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Japanese Seismic Design Codes Prior to Hyogoken–Nanbu Earthquake

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Abstract

A large number of concrete bridges were damaged by the Hyogoken–Nanbu earthquake. Many of these bridges were found to have crumbled after suffering shear failure. This indicates that the original seismic design of these structures was inappropriate. Bridges that did not suffer shear failure, however, generally survived collapse even if they were severely damaged by bending moment. It has been confirmed that concrete structures do not actually collapse if they have sufficient resistance to shear failure, even if the displacement they experience is far beyond the bending yield displacement. This justifies the latest seismic design methods, in which collapse under the influence of extreme seismic motion is prevented by making the resistance to shear failure adequate enough to ensure the survival of each member. These latest seismic design codes and the changes in the codes are described.

CURRENT SEISMIC DESIGN STANDARDS

Concrete standard specifications of the Japan Society of Civil Engineers (JSCE) [1]

In the Design Version of the JSCE Concrete Standard Specifications (CSS), the overall design principle was updated to a **limit state** design method in 1986. At the same time, the method of seismic examination was completely revised on the basis of the latest findings. The 1991 version retains the new findings included in the 1986 revision almost unchanged. The new concept, made explicit in the revised standards, is that the aim of seismic design is to ensure both safety during an earthquake and usability after the earthquake. Further, the state of damage after an earthquake is divided into four categories: 'sound and maintainable, 'minor damage,' 'intermediate damage' and 'severe damage.'

A structure suffering 'intermediate damage' can remain in service if repaired or inspected within a certain appropriate period after the earthquake. A structure that suffers 'severe damage' is supposed to be repaired and reinforced at the earliest possible opportunity. According to their public utility, economic efficiency, importance in the earthquake aftermath and required durability, it is generally recommended that civil engineering structures should, if damage is unavoidable, suffer no more than 'minor damage' after a design earthquake.

This design earthquake, which is the one assumed in the design of a structure before it is built, is usually defined as the largest earthquake expected to occur once during the service life of the structure. The inertial force assumed to act on a structure is its weight plus the weight multiplied by a design seismic factor. The lateral seismic design factor normally adopted is a standard figure of 0.2 corrected, if necessary, according to the local conditions, the ground characteristics, the natural frequency of the structure, the need for post-earthquake usability and the seismic response of members not considered in the calculation. (See Table 1 and Fig. 1.) It should be noted, however, that the level of seismic motion assumed in these standards is significantly less than that experienced in the Hyogoken–Nanbu earthquake at the locations where damage was most severe.

In the CSS, seismic motions greater than the design level are handled by bringing in a safety

Table 1. Correction coefficient for design lateral seismic factor[1]

Correction coefficient	*Classification*	*Value*
Location (ν_1)	• Highly active seismic area	1.0
	• Area with average seismic activity	0.85
	• Area with low seismic activity	0.7
Ground (ν_2)	• Class 1 Bedrock	0.9
	• Class 2 Diluvial ground	1.0
	• Class 3 Alluvial ground excluding soft ground	1.1
	• Class 4 Soft ground	1.2
Structure's natural frequency (ν_3)	See Fig. 1	0.5 to 2.0
Structure's critical status (ν_4)	• Sound and maintainable	1.0
	• Minor damage	0.7
	• Medium damage	0.55
	• Severe damage	0.4
Seismic effect of members not considered in calculation (ν_5)	• Structure in which members other than main ones cannot be expected to have an seismic effect	1.0
	• Structure in which some seismic effect can be expected of members other than main ones	0.85
	• Structure in which members other than main ones can be expected to have an seismic effect	0.7

factor for shear forces that is larger than the one used for bending moment, as well as introducing other improvements in seismic resistance into the structural specifications such as stipulating a minimum tie reinforcement ratio of 0.2%. With these specifications to provide sufficient plastic deformation performance and toughness, the CSS ensure a structure against collapse. In other words, collapse due to a severe bending moment is generally interpreted as resulting from bending tensile fracture, so the safety factor against bending moment is only 1.15 ($=1.0 \times 1.15 \times 1.0$) in total, but the safety factor against shear force is 1.79 ($=1.3 \times 1.15 \times 1.2$) for concrete and 1.38 ($=1.0 \times 1.15 \times 1.2$) for the tie bars. The CSS specify that each safety factor be multiplied by 1.25 ($=1.5/1.2$) within a range from a point of

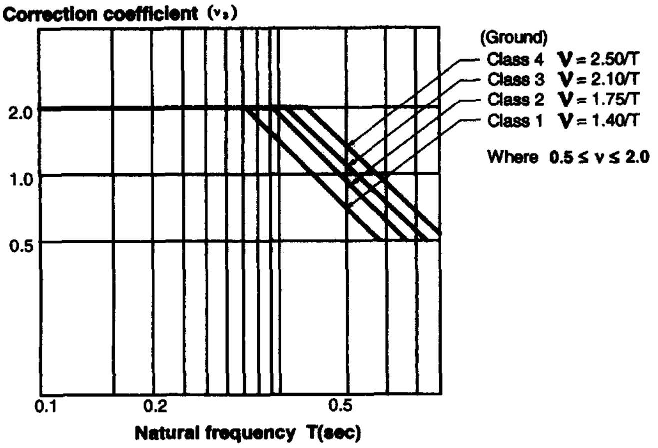

Fig. 1. Correction coefficient based on structure's natural frequency.

possible plastic hinge formation during an earthquake at structural member connections to a height equivalent to the column width. The CSS, thus, expect a structure to resist shear failure and to this end specify enhancement of the strength of columns such that they survive seismic motions greater than the design value.

Safety evaluation manual for seismic design of major outdoor civil engineered structures at nuclear power plants [2]

This manual published by the Nuclear Power Committee of the JSCE in 1992 is a typical work that systematically incorporates the latest improvements in this field. The manual employs a **limit state** design method in accordance with the JSCE's Concrete Standards Specifications.

The seismic design of nuclear power-related facilities is divided into four levels of importance as regards the potential for environment damage due to radiation leaks in an earthquake. Of the major external civil engineering structures, those related to emergency water intake belong to Class A_s. These must remain functional during an emergency and safety in this respect is evaluated by checking whether a structure has sufficient sectional strength to withstand bending and shear, as currently specified by the manual. It must also be ensured that the stress arising in piping and equipment under standard seismic motion S_1 should not exceed the elastic range and the critical measure used to check this is the yield strength of the reinforcement.

Standard seismic motion S_2 is the motion that occurs when a design limit earthquake, or an earthquake of the maximum magnitude conceivable given the geological structure, acts on the open base level. Standard seismic motion S_1 is that which occurs when a design maximum-strength earthquake, or an earthquake of greatest magnitude according to past earthquake records or behavior of active faults, acts upon the open base level. These motions are calculated for the location of each structure by taking into account seismic activity in the surrounding area. In determining these standard seismic motions, consideration must be given to the maximum amplitude, frequency characteristics and duration of seismic motion as well as changes in the amplitude envelope with time (Fig. 2).

A dynamic method of analysis that treats the ground and structure as a compound system is basically used to calculate the sectional forces imposed by seismic motion. The safety evaluation entails checking whether or not the sectional force or load stress exceeds the sectional strength or the material's allowable stress.

Design standards for railroad structures (concrete structure) [4]

These standards, established in 1992, are based on the limit state design method in compliance with the JSCE's Concrete Standard Specifications. The seismic design method stipulated in these standards features a calculation for determining the amount of tie reinforcement required to maintain the necessary strength as a

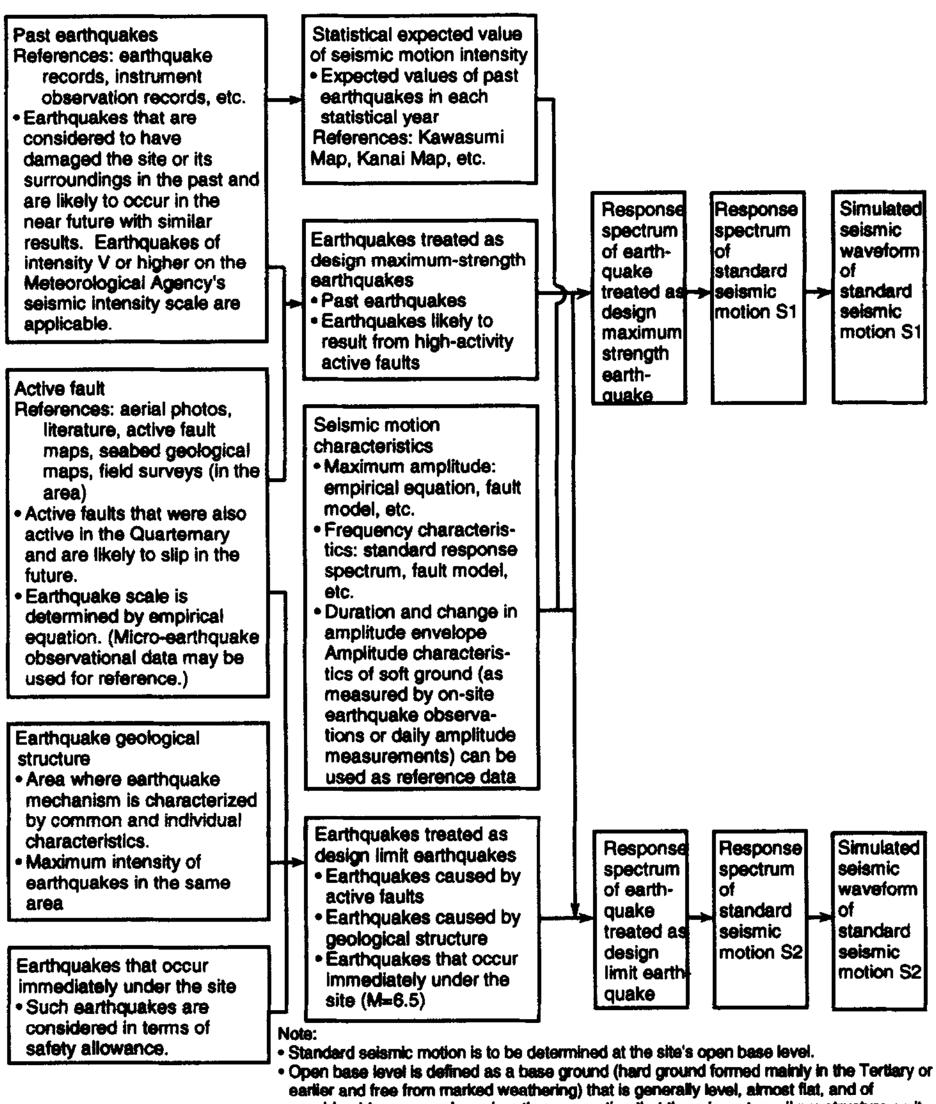

Fig. 2. Flow chart of standard seismic motion determination.[3]

function of a member's shear span ratio, tensile reinforcement ratio, axial compression strength and effective height. This directly incorporates a member's strength factor into seismic design.

These standards also considerably increase the design seismic factor and specify use of a standard design seismic factor of 1.0 for bridges and other structures, a value that provides for an elastic response. Accordingly, this may be described as seismic design system that takes rational account of the latest technological advances to cope with extreme seismic motions that occur only very seldom.

Seismic design section of specifications for road bridges [5]

In the 1990 specifications for road bridges, the bridge piers explicitly reflect recent seismic design principles by stating that the seismic design of bridges should ensure traffic safety in earthquakes. Further, they clarify that the aim is to construct bridges that maintain structural integrity when a relatively likely earthquake of medium intensity occurs and that do not collapse when a rare earthquake, such as the 1923 Great Kanto earthquake, occurs.

The standards recommend that, in addition to the conventional evaluation of allowable stress, a bridge's actual ultimate strength be checked against an equivalent lateral seismic factor, which is to be calculated from an elastic response lateral seismic factor of 1.0 taking into account plastic deformation of the bridge piers. To be more specific, if bending fracture is judged likely to precede shear fracture, the allowable plastic factor should be calculated using a safety factor of 1.5 under the assumption that the ultimate displacement is defined as the point when the strain in concrete under compression reaches the ultimate strain. Then the corresponding equivalent lateral seismic factor should be calculated so as to determine the actual force that will act on the section.

DESIGN SEISMIC FACTOR

It was in 1926 that the design seismic factor was first defined in modern Japan, when it was used to calculate the seismic load on bridges in the Proposed Specifications for Road Structures[6] published by the Civil Engineering Bureau of the Ministry of Home Affairs. The intention was to check the safety by calculating the stress on each part of the bridge based on a maximum seismic factor. This lateral seismic factor ranged from 0.10 to 0.30 depending on local conditions and the type of ground. The vertical seismic factor was established as half of the lateral seismic factor. This framework served as a guideline for many years, until the response characteristics of a structure came into consideration for bridges with piers higher than 25 m in the form of correction coefficient based on the structure's natural frequency. This was in the Seismic Design Guidelines for Road Bridges compiled in 1971. In 1980, this standard came into force for bridges with pier heights exceeding 15 m, but in 1990 the height limitation was terminated. Also in 1990 verification of safety in the event of a major earthquake was added to the specifications, using a response lateral seismic factor of 1.0.

The Standard Bridge Design established by the Construction Bureau of the Ministry of Railroads in 1930 was the first standard that provided a specific design seismic factor for railroad structures. It specified a lateral seismic factor of 0.2 against the dead load and earth pressure. In 1955, local conditions and ground effects were assimilated into the design seismic factor. Factors related to depth and the category of railroad sector were then added in 1970. The influence of the structure's response characteristics came into consideration in 1979. However, the design seismic factor method, in essence, did not change.

The 1983 revision to the National Railways Structure Design Standards specified structural requirements aimed at providing a member ductility factor not less than 4. This marked a quite remarkable improvement in minimum seismic resistance. The Railroad Structure Design Standards, as revised in 1992, introduced member ductility directly into the seismic design and specified a standard value of 1.0 when the design seismic factor equates to the elastic response. With these revisions, the seismic design system came to rationally reflect the latest technological improvements in safeguarding structures against major, but unlikely earthquakes.

The JSCE's CSS specified 0.2 as the lateral seismic factor for design, with half of this value as the vertical seismic factor, at the same time recommending that a structure be designed to limit the stress resulting from seismic forces to

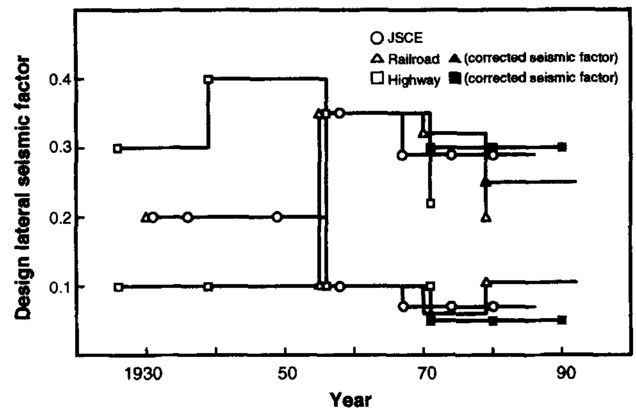

Fig. 3. Change in design lateral seismic factors (allowable stress method).

less than the allowable stress. A value of 1.5 times the ordinary allowable stress was specified for that purpose. This concept continues without changes for many years until recently, when slight variations in the design seismic factor came into use depending on differences in seismic behavior by area and ground conditions, the importance of the structure, etc. (Fig. 3)

SHEAR DESIGN

Allowable shear stress not requiring calculation of shear reinforcement

For stress not requiring calculations on the shear reinforcement, a value similar to those in other countries' standards was used up to around 1980. However, recent studies indicated that this value was too small for application to general civil engineering structures. It has recently been revealed that relative shear strength decreases with increasing sectional area and with decreasing reinforcement ratio (Fig. 4). In 1980, the CSS adopted a value significantly smaller than the conventional allowable stress as a tentative revision, followed by the introduction of an equation that quantitatively reflected these factors in 1986 (Fig. 5). The allowable values shown in Fig. 5 are those used for seismic design and are 1.5 times the ordinary allowable stress. Values given in versions of the specifications later than 1986 are design values for ordinary shear strength when the effective height of member is 3 m and the tensile reinforcement ratio is 0.5%. Values 1.5 times greater are recommended for the region between member connections, which are crucial

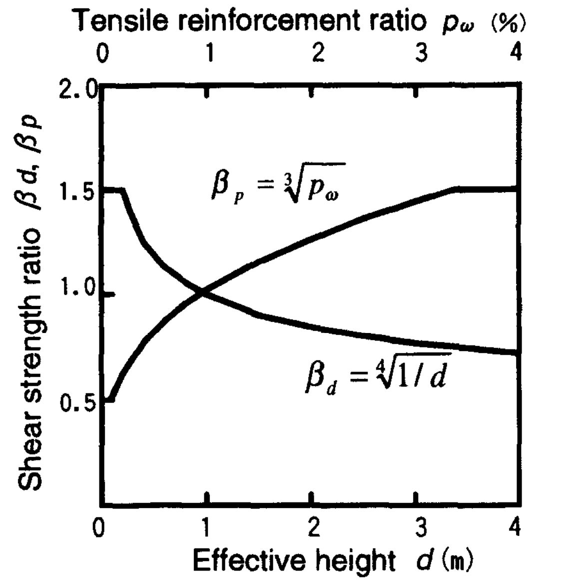

Fig. 4. Effect of dimensions and tensile reinforcement ratio on shear strength.

to seismic resistance and a height equivalent to the column width. Standards for bridges have undergone similar changes to reach the values in use today.

The concrete structures damaged by the Hyogoken–Nanbu earthquake were designed with extremely high values of allowable shear stress in light of today's knowledge. For the majority of such structures, sufficient shear reinforcing bars to meet the minimum quantity required by the structural specifications were used. On the other hand, the design requirements relating to bending moment have remained basically unchanged in principle up to today.

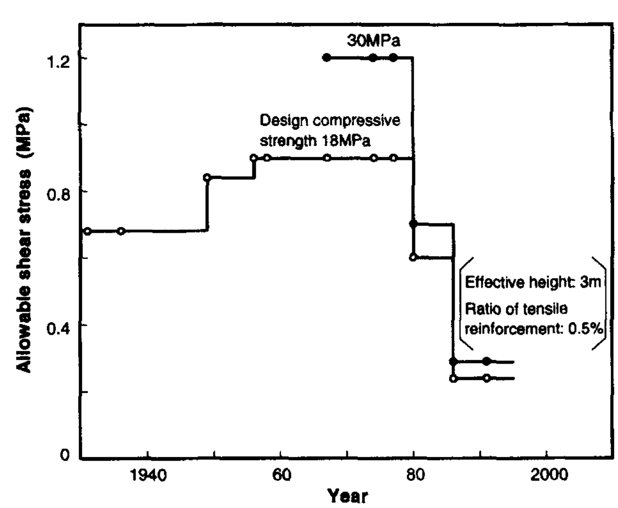

Fig. 5. Change in allowable shear stress as specified by the JSCE's CSS.

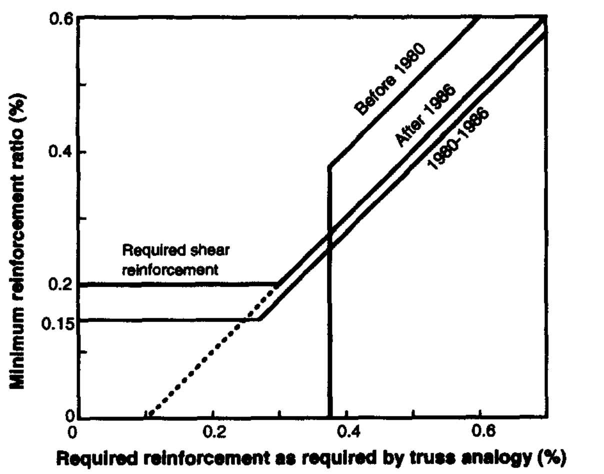

Fig. 6. Changes in shear reinforcement as specified by CSS.

Calculation of shear reinforcement

The design of reinforced concrete to resist shear forces was based on the truss analogy for many years. This analogy says that the truss formed carries all the generated shear force when diagonal cracking occurs. According to this analogy, the amount of shear reinforcing bars required suddenly increases, if the shear force exceeds the allowable shear stress of concrete (Fig. 6). Designers, therefore, made it a rule to increase the sectional area to prevent the stress exceeding the allowable shear stress when high shear is expected and is likely to exceed the allowable value. The figure shows an example for when the concrete's design compressive strength of 18 MPa. More severe lines would have to be drawn for larger strength.

When the section is increased to hold the shear stress below the allowable shear stress, the sectional area will be greater than that of nearby members in which the shear stress is sufficient. This condition, based on today's knowledge, typifies the case where shear failure leads to collapse. Such members have poor ductility and, when subject to seismic forces greater than the design seismic factor, succumb to shear failure and eventually collapse. Concrete structures with relatively large sections compared with the pier height are more likely to suffer from this type of design.

When the allowable shear stress was reduced in the CSS in 1980, a further modification specifying that 1/2 of the allowable stress should be added to the truss was introduced. In its 1986 revision, all the shear stress carried without shear reinforcement was to be added to the truss. The same was applied to road and railroad structures. This resolved the lack of continuity in the relationship between the required reinforcement and the shear force acting.

STRUCTURAL DETAILS

Minimum tie bar

When the shear stress expected to be experienced is smaller than the allowable shear stress, the minimum amount of tie bars is generally used. Regarding the amount of tie bars to be used in a column, the 1931 version of CSS specified that tie bars at least 6 mm or larger in diameter should be used and the spacing should be less than the minimum width of the column and less than 12 times the diameter of the axial bars. In the 1956 version, a requirement was added that the spacing could be up to 48 times the diameter of the axial bars. Since then, no changes have been made until recently. These requirements are intended for columns of comparatively small dimensions under axial compression.

In 1980, when a significant revision on allowable shear stress was made, it was specified that stirrups should cover 0.15% or more of the beam. This requirement has been applied also for columns. In 1986, the maximum spacing of tie bars in the region between member connections and a height equivalent to the column width was specified as one fourth of the minimum member dimension, while the ratio of tie bars in columns critical for seismic resistance was to be 0.2% or more. Seismic structural specifications for road bridges changed in basically the same way. Figure 7 shows how these rules are applied to a bridge pier with a square section, 1 m on each side; the diameter of the axial bars is 32 mm. The minimum requirement for tie bars in bridge piers damaged by the Hyogoken–Nanbu earthquake was equal to the requirement in members under axial compression, which is considerably smaller than the current value ascertained by seismic considerations.

The standards for railroad structures have been basically the same as those in CSS. In 1978, an administrative notification altered the minimum required amount of ties for columns

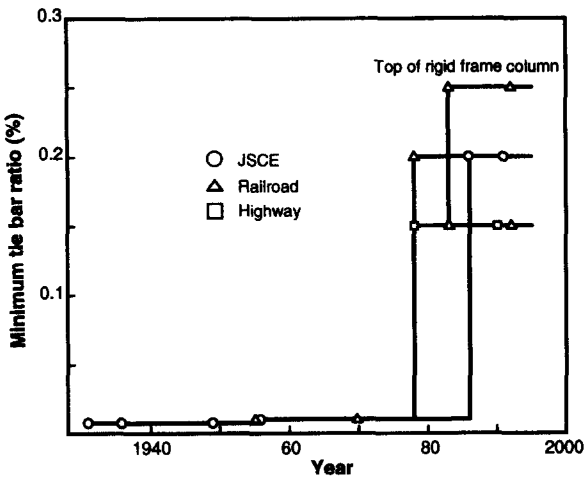

Fig. 7. Change in minimum ratio of tie bars (square section with each side 1 m and diameter of axial bars 32 mm).

to 0.2% and demanded 0.25% or more to be placed in the region between member connections and twice the effective height. In 1983, a new structural specification was added with the aim of achieving a member ductility factor of four or more. The placement of tie bars covering 0.15% or more of the sectional area of concrete was specified, while the region between member connections and a height equivalent to twice the column width was specified to receive tie bars at spacing of 10 cm or less covering 0.2% or more of the concrete sectional area and in quantities 1.2 times the calculated requirement. Between member connections at the top of a column and a point equivalent to twice the column width, tie bars should cover 0.25% or more of the concrete sectional area. In 1992, a calculation to determine the ratio of tie bars needed to maintain a sufficient ductility factor was added, nullifying the requirements for a minimum tie bar ratio, though these remain in the structural specifications.

Anchorage of tensile bars

The CSS had, since 1931, retained the principle that there should be no anchorage of tensile bars in the tensile stress region. However, anchorage in the tensile region was accepted in 1956, citing retaining walls as an example. In 1967, fearing that some cracking may lead to diagonal cracking, the CSS indicated that the anchorage length should be extended beyond the length calculated. However, no specific value was indicated for this anchorage extension. This allowed bridges to be constructed with underestimated extension in their bridge piers. The negative results of this were abundantly clear in the aftermath of the devastating Hyogoken–Nanbu earthquake.

The 1980 revision to the CSS provided specific values for this anchorage extension. The specifications called for an extension equal to the effective height of the member beyond the point indicated by calculation, and then an additional extension to the required anchorage length beyond that point. The 1980 revision also ensured that sufficient shear resistance is available at the point of anchorage, although no specific value was suggested. The 1986 version specified that tensile bars should not be anchored unless the shear strength be larger than 1.5 times the design shear force. The 1991 revision added situations where bending strength is more than double and shear strength more than 4/3 times, thereby completing the requirements in this category. Bridges are currently based on almost the same requirements.

Others

Conventional requirements call for tie bars to be placed around the axial bars and either anchored with hooks into the concrete within the columns or made as continuous spiral bars. The hooks should, in principle, be 135° bends at the ends.

Bar joint positions are specified, but instructions only cover avoiding sections under high stress where possible; no specific values were indicated. This offered a loophole for actual construction, and the consequences of such noncompliance are seen in many structures damaged at member connections after the Hyogoken–Nanbu earthquake. Bar connections should be avoided at points where forces are expected to concentrate.

CONCLUSION

The basic principle of seismic design prior to 1980 was to determine the expected stress with a design seismic factor using an elastic calculation, and to make sure it was less than the allowable stress. The standard lateral seismic factor used in design was 0.2, which, in actual usage, may then have been corrected by roughly

0.1–0.3 depending on the location, ground type, structure importance and the response characteristics of the structure. There is, thus, no quantitative guarantee about the possible effects of earthquakes causing greater loading than this. It may be, therefore, concluded that some surviving structures were in fact designed empirically with sufficient seismic resistance despite the specifications. During the period of Japan's rapid economic growth, a great number of structures were built in a short time, making it impossible for experienced engineers to prudently apply seismic design considerations to ordinary structures. This consequently increased the importance of the design standards themselves.

Under the pre-1980 specifications, the allowable shear stress for concrete was significantly greater than what is acceptable today. The shear stress for structures that should fail in the bending mode, as calculated by design seismic factor, was set below this earlier value of allowable shear stress and according to the latest methods might be expected to suffer shear failure. Tie bars were not specified with the aim of improving seismic resistance, so the required minimum was extremely low. Structures susceptible to shear failure not only suffer from low strength, but also lack enough bearing capacity to hold up the superstructure at the time of failure.

Before 1980, the requirements for intermediate anchorages for tensile bars were inadequate. The consequences of this became clear as many structures damaged by the Hyogoken–Nanbu earthquake suffered serious damage at anchorages of tensile bars at the intermediate height of piers. A lack of quantitative rules for the location of bar joints resulted in many cases of such damage and many bar fractures were witnessed at connections.

Many concrete bridge piers were subject to forces that exceeded the yield point of the reinforcement when the earthquake struck and many bridges collapsed as a result of shear failure of piers. The major reason for such collapses was an inappropriate seismic design method at the time of construction. Those which did not suffer shear failure generally survived collapse even if badly damaged as a result of the bending moment. This bears out that understanding that concrete structures do not collapse if they are sufficiently protected against shear failure, even if they are subject to a displacement considerably larger than the bending yield displacement. This may as well justify the appropriateness of the latest seismic design method for concrete structures, which assumes collapse in the event of an extreme seismic event can be prevented by ensuring sufficient ability to withstand against shear failure.

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