effort must also be made on further improvement of precision. It is a good that design calculation precision and quality is enhanced by computer, but one should note that no good design comes out of input of incorrect data.

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