COMPARISON OF PERFORMANCE REQUIRED BY BRIDGE DESIGN CODES IN VARIOUS COUNTRIES

by

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ABSTRACT

The authors carried out a comparative analysis of the regulations for bridge design codes in various countries, as for required performance and seismic performance etc. In Japan, the revision of the Design Specifications of Highway Bridges to performance based design is now under study. This comparison analysis will make a significant contribution to the revision of the Specifications. The end of this report concludes with a summary of principal contents and an outline of future issues.

KEY WORDS: Bridge Design
Performance Based Design
Required Performance
Seismic Performance

1. INTRODUCTION

Around the world, the adoption of performance based codes is being debated by many organizations in architectural field and has been partially completed. In Japan, new Building Standard Law, that is performance based standard, will be enforced the beginning in 2000 and a trial of performance based ordering of paving work is being carried out. Performance based design clarifies the performance that structures require, and a number of suitable verification methods to be used to verify their performance have been proposed.

In Japan, the revision of the Design Specifications of Highway Bridges to performance based code is now under study. A comparative analysis of the performance required by bridges under codes in various countries will make a significant contribution to this study.

To study the requirements for performance based design in bridge design codes, the authors performed a comparative analysis of the regulations for general bridge design, required performance and seismic performance in present bridge design codes in various countries. Excluding the articles specifying seismic design in the codes of various countries, there are no complete codes that can be defined as performance based codes. Consequently, the authors focused and arranged at terminology such as "philosophy" and "requirements" in various countries' codes.

The study dealt with the following codes.

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Regulations and required performance for general bridge design compared were those stipulated in the LRFD version of the AASHTO, Eurocode, Caltrans and the revised edition of the Design Specifications of Highway Bridges (Japan, now under study). And the seismic design in the LRFD version of the AASHTO, Eurocode, Transit Bridge Manual (New Zealand) and Design Specifications of Highway Bridges were compared.

(1) United States of America;
- AASHTO LRFD Bridge Design Specifications (1998, 2nd Edition)[1)] [AASHTO]
(2) European Union;
- Draft prEN1990: Basis of Design (1999)[2)]
- Preliminary Draft EN1997-1: Geotechnical Design, Part 1 General Rules (1999)[3)]
[Draft EN]
- ENV1998: Design provisions for earthquake resistance of structures (1994)[4)] [ENV1998]
(3) State of California, U.S.;
- Memo to Designers 20-1/Seismic Design Methodology (1999)[5)]
- Caltrans Bridge Design Specifications (1990)[6)]
- Seismic Design Criteria, version 1.1(1999)[7)]
[Caltrans]
(4) New Zealand;
- TNZ Bridge Manual for Design and Evaluation (1995)[8)] [TNZ]
(5) Japan;
- Revised edition of the Design Specifications of Highway Bridges (200X)[9),10)] [Draft JRA]

2. OVERVIEW OF PERFORMANCE BASED DESIGN

The term of performance based design is interpreted in various ways. A broad categorization of codes that are called performance based design places the following two classes.

In one case, following the arrival of the age of mega-competition as a consequence of the appearance of a single world market after the collapse of the Soviet Union that ended the cold war, the international community needed common rules. And they adopted "performance" as a key word accepted by all nations. The architectural field has past provided many performance elements including seismic, safety, fire resistance and human-friendly performance. And constraints applied by regulations and laws by governments on these various performance elements are obstacles to international trade. In the Nordic countries, a movement to establish "performance" as a key word that would destroy these obstacles began in the early 1970s. This movement spread to England in the 1980s, to Australia and New Zealand in the 1990s, and in recent years has been reflected in the revision of the Building Standard Law of Japan. So based on the hierarchical character of regulations (or required performance), the degree to which they are endowed with legally constraining is debated. Figure 1 presents a hierarchy of regulations based on the Nordic 5 level system[11)]. The only legal objection is generally whether or not a building's performance is guaranteed and designers are given greater freedom to decide on the method to verify their performance. In this case, performance based design can be interpreted as synonymous with free design method.

In the second case, the need for developed seismic technology has been pointed out and great progress has been achieved based on the lessons learned from a series of earthquake disasters beginning with the Loma Prieta (1989), Northridge (1994) and Hyogo-ken

Nanbu Earthquake (1995). Because, partly as a result of its technological difficulty, the limits to the capacity of structures against a large earthquake were not adequately explained to owners (or users) who were not professionals, they fell into the illusion that most buildings could withstand any conditions. Awareness of this problem triggered a movement to clarify performance as a design index that can gain the mutual approval of owners and designers. Vision 2000[12] that the Structural Engineers Association of California (SEAOC) rushed to codify performance that structures require in the form of a performance matrix. Figure 2 shows an example of a performance matrix. The horizontal axis is called performance level, that is corresponding to performance (equal to limit states), and the vertical axis is called hazard level.

These movements appeared first in the architectural field, and it can be stated that at this time, codes incorporating these approaches have not been established in bridge design field anywhere in the world. Since the Northridge and Hyogo-ken Nanbu Earthquake, performance requirements related to seismic performance has been carried out based on the results of technology development, but nothing has been done outside of the seismic design field. Therefore, a comparison of required performance for bridge design had been carried out by focusing and arranging design philosophies or requirements in various codes.

3. COMPARISON OF PERFORMANCE REQUIRED AND OTHERS BY BRIDGES

Table 1 presents required performance, foundation design, etc. that are stipulated in codes for four regions, countries, or states. The followings are special mentions.

- Because the characteristics of the codes differ, required performance of bridges is stipulated from their respective perspectives.
- Because the AASHTO and Draft JRA deal directly with bridges, the required performance is specific.
- Eurocode is not classified by structural form; rather by each material used in civil engineering and architectural fields. Consequently, the Draft EN does not directly specify required performance for bridges.
- Caltrans stipulates the basis of seismic design on the premise that it will be applied to "normal or standard bridges." The regulations excluding seismic provisions of the Caltrans are fundamentally correspondent to the AASHTO.
- Draft JRA is based on a proposed revision that is now under study. Consequently, as in the case of the Draft EN, finally its configurations and contents may change.

(1) Required Performance
As for the Draft EN, the strict required performance can not be described, because a structure has not been specified on the character of the code. Therefore, in the Draft EN, the general limit states are stipulated, substituting with the concrete required performance. In the Draft EN, ultimate limit state and service limit state has been set. The former limit state also contains not only structural safety but also human safety, and appearance is also included in the latter limit state.

The required performance of the Caltrans for large earthquakes for bridges is simple and concrete, and only collapse limit state of a structure, this is a kind of ultimate limit state, is included. In the AASHTO, economical efficiency, beauty and landscape are emphasized in addition to safety and serviceability of structures. To ensure the safety of bridges, ductility, redundancy and importance are considered in the code. And, the performance of serviceability is used for the wide meaning, including durability, inspectability, maintenanceability, amenity and deformability, etc.

In the Draft JRA, it would be specified almost equal required performance of the AASHTO. As an examination of economical efficiency, it would be epoch-making to recommend the evaluation of life cycle cost of a bridge. Still, in the design of substructures and seismic design of the Draft JRA, performance I～III and seismic performance I～ III would concretely be set as required performance for the safety of bridges.

The design of bridges which harmonize with beauty, landscape and environment would be emphasized, as shown in Table 1.

(2) Basis of Design

Ductility, redundancy and importance have been taken up as the basis of bridge design. In all codes, ductility in seismic design has been evaluated. In the AASHTO and Draft EN, redundancy as a bridge structure has been noticed, and in the Caltrans and Draft JRA, redundancy as a road network after earthquake has been noticed. Importance has been applied to seismic design for all codes.

(3) Design Service Life

A life is distinguished by service life and design life. Service life is correspondent to what is called design service life. Design life is correspondent to reference period of ISO2394(1998)[13], and design life is a period as a base for calculating fluctuating loads.

In the Draft EN and Draft JRA, the design service life of bridges is shown as 100 years. On the other hand, in the AASHTO, service life has been not specified, generally it has been understood with 100 years. Still, design life has been specified as 75 years.

(4) Life Cycle Cost

In the AASHTO, life cycle cost has been not described. In the U.S., the Life Cycle Design and Performance (1997)[14] that has been prepared by the U.S. Army Corps of Engineers is the regulation that stipulates the basic framework for performance based design applied to public works projects. It stipulates that engineering decisions should not do simply minimize initial costs or maximize reliability that ignores costs, and demands that designers perform evaluations based on life cycle cost of an entire project.

The design of highway bridges in the U.K. is based on BS5400 (Code of Practice for Bridges, BSI) and conforms with many Government Departmental Standards in the Design Manual for Roads and Bridges (DMRB). The code BD36 in the DMRB provides the method of comparing life cycle cost (the design service life in the U.K. is 120 years premised on appropriate maintenance).

The Draft JRA is the first code to clearly stipulate the concept of evaluating the optimum proposal based on total cost during service life.

(5) Limit States

Limit States are engineering indices for checking whether structures satisfy their required performance. In the AASHTO, service limit state, fatigue limit state, strength limit state and extreme event limit state are set. Each limit state means the following;

- Service limit state: under usual service conditions, stress, deformation and crack are limited.

- Fatigue limit state: stress width by repetition traffic loads is limited.
- Strength limit state: for load combinations which encounter during design life, the strength and stability of the local and whole of a structure are compensated for. Though damages are received, the whole structural system is maintained.
- Extreme event limit state: for a large earthquake, flood and the collision of a ship, the residual capacity of a bridge is guaranteed.

The basis of seismic design of the Caltrans is based on the deformation. The purpose of seismic design is to check the deformation occurring by a large earthquake, and in the Caltrans, only collapse limit state is specified. Collapse limit state of a structure is defined as a state not resisting to its dead weight, by generating large deformation.

In the Draft JRA, limit states are not stipulated, because of using an allowable stress design (WSD) method.

(6) Basis of Verification Method

The verification equation of a limit state in the AASHTO is basically provided as the following.

$\Sigma\ \eta_i \gamma_i Q_i \leqq \phi R_n = R_r$

Where:

$\eta_i = \eta_D \eta_R \eta_I \geqq 0.95$, or $\eta_i = 1/(\eta_D \eta_R \eta_I) \leqq 1.0$

γ_i: load factor

Q_i: load effects

ϕ : resistant factor (factor that is multiplied by nominal resistant force)

R_n: nominal resistant force

R_r: resistant force multiplied by resistant factor (design resistant force)

Here, the factor η_D for ductility and the factor η_R for redundancy are used to verify strength limit state, and the factor η_I for importance is used to verify extreme event limit state. As the AASHTO, in future bridge design, the effects of ductility and redundancy must be represented by a factor in some way in codes. The design method of the Caltrans converted from conventional force based design method to displacement based design method. In seismic design, it seems to introduce by and by this design method.

(7) Load

As shown in Table 1, the classification of loads is different by each code at present. The classification of the Draft EN is fundamentally similar to the regulations of ISO2394. Hereafter, it is required to match the ISO code.

In the Draft JRA, the effect of salt attack, effect of corrosion, fatigue by the repetition of live loads would be newly considered as other actions and effects.

(8) Foundation Design

In the AASHTO, three limit states except for fatigue limit state have been checked. In the calculation of resistance force (design vertical bearing capacity), resistant factor $\phi \lambda_v$ is considered. Here, ϕ is the factor which depends on the estimation method of bearing capacity, and λ_v is the factor which depends on construction management and estimating system of bearing capacity. In the Eurocode, such resistant factor is not given. The desired value of the safety index β in calculating this resistant factor is 2.0 ~ 3.0, according to Appendix A[16] of Barker et al (1991).

The foundation design of the Draft EN has been specified at EN1997-1(Geotechnical design). The MFA (Material Factor Approach) and RFA (Resistance Factor Approach) have been proposed as partial design formats of ground resistant in Draft EN. The design resistant force R_d based on each approach is represented by the following equations.

Approach of MFA: $R_d = R(X_R/\gamma_M, a_{nor})$

Approach of RFA: $R_d = R(X_R, a_{nor})/\gamma_R / \xi$

Where:

X_R: characteristic value of ground materials

γ_M: material factor

γ_R: resistant factor

a_{nor}: nominal value of geometric quantity

ξ : correlation ratio

In the MFA approach, the partial factor γ_M, as shown in Table 1, is directly considered in ground parameters. The RFA approach is used for the design of piles, and it is similar to the LRFD approach. The correlation ratio ξ is a factor considering the frequency of in-situ loading tests.

In the Caltrans, both of SLD (Service Load Design) and LFD (Load Factor Design) method are ever used as design methods. The resistant factor ϕ has been applied to the latter method. Here, the ϕ is divided in every Groups (combination of loads), and in the Group VII (present SDC) including an earthquake action, factor of 1.0 is applied, and in other Groups, factor of 0.75 is applied. There is no relation between ground parameters and estimation methods of bearing capacity.

In the present JRA, because the reliability of bearing capacity of a pile by in-situ loading tests is higher than that of empirical estimation method, the correction factor γ is applied to estimate bearing capacity to reflect the effect it. And, the introduction of the factor which represents quality and quantity of ground parameters has been also examining as future problem.

4. COMPARISON OF SEISMIC PERFORMANCE

Table 2 shows the comparison of seismic regulations in various codes. Many studies such as that by Backle (1996)[17] or NCEER (1997)[18], have compared and considered regulations in seismic design codes in various countries. This report adds considerations from the perspective of international harmonization and locality of codes. The New Zealand's TNZ Bridge Manual (1994) instead of the Caltrans is shown in Table 2.

(1) Basis of Seismic Design

The AASHTO clarifies seismic performance for two levels: medium earthquakes and large earthquakes. The ENV1998 and TNZ mention to traffic functions after an earthquake. The present JRA on the other hand treats differences in seismic performance required according to the importance of a bridge as basis of seismic design.

(2) Seismic Performance

Because in the AASHTO, the effects of earthquakes consider only extreme event limit state as stated in Table 1, the seismic performance required has been interpreted as only this limit state. As shown in 1) of Table 2, the AASHTO has adopted a two-level design seismic force, but it is not clear to the authors whether or not, in practice, this extreme event limit state is verified for both medium and large earthquakes. The limit states in the Eurocode are basically service limit state and ultimate limit state. In conformity with this concept, the ENV1998 applies these two limit states to post-earthquake service and post-earthquake safety respectively, with the former verified based on displacement and the latter verified based on strength of a structure. In the TNZ, it stipulates, not clarified in the Bridge Manual, that an earthquake equivalent to an ultimate limit state with return period of 450 years be considered[19]. Based on the earthquake motion hypothesized by this level of earthquake, the seismic performance is specified for small and large cases. The present JRA combines three limit states, post-earthquake service, post-earthquake restoration and post-earthquake safety with bridge importance A and B. Judging from bridge importance, two seismic performances, post-earthquake service and post-earthquake restoration (for a case of class B bridges), and post-earthquake service and post-earthquake safety (for a case of class A bridges) are combined with level 1 and level 2 earthquake motion respectively.

(3) Importance

Although definitions of bridge importance differ, they are categorized in two or three classes in all codes. Categorizations based on importance are, in the case of the AAHSTO, reflected in the factor for importance η_i in the calculation equation for the design load Q. Importance in the ENV1998 and TNZ is related to adjusting the return period of design earthquake motion for a normal bridge, but for practical design procedures, the design earthquake force of a normal bridge is increased or lowered. In the case of the present JRA on the other hand, it is reflected in differences in the required seismic performance indicated in h) and in differences in the safety factor used to decide an allowable ductility factor. Considered from this perspective, in contrast to the AASHTO, ENV1998 and TNZ, the specifications of the JRA are detailed.

(4) Zone Classification

The AASHTO has set the value A for a construction site based on the hazard map of return period of 475 years (10% probability of exceedance during service life of 50 years). The seismic zone has been set as class 1 to class 4 according to the value of A. This zone factor has been used to establish differences in required analysis methods, detailed of bridge piers, and bridge abutments and foundations design method. The ENV1998 has not set specific zone factors, but the design ground acceleration ag reflects the degree of earthquake risk in each nation. The TNZ has provided the contour map from 0.4 to 0.8 as its zone factor Z. In New Zealand, instead of the present deterministic hazard map, the distribution map of maximum ground acceleration based on a probabilistic risk analysis would be used as the zone factor (Figure 3).

(5) Design Earthquake Motion

The AASHTO has set two design earthquake levels. For a medium earthquake, it is based on the value of A for return period of 475 years (10% probability of exceedance during design service life of 50 years). For a large earthquake, the maximum credible earthquake (MCE) is considered to be an earthquake with return period longer than the design earthquake: an earthquake with return period of 2,500 years for example. The ENV1998 defines a large earthquake as an earthquake with return period of 475 years, but regarding a medium earthquake, it simply stipulates: "earthquake activity with an appropriate return period shall be evaluated." In the provisions of the TNZ, one level earthquake motion of return period of 450 years is considered. The present JRA defines a medium and large design earthquake motion as earthquake motion with a high probability of occurring during the service life of a bridge and strong one with a low probability of occurring respectively, but does not stipulate specific return periods.

(6) Elastic Response Spectrum

The part m) of Table 2 presents the basic equation for the elastic response spectrum in each code. Here, only the TNZ uses non-linear spectrum (yield seismic intensity). New railway seismic codes issued in 1999, in Japan, use non-linear response spectrum[20].

(7) Load Reduction Factor

The AAHSTO and ENV1998 establish reduction factors R or q based on the importance of a bridge and the type of a substructure as ways to reduce the elastic seismic force accounting for the non-linear response of ductile members. The TNZ on the other hand, has not set a load reduction factor because it has used non-linear response spectrum for allowable ductility factor μ (= 1 to 6). The present JRA differs from the other codes in that it applies the law of energy conservation to evaluate the load reduction as the reduction of design lateral force.

(8) Over strength Factor in Capacity Design

A designer must guarantee appropriate differences in the safety (hierarchy of strength) of non-linear members and ductile members in order to count on non-linear behavior of ductile members that the designer intends. In the design of a foundation, the design lateral force that a factor is multiplied by the ultimate lateral capacity of the pier is used. The factor is an over strength factor. The over strength factor is quantitatively related to the value of q in the ENV1998. The TNZ only refers to the capacity design principles in the NZS4203:1992; it does not specify a specific factor value. Because the present JRA provisions regarding the evaluation of the design lateral force used for the design of a foundation specifies that it must be 1.1 times the ultimate lateral capacity of the pier, it is generally understood that capacity design is introduced under present conditions.

6. SUMMARY AND FUTURE ISSUES

The authors have been studying the performance regulations to compare specifications for general bridge design and specifications for seismic design in codes in various countries. This report concludes with a summary of principal items and an outline of future issues.

(1) The design methods adopted by the codes surveyed include LRFD, LSD and WSD method. They should be unified in order to clarify the safety of a structure. The ISO code concerning design methods for structures is ISO2394, and the goal is the LSD or LRFD method of partial factors format that is stipulated in ISO2394.

(2) Under present conditions, stipulated contents that define the required performance for bridge design varies between the codes. The stipulated contents and required performance are hierarchical, and it is necessary to accompany debates concerning the extent to which they provide legal constraining by unifying them based on the triangle structure shown in Figure 1.

(3) As in the case of the AASHTO, it is important that in codes the effects of ductility and redundancy are represented by factors in some way in future bridge design.

(4) The code that refers to life cycle cost is only the Draft JRA. This concept is indispensable for codes that advocates performance based design. Design service life is clearly defined by the Draft EN. It is, therefore, necessary to clarify the grounds for setting the service life 100 years for bridges.

(5) The load categorization in the Draft EN differs from that in the other codes. In the Draft EN that is based on ISO2394 (1998) and is internationally unified. Future load categorization and combinations applied in bridge design should comply with the ISO code.

(6) As for foundation design, only the MFA approach of the Draft EN has considered partial factors for ground parameters. The relation between the estimation method of bearing capacity and resistant factor is detailed in the AASHTO. In the Draft EN (RFA approach) and present JRA, the bearing capacity with in-situ loading tests has been estimated, accounting for a resistant factor.

(7) The basis of seismic design is, in many cases, focussed on serviceability, safety and inspectability after earthquakes. As pointed out by Buckle (1996)[17)], the future issues are to quantitatively represent this using engineering indices.

(8) Regarding importance categorization, the JRA reflects importance in differences in seismic performance, but in the other codes, it is linked to differences in the value of design seismic forces.

(9) Except for the TNZ, the other codes set two levels of earthquake motion. The JRA actually provides for the performance of two design levels while the other codes stipulate verifications against a large earthquake and practically eliminate verifications against a middle earthquake. A two-level design method is more rational than one-level. It is necessary to study the matter to determine whether it is necessary for practical designs accounting for the importance of bridges.

(10) Excluding the JRA, the others evaluate earthquake motion probabilistically in the sense that they all account for return period. ISO/CD3010 (1999)[21)], going one step further, the uncertainty of earthquake motion based on probabilistic theory has been accounted as load factors. Because internationally, research has progressed from conventional MCE based deterministic risk analysis to probability analysis, the movement to introduce probability theory to the evaluation of design earthquake motion has gained momentum.

(11) At this time, only the TNZ has accounted for non-linearity in response spectrum. This method has been adopted in railway bridge codes in Japan. After the earthquakes in Turkey and Taiwan, the evaluation of active faults has become an important research theme. The Caltrans has stipulated that the spectrum be increased by a maximum of 20 % depending on the distance from a fault.

(12) The AASHTO and Draft EN have used load reduction factors (R value or q value) to reduce an elastic response value. A particular characteristics of the latter code is that an over strength factor used for capacity design has been represented as a function of the q value. The introduction of LSD method to seismic design is counted on to provide for the evaluation of load factors and evaluation method of rational over strength factors.

In conclusion, the authors wish to express their opinion regarding precautions that should be considered during revision of the present JRA. As stated at the beginning of this report, performance based regulations in future codes is essentially international trends. It is also necessary to achieve harmonization with international rules, and ISO2394 and ISO3010 codes must be considered as closely as possible. In this sense, an LSD method based on partial factors format (or an LRFD method) including seismic design specifications should be introduced to the future JRA. On the other hand, differences in the safety level of structures and natural environments, and regional and national differences in the stock of design and construction technology, and the level of engineers are the matters that must be recognized as localities, and which differ from items that should be unified. The revision of the present JRA must be carried out to achieve performance based design and to consider international harmony and locality.

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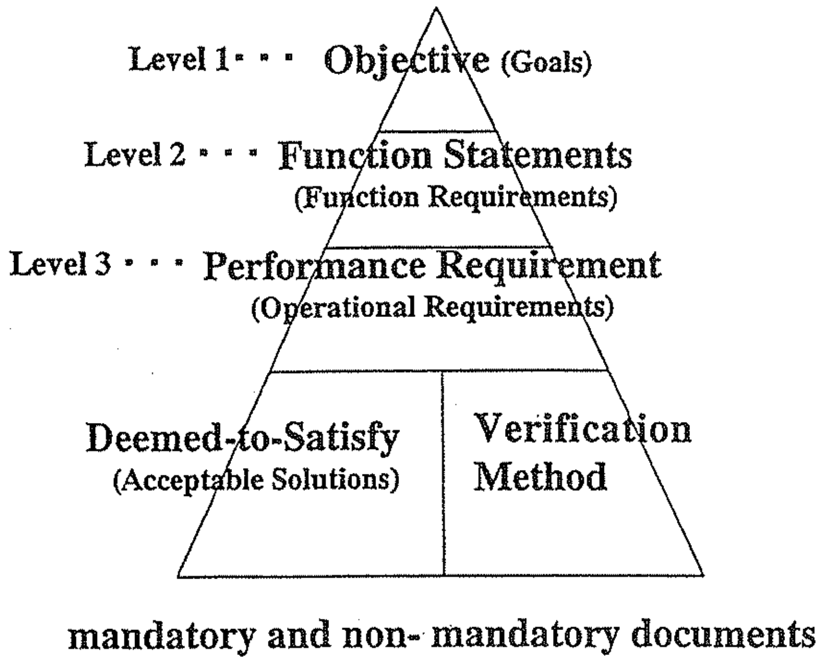

Figure 1 Nordic Five Level System

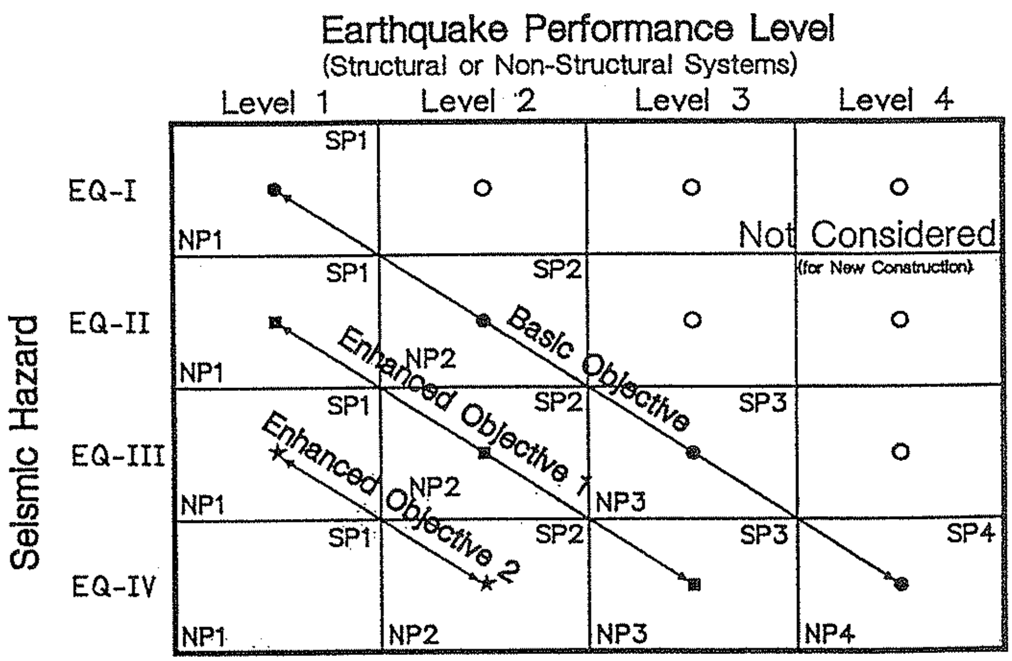

Figure 2 Typical Seismic Performance Objective for Buildings

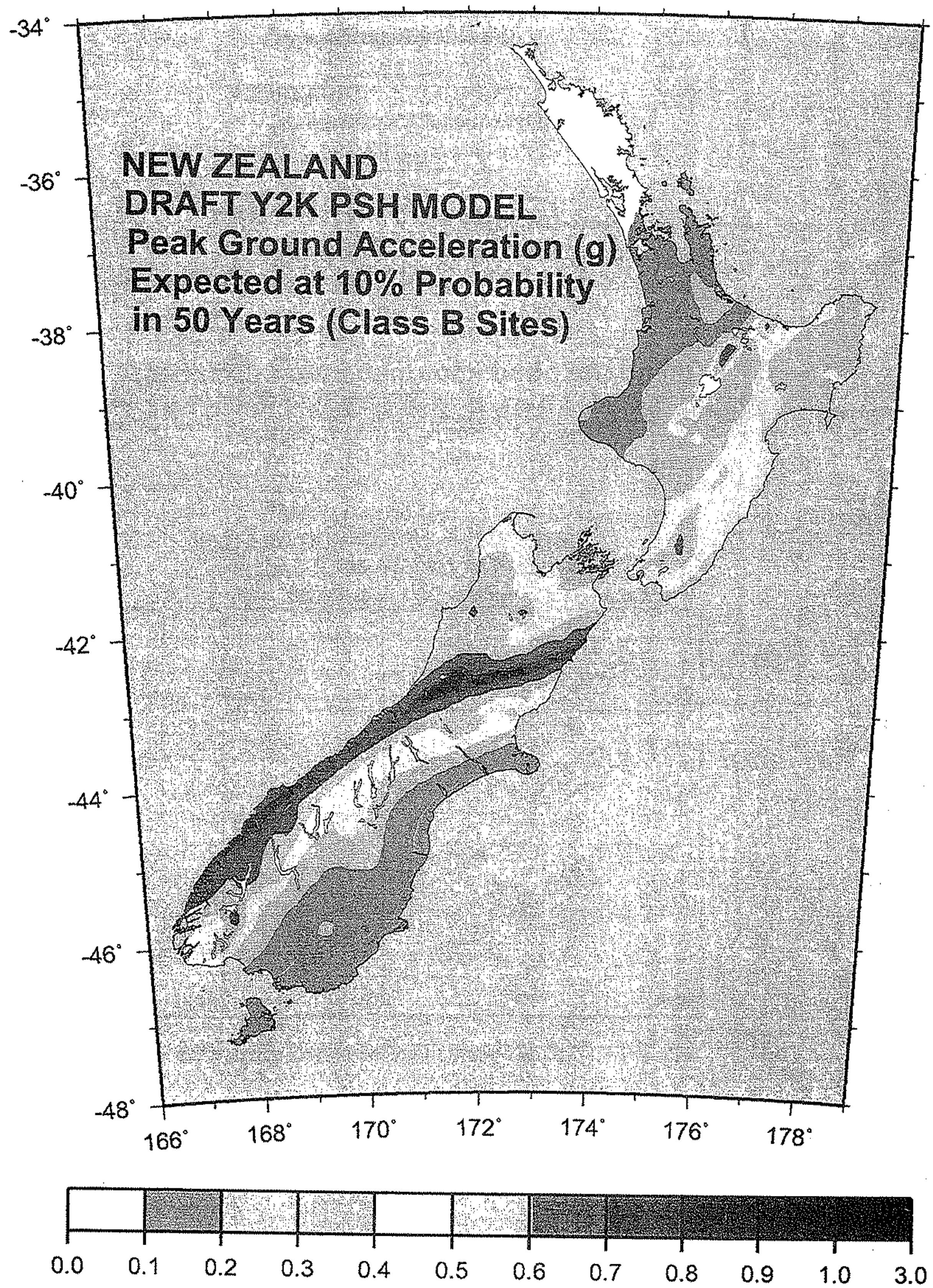

Figure 3 Distribution Map of Maximum Ground Acceleration Based on Probabilistic Risk Analysis

Table 1 Comparison of Required Performance for Bridges in Various Countries

Region/Country	United States of America	European Union	State of California	Japan
a) Code	AASHTO LRFD Bridge Design Specifications (1998, 2nd Edition)[1] [AASHTO]	Draft prEN1990: Basis of Design (1999)[2] Preliminary Draft EN1997-1: Geotechnical Design, Part 1 General Rules (1999)[3] ENV1998: Design provisions for earthquake resistance of structures (1994)[4] [Draft EN]	Memo to Designers 20-1/ Seismic Design Methodology (1999)[5] Caltrans Bridge Design Specifications (1990)[6] Seismic Design Criteria, version1.1 (1999)[7] [Caltrans]	Design Specifications of Highway Bridges (200X) A: Common rules B: Seismic Design [Draft JRA]
b) Organization	American Association of State Highways and Transportation Officials	CEN (European Committee for Standardization)	State of California, Department of Transportation (Caltrans)	Ministry of Construction
c) Code Category	Optional domestic standard	Regional standard	Compulsory state standards	Compulsory domestic standard
d) Structure Category	Expressway Bridges	Civil engineering and building engineering	Highway bridges (B; applied to normal or standard bridges)	Highway bridges
e) Region	North America (United States, Canada)	Europe	State of California	Japan
f) Design Method	LRFD : Load Resistance Factor Design	LSD: Limit State Design	Service Load Design+Load Factor Design	Allowable stress design method
g) Required Performance	● Safety ● Serviceability ● Economy ● Bridge Aesthetics	The following two limit states are set. a) Ultimate limit state - Safety of a structure and its parts - Safety of people b) Service limit state - It must fulfill the roles of a structure (or part of a structure). - Comfort from perspective of people - External appearance	All bridges must be designed so that they can withstand deformation generated by design earthquakes. Structural members must be designed to guarantee adequate strength and ductility to avoid failure under MCE.	It must satisfy the following during the design service life: [1] Safety of a structure [2] Harmony with purpose of use [3] Economic effects [4] Harmony with the environment
h) Basis of Design (ductility)	Confirmation of ductility is important for a bridge structural system. The factor η_D is considered at strength limit state.	Structural systems that could possibly fail unexpectedly shall be avoided as much as possible.	The benefits of ductility and post-elastic strength are considered in design in order to satisfy performance with a small investment.	Ductility of a member is considered at the verification against a large earthquake.
(redundancy)	A multi-load path and continuous bridge structure should be used. The factor η_R is considered at strength limit state.	A structural form or design method that provides for resisting to accidental loss of its constituent elements or accidental damage to its parts shall be selected.	If practical, the entire bridge system must be provided with redundancy in the form of alternate routes for loads.	It is important to enhance the redundancy of a highway network.
(importance)	Bridges are classified as critical, essential or other for seismic design. The factor η_I is considered at strength limit state.	Importance is classified three classes in seismic design; upper of medium, medium, lower of medium.	By desired seismic performance level, two classes are set. ● Ordinary bridges (ordinary or not ordinary) ● Important bridges Still, the Caltrans is applied to ordinary bridges.	According to road category and bridge functions and structures, bridges are categorized two classes in seismic design. ・ bridges of standard importance (Class A bridge) ・ bridges of specially high importance (Class B bridge)
i) design service life	Not mentioned. Generally, considered at 100 years.	Example for design service life; ● general structures: 50 years ● bridges: 100 years	Same as the AASHTO.	An adequately long and appropriate design service life shall be established. The standard design service life may be 100 years.
j) life cycle cost	Not mentioned.	Not mentioned.	Same as the AASHTO.	The study of economic effects should be performed to determine structural form and materials, accounting for not only initial costs, but also approximate life cycle cost.
k) limit states	● Service Limit State ● Fatigue and Fracture Limit State ● Strength Limit State ● Extreme Event Limit State	● Ultimate Limit State ● Service Limit State	Collapse Limit State	Not mentioned.

l) basis of verification	$\Sigma \eta_i \gamma_i Q_i \leqq \phi R_n = R_r$ Where, $\eta_i = \eta_D \eta_R \eta_I \geqq 0.95$, or $\eta_i = 1/(\eta_D \eta_R \eta_I) \leqq 1.0$, γ_i : load factor, Q_i : load effects, ϕ : resistant factor, R_n : nominal resistant force, R_r : design resistant force	Ultimate limit state : Ed≦Rd Service limit state : Ed≦Cd Where, Ed : design value of load effects, Rd : design value of resistant force, Cd : limit value	$\Delta_D < \Delta_C$ Where: Δ_D: larger of the two displacements generated by an analysis of an independent system or of an overall system. Δ_C: displacement of the frame at the time that the first plastic hinge of a structure reaches the performance limit.	Verification of strength: Ed≦Rd Verification of displacement: Ed≦Cd Where, Ed : design value of load effects, Rd : design value of resistant force or strength, Cd : limit of deformation
m) load (classification)	● Permanent load ● Temporary load	● Permanent load ● Fluctuating load ● Accidental load ● Earthquake load	Same as the AASHTO.	● Primarily load (P) ● Secondly load (S) ● Special load corresponding to primarily load (PP) ● Special load (PA) ● Others
(live load)	Total weight (tf) : 18.1, 32.6, 13.6, 24.5	Not mentioned.	Same as the AASHTO.	Design vehicle load is 245 kN.
(wind load)	Design basic wind speed : 160km/h (=44m/s) Wind load : 0.0036Mpa (=360kgf/m^2 Columns and Arches), 0.0024Mpa (=240kgf/m^2 Girders)	Not mentioned.	Not mentioned.	Design basic wind speed : 40m/s (40 % exceedance of probability of 50 years) Wind load (kgf/m^2) : 150 (circle · ellipse), 300 (square)
(load combination and load factor)	It stipulates combinations of loads and load factors of 11 kinds for each of the following 4 limit states. a) Service Limit State I to III b) Fatigue and Fracture Limit State c) Strength Limit State I to V d) Extreme Event Limit State I to II	For ultimate limit state, ● Permanent design state (under service) ● Temporary design state (under construction and restoration) ● Accidental design state (under fire, explosion and collision) ● Earthquake design state (under earthquake)	18 of load combinations and load factors are set, under the following conditions. ● SLD(Service Load Design) (8 cases) ● LFD(Load Factor Design) (10 cases)	Main load combinations for superstructures: ● Primarily load ● Wind load Main load combinations for substructures: ● Primarily Load (excluding live load and impact) +earthquake
n) materials (concrete and metal)	Concrete mix characteristics by class and minimum mechanical properties of metal are stipulated.	Characteristic value of materials strength is defined as 5 % of material parameter fluctuating or 95 % of fractile value.	Same as the AASHTO.	As for concrete, minimum design basic strength is specified, to confirm durability of concrete.
(ground materials)	Not mentioned.	Characteristic value is selected as careful estimation of ground parameter at limit states.	Same as the AASHTO.	Characteristic value of ground parameters is mean value, excluding unusual data.
0) foundation design (verification method)	$Q_R = \phi \cdot Q_{uk}$ (pile foundation) Where, Q_R : design bearing capacity, ϕ : resistant factor, Q_{ult} : ultimate bearing capacity Still, ϕ is applied at strength limit state. At other limit states, ϕ = 1.0.	● MFA approach : $R_d = R(X_R/\gamma_M, a_{nor})$ ● RFA approach : $R_d = R(X_R, a_{nor})/\gamma_R/\xi$ Where, R_d : design resistant force, X_R : characteristic value of ground parameter, γ_M : material factor, γ_R : resistant factor, a_{nor} : nominal value, ξ : correlation ratio	$Q_R = \phi \cdot Q_{ult}$ Where, Q_R : design bearing capacity, ϕ : reduction factor, Q_{ult} : ultimate bearing capcity Still, ϕ is applied to load factor design.	$Ra = \gamma \cdot Ru/n$ (pile foundation) Where, Ra : allowable bearing capacity, γ : correction factor of safety factor, difference from estimation method of ultimate bearing capacity, Ru : ultimate bearing capacity, n : safety factor
(partial factor of ground parameter)	Not mentioned.	Partial factor: γ_M ● For tan ϕ, c': 1.25 ● For cu, qu, 1.40	Not mentioned.	Not mentioned.
(estimation method of bearing capacity and partial factor; for example driven pile)	Resistant factor $\phi \cdot \lambda_V$ ● Empirical estimation method : ϕ =0.45～ 0.70 ● In-situ loading test : ϕ =0.80 ● Analysis of wave motion : ϕ =0.65 Where, λ_V : factor of construction management and evaluation method of bearing capacity (=0.8～1.0)	Resistant factor γ_R : 1.30 Still, correlation ratio varies 1.4 ～ 1.0 according to the frequency of in-situ loading tests.	ϕ : reduction factor (LFD) ● Group 1～6 (not including earthquake) : 0.75 ● Group7 (earthquake) : 1.00	Correction factor of safety factor γ (To account for difference from estimation method of ultimate bearing capacity) ● Case of empirical method : 1.0 ● Case of in-situ loading tests : 1.2
(flood)	Risk of flood : return period of 100 years Pressure of flow : $P(MPa) = 5.14 * 10^{-4} C_D V^2$ Factor : $5.14*10^{-4} C_D (tf \cdot s/m^3) = 0.04$ (circle), 0.07 (square)	Not mentioned.	Not mentioned.	Risk of flood : return period of 100～200 years Pressure of flow : $p(f/m^2) = P/A = K \cdot V^2$ Resistant factor K=0.04 (circle), 0.07 (square)

Table 2 Comparison of Seismic Performance for Bridges in Various Countries

Region/Country	United States of America	European Union	New Zealand	Japan
a) Code	AASHTO LRFD Bridge Design Specifications (1998, 2nd Edition)[1] [AASHTO]	ENV1998: Design provisions for earthquake resistance of structures (1994)[4] Part1-1: General rules Part2: Bridges [ENV1998]	TNZ Bridge Manual for Design and Evaluation (1995)[8] [TNZ]	Design Specifications of Highway Bridges (200X) Seismic Design [Draft JRA]
b) Organization	American Association of State Highways and Transportation Officials	CEN (European Committee for Standardization)	Transit New Zealand	Ministry of Construction
c) Code Category	Optional domestic standard	Regional standard	Compulsory domestic standards	Compulsory domestic standard
d) Structure Category	Expressway Bridges	Civil engineering and building engineering	Highway bridges	Highway bridges
e) Region	North America (United States, Canada)	Europe	New Zealand	Japan
f) Design Method	LRFD : Load Resistance Factor Design	LSD: Limit State Design	Service Load Design+Load Factor Design	Allowable stress design method
g) Basis of Seismic Design	For medium earthquakes: it shall remain undamaged by resisting within the elastic range of members. For large earthquakes: all or parts of a bridge shall not fail. Locations forecast to be damaged shall be ones whose damage can be detected, can be easily approached, and can be easily inspected and repaired.	The design philosophy requires general requirements that emergency vehicles can drive on the bridge with stipulated reliability after design earthquake.	It must be guaranteed that bridges can safely hold its traffic functions after an earthquake. To permit this, bridges are classified according to their importance, and the risk factor related to the earthquake return period (value of R) are assigned to a bridge.	According to road category, bridge functions and structures, bridges are categorized as bridges of standard importance (Class A bridge) and bridges of specially high importance (Class B bridge), and the goal is to guarantee the seismic performance required for each class.
h) Seismic performance (limit states)	Extreme event limit state: The survival of the structure of a bridge shall be guaranteed against earthquake, flood, or impact by a ship.	[1] Minimization of damage = Service limit state (evaluated by displacement) [2] Unseating prevention provisions = Ultimate limit state (evaluated by bearing capacity and ductility)	[1] Emergency vehicles can cross a bridge even if it is damaged by an earthquake. [2] After an earthquake smaller than the earthquake hypothesized by design, it must only be slightly damaged and not obstruct traffic. [3] After an earthquake larger than the earthquake hypothesized by design, the failure of a bridge must be prevented even if the damage is widespread. And emergency vehicles must be enabled to cross a bridge after emergency measures and long term measures must be possible.	[1] Service limit state after an earthquake. To be sound (Class A and B bridges) [2] Restoration limit state after an earthqua To permit limited damage (Class B bridge) [3] Safety limit state after an earthquake To prevent fatal damage (Class A bridge)
i) Importance	Bridges are classified as critical, essential or other for seismic design. It is treated as one of the factors that is part of the load conversion constant ηi in the equation used to calculate the design load Q ($Q = \Sigma \eta i \gamma iQi$), and the factor ηi for the importance is stipulated at (= 0.95 to 1.05)	Three kinds of importance factor γi is the following. [1] Greater than average (γi= 1.30) [2] Average (γi= 1.00) [3] Less than average (γi=0.70) γi is multiplied by design seismic load AED.	Bridges are categorized in 3 R-values, according to their importance that is determined by traffic volume, road functions, and availability of an alternate route. The following are return period indicated by the R-values. R = 1.3: return period of 1,000 years R = 1.15; return period of 700 years R = 1.0: return period of 450 years	There are Class A bridges and Class B bridges Differences in importance of bridges must be reflected in differences in the seismic performance and in differences in the safety factor for type I and type II earthquake motion of each bridge.
j) Ground Classification	The soil types are categorized as four types: to set a site constant (S) at I, II, III and IV.	There are 3 subsoil classes: A, B, and C.	Three categories are set: Site subsoil category (a) ～ (c). This site category reflects the three yield seismic spectra in part n) in this Table (parameter: displacement ductility factor μ).	Three categories: I, II, and III
k) Zone Classification	It is categorized in four categories from 1 to 4 according to the A-value (acceleration coefficient).	Design ground accelerations ag for each nation are set in the NAD (National application document) of each nation.	It is set from 0.6 to 1.2 as zone factor Z.	The modification factor Cz is provided at values of 1.0, 0.85, and 0.7 according to the 3 zone categories.

l) Design Seismic Force	Medium earthquake (design earthquake): Earthquake with return period of 475 years (design service life: 50 years, exceedance probability 10%) Large earthquake: Earthquake longer than the return period of the design earthquake. An earthquake with return period of approximately 2,500 years is called the maximum probable earthquake.	Medium earthquake: Earthquake motion during appropriate return period is evaluated. Large earthquake: Return period of 475 years.	Return period of 450 years.	[1] Earthquake motion with a high probability of occurring during a bridge's service life. [2] Large earthquake motion with a low probability of occurring during a bridge's service life. (Type I and Type II) Type I earthquake hypothesizes a plate boundary earthquake such as the Kanto Earthquake of 1923. Type II earthquake hypothesizes an inland intraplate earthquake such as the Hyogo-ken Nanbu Earthquake of 1995. There is no concept of return period.
m) Elastic Response Spectrum	CSM = 1.2 AS/Tm$^{2/3}$ ≦2.5A Where: C_{SM}: elastic response spectrum A: A-value S: site factor Tm: natural period of the mth mode (sec)	Se(T) = (ag, S, T, η, β0, k1, k2) Where: Se(T): elastic response spectrum ag: design ground acceleration accounting for reference return period S: ground parameter T: natural period of a single degree of freedom system η: modification factor for damping β0: response multiplier of a structure k1, k2: correction factor of spectrum curve	Maximum lateral force V: V = CμZRSpW ≧ 0.05 W Where: Cμ: basic value of acceleration coefficient Z: zone factor (0.6 to 1.2) R: risk factor (1.0 to 1.3) Sp: structural performance coefficient (0.67 to 0.9) W: dead load	[1] Earthquake motion with a high probability of occurring during a bridge's service life. Kh = Czkho [2] Large earthquake motion with a low probability of occurring during a bridge's service life. (Type I and Type II) Khe = Czkheo
n) Load Reduction Factor	The R-value (response modification factor) (1.5 to 5.0) is selected, according to a substructure type and a bridge's importance.	The q-value (behavior factor) (1.0 to 3.5) is selected, according to a substructure type and required ductility of members.	The value of displacement ductility factor μ that is the basis for the basic value (Cμ) of the non-linear response spectrum shown in part m) in this Table is set from 1 to 6.	The design lateral force coefficient khc is reduced as the equivalent lateral force coefficient khe according to the allowable ductility factor μa of a bridge pier. khe = khc /[2μa-1]$^{1/2}$
o) Over Strength Factor	Concrete members: 1.3 times nominal bearing capacity Steel members: 1.25 times nominal bearing capacity	Plastic moment factor (yo): yo = 0.7 + 0.2 q (when q = 3.5, yo = 1.4) Where: q: behavior factor	It is based on Capacity Design Principles in NZS4203.	In the case of foundation design, the design lateral force is set as 1.1 times the ultimate lateral capacity of the pier that a foundation supports.