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Notice: The conclusions presented in this paper are those of the authors and not necessarily those of any sponsor.

Implementation of the Analytical Capabilities Required for the Aseismic Design of Bridges

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ABSTRACT

The design of a highway bridge located in a region of high seismic risk must include a detailed and accurate analysis of the bridge to determine its maximum anticipated seismic loads. To comply with newly developed code requirements and to ensure the utmost confidence in the predicted response, the seismic analysis should be performed by using the appropriate analytical procedures. The recently developed computer program Seismic Analysis of Bridges (SEISAB) used to conduct seismic analyses that comply with both the current AASHTO specifications and the Applied Technology Council seismic design guidelines is described. In addition, a description is given of the single-mode spectral method developed for the new guidelines for a specific category of bridges with low to moderate seismic vulnerability. An example is included to demonstrate the applicability of this method to a two-span bridge. A second example is included to illustrate how SEISAB-I was used to conduct a response spectrum dynamic analysis on a six-span curved bridge. Included also is a description of the nonlinear dynamic analysis capabilities to be included in the next version, SEISAB-II. The implementation of SEISAB-I through workshops funded by the National Science Foundation and the acceptance of the program based on trainee evaluations are also briefly described.

Both the current AASHTO bridge specifications (1), which were upgraded following the 1971 San Fernando earthquake, and the more recently adopted AASHTO Seismic Design Guidelines for Highway Bridges (2) require that a single-mode or multimode response spectrum analysis be conducted in the design of bridges to be located in zones of higher seismic activity. Because the analytical procedures involved in seismic analyses are new to many bridge designers, it has been difficult to implement these new methodologies within the United States. Recognizing this problem, the National Science Foundation elected to fund a project to develop the computer program Seismic Analysis of Bridges (SEISAB) and conduct pilot workshops to aid in this implementation effort.

In addition to being used as a design tool to facilitate the implementation of the new design codes, SEISAB is also being extended to bring to the profession the nonlinear capabilities that were developed at the University of California, Berkeley, as part of an investigation into the adequacy of bridge structural resistance to seismic disturbances (3). These nonlinear capabilities of SEISAB are being designed for use by the researcher or bridge designer involved in the following design-related activities:

1. Conducting parametric studies to establish procedures and design coefficients for new or improved aseismic design specifications,
2. Conducting detailed dynamic analyses on complex bridges,
3. Investigating newly developed aseismic design strategies that include energy dissipation, and
4. Developing design procedures that include the complex effects of soil-structure interaction.

Extending SEISAB to include both newly developed elements unique to bridges and nonlinear analysis capabilities provides a vehicle for implementing the state-of-the-art methodologies emerging from the universities for the bridge engineering profession.

In line with the primary objective of developing a usable design tool, SEISAB-I was developed by incorporating a problem-oriented language written specifically for the bridge engineer (4,5). The free-format SEISAB language consists of simple, easy-to-remember commands natural to the bridge engineer in describing a bridge. Using a minimum amount of user input data, the program generates a mathematical model completely. SEISAB-I, which contains linear dynamic analysis capabilities, was well received in its initial pilot workshop in which it was presented to a selected group of highly qualified bridge engineers from the California Department of Transportation. Three subsequent workshops that included the use of SEISAB-I for both the design and the retrofitting of bridges were equally successful.

BACKGROUND

FHWA recently sponsored a series of workshops entitled Seismic Design of Highway Bridges to implement the latest principles of aseismic design (6). During these workshops, it was obvious that one of the most complicated tasks for a bridge engineer in attempting to apply these new design principles is conducting the dynamic analysis of the structural system. This problem faces most bridge designers today, whether they use the current AASHTO design specifications or the newly adopted Applied Technology Council (ATC) seismic design guidelines (2). The introduction of structural dynamics to the bridge design process requires that bridge designers learn both the basic principles in dynamics and the use of computer programs having dynamic analysis capabilities. This also implies that the designer has had at least introductory training in the art of mathematical modeling.

Because of the new concepts introduced in the AASHTO and ATC-6 design specifications, a major effort is required to train practicing bridge engineers in the latest principles of seismic design. In addition, if this training is to broaden the base so that further advancements in seismic design can be made, it must stimulate the interest of the profession as a whole.

Although the application of structural dynamics to the bridge engineering field is somewhat in its infancy, it has become apparent that certain types of bridges may be idealized so as to be more easily analyzed mathematically. Penzien and Imbsen developed the single-mode spectral method (SMSM) presented in the ATC-6 guidelines in an effort to simplify the task of implementing structural dynamics within the field of bridge engineering (7).

The SMSM is used to calculate the seismic design forces of a bridge that can be characterized as having its major dynamic response in a single mode of vibration. This method, although quite rigorous from a theoretical point of view, reduces a complex dynamics analysis to the performance of just two statics analyses. The first statics analysis is conducted to obtain the structural period and its corresponding displaced shape, the second to apply inertial forces consistent with that displaced shape. The intensity of the inertial forces is determined from a response spectrum selected for the bridge site by using the calculated structural period.

The SMSM as formulated can be applied to many types of bridges, including those with either continuous or discontinuous superstructures. Boundary conditions at the abutments and piers can be modeled to include the effects of the foundation. A bridge engineer can readily apply the SMSM by using a hand calculator and conventional statics structural analysis procedures. For the more complex bridges in the higher seismic zones, the seismic design guidelines recommend the multimode spectral method (MMSM), which is a response spectrum analysis. The SEISAB-I user has the option of using either the SMSM or the MMSM.

DEVELOPMENT OF THE SMSM FOR CONTINUOUS BRIDGE SYSTEMS

Bridges are generally continuous systems made up of many components, each component having distributed mass and elasticity and contributing to the overall response of the system. The response displacement of continuous systems, such as the one in Figure 1, can be shown at any time to be a linear combination of the individual modes of vibration. Restricting the number of modes to one and recognizing that the true vibration shape is unknown results in the following displacement approximation for transverse displacements:

$$\begin{aligned} v(x,t) &= v_s(x)v(t) \\ &= v_s(x)A \sin(\omega t - \phi) \end{aligned} \tag{1}$$

where

$v_s(x)$ = assumed vibration shape,
$v(t)$ = generalized coordinate representing the amplitude,
A = arbitrary scaling factor, and
ω = circular frequency.

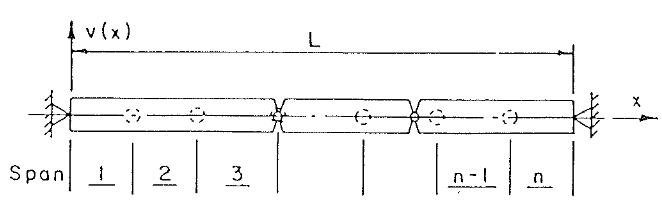

FIGURE 1 Typical bridge configuration.

Because the true mode shape is unknown, the best approximation to it should be obtained.

The process of selecting the closest possible approximation of the shape function, such as the approximation shown in Figure 2, can be facilitated by taking advantage of the free vibration displacements that result from inertial forces. Because inertial forces are proportional to the mass distribution, a transverse distributed load proportional to the mass should produce a good approximation of the true mode shape. Because the mass is usually distributed uniformly in bridge decks, application of a uniformly distributed load, p_o, shown in Figure 3, will displace the bridge deck into the approximate shape of the mode. This method of obtaining an approximating shape results in the consistency of $v_s(x)$ with the support conditions and intermediate expansion joints in the deck.

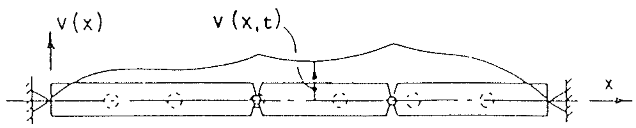

FIGURE 2 Displacement function.

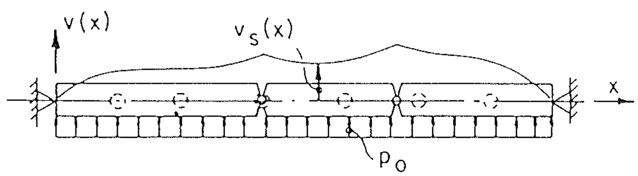

FIGURE 3 Mode shape due to uniform static loading.

Determining Period of Assumed Mode Shape

The vibration period associated with the assumed mode shape can be determined by using Rayleigh's method, which consists of equating the maximum strain energy with the maximum kinetic energy. The maximum strain energy is the stored internal energy resulting from the application of the load p_o and is equal to the work done on the system in displacing the bridge deck into the displaced shape $[v_s(x)]$, which can be expressed mathematically as follows:

$$W_E = (1/2)\int_o^L P_o v_s(x)dx = (P_o/2)\,\alpha \tag{2}$$

where

$$\alpha = \int_o^L v_s(x)dx \tag{3}$$

The kinetic energy (K) is expressed as follows:

$$\begin{aligned} K &= (1/2)\int_o^L m(x)[\dot{v}(x,t)]^2\,dx \\ &= (1/2)\int_o^L m(x)[\omega\cos(\omega t-\phi)v_s(x)]^2\,dx \\ &= (\omega^2/2g)\cos^2(\omega t-\phi)\int_o^L w(x)[v_s(x)]^2\,dx \end{aligned} \tag{4}$$

where $w(x)$ is the weight distribution along the deck. Equation 4 will be at its maximum when $\cos^2(\omega t - \phi)$ is equal to 1, or

$$K_{max} = (\omega^2/2g)\int_o^L w(x)[v_s(x)]^2\,dx = (\omega^2/2g)\,\gamma \tag{5}$$

where

$$\gamma = \int_o^L w(x)[v_s(x)]^2\,dx \tag{6}$$

Equating Equation 2 to Equation 5 and noting that $T = 2\pi/\omega$ results in the following:

$$T = 2\pi(\gamma/P_o\,\alpha g)^{1/2} \tag{7}$$

Pseudoinertial Loading

The maximum value of $v(t)$ can be obtained by applying the concepts of the response spectrum method. The equation of motion for a continuous system approximated by a single generalized coordinate is found by using Hamilton's principle, which states that the first variation of $(K - V)$, where K is the kinetic energy and V is the strain energy, plus the first variation of all nonconservative forces (W_{nc}) is equal to zero. Mathematically, calculating the first variation of I will produce the equation of motion:

$$I = \int_{t_1}^{t_2}(K - V)dt + \int_{t_1}^{t_2} W_{nc}\,dt \tag{8}$$

It can be shown (8) that the first variation of Equation 8 will result in the following:

$$m^*\ddot{v}(t) + c^*\dot{v}(t) + k^*v(t) - p^*_{eff}(t) = 0 \tag{9}$$

where

$$m^* = \int_o^L m(x)[v_s(x)]^2\,dx \tag{10}$$

$$c^* = \int_o^L c(x)\,[v_s(x)]^2\,dx \tag{11}$$

$$k^* = \int_o^L E^*I^*(x)[\partial^2 v_s(x)/\partial x^2]^2\,dx \tag{12}$$

$$p^*_{eff} = -\ddot{v}_g(x,t)\int_o^L m(x)v_s(x)dx \tag{13}$$

$E^*I^*(x)$ is the equivalent bending stiffness of the deck and $\ddot{v}_g(x,t)$ is the horizontal ground acceleration. Dividing Equation 9 by m^*, noting that c^*/m^* is $2\xi\omega$ and k^*/m^* is ω^2, and defining

$$\beta = \int_o^L w(x)v_s(x)dx \tag{14}$$

results in

$$\ddot{v}(t) + 2\xi\omega\dot{v}(t) + \omega^2 v(t) = -\ddot{v}_g(x,t)(\beta/\gamma) \tag{15}$$

By using the response spectrum method with a desired acceleration spectrum, noting that $S_d = S_a/\omega^2$, and given that $C_s = S_a/g$, $v(t)_{max}$ is calculated by the following equation:

$$|v(t)|_{max} = (\beta C_s g/\gamma\omega^2) \tag{16}$$

Substituting Equation 16 into Equation 1, $v(x,t)$ becomes

$$v(x,t)_{max} = (\beta C_s g/\gamma\omega^2)v_s(x) \tag{17}$$

Equation 17 defines the maximum spectral displacements of all points on the bridge deck due to an assumed acceleration spectrum. The pseudoinertial load $(\overline{F}_I)$ that is associated with this displacement and that approximates the inertial effects is found by noting that $S_a = \omega^2 S_d = \omega^2 v(x,t)_{max}$:

$$\begin{aligned} \overline{F}_I &= m(x)a \\ &= m(x)S_a \\ &= m(x)\omega^2 S_d \\ &= m(x)\omega^2 v(x,t)_{max} \\ &= [C_s\beta w(x)/\gamma]\,v_s(x) \end{aligned} \tag{18}$$

When the inertial load defined by Equation 18 is applied to the deck as a uniformly tranverse distributed load, as shown in Figure 4, the resulting static forces become the pseudoseismic forces.

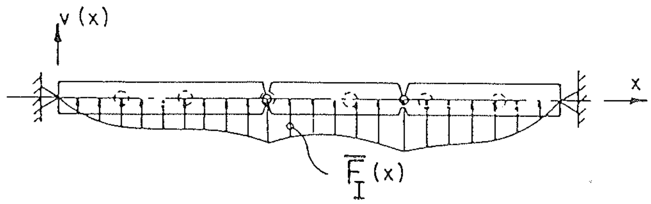

FIGURE 4 Pseudoinertial loading.

Summary of SMSM Procedure

The SMSM procedure is performed in the following steps:

1. Apply a uniformly distributed load (p_o) transversely to the bridge deck and calculate the displacements of the deck. The displacements will define $v_s(x)$.
2. Using $v_s(x)$, calculate α, γ, and β by using Equations 3, 6, and 14, respectively.
3. Calculate the period of the approximating vibration shape by using Equation 7.
4. Select an acceleration spectrum with damping ratio ξ and compute the dimensionless seismic coefficient (C) associated with the period calculated in step 3. Use C to compute the pseudoinertial load by using Equation 18.
5. Apply the pseudoinertial loading transversely to the bridge deck and compute the displacements and forces for design.

Comparison of SMSM and MMSM on Two-Span Bridge

The applicability of the SMSM can be demonstrated by comparing the results obtained from its use with those obtained from using the MMSM. The South Turlock Overcrossing is used for the comparison. The two-span straight bridge, which is supported on a single-column bent, is subjected to transverse earthquake loadings. Two separate test cases are considered for the transverse constraints at the abutments. In the first case, the abutments are fixed in the transverse direction, whereas in the second case, springs in the transverse direction are inserted to model the flexibility of the soil at the abutments. A spring coefficient of 1.0 x 10 kips/ft is used for the soil flexibility. Longitudinal movement of the superstructure is permitted in both cases.

SEISAB is used to perform the SMSM and the MMSM for both cases. The results of the SMSM and MMSM response analyses are tabulated for test cases 1 and 2 in Tables 1 and 2, respectively. The coefficients obtained by evaluating Equations 3, 6, and 14 for the SMSM are included in the tables. The first 10 modes are included in the results for the MMSM. The first transverse mode of response from the MMSM is also included in the tables for comparison.

The results from both test cases show that the structural period was closely approximated by using the SMSM. In addition, the SMSM calculations of the transverse shear force at the abutments and the bent are also close to the forces obtained by the MMSM.

TABLE 1 Test Case 1 Comparison

				Response Spectrum (kips)		Normalized Transverse Displacements		
Location	Displacement Due to p_o[Y(x)] (ft)	Pseudoinertial Loading ($\bar{F}_I$) (kips/ft)	Forces Due to $\bar{F}_I$ (kips)	First Transverse Mode	Three Transverse Modes (RMS)	Uniform Load p_o	Inertial Load $\bar{F}_I$	First Transverse Mode Shape
Abutment 1	0.0	0.0	278	287	291	0.0	0.0	0.0
Span 1								
One fourth	0.0122	3.582				0.524	0.499	0.488
One half	0.0203	5.960				0.873	0.846	0.839
Three fourths	0.0233	6.841				1.000	0.992	0.989
Bent 2	0.0232	6.811	1,045	1,134	1,134	0.988	1.000	1.000
Span 2								
One fourth	0.0233	6.841				1.000	0.922	0.989
One half	0.0203	5.960				0.873	0.846	0.839
Three fourths	0.0122	3.582				0.524	0.494	0.488
Abutment 3	0.0	0.0	278	287	291	0.0	0.0	0.0

Note: $\alpha = 5.460\ ft^2$, $\beta = 34.644\ kip{\cdot}ft^2$, $\gamma = 0.749\ kip{\cdot}ft^3$, $T_{SMSM} = 0.41$ sec ($T_{MMSM} = 0.40$ sec).

TABLE 2 Test Case 2 Comparison

				Response Spectrum (kips)		Normalized Transverse Displacements		
Location	Displacement Due to p_o[Y(x)] (ft)	Pseudoinertial Loading ($\bar{F}_I$) (kips/ft)	Forces Due to $\bar{F}_I$ (kips)	First Transverse Mode	Three Transverse Modes (RMS)	Uniform Load p_o	Inertial Load $\bar{F}_I$	First Transverse Mode Shape
Abutment 1	0.00707	2.066	355	301	349	0.286	0.221	0.201
Span 1								
One fourth	0.01701	4.971				0.688	0.636	0.616
One half	0.02316	6.768				0.937	0.911	0.900
Three fourths	0.02472	7.224				1.000	1.000	1.000
Bent	0.02415	7.057	1,196	1,253	1,254	0.977	0.988	0.993
Span 2								
One fourth	0.02472	7.224				1.000	1.000	1.000
One half	0.02316	6.768				0.937	0.911	0.900
Three fourths	0.01701	4.971				0.688	0.636	0.616
Abutment 3	0.00707	2.066	355	301	349	0.286	0.221	0.201

Note: $\alpha = 6.600\ ft^2$, $\beta = 41.877\ kip{\cdot}ft^2$, $\gamma = 0.9093\ kip{\cdot}ft^3$, $T_{SMSM} = 0.41$ sec ($T_{MMSM} = 0.41$ sec).

SEISMIC ANALYSIS OF SIX-SPAN CURVED BRIDGE WITH SEISAB

SEISAB was developed to meet the need for a computer program with MMSM capabilities written specifically for bridge designers. By using SEISAB, a complete, lumped-parameter structural model can be generated with only a few simple, free-form input commands. To illustrate the use of SEISAB, a response spectrum analysis was performed on a six-span curved bridge. An ATC-6 acceleration spectrum was used for the dynamic loading.

Description of the Bridge

The bridge is six-span curved box-girder bridge with single-column bents. The prismatic superstructure is continuous, with the exception of span 3, which contains an intermediate hinge. The intermediate hinge is outfitted with earthquake restrainer units to provide longitudinal restraint. Shear keys at the hinge provide transverse restraint between the two superstructure sections.

The seat-type abutments are radially oriented with transverse abutment-to-superstructure shear connections. Longitudinal restraint at the abutments is provided by restrainer units. The radially oriented, single-column bents are founded on pile groups.

Modeling and Program Input Details

As is conventionally done, the SEISAB program models a bridge by lumping properties at discrete locations along the superstructure and columns. The structural characteristics of the bridge are input into SEISAB in modular blocks called input data blocks. The subheads in this section are arranged according to data blocks to illustrate the SEISAB commands required to conduct a seismic analysis of a six-span curved bridge.

Initiating a Response Spectrum Analysis

The user initiates a response spectrum analysis by specifying the appropriate command in the SEISAB data block. In addition, the number of intermediate node points to be used on the superstructure and columns (i.e., the degree of accuracy of the analysis) may also be specified. Because the curved geometry of this bridge would result in coupling effects, the default number of three nodes on the superstructure was increased to 4. The input in the SEISAB data block is as follows:

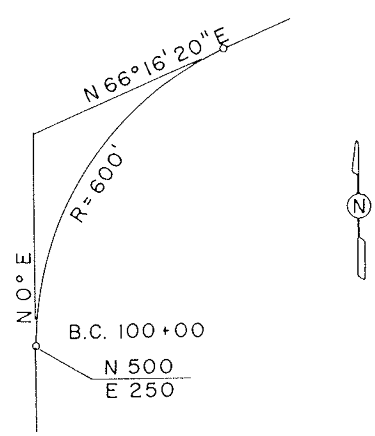

FIGURE 5 Horizontal alignment.

```
SEISAB 'RESPONSE SPECTRUM ANALYSIS, 6-SPAN CURVED
  BRIDGE'
RESPONSE SPECTRUM
SUPERSTRUCTURE JOINTS 4
```

Describing the Horizontal Geometry

To develop an accurate model, the location of the bridge centerline must be described correctly. This information is supplied to SEISAB in the ALIGNMENT data block. Alignment information for this bridge was taken directly from bridge plans and input into SEISAB. The alignment of the bridge is shown in Figure 5, and the input for the ALIGNMENT data block is as follows:

```
ALIGNMENT
C                               INITIAL REFERENCE POINT
STATION      100 + 0.0            INFORMATION
COORDINATES  N 500.0            E 250.0
BEARING      N 0 E
C                               CURVE INFORMATION
BC        10000.0
RADIUS    R 600.0
BEARING   N 66 16 20 E
```

Superstructure

The stiffness and mass characteristics of the superstructure were obtained from its cross-sectional properties. Because the spans are prismatic, only the properties of span 1 were input. The torsional moment of inertia was calculated by using expressions based on thin-walled, enclosed regions. The input for the SPAN data block is as follows:

```
SPANS
LENGTHS   100.0, 143.0, 3*117.0, 100.0
AREAS       86.0   $ PROPERTY GENERATION WILL BE USED
I11        862.0   $ FOR SPANS 2-6.  ALSO, PROGRAM
I22      13000.0   $ DEFAULTS WILL BE USED FOR THE
I33        360.0   $ MODULUS AND DENSITY.
```

Defining the Structural Members

Another user input feature of SEISAB is that any structural member that appears at more than one location in the bridge is described once in the DESCRIBE data block and then placed at the appropriate locations. The structural members in the six-span bridge that required defining are the bent columns and the longitudinal restrainers. Because the five columns are identical in cross section, only one needed to be defined. The input in the DESCRIBE data block is as follows:

```
DESCRIBE
COLUMN     'TYPE 1'    "TYPICAL PRISMATIC COLUMN"
AREA        33.0
I11        146.0
I22         73.0    $ PROGRAM DEFAULTS WILL BE USED
I33        143.0    $ FOR THE MODULUS AND DENSITY
RESTRAINER 'TYPE 1' "GALV. H.S. ROD"
LENGTH       5.0
AREA         3.068E-03
E            2.010E+06
RESTRAINER 'TYPE 2' "GALV. STEEL CABLE"
LENGTH      20.0    $ PROGRAM DEFAULTS WILL BE USED
AREA         0.01   $ FOR THE MODULUS
```

Abutment Information

The modeling of the bridge's two abutments was accomplished through the ABUTMENT data block. The connectivity between the superstructure and the abutment was assumed to provide translational constraint in the transverse and vertical directions and rotational constraint about a horizontal axis perpendicular to the centerline of the abutment. The shear keys provided the translational constraint, and the width of the superstructure was assumed to provide the torsional constraint. The input in the ABUTMENT data block is as follows:

```
ABUTMENT STATION  100 + 0.0
ELEVATION         152.5  155.5
WIDTH NORMAL      35.0   $ GENERATION IS USED FOR
                           ABUT 7
RESTRAINER NORMAL LAYOUT 'TYPE 1' 8.0, 8.0 'TYPE 1'
  AT 1,7
```

Bent Information

The number, type, and spacing of bent columns are specified in the BENT data block. In addition, the user may also input into this data block the type of connectivity to the superstructure, the column end conditions, and the locations of any restrainers. The bridge under consideration has only single-column bents, and the columns are oriented radially to the superstructure. The column end conditions are fixed at both ends. Many program defaults in the BENT data block have been used for this bridge. The required input for the BENT data block is as follows:

```
BENT
ELEVATION TOP   153.0, 153.5, 154.0, 154.5, 155.0
HEIGHT            25.0  $ HEIGHTS GENERATED FOR OTHER
                              BENTS
COLUMN  'TYPE 1'   AT 2 3 4 5 6
```

Foundation Information

Modeling the connection of the columns and abutments to the foundation may be accomplished either by assuming complete fixity or by allowing for a flexible support. Complete fixity is a program default, whereas movement of the column bottoms or abutments or both is allowed by modeling soil as uncoupled springs. These soil springs are input into the FOUNDATION data block. The direction of the springs is normal and tangential to the centerline of the bent. The input for the FOUNDATION data block is as follows:

```
FOUNDATION
   AT BENT   2 3 4 5 6
       KF1  4.084E+08
       KF2  4.084E+08
       KM1  2.704E+10
       KM2  1.292E+10
       KM3  2.220E+10
```

Span Hinge Information

Discontinuities in the superstructure between bents are input into the HINGE data block. The mathematical modeling of the expansion joint or hinge is done by using a special zero-length element that has the unique property of being able to release the moment along the centerline of the hinge. Translational connectivity is specified for a horizontal axis perpendicular to the centerline of the superstructure at the location of the hinge. In addition, longitudinal restrainers may be placed across the hinge.

Because the joint has transverse shear keys, the transverse force condition is input as fixed. Longitudinally, the only restraint is provided by the restrainers. The width of the bridge is sufficient for transmitting torsional moment across the hinge. The input for the HINGE data block is as follows:

```
HINGE
AT   3   102.00          $ HINGE IS IN SPAN 3;  102 FT
WIDTH NORMAL   33.5          FROM BEGIN.
TRANSVERSE FIXED
REST NORMAL LAYOUT 'TYPE 2' 4.5,4.0,4.0,4.5 'TYPE 2'
  AT 1
```

Earthquake Information

The last data block, LOADINGS, specifies information about the loads applied to the bridge. The required loading for a response spectrum analysis is an acceleration spectrum. The SEISAB program has the ATC-6 spectra stored away; therefore, because the default is not applicable in this case, the only input needed to define the acceleration spectrum is the soil type. Soil type 3 (30 ft or more of soft to medium stiff clays) is present at the bridge site. Two loading cases are desired, one along an axis connecting the two abutments (in a chord or longitudinal direction) and one transverse to that axis. Because both of these loading cases are required by ATC-6, they are included in SEISAB as a program default and no input is needed. The input for the LOADINGS data block is as follows:

```
LOADINGS
   RESPONSE SPECTRUM
   SOIL TYPE III
```

EXTENDED NONLINEAR DYNAMIC-ANALYSIS CAPABILITIES OF SEISAB-II

In regions of high seismicity it has generally not been economically feasible to design and build bridges that resist earthquake loads elastically. Thus, in order to achieve acceptable performance, designers have relied on the postelastic behavior of certain components. This has generally meant that columns or piers could be expected to yield during a major earthquake. This design strategy also requires that other nonductile components, such as bearings, be designed to resist seismic forces elastically.

Recently, however, there has been growing interest in using different design and retrofitting strategies in regions of high seismicity (9,10). These strategies, which use concepts such as isolation, energy dissipation, and restraint, often employ special bearing devices designed to behave nonlinearly during a major earthquake. However, many of these design strategies are relatively new and lack histories of performance during an earthquake. In addition, the effect of these strategies cannot be adequately evaluated by experimental research that investigates only the performance of isolated components. Because full-scale testing of prototype designs is expensive, such testing is not usually economically feasible. Therefore, analytical techniques must be relied on if these new aseismic design strategies are to be properly evaluated. In many cases the analytical methods currently used to evaluate these strategies are based on simple, single-degree-of-freedom idealizations of a given bridge. However, the geometry and articulation of most bridges makes the validity of such simple idealizations questionable, especially in view of the presence of other nonlinear components in the bridge. To properly evaluate these new strategies and identify those situations where they will be the most beneficial, bridge designers must be able to realistically analyze the true nonlinear behavior of various types of bridge structures employing many different design and retrofitting strategies.

Nonlinear dynamic-analysis capabilities would also facilitate much of the research recommended in recent workshops on the seismic aspects of bridge design (11,12). The objectives of much of this recommended research could be accomplished more efficiently if such analytical capabilities were readily available in a form that practicing bridge engineers could use.

The nonlinear program SEISAB-II will consider, along with the nonlinear behavior of bridge bearings, the effects of column flexural yielding and the formation of plastic hinges. An efficient method for considering column yielding in a finite-element computer program is to use nonlinear beam elements in which flexural yielding can occur at the ends of each element. An axial load and biaxial moment interaction yield surface can be described by using the conventional theory of ultimate strength of reinforced concrete. By assuming a transition from ideally elastic behavior to ideally plastic behavior at the yield surface, engineers can write (and have written) algorithms that include the nonlinear behavior of reinforced bridge columns (13-15).

CONCLUSIONS

Previous efforts by Imbsen et al. (6) to implement computer programs capable of assessing the dynamic response of bridges indicated that there was a need to develop both simplified methodologies and a computer program written specifically for bridge designers.

A simplified procedure such as the SMSM, which is applied by using conventional techniques of statics analysis, is easily understood. The procedure is applicable to bridge configurations that can be characterized as having their major dynamic response in the first mode of vibration. The current AASHTO seismic design guidelines (2) recommend that this method be used on such bridges in zones of both moderate and high seismic activity (i.e., seismic performance categories C and D). The guidelines also recommend that the SMSM be used on all bridges in a zone of moderately low seismic activity (i.e., seismic performance category B). The SMSM, included in SEISAB, was formulated by using the pseudo-dynamic-analysis procedures described in this paper. This procedure was included in SEISAB-I to provide the bridge designer with an easy-to-use analytical tool capable of handling space-frame structures required for lateral static loadings.

For the more geometrically complex bridges (e.g., curved alignments, varying column lengths, highly skewed supports) the response spectrum method (i.e., the multimode response method) is recommended as a minimum for the response analysis. Although most general frame analysis computer programs (e.g., STRUDL, SAP, EASE, and NASTRAN) have the capabilities needed to conduct an adequate response spectrum analysis, they tend to be quite difficult for most bridge designers to use. To model a bridge and select the appropriate commands from the ensemble of commands typically available in these general analysis programs, a designer must have a working knowledge of structural dynamics. In addition, boundary conditions at intermediate expansion joints and supports on a skew are difficult to model by using these general analysis programs. A special element developed by Tseng and Penzien (13) for intermediate hinges has been incorporated into SEISAB-I to model force releases that will accommodate movements along the bridge centerline and moment releases that are directed along a skewed support centerline nonorthogonal to the bridge centerline. In addition, because SEISAB-I has been developed specifically for bridges, it automatically generates a bridge model that simulates the inertial characteristics of a vibrating bridge. A special element has also been developed and included in SEISAB to model the stiffness characteristics of a bent cap embedded in the superstructure. Other features that are unique to bridges have also been included in the program to make it a convenient, easy-to-use program for conducting a seismic analysis of a bridge. SEISAB's output results have been tailored and formatted to report only those qualities needed for model verification and design.

Pilot workshops sponsored by the National Science Foundation were given initially to bridge engineers in California who were familiar with seismic design and subsequently to engineers less familiar with seismic design. Both groups of engineers indicated an overwhelming acceptance of SEISAB-I. The availability of SEISAB-I concurrent with AASHTO's adoption of seismic design guidelines and completion of the ATC seismic retrofitting guidelines (ATC-6-2) has also contributed to the acceptance of SEISAB. The four 2-day intensive workshops that have been conducted to date have included hands-on experience in using SEISAB on problem assignments for both seismic design and retrofitting. Because these workshops are geared toward the engineer involved in bridge design on a day-to-day basis, they fill a specific need by helping to equip bridge engineers with the skills needed to apply the newly developed methodologies and guidelines for aseismic bridge design.

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Prototype Prestressed Wood Bridge

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ABSTRACT

The transverse prestressing of wood was conceived of in 1976 as a method for rehabilitating nailed laminated wood decks. Using high-strength prestressing steel, a permanent pressure is introduced normal to the direction of the laminations to provide high interlaminate shear strength and improved load distribution. The success of this new concept in rehabilitation resulted in its becoming the subject of a major research and development program conducted by the Ontario Ministry of Transportation and Communications (MTC). The extensive work performed by MTC over the past 7 years has led to the formulation of a set of comprehensive design specifications for prestressed wood. The objective of this paper is to outline the design, construction, and load testing of the world's first new prestressed wood bridge. The bridge was designed by MTC and constructed by the Ontario Ministry of Natural Resources (MNR) over the West River, on a logging access road, near Espanola, Ontario, in 1981. The design process with reference to the new design specifications, which have since been adopted by the Ontario Highway Bridge Design Code, is discussed. The field construction is outlined highlighting the prefabrication and assembly of the prestressed wood superstructure. The load testing of the bridge in 1982 and the subsequent evaluation of the test results are described. The MNR determined that the West River bridge cost only two-thirds of the steel structure originally proposed for that site. The load testing and subsequent evaluation indicated that this prestressed wood bridge is an extremely rigid structure with considerable reserve strength.

The transverse prestressing of laminated wood decks was conceived of in 1976 (1) as a method of rehabilitating existing nailed decks. The success of this new concept in rehabilitation resulted in a major research and development program (2) conducted by the Ontario Ministry of Transportation and Communications (MTC). Extensive research and development work led to the formulation of a comprehensive set of design specifications (3,4) devoted entirely to the design of prestressed wood decks. These new specifications have been included in the 1983 edition of the Ontario Highway Bridge Design Code (OHBDC) (4).

To evaluate the effectiveness of these new specifications, MTC and the Ontario Ministry of Natural Resources (MNR) designed and constructed the first new prestressed wood bridge in 1981. The objective of this paper is to outline the design, construction, and load testing of this prototype prestressed wood bridge.

The design analysis, with reference to the new OHBDC specification, and several computer analysis techniques are described in this paper. The fabrication and erection procedures are also outlined with particular emphasis on the field construction conducted by the MNR field construction crew. The load testing and subsequent evaluation of the completed bridge, performed by MTC in 1982, are also summarized.

STRUCTURAL DESCRIPTION

The main objective of the structural selection was to optimize the use of the prestressed wood concept while minimizing on-site construction requirements. The use of this prototype to demonstrate the design flexibility of the prestressed wood system was of secondary importance.

The bridge is located on the MNR Fox Lake logging access road near Espanola, Ontario. It is believed this bridge, which spans the West River near the