ASEISMIC DESIGN OF BRIDGES

by

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Synopsis

Aseismic design of bridges at present, and the general nature of current bridge codes. Limitations of existing design regulations. Dynamic design concept and tendency to evaluate the qualitative behaviour of bridges. Dynamic analysis of mathematical bridge models, tests of real bridges and physical models. Observed behaviour of bridges. Basic geometrical and structural characteristics affecting the seismic behaviour of bridges. Seismic stability of bridges. Concept of dynamic equivalence.

INTRODUCTION

As observations and performed surveys indicate, past earthquakes have caused severe damage to bridges. The primary damage was suffered by the bridge substructure as well as by the superstructure.

One of the common scales for determining the intensities in areas affected by an earthquake is the Modified Mercalli scale, 1956 Version. This scale with maximum intensity XII, lists damage to bridges starting at Intensity X. Therefore, it is considered that bridges may be destroyed during a major earthquake. It is evident, however, that the fall or temporary destruction of a bridge or its parts means traffic interruption. It may also greatly handicap safety measures at the time of an earthquake and the supplying of needs afterwards. In addition, such destruction may endanger those either on or under the bridge at the time of the shock.

According to the conventional earthquake resistant design accepted in many countries, a bridge should carry a set of static lateral loads,the magnitude and distribution of which are specified by codes (Ref. 1).

Most bridge codes require that lateral forces be computed as the seismic coefficient times the weight. Usually, this coefficient is assumed to be constant throughout the height of the structure. This implies a uniform horizontal acceleration, therefore a rigid bridge structure on a rigid base. But inertia forces produce deformations in both the bridge structure and the ground. Therefore, the method is implicitly contradictory.

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According to the present status of design in Canada and in the United States, highway bridges up to a 400 ft. span, which may be affected by earthquakes, are designed by the method presented in the AASHO specifications (Refs. 2 and 3).

Seismic stresses in bridges are computed with the assumed horizontal force

$$EQ = C D \qquad (1)$$

in which D is the dead load weight of the structure, and C is a coefficient equal to

0.02 for rock foundation,
0.04 for spread footing, and
0.06 for pile foundation.

Specifications generally define the static loading which a bridge structure must resist in regions where earthquakes occur. These loadings, however, do not indicate what the response of a particular bridge to a particular ground motion will be.

Instead, it is the intention of the specifications that if bridges are designed to resist the specified static loads, then they will not suffer great damage during ground motion expected to occur within some period of time. Obviously, simple formulas can be applied only for a very narrow range of bridge structures.

Although the formulas do not describe actual inertia loading (in many cases, they are overconservative, while in some cases, just the opposite), they do provide some guidance to designers.

However, it may be stated that improvement in the analysis of seismic forces affecting bridges and studies of typical destructions may result in designs providing greater safety.

According to the statical design of supports, the superstructure of a bridge is considered independently, and its participation with the supports is expressed only during transmission of the inertial forces. From the point of view of the dynamic theory, such an approach cannot be accepted. This is because the design should consider the proper forms of the vibrations of the entire structure during an earthquake, with the integral cooperation of all the separate elements.

It is also necessary to take into account the free vibrations of the individual parts of the structure, and to investigate the dynamic equivalence of the separate carrying elements of the structure. In bridges, such individual behaviour of separate elements may be found to be exceptionally strong. A marked difference between the dynamic properties of the substructure and the superstructure complicate the character of their mutual performance during earthquakes. This may explain why the damage of bridges is very often localized at connections between the supports and the superstructure, generally in the form of displacement of bearings and the fracture of bearing pads. An analytical checking of girder bridges indicates that this damage is hardly explained by the usual magnitude of the seismic loads determined by the dynamic method. Evidently, in this case, more complicated causes of superposition of different types of vibrations in the separate parts of a bridge take place.

In order to arrive at some basis for the dynamic aseismic design of bridges, there may be the following methods, or a combination of them:

a) Dynamic analysis of the mathematical models

b) Dynamic analysis of the physical models

c) Observations on the seismic performance of real bridges

Suspension bridges are not considered in this paper, on account of their special structural characteristics which require separate investigation into their behaviour during earthquakes.

DYNAMIC ANALYSIS OF THE MATHEMATICAL BRIDGE MODEL

The objective of the dynamic analysis is to develop a method for predicting seismic effect on the proposed structural system of a bridge. This system should include the superstructure, substructure and foundation.

Dynamic analysis of the mathematical bridge model is based on the application of a digital computer and the structural idealization of the bridge system. In practice, it is necessary to idealize a bridge structure, so that a mathematical model can be formulated and the response of this model to suitable ground motion can be determined.

Because of the complexity of the problem, a practical approach is to simplify a bridge system into a physically analogous system to which the theory of finite degrees of freedom can be applied.

For the purpose of structural analysis, the method of generation of artificial models or pseudo-earthquakes may be applied successfully, (Ref.4). A procedure has been developed for generating, on a digital computer, a random function that has the pertinent known properties of recorded accelerograms of strong earthquake ground motion. Because of the similarity of the velocity spectra of the real and artificial earthquakes, it is possible to use pseudoearthquakes in the analysis and design of bridges to resist earthquakes. These accelograms can provide ground motion of any specified intensity and duration.

In order to formulate a mathematical model of a bridge structure, it is necessary first to proceed with the idealization of the structure into a mechanical model. This is done by using the lumped mass method, according to which the distributed masses are considered as concentrated in a finite number of points. Oscillators with a finite number of degrees of freedom are thus obtained, and a model analysis can be performed considering the linear,as well as the non-linear behaviour of the bridge.

The differential equations of motions describing the dynamic behaviour of the mathematical model may be written in the following general form

$$[m]\{\ddot{x}\}+[c]\{\dot{x}\}+[k]\{x\} = -\ddot{x}_s\{m\} \qquad (2)$$

where $[m]$ = a mass matrix, $[c]$ = a damping matrix, $[k]$ = a stiffness matrix, and $\{\ddot{x}\}$, $\{\dot{x}\}$ and $\{x\}$ are the vectors defining the acceleration, velocities and displacements of the lumped masses and $\ddot{x}_s$ is the ground acceleration.

Matrices $[m]$, $[c]$, and $[k]$ must be specified, the difficulty of which depends on whether the analysis to be performed is linear or nonlinear.

In linear analysis, matrices $[m]$ and $[k]$ may be expressed from the detailed plans of the bridge structure. The terms corresponding to damping and forced vibrations are disregarded when computing the natural frequencies and vibration modes. The resulting eigenvalue problem

$$[\ [k] - \omega^2[m]\ ]\ \{x\} = 0 \qquad (3)$$

may be solved by standard techniques to obtain the mode shapes and associated natural frequencies (ω_i).

Nonlinear analysis of the bridge structure is more difficult than linear analysis, since mode superposition cannot be used. In this case, the original set of coupled differential equations must be solved directly by a step-by-step integration procedure.

The mass matrix $[m]$ may be calculated from the design data, but neither the stiffness nor damping matrix may be defined as simply as in the linear case.

Actually, the stiffness and damping matrices should be combined to form a nonlinear force-deformation relationship; however, the known proposals are not entirely satisfactory, (Ref. 5). In an attempt to determine the actual force-deformation relationship, single-degree of freedom systems were tested into the nonlinear range, (Ref. 6).

Current methods of formulating mathematical models of bridge structures for aseismic design have not yet been developed to the point that accurate models can be formulated for most types of bridges. Due to the complexity of the actual structure and somewhat inexactly defined properties of the material, the theoretical determination of dynamic characteristics on mathematical models does not really provide exact results.

The computation involved in performing dynamic analysis with even a simple mathematical model is extensive. Therefore, it is important that the mathematical model should be as simple as possible, without omitting any structural characteristics of the prototype that affects its dynamic behaviour appreciably.

It is evident that progress in this area can be made by applying additional experimental dynamic testing of the already built structures and models, (Ref. 7).

DYNAMIC TESTS OF REAL BRIDGES

Dynamic tests of real bridges have been conducted in order to determine dynamic properties and to establish mathematical models that can represent the dynamic behaviour of the prototype structures.

The quantities normally determined by a dynamic test of a structure are: resonant frequencies, mode shapes, and damping capacities.

At present, the equipment most commonly employed in dynamic tests is eccentric mass-type generators, as developed by the Earthquake Engineering Laboratory at the California Institute of Technology.

Resonant frequencies of bridges are primarily affected by its stiffness and mass characteristics, and determined by sweeping the frequency range of the vibration generators.

Near resonance, where the slope of the frequency-response curve changes rapidly, the frequency interval steps are as small as the speed control permits. Plotting the vibration response at each frequency step results in a frequency-response curve.

In general, resonant frequencies may be obtained with sufficient accuracy by reading the frequencies corresponding to peak amplitudes from resonant curves.

Mode shapes may be determined at each of the resonant frequencies, after they are found. It is generally necessary to measure the vibration amplitude of all the required points simultaneously.

The damping capacity of the bridge structure cannot be calculated, and may be obtained only by experimental tests. Therefore, in formulating a mathematical model, it is important that some experimental results of the damping capacity of a bridge structure similar to the one being modeled be available, (Ref. 8).

Many such tests were performed for buildings, but not for bridges.

From the point of view of the theory of seismic stability, it is desirable to have data on dynamic characteristics separately for supports and superstructure.

MODEL TESTS

The determination of realistic quantitative data by model investigation may provide difficulties often impossible to overcome. However, the model test may provide very useful qualitative information regarding the expected dynamic behaviour of the bridge.

Also, since tests on real structures of necessity are conducted at low amplitudes, little information on the nonlinear behaviour of structures is found from such tests.

However, to study nonlinear behaviour and energy absorption characteristics, dynamic tests may be conducted on small model structures vibrating at large amplitudes. Data from such tests can be extended to an analytical evaluation of the nonlinear behaviour of full-scale models.

These types of experiments can be made by using vibrating or shaking tables. Dynamic inertia forces are set up by displacing the table horizontally against coil springs and then suddenly releasing a tripping mechanism. In this way, horizontal accelerations can be obtained. By changing the number and characteristics of the springs, the natural period of vibration of the table system can be varied. Some benefits from these experiments may also be obtained to evaluate the role of the foundation in the structural scheme of the bridge.

Instrumentation for dynamic tests requires that accurate measurements of structural response be simultaneously made at various points and recorded vs. time. Because of this need for simultaneous recording to establish mode shapes and other structural relationships, electrical transducers recording on multi-channel oscillographs are necessary for most model tests.

OBSERVED BEHAVIOUR OF BRIDGES DURING EARTHQUAKES

The behaviour of bridges during earthquakes in the past may be of great assistance in new designs.

It is necessary to investigate the actual deformation performance and capacities and try to relate them to future designs. Having the deformation capacities found from tests considering the usual proportions of members and connections used in bridges, it will be possible to design structures able to resist major earthquake forces.

At the present time, substantial data concerning the behaviour of bridges has been collected after earthquakes as: in Japan 1898 and 1923, California 1906, Chile 1929, New Zealand 1929 and 1931, India 1931, Japan 1948, California 1952, Yugoslavia 1963, Alaska 1964 and California 1971.

The analysis of this data is of great interest to the development of the theory of aseismic stability of bridges.

The damage and destruction of bridges during earthquakes indicate that the main factors are: the type of structure, the length of the span, the material used and the foundation conditions, (Refs. 9, 10, 11 and 12).

Apparently, shaking has little effect on short or medium span highway bridges designed according to present day standards. However, bridges of these spans may be destroyed during a major earthquake and the cause of their destruction will very likely be the differential movement of the soil, rather than the acceleration force. But bridges of long spans and very high structures are susceptible to the acceleration forces of earthquakes.

The following basic factors have a strong influence on earthquake damage to bridges:

1) In most bridges, each pier foundation acts as an independent unit;

2) Piers and superstructures are not monolithic, but connected by bearings;

3) The substructure of the bridge is often influenced by the soil masses, such as embankment, cutting or slope;

4) The type of supporting soil has a great influence on the behaviour of structures. Bridges supported by soft soils suffer the greatest damages, and bridges supported by denser foundation material may suffer from sheared backwalls and wingwalls, severely cracked pier shafts, and broken anchor bolts.

The most frequently observed damage to bridges is the shortening of the free span between substructure units, so called "crowding". Apparently, crowding is caused by the longwave action and extended duration of the earthquake, followed by the subsequent displacement of highway approach fills and river bottoms. Crowding is characterized by the tilting of piers with their tops toward the bridge ends, the cracking of the inside faces of pier shafts at ground elevations, the shearing-off of abutment back-walls, the cracking of the inside faces of steps in pier caps, and the displacement of beams from bearing assemblies.

As the experience of the Alaskan earthquake indicates, simple span bridges having a heavy reinforced concrete superstructure and flexible piers, were poorly equipped to withstand large soil displacements, or the acceleration forces of the earthquake, (Ref. 13). The extent of their destruction was roughly proportional to the total length of the bridge. The damaging movements were along the centerlines of roadways with the substructure units moving closer together. Such behaviour cannot be explained by referring to the direction of earthquake wave motion, because bridges of the same type in the same areas experienced the same displacements, though the directions of their longitudinal axes varied.

The primary longitudinal movements were due to two factors: the piers oscillated in their most flexible planes, irrespective of the direction of the earthquake waves, and the differential displacements of the approach fills were in the longitudinal direction.

During their collapse, the above substructure displacements resulted in the movement of the superstructure units as far as 10 ft. longitudinally, but in only slight movement in the transverse direction.

The most common types of damage or destruction of the substructure of a bridge may be summarized as follows:

a) Displacement and failure of abutments due to increased soil pressure,

b) Settlement and failure of the piers and abutments founded on piles,

c) Displacement of the masonry in piers and abutments at horizontal joints in the longitudinal or transverse direction, or in both,

d) Opening of horizontal joints at intermediate piers, accompanied by the inclination and overturning of these parts,

e) Cracks and failure of side walls,

f) The destruction of bearing pads,

g) Differential settlement of piers and abutments situated on soft ground.

As to the behaviour of the superstructure during earthquakes, the shearing-off of the anchor bolts of girders and trusses is one of the most prevalent failures. In many cases, the estimated force necessary for the cutting of these anchor bolts is greater than the weight of the superstructure.

One of the reasons for such failures seems to be the unequal movement of the substructure. When the displacement of the top of the abutment exceeds a certain amount, the girder acts as a strut. The parapet wall is cut off by the strut action of the girder, and in many cases, the anchor-bolts are cut off by a similar action.

Displacement and fall of the superstructure from piers and abutments may occur in either the longitudinal or transverse direction, or both.

CONCLUSIONS

The observed behaviour of bridges during earthquakes is closely connected to the choice of the working scheme of the bridge. The basic requirement for a rational aseismic design of a bridge as a whole, should be in accordance with the location and the topographical and geological conditions. It is also necessary to clarify the general pattern of the seismic vibrations of bridges to develop the dynamic scheme. The design may be based on a rather crude estimate of the expected ground motion.

To resist large earth displacements, bridges should have a substructure with mass and stiffness compatible with the mass and stiffness of the superstructure units. Generally, the design study should result in such a working scheme which closely represents the qualitative character of the vibrations of the structure and simultaneously is simple enough for practical purposes.

Substructure

Aseismic properties of foundations depend on the latter type, and the actual properties of the supporting ground.

A theoretical analysis indicates that for short massive supports, having relatively great transverse dimensions and stiffness, a substantial part of the displacement under vibrations is caused by the inclination of supports due to elastic deformation of the supporting soil. These displacements may be greater than the displacements due to deformations such as bending, or movements of the support, itself. This fact was confirmed by a comparison between the numerical values of the periods of free vibration of supports, obtained by experimental investigations.

During the determination of the periods of free vibration of the bridge supports, in a number of cases it is necessary to consider the elastic deformations of the ground. It will be necessary to develop the design formulas and determine the limits when the accounting of ground yielding has practical meaning.

Much more complicated is the problem regarding the types of deep foundations. In this field, the following problems should be investigated: the depth of the spreading of seismic action on supports, consideration of the inertial forces developed by the deep part of the foundation, the degree of stiffness of supports, and its influence on the intensity of the seismic action.

A similar situation exists with regard to pile type foundations. The different types of piled foundations currently used in practice require special investigation, not only with respect to analytical methods, but also in the light of purposefulness and conditions of their use in the seismic areas.

During earthquakes, oscillating lateral movements of the soil relative to bridges on piles may cause the soils around the upper portions of the piles to break contact with these. This fact should be considered in assigning friction capacities to piles.

Piers and abutments should be designed preferably as reinforced concrete structures having adequate reinforcement in construction joints and sufficiently connected with the foundation to prevent any shifting from it.

Superstructure

The resistance of the bridge superstructure to earthquake damage depends more on the type of framing and detailing of connections and joints than on the structural design. The type of bridge, the position of movable and fixed supports, must be carefully determined after comparing various alternatives regarding aseismity. Actually, as observations indicate, structural details are the critical factors in the earthquake resistance of the superstructure, (Ref. 14).

Apparently, rigid frame type bridges having spill-through abutments have excellent earthquake resistance. They will oscillate with the ground, yet allow differential approach fill movement with little or no damage to the structures. Abutments should have minimum backwall areas in contact with the ground and should be designed for passive earth pressures.

However, rigid frame bridges with retaining-type abutments may be severely damaged by differential ground movements, though the structure may be usable after a major earthquake.

The resistance of these bridges to damage is increased by designing the retaining abutments for passive earth pressures of the soils retained.

A more practical method of preventing damage is to provide elastic sections between the walls and the approach fill, using corrugated sheet metal or expanded polyethylene resistant to chemical attack by the soil.

Multiple span continuous bridges possess high earthquake resistance properties. Bridges of this type should have their bearings fixed against horizontal movements at as many piers as expansion stresses will permit, and relatively free movements at abutments. By these means, the structure will be somewhat isolated from the differential approach fill movements. Damage to these structures by high-intensity earthquakes may range from cracked abutment backwalls and wingwalls to the punching of beams through the backwalls and the tilting and cracking of pier shafts.

Cantilevered bridges should have as many spans as possible fixed against horizontal movements at their piers, and should be detailed to allow expansion at the abutments.

Skewed and curved bridges provide less resistanct to earthquakes compared to the previously considered types.

Longitudinal movements of skewed or curved structures cause differential transverse movements at the superstructure joints. Beam bearings for these structures should be detailed to prohibit any transverse movement. Effective restraint against transverse movement will make these structures as resistant to differential fill movement as comparable square structures.

The ribs and bolts should be so designed that either of them independently should be able to resist the total horizontal force due to an earthquake.

The movable bearings should be designed to allow displacement caused by temperature variation, but no to permit any excessive displacement due to an earthquake. Special stoppers should be provided on both types of bearings to prevent the fall-down of a superstructure.

Other precautions include an increase in the distance between a bearing and the front edge of an abutment or pier. Generally, the top parts of piers and abutments should be sufficiently wide to allow differential longitudinal movements between the superstructure and substructure.

In the case of multiple simple spans, girders or trusses should be connected to each other, as well as to the parapet wall of an abutment, with sufficient clearance between them.

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