Journal of Japan Association for Earthquake Engineering, Vol.4, No.3 (Special Issue), 2004

DAMAGE ANALYSIS AND SEISMIC DESIGN OF RAILWAY STRUCTURES FOR HYOGOKEN-NANBU (KOBE) EARTHQUAKE

Akihiko NISHIMURA

Member of the JSCE, Dr. Eng., Vice-president, Representative Director,
JR Souken Engineering Co., Ltd., 2-8-38 Hikari-cho, Kokubunji-shi, Tokyo 185-0034, Japan,
nisimura@jrseg.co.jp

ABSTRACT: The devastation wrought on railway structures by the 1995 Hyogoken-Nanbu earthquake has shown that there is an urgent need for developing a new seismic design of railway structures. After the earthquake, many groups set to investigate the reasons for the damage using static and dynamic analyses. This paper presents causes of seismically induced damage to bridges elucidated by some of these analyses of the earthquake, a contemporary seismic design method for railway structures, and issues related to the new seismic design method.

Key Words: Damage analysis, seismic design, earthquake disaster.

INTRODUCTION

The 1995 Hyogoken-Nanbu (Kobe) earthquake in Japan caused severe damage to railway structures, including completely collapsing. This extensive damage emphasized the need to develop new procedures and specifications to assess existing structures and to improve seismic designs for new structures. This disaster has thus become fixed in memory for the same reason as the 1923 Great Kanto earthquake. Majority of Japanese civil engineers were over confident that such serious structural damage would not be widespread in the event of a large earthquake because of progress in seismic design.

In the pre-Kobe earthquake, seismic design has been based exclusively on the seismic coefficient method, which was firstly adopted for civil structures in the early Showa era (1930's). Although the methodology has been modified after every major earthquake disaster, basic philosophy has remained unchanged. The Niigata earthquake (1964) caused railway bridges to collapse, but there was no heavy damage or collapse to be experienced until the event of the Hyogoken-Nanbu earthquake. The devastation wrought on civil structures by this earthquake indicates to civil engineers that the conventional seismic design method is still inadequate. This leads to conclude that this seismic coefficient method could be used to design bridges that withstand only levels of the past large-scale earthquakes. In this article, the lessons on seismic design learned from this latest earthquake and a new seismic design methodology are described.

CAUSES OF DAMAGE TO VIADUCTS IN THE HYOGOKEN-NANBU EARTHQUAKE

Outline of Damage

The Hyogoken-Nanbu earthquake caused considerable damage to railway constructions such as elevated bridges, embankments, and other civil structures. Its impact affected many of railway, the Sanyo Shinkansen line, Tokaido Main line, Hanshin-Kobe line, Itami line, Hanshin Main line, and others. This section focuses on the damage of concrete bridges on the Sanyo Shinkansen line. The damage extended from the epicenter in northeast towards as far as the cities of Akashi and Takatsuki, as shown in Fig. 1.The damage also caused to concrete bridges in the section between Osaka and Himeji, particularly concentrated on several kilometers between Shin-Osaka side of the portal of Rokko tunnel and starting point, and between Nagasaka tunnel and Nishiakashi. And, it also spread over elevated bridge between Kyoto and Shin-Osaka, nearby Takatsuki on the Tokaido Shinkansen line. The forms of damage to concrete bridges can be summarized as follows. Floor beams and/or slabs of reinforced concrete viaducts and abutments collapsed after supporting columns failed. About 1,200 columns were collapsed and further 3,400 were damaged in total in the region. Concrete viaducts and bridges collapsed completely at many locations, as shown in Fig. 1. All the viaducts were rigid-frame structures, and some of them collapsed because their columns failed after shear cracks developed either at the top or at the bottom of columns due to huge horizontal seismic loads.

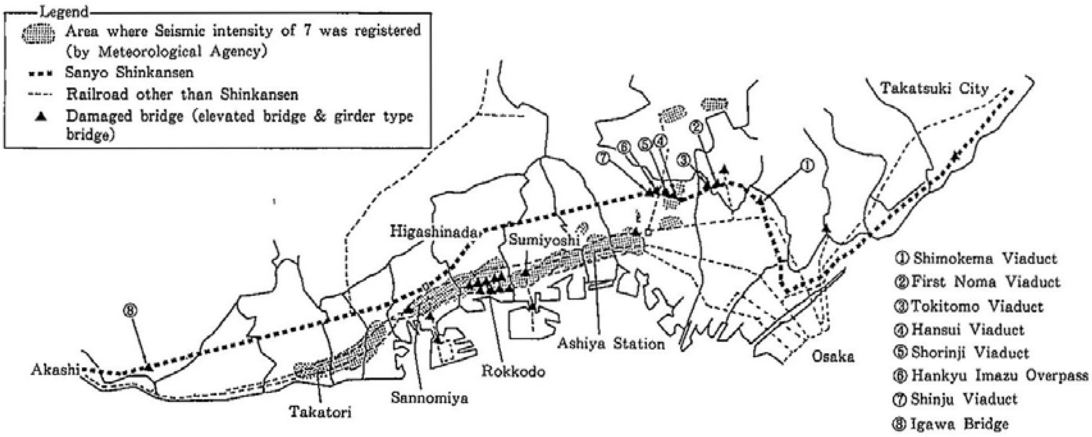

Fig. 1 Location of major damaged railroad

Causes of Damage

Consideration of Yield Strength and Deformation Capacity

Damage is categorized as pattern “S” and “M” that indicate the causes of failure, “shear” and “flexural”, respectively, as shown in Fig. 2. It is also classified into 4 levels A to D (with D: non-damaged). Shear failure is generally brittle in concrete therefore the structure is completely destroyed if it suffers seismically induced shear failure.

The strength and deformation capacity of viaducts on the Sanyo Shinkansen line were verified according to "Standards for Design of Railway Constructions (Concrete Constructions)". Horizontal seismic coefficients were calculated by static linear analysis corresponding to the strength of flexural yield, flexural ultimate, and shear of each framework member. In this analysis, the strength of each member was determined by the amount of reinforcement and so on specified in the verification of failed structures. Strength coefficients of materials were obtained from the measurements taken just after the earthquake. Patterns of damage found in the site investigations, and analysis results are shown in Fig. 2. The analysis was carried out in the sections along with and perpendicular to longitudinal bridge axis. Each framework member was classified into damage categories, as follows:

1) Shear failure: $k_h(M_y) > k_h(V_y)$
2) Shear failure beyond flexural yield: $k_h(M_y) < k_h(V_y) < k_h(M_u)$
3) Flexural failure: $k_h(M_u) < k_h(V_y)$

where, $k_h(M_y)$ is the horizontal seismic coefficient at flexural yield strength, $k_h(M_u)$ is the horizontal seismic coefficient at flexural ultimate strength, and $k_h(V_y)$is the horizontal seismic coefficient at shear yield strength.

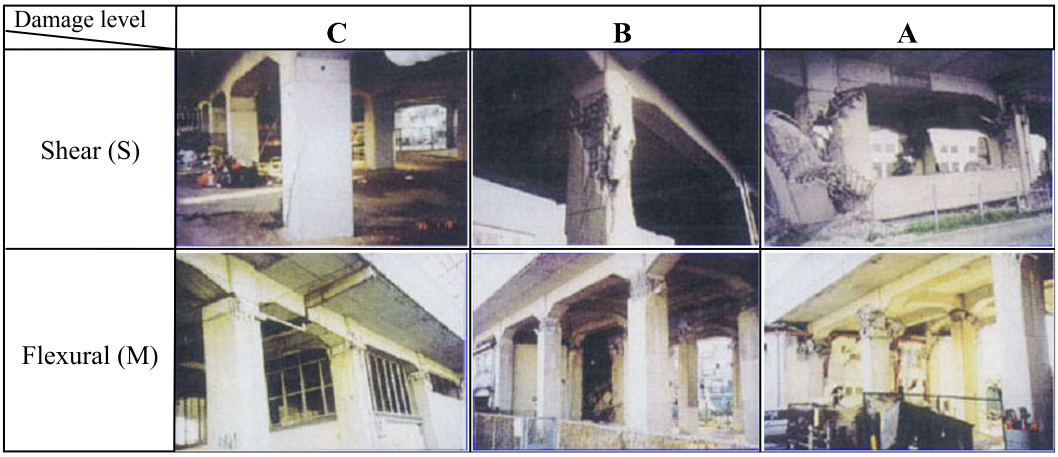

Fig. 2 Patterns and levels of damage

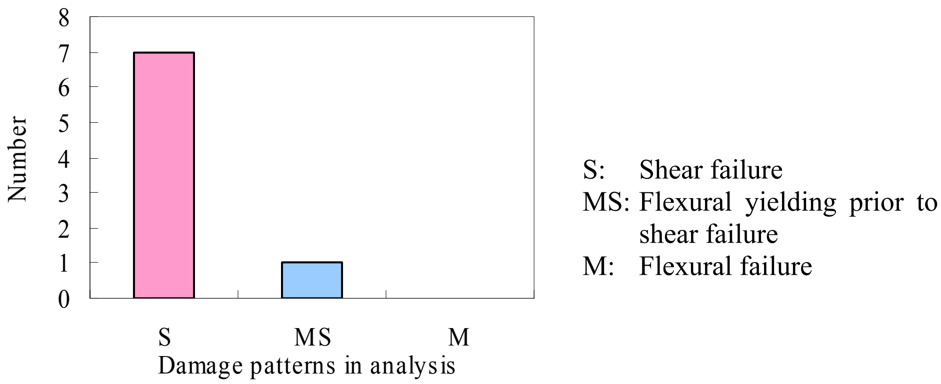

Fig. 3 Relationship between observed damage classifications and analytical damage patterns

All the viaducts that completely collapsed in the earthquake (categorized as SA) are thus assumed to have suffered shear failure at level A. Fig. 3 shows that analytical damage patterns and observed damage classifications of completely collapsed viaducts are well corresponded together.

Study by Seismic Response Analysis

Fig. 4 shows a geological cross section at location where a shinkansen viaduct collapsed. Most damage occurred on diluvium that is newly formed with generally low strength. Thus, characteristic of subsurface layer is considered as one of causes of seismically induced damage.

Two collapse cases of Hansui and Shimokema viaducts, which are located 7 km apart and two non-collapse cases of No. 2 Danjo viaduct near Hansui viaduct and No. 2 Noma viaduct on a diluvium formation were analyzed. They are all 2-layer and 3-span rigid-frame structures as schematically shown in Fig. 5. All of them have a 1.2 m-diameter and 8 m-deep cast-in-place pile foundation except No. 2 Noma viaduct has a spread foundation, as shown in Fig. 6 with their boring logs.

An outline of the analysis schematically shown in Fig. 7 is described as follows. Firstly, the response of subsurface layers to seismic waves propagating from the hypocenter was calculated. Then, it was applied to analytical models that take account of soil-structure interactions between ground, foundations, and structures. This yielded a dynamic response of the structure. Stiffness of structural members was fixed according to the load-displacement relationship for a concrete structure.

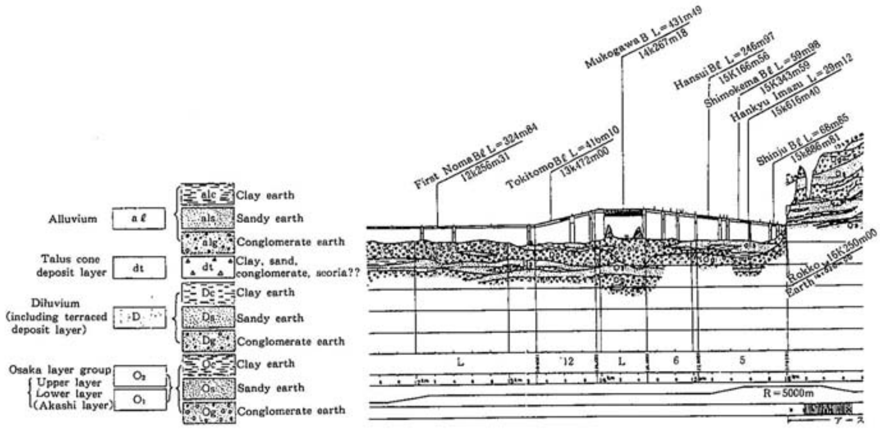

Fig. 4 Geological section from the Sanyo Shinkansen Shin-Osaka station to Shin-Kobe station

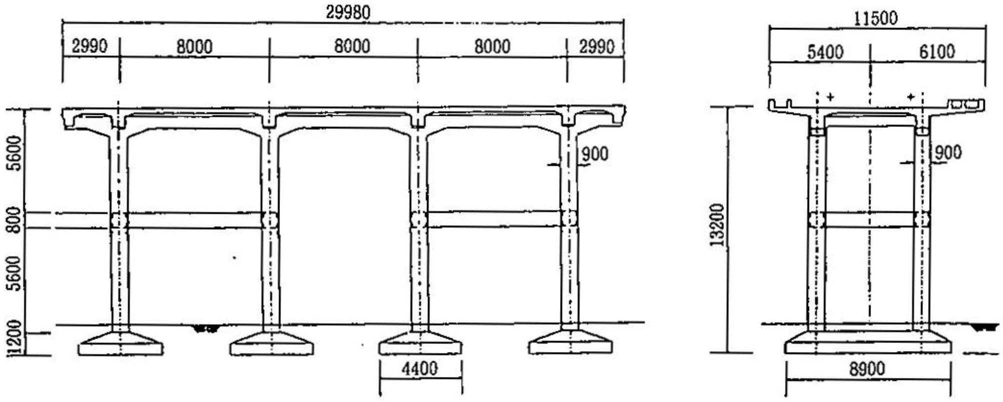

Fig. 5 General drawing of elevated bridge

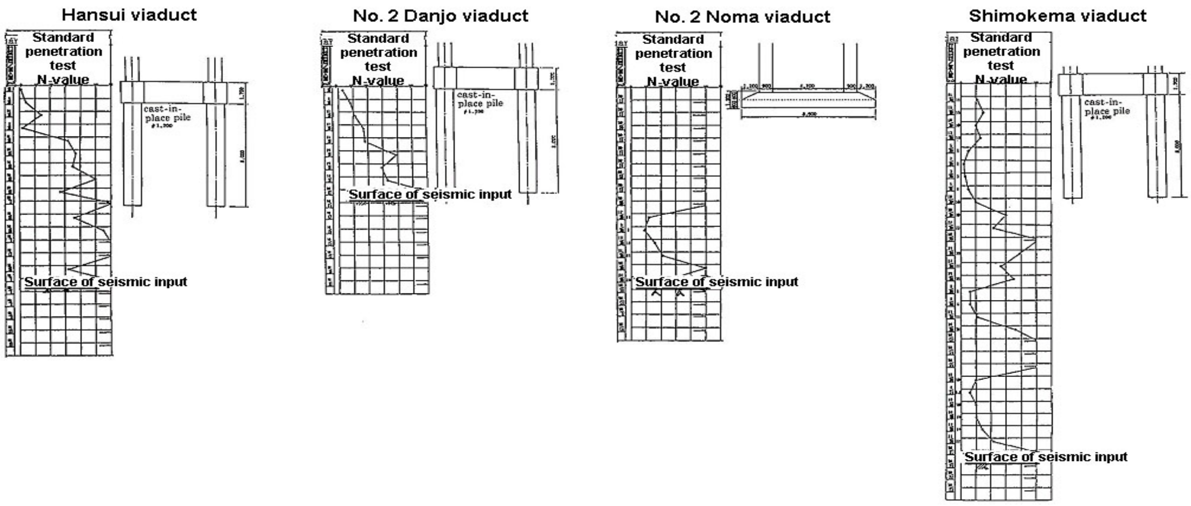

Fig. 6 Foundations and soil boring logs

The EW component of acceleration measured at GL-83 beneath Kobe Port Island was reduced with the distance from the epicenter to each target viaduct. This modified seismic motion was then applied as an input seismic wave to the structure.

Seismic response of surface layers is computed by using one-dimensional effective stress analysis program. Soil properties were determined according to the "Japanese Design Code for Railway Structure Foundations". Table 1 shows maximum values of seismic wave on the ground surface at each viaduct. It is observed that acceleration, velocity, and displacement varied with the differences in subsurface layers beneath viaducts. The analytical results are shown in Fig. 8 as ratios of bending moment and shear force strength to sectional forces of seismic response analysis on the vertical axis of diagram. Ratios less than unity represent collapse.

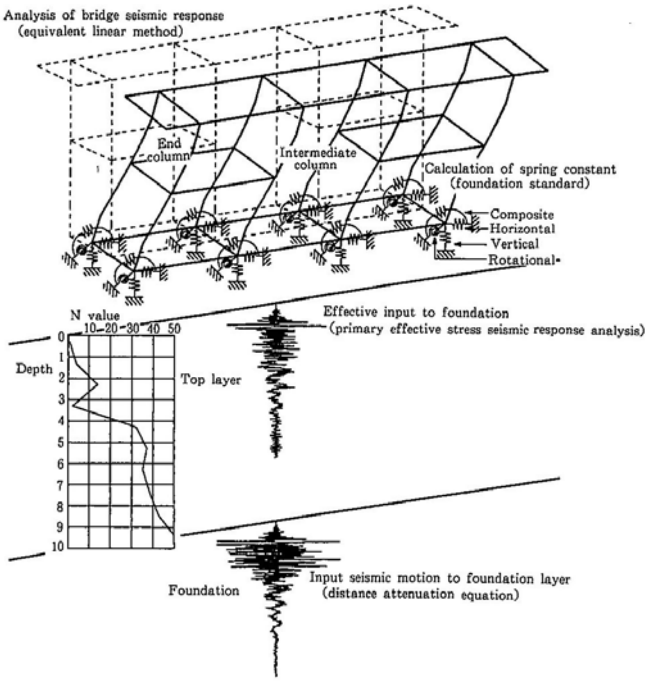

Fig. 7 Schematic diagram of the analysis

Table 1 Maximum values of surface seismic motion

Position	Peak acceleration	Peak velocity	Peak displacement	Peak displacement
Shimokema	235(280)	42.2(27.4)	19.0(11.1)	Value in parenthesis is seismic base layer seismic motion used in analysis
Hansui	256(309)	34.7(30.2)	15.5(12.2)	
No.2 Danjo	277(309)	29.8(30.2)	12.7(12.2)	
No.2 Noma	309	30.2	12.2	

Ranges of value represent deviations of sectional force strength by different proposed formulas. The figure shows clearly that collapsed viaducts have smaller value for shear. Viaducts failed due to shear forces varied with different characteristics of subsurface layers.

Analytical Prediction of Damage Origin

All of the structures investigated were designed to withstand a horizontal seismic coefficient of 0.2 under the Standard. Therefore, many of them were devastated by a recorded horizontal seismic force of larger than 0.2. Further, where the beam of shinkansen rigid-frame viaducts collapsed, greater damage occurred that one factor of safety of the bending moment is smaller compared to that of shear strength.

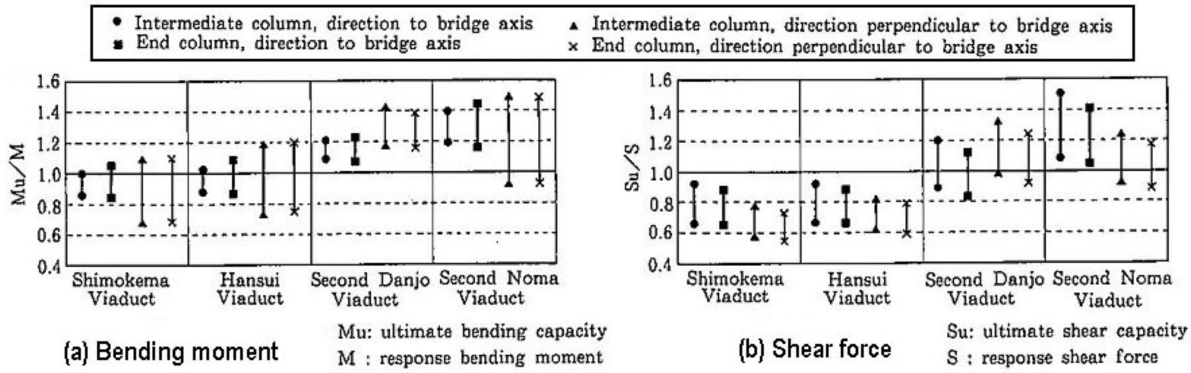

Fig. 8 Ratios of sectional forces for viaducts during an earthquake

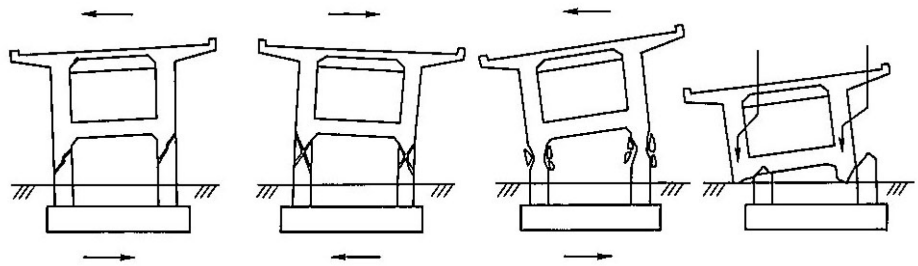

Fig. 9 Collapse process of a rigid-frame viaduct

Fig. 9 schematically illustrates a mechanism of damage to a rigid-frame viaduct. Shear cracks that developed on the top or bottom of middle beam advanced rapidly under a strong earthquake motion. Ultimately, spalling concretes fell out. Consequently, the structure could no longer support its own weight then upper structure slipped down perpendicular to the longitudinal bridge axis.

SEISMIC DESIGN FOR RAILWAY STRUCTURES

The facts mentioned above indicate that following procedures are important to seismic design for railway structures.

1) Taking inland earthquake into account
2) Evaluating safety of members by considering failure modes of structures
3) Necessary to use dynamic analysis methods and considering dynamic behavior of surface ground in response analysis of structures.

Seismic design of a railway structure should therefore be carried out according to the following procedures. Firstly, damage degree of the structure (seismic performance) should be identified in respect to damage control. Secondly, surface ground responses are analyzed by inputting a design earthquake motion in the base ground. Thirdly, responses of the structure are analyzed with the input of obtained surface ground response. Finally, seismic performance of the structure can be checked based on the obtained structural responses.

Setting of Design Earthquake Motion

Setting of Earthquake Motions for Bedrock

There are two types of design earthquake motion, L1 and L2 to be determined for bedrock. L1 earthquake motion has a recurrence probability of a few times during service life of a structure. It has approximately the same level as the acceleration spectrum corresponding to high quality ground, which used to be adopted in allowable stress designs. The maximum response value of acceleration is 250 gal corresponding to a damping coefficient of 5%. L2 earthquake motion that is caused by a near-land-large-scale interplate earthquake or an inland earthquake near structure with high intensity has lower occurrence probability. It is classified into 3 following types.

1) Spectrum I: acceleration spectrum corresponding to near-land interplate earthquakes of magnitude 8.0 and epicenter distance of 30 to 40 kilometers.
2) Spectrum II: acceleration spectrum based on statistic analysis of the earthquake data recorded in the past inland earthquakes caused by active faults.
3) Spectrum III: also representing the motions caused by active inland faults, but based on the analysis of active faults when such a model of active fault is available.

Setting of Design Earthquake Motions on Ground Surface

Table 2 presents 8 types of soil profile used in the design. Depending upon each soil profile, design acceleration response spectra on the ground surface are determined corresponding to L1 earthquake motion, Spectrum I and Spectrum II of L2 earthquake motion. Fig. 10 is an example for Spectrum II.

Table 2 Soil profile types

Soil Profile Types	Period (sec)	Soil Profile Names/Generic Descriptions
G0	-	Hard rock
G1	-	Bedrock
G3	- 0.25	Diluvium
G4	0.25 – 0.50	Dense soil
G5	0.50 – 0.75	Dense soil to soft
G6	1.00 – 1.50	Very soft soil
G7	1.50 -	Extremely soft soil

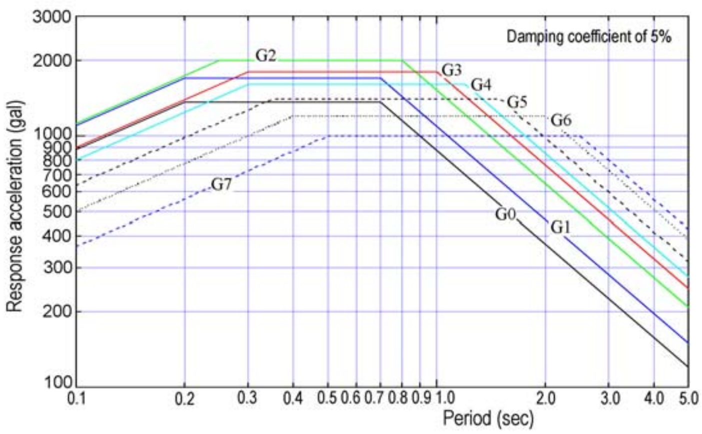

Fig. 10 Design response spectra of acceleration on ground surface for Spectrum II

Seismic Performance of Structure

Setting of Seismic Performance Level for Structures

Corresponding to presumed levels of repair and reinforcement of structures that may be required after an intense earthquake, seismic performance can be categorized into 3 levels as briefly described in Fig. 11.

These performance levels are mainly defined by the degree of ease to recovery structures after an earthquake. Therefore, the relationship between the earthquake motion level and seismic performance was established as follows.

For L1 earthquake, structural seismic performance SPI should be satisfied by all the structures.
For L2 earthquake, SPII should be satisfied by more important structures, and SPIII by others.

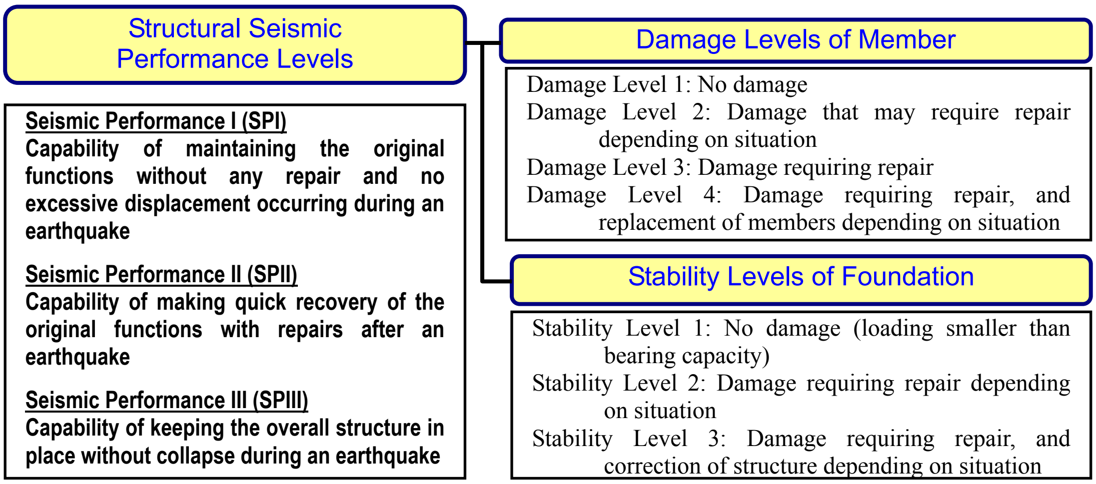

Fig. 11 Relationship among seismic performance levels, damage levels of member and stability levels of foundation (bridges and viaducts)

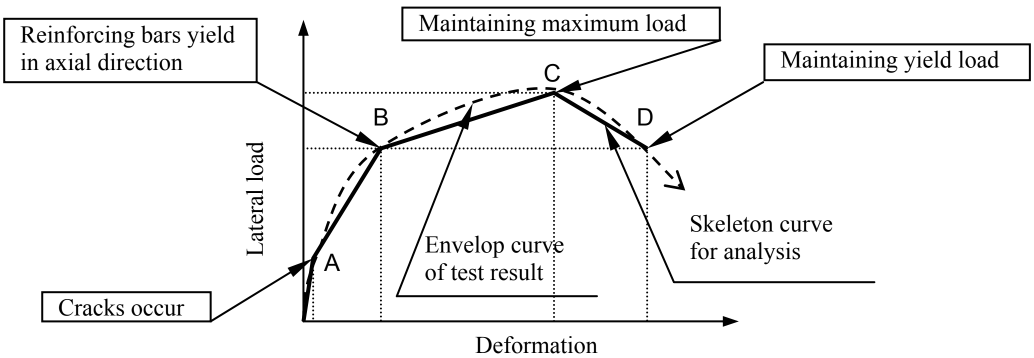

Fig. 12 Relationship of lateral load-deformation relation for reinforced concrete members with a general level of compressive axial force

Damage Levels of Members

It is considered appropriate to determinate damage levels of a member by considering the relation among member properties, damage state, and repair method. Fig. 12 shows a load-deformation relationship of a member in the case of flexural failure occurring first under an exerting compressive axial force. Considering its characteristics, damage level of the member is defined corresponding to deformation range as follows.

1) Damage Level 1: before point B
2) Damage Level 2: from point B to C
3) Damage Level 3: from point C to D
4) Damage Level 4: after point D

Once a relationship between damage level and deformation is established, the amount of deformation that may be directly calculated from a response analysis becomes a relevant index for checking damage levels. If nonlinear behavior of a member is evaluated with a mechanical model of bar, generally, rotation angle or curvature for the section of plastic hinge is taken as an index for checking member. The relationship between them is shown in Table 3.

Table 3 Relationship between damage level of member and rotation angle

	Limit Values of Rotation Angle
Damage level 1	υ_{yd} : Yield rotation angle of member
Damage level 2	υ_{md} : Rotation angle of member corresponding to the maximum deformation resulting from the peak lateral loading
Damage level 3	υ_{nd} : Rotation angle of member corresponding to the maximum deformation being able to resist the yield lateral loading
Damage level 4	υ_{ud} : Rotation angle of member for limiting the excessive deformation in axial direction

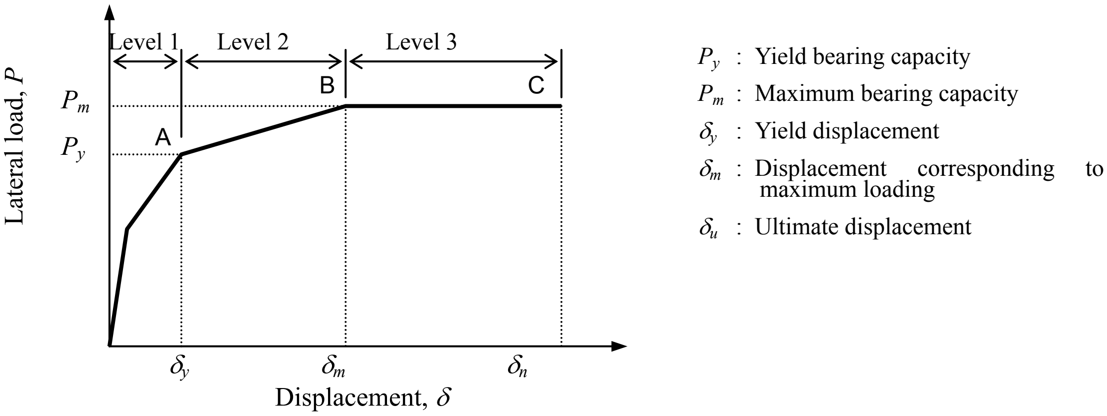

Fig. 13 Load-displacement curve with stability levels of a foundation

Stability Levels of Foundation

In order to ensure seismic performance for an overall structure, stability level of foundation should be determined with two aspects: damage level with respect to the stability of foundation itself, and damage level of members constituting it.

For evaluating these items, two indexes that are response ductility ratio defined as a ratio of seismic response displacement to yield displacement obtained from the load-displacement curve of foundation, and residual displacement should be used. Using displacement indices in the load-displacement curve of foundation with stability levels generally illustrated in Fig. 13, the stability levels of foundation can be determined as follows.

1) Stability Level 1: In principle, load acting on the foundation should be less than its yield bearing capacity and no excessive displacement occurs. Stress resultant of members composing the foundation should not exceed yield strength.
2) Stability Level 2: Bearing capacity should be maintained sufficiently even though either subgrade supporting foundation or members composing the foundation or both of them are deformed plastically. There is neither displacement detrimental to maintenance of structural functions, nor residual displacement to be allowable after an earthquake.
3) Stability Level 3: Bearing capacity should be maintained sufficiently enough to protect the structure from collapse because of damage of bearing subgrade or structural members.

Besides the values of stability level are set corresponding to the types of foundation.

Limit Values

Based on the consideration explained above, the parts of a rigid frame viaduct where damage may occur, are illustrated in Fig. 14, and an example of the relationship among the limit values of structure's seismic performance levels, member's damage levels and foundation's stability levels is shown in Table 4.

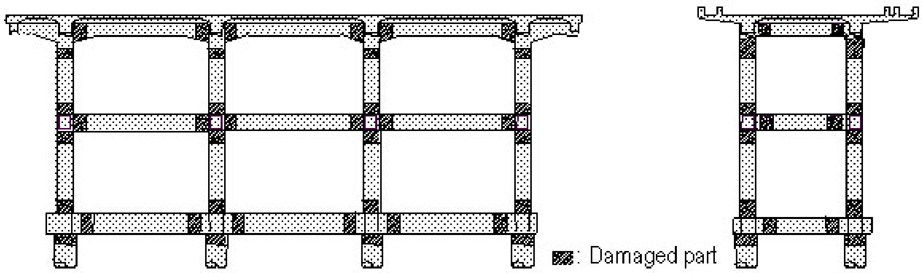

Fig. 14 Illustration of damaged parts of a rigid frame viaduct

Table 4 An example of the relationship among the limit values of structure's seismic performance levels, member's damage levels and foundation's stability levels (rigid frame viaduct)

Structure		SPI	SPII	SPIII
Damage level of member	Superstructure girder and underground beam	1	2	3
	Other beams	1	3	4
	Columns	1	3	3
Stability level of foundation		1	2	3

SAFETY (SEISMIC PERFORMANCE) CHECKING OF STRUCTURES

Static nonlinear (i.e. "pushover") analysis is stipulated to apply in the checking process. Its procedures are: i) modeling overall structure (from superstructure to foundations) to a frame structure, and subgrade supporting foundation to a spring system; ii) setting strengths and deformation behaviors for structural members and subgrade reactions based on mentioned above; iii) calculating structural displacements by increasing seismic load incrementally and plotting the relationship between seismic load and displacement. By indicating various critical steps in the load-displacement curve, the failure of overall structure can be grasped. Such critical steps include the steps where structural capacity reaches the limit values of yield, maximum and ultimate. The ultimate displacement can be determined by comparing calculated displacement with the limit value listed in Table 3. The ultimate displacement for overall structure is determined from the displacement when the capacity of a certain member belonging to superstructure or foundation reaches the limit value of ultimate state. Therefore, a structure is judged safe if its ultimate displacement is larger than the response displacement calculated by dynamic analysis method, that is, the designed seismic performance satisfies the seismic performance objective.

Furthermore, the judgment of each member's damage level and foundation's stability level should be conducted by checking the deformation state of the step in the pushover analysis, whose displacement is as same as that calculated by the dynamic analysis method. The main contents about this checking are described as follows.

Checking Damage Levels of Members

In checking damage level of a concrete member, failure mode should be judged at first. If shear stress calculated is smaller than shear strength when flexural strength is reached, the failure mode is defined namely as flexural failure mode, inversely as shear failure mode. In the codes, it is stipulated that real strength of reinforcing bar should be used in the judgment of failure mode.

Damage level of a member with flexural failure mode can be judged with the deformation calculated from a static nonlinear analysis. However, in the case of shear failure mode, judgment can only be done according to the strength. That means, deformation behavior of a member with shear failure mode should be set to linearity in the overall structural model for static nonlinear analysis.

Checking Stability Levels of Foundation

In the approach, following items are stipulated for checking the stability level of foundation.

1) Response ductility ratio of foundations;
2) Damage levels of the members composing foundations;
3) Residual displacement of foundations.

This residual displacement is taken as a main index for checking Seismic Performance II. Therefore, allowable value of the residual displacement should be limited within a small range so that operational function of train could be quickly recovered.

All the items above are checked based on results obtained from the static nonlinear analysis.

CONCLUSIONS

This paper presents a seismic damage analysis of railway structures due to the 1995 Hyogoken-Nanbu earthquake in Japan and a new seismic design established based on the lessons learned from the analysis results, with basic principles and some important advances for the design.

Adequacy of the methodology should be confirmed through precise analysis of real damage examples from the past earthquakes. The current seismic design methodology will be improved and become more and more perfect along with achievements from modern researches in the near future.

Moreover, the methodology becomes rather complicated because of the consideration of non-linearity of both structure and subgrade. In order to avoid meaningless complication, approaches for seismic design are essentially used to express damage level of structures. Therefore, damage state of a designed structure during an intense earthquake could be anticipated.

At last, it is noticed that the precision of input parameters concerning structure and subgrade and the computing accuracy should be appropriate to the execution of computer. Even though the level of design method is promoted, a design using incorrect input data cannot be considered as a good one.

REFERENCES

Nishimura, A. (2002). “Seismic design for railway structures, earthquake resistant design codes in Japan.” *JSCE*, January 2002.

Nishimura, A. (1999). “Earthquake resistant performance and design method for foundation.” *RTRI Report*, Vol. 13, No. 3, March 1999.

(Submitted: March 31, 2004)
(Accepted: July 2, 2004)