

Earthquake-Resistant Design for Civil Engineering Structures
Earth-Structures and Foundations in Japan

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Chapter 1. Preface

Unfortunately situated along the circumpacific earthquake zone, Japan has earned fames also for haunting earthquakes of grand scales. The Japanese engineers have been compelled, therefore, to establish a practice to take into account earthquake effects as one of the major external forces to be considered in the design of almost any type of structure. The practice demands various careful treatments especially for such public utilities as earth-structures, foundation works, bridges, dams, harbor facilities and water supply systems; each treatment embodying time-honored experiences plus results of scientific researches of Japanese engineers.

As a token to commemorate the 2nd world conference on earthquake engineering, the following six authorities, namely, the Japanese Society of Soil mechanics and Foundation Engineering, the Bridge and Structural Committee, Japan Society of Civil Engineers, Japanese National Committee on Large Dams, Port and Harbor Bureau, Ministry of Transportation, Japan Waterworks and Sewerage Association and Earthquake Engineering Research Committee, Japan Society of Civil Engineers compiled histories of earthquake damages, current aseismic design procedures, important results of research activities and other related data in their fields.

Though every one of the presented data is deemed a contribution of great interest to enriching the aseismic engineering at large, it is not possible to present them at the conference in their entirety in the limited space assigned to each author. The writer will therefore confine himself to trying only a general review of these data. For further details, readers are referred to the recent publication of the Japan Society of Civil Engineers, titled "Earthquake Resistant Design for Civil Engineering Structures, Earth Structures and Foundations in Japan".

Chapter 2. Earth structures and foundations

§1. Current methods of earthquake resistant design

1) Design seismic-coefficient

For the purpose of earthquake-resistant design of foundation for building and civil engineering structures, including earth structures, the so-called seismic-coefficient method is commonly used at present. In the seismic-coefficient method the force equal to the weight of a mass multiplied

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by the specified value of seismic-coefficient is assumed to act statically at the center of gravity of the mass, and the stability of a structure is checked under the above loading condition. In general, the action of the seismic force is considered in horizontal direction alone, whereas in some cases the vertical force is also taken into account.

Values of design seismic-coefficient specified in different fields of construction engineering will be shown in the followings.

a) Building foundation

Earthquake force is assumed to act in horizontal direction alone and the value of design seismic-coefficient, k , is determined within the specified range of 0.096 - 0.300 depending upon the ground condition and the type of the construction of the building.

b) Bridge

See Chapter 6.

c) Dam

See Chapter 4.

d) Port and harbor structure

See Chapter 5.

e) Embankment

For earth structures such as river dike, coastal levee or highway embankment, no effect of earthquake action is taken into account. The reasons for this treatment are that (1) river dikes and highway embankments may be easily repaired even when they are damaged and (2) furthermore, for river dike which is to be designed with due consideration of flood, it may be permitted to assume that flood and earthquake seldom occur simultaneously.

2) Earth pressure

To compute earth pressure upon a retaining wall under the action of earthquake force, Coulomb's earth pressure formula is used in general, with the assumption that the back-fill is acted upon by horizontal acceleration $k \times g$ (g : acceleration of gravity) or both by horizontal and vertical accelerations. However, when conditions are favorable, the increments of earth pressure due to earthquake are sometimes disregarded.

3) Soil reaction

Soil reaction acting on the base of footing under the influence of earthquake forces is usually assumed to be distributed in a trapezoidal or triangular form.

4) Bearing capacity of soil

For building foundations it is permitted during earthquake to double the allowable bearing value of the subsoil or of the pile for normal case. This concept is, however, not always supported in other fields of construction and the ways of estimating bearing capacity under the action of earthquake are quite different.

5) Resistance against sliding

The resistance against the sliding of a structure is also examined when the structure is subjected to earthquake forces.

6) Earth structures

In design computation of earth dam, it is assumed that the earthquake force acts in horizontal direction and the factor of safety is calculated by taking the ratio of total shearing resistance to developed shearing force along the most unfavorable circular surface of sliding. Furthermore the pore water pressure is also taken into account in the above computation.

The angle of slope of the downstream side of a rock-fill dam for slag-dump of an ore mine is determined so that the following condition is satisfied:

$$i < \phi - \tan^{-1} k$$

wherein

i : angle of slope

ϕ : angle of internal friction of rock-fill material (assumed to be equal to angle of repose)

k : design seismic-coefficient

7) Factor of safety

In some fields the factor of safety for a structure is reduced during an earthquake in a similar way to above stated building foundations. This idea is based on the momentary character of earthquake and the economy of construction. Since, however, the effect of earthquake force has not been fully revealed as yet, the reduction of the factor of safety during an earthquake is often disfavored.

For an earth structure such as earth dam, the factor of safety may be equal to unity during an earthquake, which is determined as mentioned before as the ratio of total shearing resistance to shearing force developed along the most unfavorable circular surface of sliding. In general, thus designed earth structure provides the factor of safety equal to 1.5 to 1.7 in normal cases.

The factor of safety during an earthquake as to sliding of gravity-type quaywall may be reduces to unity, while for some railway structures such as retaining walls the factor of safety for earthquake force is often considered as same as in normal case.

§2. Problems in earthquake resistant design

Because of the facts that the effects of earthquake, especially on earth structures and foundations, are not precisely known as yet and that the designing is inevitably influenced by economy, it may be said that the general policy for earthquake-resistant design in Japan is likely to be underlain by a philosophy of resignation.

The reduction of the factor of safety during an earthquake as stated in the previous paragraph may not necessarily be justified, if a great earthquake loosens the ground or causes an abrupt increase of pore water pressure and a loss of strength of subsoils. However, as to the behavior of the subsoil beneath a structure or the change of pore water pressure due to earthquake, many things remain still unknown.

On the other hand, the full-scale and model tests under static loadings carried out in the National Railways show that some assumptions in conventional theories of stability computations are not adequate. For instance, it has been an accustomed practice to check the stability of a shallow footing for overturning and sliding separately; however, it is shown that the resistance against sliding changes with the magnitude of overturning moment and, consequently, the conventional practice to assume a constant resistance against sliding is not correct. Both effects of overturning and sliding are thus closely related to each other and should not be dealt with separately.

The usual assumption, that soil reaction upon the wall surface of a well or a caisson acts purely in horizontal direction, distributions in a parabolic form and is not allowed to exceed the value of Rankine's passive pressure, is far from the actually observed result. In reality, the reaction at the wall surface is not horizontal but involves a vertical component due to friction and adhesion. Even when the magnitude of this reaction exceeds far more than Rankine's passive pressure, the soil does not perfectly reach the state of yielding.

Fortunately, such inadequacies in the theory of stability computations are gradually being removed and corrected to some extent by analysing results of static tests; nevertheless, as for deformation characteristics and strength of subsoils under the action of dynamic forces such as earthquakes, it could not be overemphasized that many unsolved, important problems should be studied in the near future.

While various attempts have been made with respect to vibration tests of model foundations, satisfactory conclusions have not yet been obtained.

It is expected that many uncertainties would be made clear if full-scale foundations were subjected to strong, artificial vibration and their behaviors were carefully observed. Unfortunately, however, it is quite difficult in the present state of techniques to generate and control at one's disposal such a strong vibrational motion that would cause damages to the specimen. Consequently, in addition to the development of basic theories, the establishment of empirical earthquake-resistant design method based on analyses of observed earthquake damages is most important under the existing circumstances. In order to promote such an empirical studies the Japanese Society of Soil Mechanics and Foundation Engineering should like to make proposal to World Conference on Earthquake Engineering to establish an international standards of questionnaire on earthquake damages. It will

be a great help to have unified and comprehensive information on earthquake damages at every part of the world.

Chapter 3. Water supply works

§1. Introduction

Aseismic structures for the water supply works in our country are designed in accordance with the standards determined by the Japan Waterworks and Sewerage Association in 1953. These standards specify the general principles of aseismic structures and the detailed treatises of special structures for water supply works. Some notes about the latter will be given here.

§2. Detailed treatise

A. Water source installations

(1) Careful precaution shall be exercised so that the outlets and out-flow channels, either of open-channel or of tube type, may not be clogged by surface soils and rock debris originating from adjoining hills at the time of earthquake.

(2) Intake tower

(a) The most desirable type of an intake tower is a reinforced concrete structure resembling a cylindrical appearance.

(b) The intake tower shall be given a reasonably deep embedment into a strong foundation. When a sunk-well technique is applied, the embedment depth shall be sufficiently large and the bottom lined with concrete to a considerable thickness.

(c) The conduits leading away from the intake tower shall be, if possible, iron pipes embedded deep below the water bottom.

(d) It is desirable to make a foot bridge approaching the intake tower as light as possible so that the tower may remain free from any extra load.

(e) It shall be avoided to locate the intake tower at such places where there is a possibility of burial due to collapse of river embankment or hill slopes, or of scouring due to currents.

(f) In computing the stability of an intake tower, a particular emphasis shall be placed on the buoyancy so that the safety of the structure may be guaranteed against earthquake shocks when the interior is empty.

(3) For such intakes of gate and pipe type the same sort of care shall be exercised as for the case of a reservoir.

(4) The intake pipe crossing an embankment shall be made of iron, laid upon a strong solid foundation, well protected with concrete, and re-filled with special care.

(5) As an infiltration gallery, a perforated reinforced concrete pipe equipped with sockets of sufficient strength and considerable depth shall be laid on a homogeneous foundation: the spigot shall be inserted sufficiently deep into the socket and its periphery protected with strong wooden frames or other devices.

(6) It is desirable that the attached manholes and junction wells shall be placed on an especially strong foundation, and the joints connecting 2 or 3 succeeding pipes immediately adjoining them on both sides shall be dressed in comparatively long thimbles.

(7) The portions of an infiltration gallery other than the infiltration pipes, including the part crossing an embankment, shall be made of high-qu-

ality pipes without openings and the joints shall be completely water tight.

(8) The side walls of a shallow well shall be, as a rule, of a cylindrical form and built of reinforced concrete.

(9) A deep well casing shall be equal or superior to the JIS-G3427-type gas pipe; the joints and strainers shall also possess sufficient strength.

(10) Deep well casings shall project beyond the floor slab of the pump chamber.

(11) It is not advisable to shape the upper portion of a deep well casing similar to that of a shallow well in order to use it also as a pump well.

B. Water conveyance installations

(1) In principle, an open channel and a culvert shall be built of reinforced concrete with the combination of the bottom and side walls behaving as a monolithic unit.

(2) It shall be avoided to place them on or inside the refilled soil. When otherwise impossible, the techniques applied to purification installations shall be employed.

(3) The expansion joints shall be installed at approximately 10 to 20 m intervals; they shall also be used at such places where different geological formations meet, and before and behind a bridge, weir, manhole and gate, etc.

(4) The route of a tunnel shall avoid such places as a hill slope which exerts a biased load.

(5) Since the entrance and exit of a tunnel are liable to collapse, they shall be protected adequately.

(6) At such places where there is a fault line running in the vertical direction or where sharply differing geological formations meet, the tunnel lining shall be insulated and the lining before and behind this position particularly strengthened.

(7) Since an aqueduct structure is top-heavy, the design shall be so arranged as to prevent it from falling off the bearing plate due to a horizontal thrust of an earthquake. For such purposes it is advisable to extend arms from bridge piers or bearing posts, and connect the bridge piers or bearing posts to the aqueduct beams with diagonal members so that both tensile and compressive stresses may be transferred directly to the piers, bearing posts or beams.

(8) The piers or bearing posts of an aqueduct bridge shall be equipped with appropriate transversal straddling post to prevent overturning due to earthquake.

(9) The post of the aqueduct bridge pier and the foundation shall be connected adequately.

(10) The abutment (of an aqueduct bridge), which is generally vulnerable to earthquake shocks, shall rest on a firm solid foundation with a large embedment depth.

(11) Since the connection of the conduit and the aqueduct constitutes one of the most vulnerable points, special care shall be exercised to make it a perfect earthquake-proof structure, or otherwise an efficient expansion joint shall be installed at this position. In cases where an intensive earthquake shock is expected, a further safety shall be guaranteed by installing two joints at a relatively close interval.

C. Installations for purification and distribution

(1) A structure of this category shall not be placed upon a refilled earth or near a slope toe which is apt to collapse. If otherwise impossible, it shall rest on a pile foundation or reinforced concrete columns, either supported on the original ground, and the bearing capacity of the refilled earth shall be disregarded in the process of computation.

(2) For the sloping walls of a sedimentation basin which are inevitably built on an artificial fill, the soil shall be adequately compacted and the construction shall start only after an adequate amount of settlement and hardening of the fill has taken place. In the regions where the value of seismic coefficient exceeds 0.3, the structure shall on no account be placed on an artificial fill.

(3) The tops of the foundation piles shall penetrate into the structure as deeply as possible (30 to 50 cm, if possible).

(4) It is not advisable to construct a unit structure astride a composite foundation which consists of layers with different bearing capacities. If otherwise unavoidable, it shall be divided into separate units in proportion to the capacities of the supporting layers.

(5) If it is necessary to place a unit structures astride a composite foundation, the layer of lower capacity shall be improved by pile driving or other treatments to equalize the bearing capacities of the entire layers before erecting the structure, and more-over, expansion joints shall be installed at the boundaries of the neighboring structures.

(6) Any type of structure having simpler appearance both in plan and profile and devoid of abrupt changes in configuration is advantageous in resisting earthquake shocks. Accordingly a large haunch shall be placed at the rigid joints.

(7) Particular care shall be exercised in designing the connecting points of different structures since they are vulnerable to earthquake shocks.

(8) For better resistance against earthquake shocks a circular plan shape is generally preferable as a basin-well structure to an angular one.

(9) The floor and side wall of the basin-well shall be built of reinforced concrete in accordance with the earthquake-proof design and rest on a solid foundation so that their combination may act as monolithic unit.

(10) The side wall and floor of the basin-well shall be made water-tight by applying high-quality concrete elaborately. In case a water-proof mortar coating is applied to a purified water storage basin or a basin of a large depth, it shall be reinforced with thin bars arranged densely both in longitudinal and latitudinal directions. The same precaution will also improve the safety of the protection concrete or mortar coating for a water-proof layer of asphalt or other materials.

(11) On the side walls of the basin well the expansion joints shall be placed near the corners. At other portions the expansion joints shall be inserted at intervals of 6 to 9 m for a thin wall and 15 to 18 m for a thick wall (10 m ordinarily and 15 m in maximum). The gap of the joint shall be ordinarily 1 to 3 cm.

(12) A canopied basin shall be designed in such a manner as to allow safe entrance and exit even at the time of earthquake, and hence every arrangement shall be provided to satisfy this condition.

(13) Since the doorways and inspection windows are vulnerable to local damages due to earthquake they shall be reinforced with special care.

(14) The foundation of a water tower shall rest on a ground of particular reliability. It is desirable even on a favorable ground to secure as great an embedment as possible.

(15) A cylindrical structure of a water tower is advantageous to safety against earthquake.

(16) The riser portion of an intake pipe which is housed inside the water tower shall be supported along the wall surface. It shall be born, if available, by the columns in the center which support a canopy for a large tower. It is advisable to connect the columns with horizontal diagonals.

(17) If further the inside face of the water tower wall is made watertight by lining with steel plates more than 3 mm thick, a fear of leakage will be much reduced assuring extreme safety against earthquake shocks.

(18) The material for an elevated tank shall be steel in preference to reinforced concrete in view of safety against earthquake shocks.

(19) The riser portion of various types of attached pipes to an elevated tank shall be equipped with earthquake-proof joints near the ground surface.

(20) The elevated tank shall be provided with as many supporting columns as possible, with horizontal and diagonal members adequately arranged to prevent buckling.

(21) The tank shall be tied fast to the supporting frames.

(22) A reclining type is recommendable as safe for a pressure tank.

D. Pipeline and appurtenances

(1) The most preferable earthquake-proof method is to select a reliable ground for the site of a pipeline route.

(2) The route shall avoid an abrupt bend either horizontal or vertical. When an abrupt bend is included due to unavoidable circumstance, it is necessary for safety to place a supporting platform equipped with earthquake-proof joints before and behind it.

(3) It shall be avoided to lay a pipeline on refilled earth. If otherwise unavoidable, the technique specified for purification installations shall be employed.

(4) The embedment-depth of pipeline shall be arranged in such a manner as to allow convenient maintenance as well as easy repair.

(5) A solid uniform ground is desirable along entire route. A soft ground shall be adequately reinforced by pile driving, ladder struts or concrete foundation even for a small pipeline.

(6) At such places where soft and firm grounds join, an earthquake-proof joint shall be installed near the latter.

(7) At such places along the trunk route where the specified design coefficient is more than 0.3, it is safe to install expansion joints at the rate of one for every 3 joints.

(8) It shall be avoided to lay the trunk route of a pipeline on the reclaimed area, vicinity of ditches, river banks, beaches, cliffs or bulkheads.

(9) At such places where the route crosses over a sewage pipe or other establishments, a gap shall be maintained between them.

(10) The distribution pipes shall not have a dead end; the pipe terminals shall be connected to each other.

(11) The distribution pipelines shall all be equipped with gate valves

installed in such a manner that in event of damages inflicted on any part of the system the suspension of water supply operation could be confined to as small a region as possible.

(12) Even such types of pipe as a delivery pipe and a trunk distribution main pipe which do not possess branch pipes shall be equipped with sluice valves at an interval of approximately 1000m.

(13) It shall be avoided to lay the specials continuously. A cut or straight pipe shall be inserted between 2 adjoining specials.

(14) Near the position of a tee or cross pipe along a key route it is desirable to install earthquake-proof joints at the two pipelines meeting at right angle.

(15) When inserting along the pipeline such appurtenances as a fire hydrant or sluice valve, etc., which present different characteristics of vibration due to earthquake from that of the pipe, it is desirable to install earthquake-proof joints before and behind it.

(16) The bends, dead ends or tee pipes shall be protected by placing a concrete platform in direct contact to the outer surface of them or driving reinforced concrete piles. It shall be avoided, however, to install the bends inside a concrete block.

(17) Various types of pipes which penetrate through the surrounding wall or bottom slab shall not touch the wall body directly. When it is necessary to install them in direct contact to the wall in order to ensure water-tightness, an earthquake-proof joint shall be installed apart from but near the wall. The same procedure is recommendable between a pump and a suction or pressure-conveying pipe.

(18) A suction pipe which is suspended down a pump well shall be fixed to the wall with strong metal fittings to prevent it from vibration in a different manner from the wall.

(19) A pipeline running on the ground surface for a great length shall be equipped with a fixing platform at an interval of approximately 300 m to fix the pipeline to the subsoil (namely, fixing with iron bars to a dented platform), together with expansion joints set between them.

(20) In case a high-quality cast-iron pipe is used and unless an effective earthquake-proof joint is employed a lead socket joint shall be used as a rule.

(21) Special care shall be exercised in treating the upper and lower portions of the perimeter of a lead connection.

(22) When using a steel pipe, a flange or electric-welded joint beside a socket lead joint is recommendable; and the former two types of joint shall have an expansion joint inserted at an interval approximately 30 to 60 m.

(23) A siphon crossing shall not be built on unfavorable ground conditions.

(24) A flexible joint durable to water pressure shall be used for a siphon crossing.

(25) The approach pipes before and behind a siphon crossing shall have as gentle a curvature as possible, and any curve shall be fixed sufficiently with a concrete platform.

(26) Sluice valves shall be palced at both ends of a siphon crossing.

(27) A siphon crossing of a large-calibered pipe or of a key trunk line may be divided into 2 or more branches for safety.

(28) Unless otherwise unavoidable, a pipeline shall not be laid on a wooden bridge.

(29) A pipe bridge shall be located away from an existing wooden bridge by at least more than 2 m upstreamside.

(30) A pipe bridge shall be built of fire-proof material, and on a soft foundation a steel plate-girder or a steel frame girder is recommended for safety.

(31) It is advantageous against earthquake shocks to build the superstructure as continuous system on relatively favorable ground.

(32) The superstructure shall be adequately anchored on the substructure with anchor-bolts in order to prevent it from falling off the position at the time of earthquake.

(33) It is safe to use a steel pipe for a bend pipe or an S-pipe which connects to a bridge-born pipe and an underground straight pipe behind the abutment. It is advisable to treat the joints of the steel pipe by electric welding.

(34) A bridge-born pipe shall be fixed to the superstructure at each span, and also equipped with an expansion joint at each span.

(35) For a bridge-born pipe made of cast-iron a flange joint shall be avoided.

(36) At the position where connection is made to the bend pipe on either end of a bridge-born pipe an expansion joint shall be placed without exception.

(37) A pipe mounting to the abutment shall be sloped by less than 45° and connected tightly to a straight pipe before or behind the bridge.

(38) The both ends of a bridge-born pipe shall not be fixed to the abutments, but allow for a considerable gap around the pipe.

(39) Special care shall be exercised to refilling the banking behind the abutment, and a foundation supporting a bend pipe shall be particularly reinforced by pile driving.

(40) A pipe bridge having the steel pipe itself as the main girder shall be a simple beam structure especially on a soft foundation and equipped with flexible joints on the abutments and piers.

(41) Gate valves shall be installed at the pipeline before and behind bridge.

(42) A gate valve shall not be placed close to a building or any other establishment in order to ensure convenient and rapid replacing and repairing.

(43) It is desirable to insert an expansion joint before and behind a gate valve.

(44) Sluice valve chamber or fire hydrant chamber shall be built of reinforced concrete or, preferably blocks of concrete or brick, so that damage repair may be convenient.

(45) It shall be avoided to use a gas pipe as a service pipe, since its screws are vulnerable to an earthquake shock.

(46) When a steel or copper pipe is used as a service pipe, a lead pipe approximately 500 mm long shall be inserted before and behind the connection to a ferrule, stop valve or hydrometer. The connection to a large tap post such as an anti-freeze common tap shall also comply with the same provision.

E. Power and pump

(1) An extra power unit of, if possible, different type shall be maintained for emergency use.

(2) When only the electric power is available, an extra unit with different source and supply route shall be provided for emergency use.

(3) The delivery end of a pump shall be directed horizontally or downward.

Chapter 4. Dams

§1. Damages to dams in past earthquakes

In Japan, from ancient days agriculture developed primarily with the cultivation of rice paddy fields. To assure supplies of water for irrigation purpose, many earth dams have been built and among the existing dams, the oldest date back to over 1,000 years. These dams were constructed based on experiences during those days. Most of the dams, approximately 50 meters in height, including other purpose dams were constructed in the past thirty years. Therefore, at the time of the 1923 Kanto Earthquake the number of high dams in existence was extremely small. Since then, Japan has been affected by several severe earthquakes. Investigations of damages to dams caused by these earthquake were important factors in the researches on earthquake resistant features in the design of dams.

In this investigation the method followed was to circulate questionnaire forms to administrators of dams. However, it was technically difficult to investigate all phases of damages. Therefore, investigations were made of the 1923 Kanto Earthquake and the 1944 Nankai Earthquake which caused comparatively heavy damages to dams. In both earthquakes, as a principle, investigations of damages were made on high dams selected in areas where the intensity exceeded scale V(80-250 gals) established by the Central Meteorological Observatory.

(1) Damages to dams in the Kanto Earthquake

A description of the dams that were damaged by the Kanto Earthquake are given below:

(A) Ōno dam

The Ōno Dam is located in Ōno, Uenohara Town, Kita-Tsurugun in Yamanashi Prefecture. The dam constructed on the Yata River, a tributary of the Katsura River, in the Sagami River System is an earth dam with a center impervious core. The height of the dam is 37.3 m above foundation and 49.1 m from the base of the cut-off wall. The center core is concrete and soil concrete. The dam body is red earth which was placed in lifts and compacted. The dam was completed in 1914 and nine years later it was hit by the Kanto Earthquake.

The dam axis runs generally in the direction of N 21° E. The left abutment curves towards the upstream. The geology of the right bank is palaeozoic strata and left bank is alluvium. The major part of the dam rests on bedrock.

From observations of toppled stone monuments in the locality of the dam, it was estimated that the earthquake intensity was about 330 gals or Intensity Scale VI. Settlement of about 3 cm took place in the center of the dam crest and about 25 cm at the curved section of the left abutment. A crack almost 20 cm wide, 6 m deep and 40 m long appeared on the crest near the left bank. On the upstream slope several cracks appeared running parallel with the dam axis, but the cracks were in the riprap and did not extend into the dam body. Several cracks appeared also on the down stream slope within the distance of about 10 m from the crest. The crack nearest to the crest was about 11 m deep and 30 m long was observed and numerous small cracks in the vicinity of the drain duct in the berm. Swelling on the slope about 18 m below the crest was noticed.

(B) Murayama-kami Dam

Murayama-Kami Dam is located in Imokubo, the Town of Yamato, Kitatama-gun in Tokyo. This is an earth dam 24 m high with a center core. The direction of the dam axis is N 10° E. This dam was completed in June 1923, just immediately before the Kanto Earthquake. The geology of the dam site is loam formation of the Tertiary Period. The core puddle is a blend of clay and sand mixed to a ratio of 2:1. The material was placed in layers of about 9 cm in thickness and compacted with rollers to 6 cm. In the other sections of the dam, earth was placed in layers of about 15 cm in thickness and compacted to about 9 cm. In the upstream slope of the dam, a layer of gravel 30 cm in thickness was laid on which concrete blocks 30 to 45 cm thick and 1.8 m square were placed. The void between the concrete blocks were filled with clay.

The earthquake intensity in this district was estimated to be in the scale of VI. The reservoir pool was 7.58 m below full water level and the effective drawdown was 5.94 m. Damages by the earthquake were movement of the concrete block facing by approximately 6 cm, destruction of a drain duct and parapet wall erected on the dam crest. Settlement of about 18 cm was observed near the center on the crest. On the downstream slope, settlement of about 1.2 m was observed near the center and a crack about 12 cm wide and 110 m long appeared in the downstream slope protection. Settlement of about 1.2 m was observed around the center of the drain ducts in the berm and in the downstream foot of the dam. Generally speaking, deflection was observed on the horizontal plane. However, cracking did not happen in the slope of the dam body.

(C) Murayama-shimo Dam

Murayama-shimo Dam is located near Murayama-kami Dam. The construction method applied to Murayama-kami Dam was followed at this dam. At the time of the 1923 Kanto Earthquake the dam had reached a height of 16.2 m of a total height of 30.3 m. Three cracks running parallel with the dam axis appeared approximately 25 m upstream of the dam center. One of these cracks was about 2.4 cm wide, 10 m deep and 68 m long. It was recognized that the foundation where this crack appeared was the weakest zone.

(2) Damages to dams by the Nankai Earthquake

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There were 7 instances in which high dams were damaged in the Nankai Earthquake. All of those dams were for irrigation purpose and aside from the Honen-ike Dam in Kagawa Prefecture, which is a multiple-arch concrete dam, the rest are earth dams. Of the dams damaged in that earthquake, descriptions are given of the Otani-ike and Honen-ike Dams.

(A) Otani-ike Dam

Otani-ike Dam located in Otani, Komatsu Town, Ehime Prefecture is an earth dam with a center impervious core. This dam is 27 m high, 8 m wide at the crest, 116.4 m wide at the base, the slopes of the upstream 1:4 and the downstream 1:3, and the total embankment 168,800 m². This dam was completed in 1920 and 26 years had lapsed when it was struck by the earthquake. The foundation of the dam is sandstone, the direction of the dam axis is N 85°E and the depth of the water in the reservoir at the time of the earthquake was 16 m.

The earthquake caused a crack on the crest approximately 84 m long and running parallel with the dam axis. In addition, cracks running parallel with the dam axis appeared on the upstream slope at distances of 12 m and 22 m from the upstream end of the crest. These cracks were respectively about 60 m and 24 m long. The culverts in the base of the dam and in the left abutment were damaged. In repairing the dam, earth was compacted where the cracks occurred and cement was grouted from the culvert.

(B) Honen-ike Dam

The Honen-ike Dam is located in the village of Gogo, Mitoya-gun, Kagawa Prefecture. Minor damages were suffered by this dam in the Nankai Earthquake. The dam is 30 m high and the length at the crest is 144 m of which the center 87 m is a multiple arch structure adjoined to concrete gravity structures on both ends. Each arch section with a span of 14.4 m, is 5.45 m thick at the crest and 8 m thick at the foundation.

The earthquake caused a crack in the center arch section near the joint at the buttress and leakage of water was observed. Leakage occurred also in the left abutment where the foundation rock (Izumi sandstone) is weak. The damages were repaired by grouting cement.

(3) Damages to dams by earthquakes

From the foregoing discussion, it is possible to draw a general view on earthquake intensity which have caused damages to various types of high dams. Maximum earthquake intensity experienced with concrete gravity dams was within the scale of VI (250-400 gals) and no damages have happened. However, there have been a few cases in which cracks appeared in the base of overflow gate piers.

Earth dams were damaged by earthquakes of an intensity scale over IV (25-80 gals), and the damages experienced are cracking, sliding of the slopes and collapsing. The degree of damage cannot necessarily be determined by the seismic intensity. Though the intensity may be the same, the degree of damage is dependent on the motion of the earthquake. But, it is recog-

nized that the seriousness of damages is greater to a dam near the epicenter than away from it. The degree of damages to an earth dam greatly depends on the physical properties of embankment materials and the appropriateness of the construction practice, and that a number of the dams reported in this investigation were constructed in an era before modern construction techniques were developed.

Arch. hollow gravity and rock-fill dams are types of structures which have developed comparatively in recent years and have not experienced serious earthquakes. Their ability to withstand earthquake shocks remains to be tested.

§2. Earthquake resistant design methods for dams

Earthquake resistant calculation methods generally followed in the design of dams is the so-called earthquake intensity method in which the weights of the dam and a part of stored water determined by the formula of dynamic water pressure, are multiplied by the seismic intensity and these forces of inertia are applied horizontally on the dam body to calculate stress and stability. This method has been in practice in Japan from the time high dams began to be constructed. Since then as years have passed improvements to the method have been made in substance, and with developments in the researches and studies of earthquake phenomenon and earthquake resistant properties of dams, at present the earthquake intensity taken into account in the design of dam is determined by various factors such as the type of dam, foundation rock, occurrence of earthquake in the past in the area where the dam is to be constructed, and the magnitude of influences on the downstream reaches. In the case of an important dam, model tests are made to study vibration features, equivalent seismic coefficient are considered dynamically, and measuring of stresses under assumed earthquake conditions are considered dynamically, and measuring of stresses under assumed earthquake conditions are conducted to examine closely the stability of a dam in an earthquake.

Table 1 is the design seismic coefficient given in the design criterion for dams established in 1957 by the Japanese National Committee on Large Dams and this criterion is commonly followed in Japan today. Half the values shown in Table 1 may be taken when the reservoir is empty because in case of an emergency the damages caused thereby not be serious.

Table 1. Design Seismic Coefficient

Type of dam	District	
	Tohoku region(Fukushima, Akita, Miyagi Prefectures); Kanto region: Chubu region; Kinki region; Southern Shikoku region	Hokkaido region; Hokuriku region; Tohoku region(Iwate, Yamagata, Aomori Prefecture); Northern Shikoku region; Kyushu region
Concrete dams and rock-fill dams	0.12 - 0.20	0.10 - 0.15
Earth dams	0.15 - 0.25	0.12 - 0.20

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In the case of a concrete dam, on the stress obtained from the above seismic coefficient, the allowable stress under static load condition only is increased by 15 to 20 %.

In the following paragraphs are described examples of earthquake resistant designs applied in existing dams.

(1) Ogochi Dam (concrete gravity type)

The Ogochi Dam is a non-overflow concrete straight gravity structure constructed on the upper reaches of the Tama River in the Kanto region. The dam is 149 m high, 353 m long at the crest and the volume content of concrete is 1,675,680 m³. This dam was built by the block system and keys were provided in both the longitudinal and transverse joints which were bonded by grouting after cooling of the mass concrete.

Stress calculations made in the design of the dam were both gravity and trial load twist methods of analyses which were conducted under the following combinations of loads.

- (a) static water pressure at full water level-silt pressure-earthquake force in the downstream direction-dynamic water pressure-dead load-up-lift pressure.
- (b) static water pressure at full water level-silt pressure-dead load-up-lift pressure.
- (c) dead load-earthquake force in the upstream direction.

In the trial load analysis, variation of stress conditions in ungrouted and grouted transverse joints and other conditions were analysed.

The equivalent horizontal seismic coefficient taken for full reservoir condition was 0.12 and that for empty reservoir 0.06. The allowable compressive stress of concrete under earthquake condition was taken at 46 kg/cm². To study the results of the design calculations, model tests were made by the photoelastic method. However, in this analysis, the earthquake force was considered as static external force. Studies are being continued on the behavior of the dam to vibration caused by natural earthquakes.

(2) Ikawa Dam (hollow gravity type)

The Ikawa Dam is a hollow gravity type concrete structure, 103.6 m high and 243 m long at the crest, constructed on the upper reaches of the Oi River in the Chubu region. The upstream and downstream faces are 1:0.55 and the section is I shaped.

Earthquake resistance studies in the design of this dam were for the following load combinations.

- (a) static water pressure at full water level-silt pressure-earthquake force in the downstream direction-dynamic water pressure-dead load.
- (b) earthquake force in the upstream direction-dead load.
- (c) earthquake force in the direction of the dam axis-dead load.

The equivalent horizontal seismic coefficient taken for full water condition was 0.12 and that for empty reservoir 0.06. For stress calcula-

tions, in the case of (a) and (b) load conditions, load distribution was assumed to be trapezoidal and in the case of (c), buttress wall was assumed to be fixed triangular slab. Stress distribution was studied by three dimensional photoelastic experiments to check and correct the computed values.

From the standpoint of earthquake resistance, case (c) was considered to be the most important one because of the existence of tensile stress, though very little, and vibration experiments with rubber models were supplemented to determine the period and mode of natural vibration. The results of this experiment generally agreed with the calculated values and the primary vibration period of the highest section in the actual dam was estimated to be in the order of 0.11 to 0.12 seconds.

After the dam was completed, vibration experiments were conducted with vibrators and records are maintained on the vibration of the dam and foundation caused by natural earthquakes.

The vibration experiments were primarily conducted to study the vibration characteristics in the direction of the dam axis. The vibration period is generally close to the value obtained in the model experiments and the damping constant showed an extremely small value of about 2%. In view of the fact that the fundamental period of the dam is short in comparison with the predominant period of the earthquake motion and consequently, the degree of resonance being rather small, it was concluded that the dam is stable against earthquakes. Vibration records of natural earthquakes are generally in agreement with the estimated values.

(3) Kamishiiba Dam (concrete arch dam)

The Kamishiiba Dam constructed on the upper reaches of the Mimi River in the island of Kyushu is a constant angle arch dam. The standard radius at the crest is 142 m, the center angle 115°, the crest length 330m, the height 110m, the maximum width 27.7 m and the minimum width 7 m.

Earthquake resistant features studied in the design of this dam are stress conditions under the following load combinations.

- (a) static water pressure at full reservoir-silt pressure-decline of temperature-earthquake force in the downstream direction-dynamic water pressure-dead load-up-lift pressure
- (b) static water pressure at low water level-rise of temperature-earthquake force in the upstream direction-dead load
- (c) earthquake force in the tangential direction at arch crown
- (d) relative displacement of both banks caused by the phase difference of seismic waves at either banks

The equivalent horizontal seismic coefficient used in the calculations were 0.12 at full reservoir and 0.06 at low water level conditions.

Stress calculations were checked by the trial load analysis method and adjustments were made in the radial direction or in the tangential direction, tangential direction and rotation.

The maximum stress was found to be in load condition (a) and in the other cases the stress was of a value much smaller and not comparable with (a).

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In the case of load condition (a), when the tensile stress exceeded 5 kg/cm^2 on the cantilever elements, recalculations were made as cracking of concrete was assumed to have occurred. Allowable compressive strength of concrete was so determined that the safety factor is 5 based on the strength of test pieces (15 cm diameter, 30 cm long) 91 days and the allowable strength of concrete was increased by 15% under earthquake conditions.

In addition to the above described earthquake considerations, in the design of the dam, studies of the vibration features by model tests, observations of earthquakes at the damsite and detailed investigations of acceleration spectrum due to seismic wave action and seismic activity in the vicinity of the damsite were conducted.

The mode and the natural period of the vibration of the dam at empty reservoir condition were studied with models made of agar-agar and those at full and empty reservoir conditions were studied on a rubber model whose specific density coincided with that of concrete. Through these experiments, the modes and periods of the vibrations of the first, second and third order were determined and further the prolongation of the natural period by the storing of water were ascertained.

For the earthquake characteristic in this region, it was found that the predominant period at the damsite was shorter when the earthquake originated in the inland and the epicenter was close, and in the case of an earthquake originating in the ocean some distance away, the period was quite similar to the dam's lowest natural period of 0.3 to 0.4 second. From the acceleration spectrum calculated from earthquake records, it was found that the equivalent seismic coefficient is 3 to 1.5 times the intensity at the foundation for a period of 0.1 to 0.4 second taking a damping constant of 0.1.

On the earthquake activities in the region, based on earthquake records of almost 50 tremors in the Kyushu region, it was estimated that the maximum intensity was in the scale of III to IV. It was also assumed that the influence would be extremely little to this damsite by inland earthquakes of a volcanic nature.

As a result of these stability calculations and experimental studies, it is believed that the stability of the Kamishiiba Dam against earthquakes has been generally satisfied. Since completion of the dam, vibrations caused by natural earthquakes and with vibrators are being studied.

(4) Miboro Dam (rock-fill dam)

The Miboro Dam is a rock-fill type of structure with an inclined impervious clay core under construction on the upper reaches of the Sho River in the Hokuriku region. The dam is 131 m high, 405 m long at the crest and the total embankment is $7,950,000 \text{ m}^3$. Design calculations were made for the following load conditions.

Stability calculations against sliding were made of the dam body (downstream filter zone and rock zone) under full reservoir condition. Stability against bearing strength under full reservoir condition were examined at the base vertically below the downstream end of the crest of the

standard section of the dam. On the stability of the upstream slope, calculations were made by the slip circle method under the following conditions: at completion of the dam, full reservoir level and low water reservoir level. Stability calculations of the downstream slope was also by the slip circle method. For the foundations, the stability against horizontal shearing force was calculated under empty reservoir and full reservoir conditions. In all of the stability studies described above, calculations were made taking into account earthquake influences as well as not taking into account such influences.

Of the load conditions that were taken into account in the design calculations, the horizontal seismic coefficient was $K_h = 0.12$ and in case of a temporary condition only one half of this value was considered.

As a result of these calculations, the minimum safety factor obtained for sliding was 1.1 and for stability of the Slopes was 1.2.

(5) Makio Dam (rock-fill dam)

The Makio Dam under construction on the tributary of the Kiso River in Nagano Prefecture will store water for the Aichi Irrigation System. This dam is a rock-fill structure with a center clay core. The dam is 106 m high from foundation rock, 264 m long at the crest and the total embankment is 2,500,000 m³. A minimum safety factor of 1.37 was obtained as a result of studies made by the modified Fellenious' method for various load conditions. A horizontal seismic coefficient of 0.15 was taken in the design calculations taking into consideration earthquake records taken at the dam site and the results of vibration experiments on a model. Acceleration meters and displacement meters are embedded in the dam and observations of earthquakes and measurements of the movement of the dam are being taken.

(6) Ojiya Dam (earth dam)

The Ojiya Dam is a center core, rolled-fill earth structure located at the downstream of the Shinano River in the Hokuriku region. It is 954 m long at the crest, 18.8 m in maximum height above ground level and 29.3 m above Tertiary bed rock.

Calculations of the stability of the dam were carried out as follows: The stability of the dam to shear force was studied at the base and on the surface of the Tertiary bed of the highest section under normal and earthquake conditions. At the same time the stability of the slopes of the dam under normal condition was examined by the slip circle method when rapid draw-down takes place on the upstream side and when there is a snow load of 1.75 t/m² working on the downstream slope and its seepage pressure is at full work.

As a result of the studies, minimum safety factors of 1.18 at the base of the dam, 1.04 on the surface of the Tertiary bed in time of earthquake shock and 1.48 on the upstream slope and 1.47 on the downstream slope under normal condition were obtained. The angles of internal friction of the soil used in the calculation were determined by laboratory shear tests. A seismic coefficient $K_h = 0.1$ was used as an earthquake force and the angle of internal friction was reduced by applying Dr. Sano's method. As the unit weight of earth below the seepage line, saturated unit weight when it

acted as external force and bouyed unit weight when it acted as resistance were adopted. The excessive pore pressure in the clay core taken in the calculation of the slope stability was 1.25 times the vertical height of the core from the point under consideration.

§3. Description on research activities

The history of research activities related to earthquake resistant feature of dams can be braodly divided into two periods. The first period is the years from the Kanto Earthquake to the termination of the Second World War. Research activities during these years in respect of earth dams was treated dynamically and of gravity dams the research was mainly directed to practical calculation methods by the earthquake intensity method. On arch dams there were no studies worthy of mention. This trend in research activities is believed to have resulted from the belief that, in view of actual damages to dams by earthquakes, dynamical clarification of an earth dam was thought essential but as for gravity dams (the highest in existence during that time was the Miura Dam which is only 84.1 m) it was recognized that they were in practicality considered rigid structures.

The second period of research activities is the year from the termination of the War to date. During this period theories of considering earthquake force as dynamic external force and researches by model experiments have developed. These activities were stimulated by definitive schedules of construction of gravity dams, arch dams and hollow gravity dams in the magnitude of 150 m in height, and dynamical researches advance greatly as it was realized that statical researches only were inadequate in the studies of earthquake resistant features of these dams. Whereas researches up to now, in a way, have dealt with the fundamental problems of vibration, in recent years the practical application of theories have been pursued. In order to obtain necessary data for this purpose, measurements of dynamic characteristics and vibration during earthquakes in existing dams and researches in the dynamic strength of structures and structural materials (concrete and earth) have been stated.

Generally speaking, the researches on the earthquake resistant features of structures have been made to understand the properties of earthquake motion expected in the region where a structure is to be built and the properties of structures that can resist such forced vibration and to find an appropriate earthquake resistant design method. It will be noted from the general description of earthquake resistant studies of dams in Japan given in the foregoing chapter our knowledge with respect to the earthquake motion at a damsite and the dynamical properties of a dam are by no means sufficient. The trend of abstractness in many of the studies is believed to be due to the inadequacy in the accumulation of concrete knowledge. Fortunately, seismometers and other measuring instruments have been installed in 17 important dams recently constructed. There are plans to maintain records of earthquake motions of the dams and foundations at the Japanese National Committee on Large Dams.

In the design of important dams dynamic studies by model tests are being conducted to estimate their earthquake resistant properties. Exciting machines and large shaking tables for model experiments essential for the above studies are installed in the Technical Research Laboratory of the Cen-

tral Research Institute of Electric Power Industry, Institute of Industrial Science, University of Tokyo and Engineering Research Institute of Kyoto University. On the dynamic properties of materials used in the construction of dams, the studies are being performed to find the non-elastic properties of the materials going a step forwards from their elastic properties.

Chapter 5. Harbor structures

§1. Fundamental items in aseismic design of quay walls

1) Design seismic coefficient

Current aseismic design of quay walls are briefly reviewed in this paper following the newly revised "Japan Harbor Engineers' Manual (1959)". As shown in the manual, in the aseismic design the whole effects of an earthquake are substituted by static force obtained by multiplying seismic coefficient to the mass in question. Consequently proper estimation of seismic coefficient K , which is the ratio of seismic force to gravity, is essential to an aseismic design. Although many factors should be considered in estimating design K -value, it is done empirically. As to locality of proposed harbor structure, K -values being different for each region of this country is given in the manual by referring to a study of Dr. Kawasumi on the regional probability of occurrence of destructive earthquakes. It is also to be noted that in the current design only the horizontal seismic coefficient is taken into consideration.

Seismic coefficient in the air should be increased in the water due to buoyancy. This increased coefficient is called apparent seismic coefficient and it is given by the following equation.

$$K' = \frac{\gamma}{\gamma - 1} K$$

where K' : Apparent seismic coefficient in the water
 K : Seismic coefficient in the air
 γ : Unit weight of the mass in the air
 (for the soil it should include the weight of water saturating the soil mass)

2) Lateral earthpressure in an earthquake

Mononobe-Okabe formula is used in the computation of lateral earthpressure in an earthquake.

$$P = \left\{ \sum \gamma h + \frac{\gamma \cos \phi}{\cos(\psi - \beta)} \right\} C$$

$$C = \frac{\cos^2(\phi \mp \psi - \theta)}{\cos \theta \cos^2 \psi \cos(\delta + \psi \mp \theta)} \left[1 \pm \frac{\sin(\phi \mp \theta) \sin(\phi \mp \beta - \theta)}{\cos(\delta + \psi \mp \theta) \cos(\psi - \beta)} \right]^2$$

where ϕ : angle of internal friction of backfill,
 β : friction angle between wall and soil,
 ψ : inclined angle of back surface of wall to the vertical,
 δ : angle of ground surface of backfill to the horizontal,
 h : height of the wall, and $\theta = \tan^{-1} K$

In this equation, following values are usually to be used. The unit weight

of sandy soil is 1.8 t/m³ above the water level in the backfill and 1.0 t/m³ below the water level in the backfill, and angle of internal friction of sand is 30° for general case and 40° for particular good condition, and friction angle between wall and soil is 15° - 20°.

Correction should be made for the lateral earthpressure below sea water level as follows. Lateral earthpressure at the water level in the backfill and that at the bottom of the wall are computed by employing seismic coefficients in the air and that in the water respectively and straight line connecting these two values of lateral earthpressure gives the lateral earthpressure distribution under water.

3) Dynamic pressure of water in an earthquake

The dynamic pressure of water in the backfill is not considered in the current design procedure, because during an earthquake the water might move together with soil particles and dynamic water pressure is to be included in the lateral earthpressure when it is computed by employing apparent seismic coefficient. Also the dynamic pressure of water in front of the wall is not considered because of the complexity of its characteristics such as difference of phase from that of wall movement of lateral earthpressure.

4) Bearing capacity in an earthquake

In addition to the value for the static state, a great increase of inclination and eccentricity of load should be taken into consideration during an earthquake. The circular sliding surface method is employed in the computation of the bearing capacity during an earthquake.

5) Lateral resistance of piles

For the lateral resistance of a vertical pile the following values are standard in design

for clayey soil < 5 ton
for clayey soil when replaced the top 2 m layer with dense sand < 7 ton
for sandy soil < 10 ton

The lateral resistance of group of piles is considered to be the total sum of lateral resistance of single piles.

Lateral resistance of coupled piles is computed from two axial forces each of which might take the ultimate bearing capacity of pile. The estimation of the bearing capacity is made by (1) loading test of pile, (2) statical bearing capacity formula, or (3) pile-driving formula. The safety factors of 2.5 are used for the cases (1) and (2) and 6-7 for the case (3).

6) Stability of slopes during an earthquake

The stability of slopes in an earthquake is analyzed by the circular sliding surface method, by taking horizontal seismic force into consideration. The lower limits of safety factors in the static and seismic conditions is to be 1.5 and 1.2 respectively when a permanent structure is proposed. These values are reduced to 1.2 and 1.0 in the case of a temporary or an unimportant structure. Usually in sandy soil or gravelly soil the base

failure need not be considered. However, the possibility of the toe failure should carefully be examined.

§2. Aseismic design of quay walls

1) Gravity type quay walls

Main external forces acting on the wall in an earthquake are lateral earthpressure, dynamic water pressure and mass force of the wall itself. The computation should be done on three possible causes of failure, namely on (1) the sliding of the wall along its base, (2) the high toe pressure in excess of bearing capacity of the foundation soil and on (3) the sliding below the foundation. The safety against sliding along the base will be secured by making the ratio between horizontal and vertical components of the total external force smaller than coefficient of friction along the base. The safety factors are to be 1.2 for static state and 1.0 for seismic condition. The bearing capacity and the sliding in the foundation soil is dealt with by the manners mentioned in §1. In addition to the above mentioned analyses it is desirable to take the following measures to prevent the damage at the corner and the approach of the quaywall and at the joint of different structures: (1) increasing design K-value by some 20% (2) providing stays or (3) connecting the walls in the direction of quaywall.

2) Sheetpile bulkheads

The aseismic design of a sheetpile bulkhead is performed on the following steps.

- (1) Computation of lateral earthpressure on the bulkhead:
(cf.2) and 3) in §1)
- (2) Estimation of necessary length of sheetpile embedment:
120 % of the length computed by the free-earth support method is adopted.
- (3) Design of tie-rod:
Tie-rod tension is computed by supposing the bulkhead as a simple beam which is supported at the sea bottom and the position of wale. Allowable stress of tie-rod is to be 900 kg/cm² and 1400 kg/cm² for static and seismic conditions respectively.
- (4) Design of sheet-section:
The maximum moment is computed for the simple beam mentioned above. Allowable stress of sheetpile is 1400 - 1600 kg/cm² and 2100-2400 kg/cm² for static and seismic conditions respectively.

These computations are illustrated in the design charts given in the manual in the case of a sheetpile bulkhead with relieving platform, horizontal forces on the bulkhead including horizontal mass force of the platform and the fill on it are supported by batter piles, design of which is the same as described in section 5) in §1.

3) Cellular bulkheads

In the aseismic design of a cellular bulkhead, the stability analysis of bulkhead itself should be included in addition to the analyses of the whole structure as a gravity type quaywall. Since the stability of the

bulkhead is considered to be depending on the lock tension of sheetpiles and internal shear of bulkhead, analyses should be performed on these items. The width of bulkhead is so determined that the shearing force S on the vertical neutral plane of bulkhead does not exceed the allowable shearing resistance Sa of bulkhead. The values of S and Sa are given in following equations.

$$S = \frac{3}{2} \frac{M}{b}$$

$$S_a = \frac{1}{F} \times \frac{1}{2} K_i \cdot \gamma \cdot H^2 (\tan \phi + f)$$

where

- M : Overturning moment due to external force acting on the bulkhead above the sea bottom
- b : Width of bulkhead
- F : Safety factor
- K_i : Lateral earthpressure coefficient of fill material (usually 0.6 is used for both static and seismic conditions)
- γ : Unit weight of fill material
- φ : Angle of internal friction of fill material
- H : Height of the wall from the sea bottom
- f : Frictional coefficient between the locks of sheetpiles

The lock tension T given in the following equation is compared with the allowable value, which is 250 ton/m for the straight sheetpile produced in this country.

$$T = K_i \cdot \gamma \cdot H r$$

where

- r : Radius of a circular cell

4) Trestle type piers

Earthquake force to a trestle type pier is only mass force of the trestle itself and surcharge on it. The following assumptions are made on the fixity of the embedded part of the pier. (1) For the trestle type pier of piling, the depth of the point of fixity is assumed to be αH, in which H is the height of trestle (neutral axis of beam) from the sea bottom and α is coefficient being equal to 0.1 for sandy soil and to 0.25 for the sandy clay or firm clay (c > 0.3kg/cm²). (2) In design of trestles of cylinder supported by piles, pile heads are assumed to be hinged. (3) In the case of the pier which has trestles of large dimensions (caisson, pneumatic caisson or well) the design of trestle is similar to that of a well.

It is a common practice to rigidly connect the trestle with the floor frame, but it is desirable to divide a pier into several blocks to prevent the different settlement. In such a case a dynamical analysis can be applied to each block, because it is considered to be a relatively simple system of vibration.

5) Trestle type piers with small retaining walls

In addition to the separate analyses of the main part and the retaining wall, the stability of a whole structure including slope and wall should be analyzed according to the procedure shown in the section 6) in preceding

article.

Chapter 6. Bridges

§1. Earthquake Damage

The following are the general findings of the survey of the damages which bridge structures suffered from five violent earthquakes occurred during the last four decades in Japan: the Kanto Earthquake (September 1, 1923; M = 7.9), Nankaido Earthquake (December 21, 1946; M = 8.1), Fukui Earthquake (June 28, 1948; M = 7.2), Imaichi Earthquake (December 26, 1949; M = 6.5), and Tokachi-oki Earthquake (March 4, 1952; M = 8.2).

- a) No damage was found in regions of seismic intensity scale III (8-25 gal). Some little damages were found regions of seismic intensity scale IV (25-80 gals), and severe damages were caused in regions where seismic intensity scale V (80-250 gals) or up. Structures located in regions of seismic intensity scale VI (250 - 400 gals) suffered much more extensive and severe damages than those in regions of seismic intensity scale V.
- b) The degree of damages is largely related to the kind of foundation, and marked damages were observed where foundations are alluvial or soft ones.
- c) As regards the aseismatic property of bridges, superstructures and substructures shall not be considered independently. For instance, while we can find some cases where the superstructure seems to have fallen down due to the damage of the substructure, there are cases where slanting or overturning of substructures shall be attributed to the extraordinary forces resulting from the downfall of the superstructure.

Details of earthquake damages of bridge structures are as follows.

1) Superstructures: Most of the damages are the downfall of girders or trusses. For example, there were some reinforced concrete girders that at last fell after moving perpendicularly to the bridge axis, and also some trusses that toppled sideways moving on rocker bearing or that fell to the ground because of the dislocation from piers. Worthy of mention is the Yoshino-River railway bridge that suffered in the Nankaido Earthquake. The superstructure of this bridge is a three-span (3× 71.2 m) continuous through truss of single track supported by piers of 20 m deep caisson foundation. In the case of this bridge, complex movement of pier tops due to the slanting and sinking of substructures caused relative displacements of truss shoes, both horizontally and vertically, which induced in the truss members stresses greater than the allowable stress. However, these stresses fortunately seemed to have not reached the yielding stress of the material and the bridge has been still in use after repairing.

There are also a few examples of reinforced concrete girders in which cracks grew due to the use of poor concrete having low strength.

2) Piers: The earthquake damage to piers is classified into the occurrence of cracks, the cutting of pier body, the overturning after cut, the sinking and lean of pier, and so forth. In the bygone earthquakes, the damage occurred mainly in the piers made of bricks, the stone masonry piers

with filling, or the plain concrete piers, and on the other hand the piers made of reinforced concrete have hardly been injured by earthquakes.

Most of horizontal cracks in a pier body generated in the neighborhood of ground surface, but the number of cut sections tends to increase with the increase in height of a pier. The damage example of bridge structures in the ruins of the Kanto Earthquake of 1923 show that the number of cut sections was 2 or 3 if the pier height was 14 m - 17 m, while a pier of about 10 m high was cut at one section and the piers with less height were not cut horizontally by the earthquake shocks.

Even the similar piers at the same bridge site revealed different states of damage according to the nature of foundations they stood. The damage to such a pier as tied with arch on two wells was found not a little. Some of the piers constructed on a slope were considered to lean or overturn at their foundations because of the sliding of the slope, the flow of sand and so on.

3) Abutments: The damage to abutments from earthquakes are roughly divided into the lean of structures, the cutting of parapet wall, the horizontal cracks and cut of an abutment body, the damage of wing masonry, the bending or cutting of anchor bolts.

Judging from the examples of the damage from the bygone earthquakes, the stability of an abutment against seismic action is improved and the damage is decreased if the portion of the abutment buried in soil is large.

Girder acts as a strut against the sliding of an abutment, and consequently, there have been many damage examples of the bending and cutting of anchor bolts or the cutting of a parapet wall.

Moreover, the damage to foundations which include wells and piles, shall be discussed and it is suggested to refer the publications of the Japan Society of Civil Engineers regarding these items.

§2. Aseismic design method

As the design specifications for highway and railway bridges in Japan are based on the similar idea, discussions will be made mainly with railway bridges here.

The seismic force applied to a structure is assumed as a statical force which is obtained by multiplying the weight of the structure by seismic coefficient. The earthquake-resistant design of structures is specified in the draft of the Design Specifications for Plain and Reinforced Concrete Structures (April 1955) proposed for adoption by the Japanese National Railways. According to these standards, the horizontal seismic coefficient is taken as 0.15, 0.20, 0.30 dividing the country of Japan into three portions A, B, and C, and the vertical seismic coefficient value to be used is half the value of the horizontal seismic coefficient. These coefficients can be modified by the use of table A and B, if the designer so desires and only with the approval of the railway authorities, to account for such items as the locality of the structure, the type of site soils, the kind of structure,

a balance between strength and stability of structure, and the respective vibrational characteristics for the structure and the foundation.

TABLE-a
Horizontal Seismic Coefficient Modification Table

Type of Calculation	Strength Calculation of a structure which vibrates freely						Stability calculation of a structure which vibrates freely, and strength and stability calculations when the effect of earth pressure is taken into consideration		
	Massive Structure			Slender Structure					
Region / Kind of Foundation	A	B	C	A	B	C	A	B	C
Class I	0.35	0.25	0.20	0.20	0.15	0.10	0.20	0.15	0.10
Class II	0.25	0.15	0.10	0.30	0.20	0.15	0.25	0.20	0.15
Class III	0.15	0.10	0.10	0.30	0.20	0.15	0.30	0.20	0.15
Class IV	0.15	0.10	0.10	0.30	0.20	0.15	0.35	0.25	0.20

TABLE-b
Classification of Foundation Soils

Class	Kind of Bed
Class I	The surface layer is an Alluvium formation 2 meters or less in thickness and directly underneath exists a hard stratum of the tertiary or earlier era extending over a fairly wide area.
Class II	The surface layer is of the Diluvium formation whose thickness ranges from 3 to 15 meters or of the Alluvium formation whose thickness ranges from 2 to 10 meters
Class III	The surface layer is of the Diluvium formation 15 meters or more in thickness or of the Alluvium formation, whose thickness ranges from 10 to 25 meters.
Class IV	Remarkably soft and weak bed, or the surface layer is of the Alluvium formation, 25 meters or more in thickness.

These coefficients are to be changed in consideration of the grade of materiality of the railway lines.

On the other hand, the Design Specifications for Steel Railway Bridges (Japanese National Railways; 1955) specify the standard seismic coefficient of 0.2 (horizontal) and 0.1 (vertical) for design of steel railway bridges. However, these values can also vary in consideration of the conditions of

bridge site. The reasons why the different values of seismic coefficient are employed for concrete structures and steel railway bridges are as follows: first, the fatal damage to steel bridge superstructures from an earthquake has not been reported so far, and second, if the constant seismic coefficient is used, the identical standard design is usable throughout the country.

Inasmuch as there has been no generally reliable theory developed or adequate actual observed data with respect to the earth pressure during an earthquake, there is no way to pin-point the resultant earthquake effect other than to use the results of experimental tests and what actual observations have been made. At the present stage, Dr. Mononobe's theory on the earth pressure under seismic action is widely used. Although the assumptions made by Dr. Mononobe might be valid at some time, the earth pressure during an earthquake varies according to the complexity of the vibration and the deformation condition of each part. Accordingly, it shall be necessary to take such items as the rigidities of foundations and structure, the characters of soil in contact with them, to find out the earth pressure during an earthquake.

(NOTE) In Mononobe's theory, the foundations and structure are regarded as one body and are tilted in the critical direction, in respect to the direction of gravity force, an angular amount of $\theta = \tan^{-1} K$. Then, making use of $W' = mg (1 - K_v) \sec \theta$ instead of $W = mg$, the earth pressure is calculated by means of a method similar to that for the normal gravitational earth pressure. In the above expressions, $K = K_h / 1 - K_v$ is the resultant seismic coefficient, where K_h = horizontal seismic coefficient and K_v = vertical seismic coefficient.

So far as the stability calculations for structures are concerned, the draft of the Design Specifications for Plain and Reinforced Concrete provide for the stabilities against sliding, overturning, and bearing capacity.

The safety factor for sliding at the bearing face and the base of foundation etc. is specified to be more than 1.5 and the resistance to sliding of pile foundation is taken as equal to the resistance at foundation base. When the battered piles are used, the resistance of soils may be added to the allowable horizontal resistance of the piles to sliding, which is defined by the horizontal component of the vector in the direction of pile axis expressing the allowable bearing power of the pile.

In regard to the safety for overturning, the point of application of the resultant force of loading on foundation base shall be in a middle third of the base width if a structure stands on soil, and be in a middle half of the base width if it stands on firm foundations like rocks or piles. When a foundation is soft and it has a fear of unequal settlement, the point of application of the resultant force mentioned above is desirable to be as close to the center of foundation base as possible. However, in designing a substructure against earthquakes, the point of application of the resultant values of foundation reaction or loading at the top of a pile exceed the allowable bearing capacity. As a rule, the normal allowable bearing capacity of soil foundations and piles is a half of the ultimate bearing capacity on the bases of the actually measured values, but the allowable bearing capacity of the clay foundation and the friction piles supported by unstable soils like clay shall be less than one-third of the ultimate value.

These allowable bearing capacity employed for aseismic design should be the value at the normal times.

The allowable stresses of materials at the time of an earthquake is differently specified for concrete structures and steel bridges. In the former, the allowable stresses may be increased 50 % for combinations of seismic and dead loads and increased 100 % for combinations of seismic, dead and live loads. On the other hand, in the design of steel bridges, the allowable stresses may be increased 75 %. This difference is originated from the fact that at the time of an earthquake the stress of steel is permitted up to its yielding stress in designing steel bridges.

The actual structures of prestressed concrete have a history of only about six years in Japan, so there is no example of earthquake damage to them and they are designed in the same way as for reinforced concrete structures. The relation between the natural frequency measured and span of the prestressed concrete highway bridges ever constructed is shown in the following table.

Span (in m)	15	20	25	30	35	40
Nat. Freq. (cps)	6.0	4.5	3.5	3.0	2.7	2.4

Since one can think of a fear that they might be in resonance with earthquake motion the amount of anchor bars is calculated by the same computation method as in reinforced concrete highway bridges and the girders are anchored in their substructures.

§3. The earthquake-resistant practice of bearing construction in bridges

The evidence obtained from past earthquake damages show that a superstructure of bridge was hardly injured by itself and the damage was found mainly in its substructure or the connection between super and substructure. To cite the instances of the latter, because of the lack in strength at the connection a girder jumps on to substructure or falls to the ground, and sometimes rollers at support are out of place.

In the light of these earthquake damages, the following countermeasures have been taken in earthquake-resistant construction of supports of railway bridges.

(1) Supports of truss bridge: In 1916, the oldest type of the bearing construction was adopted as a standard design of the shoe of truss bridges by the Japanese National Railways. However, based on the experiences from the Kanto Earthquake of 1923 the modification of design was made and then a new standard design appeared in 1925 since the modification mentioned above was not satisfactory. In the new standard design, a pin was covered by a cap to connect the upper shoe with the lower one and the shoe was anchored to the body of substructure by bolts in order to prevent truss from jumping.

The design to keep rollers from being out of place, has been used since 1937. In this practice, because the projecting parts on and underneath the chain lock is inserted into notches of both shoes with some margin, rolling-out of the whole rollers kept their spacing by a connection plate is prevented without having a hindrance to a move of the roller.

(2) Supports of plate-girder : The structure of a support of plate-girder bridge has been also modified from time to time. In 1919 and 1920, the Japanese National Railways adopted a standard design (E33 and E40 Deck plate-girder) in which there was no anchor bolt but in the light of the experiences from the Kanto Earthquake of 1923, the anchor bolts to connect the girder to a substructure were adopted after 1924.

(3) Steel post supporting plate-girder : In the overbridges constructed in 1907 through in Tokyo by the Japanese National Railways, the post of a plate-girder had two spherical bearings made from cast steel at its both ends. Since this structure had a defect in keeping the girder from jumping during an earthquake, pins were inserted at the both ends of a post to prevent uplift after 1924.

Later the spherical bearings were employed again, and the modification has been made at the point that pins were inserted at the top of spherical surface. Although there remains some ambiguity in stress distribution, this practice has been used so far in Japan.

Chapter 7. Conclusion

In the above chapters, general standards applied to the usual structures are described. However, the specially important structures are used to be designed after precise investigations on their stability during earthquakes. Some examples of the procedures for modern aseismic design are as follows.

- 1) The New East Pier in Port Kobe :
Seismic intensity to be expected at the site was determined by referring to many past earthquake records obtained by the Kobe Meteorological Observatory at its location, considering the difference of geological conditions of both location. Coefficients of base shear for each floor were decided by theoretical investigation on the vibration of structure. After the completion, various tests on the vibration characteristics of the structure were carried out.
- 2) Wakato Suspension Bridge :
Wakato Bridge is under construction at present and is 90 m + 364 m + 90 m in span length and will become the longest bridge in Japan after completion. In order to examine its seismic stability, vibration tests on the model which is 1/100 of the prototype in scale was carried out. The movement of the foundation during earthquake is assumed to be represented by a sinusoidal wave, because there is no record of strong motion earthquake at Wakamatsu, and nevertheless we must design it dynamically, for fear that there occurs resonance of the bridge with the ground motion during an earthquake. The amplitude, period and the number of ground motion of sinusoidal wave are decided theoretically as well as experimentally.
- 3) The Kamishiiba Arch Dam :
The approximate period of vibration is given by vibration test using the models, and the seismic coefficient is determined by using the record of earthquakes at the dam site. After completion, vibration due to earthquake are measured, and the conditions for design are checked by the records obtained by the seismograph.
- 4) Bridges in the Meishin Express Highway:
Tremor observation was done at the site of bridges planned, and the proper seismic coefficients are determined on the basis of its results.

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In order to design structures reasonably, it must be necessary to get the more detailed and complete informations on characteristics of the earthquake, and dynamical properties of materials and structures. And now field and laboratory tests, and theretical researches are being carried out for the purpose of getting the more attainments.