SEISMIC ANALYSIS OF CABLE-STAYED BRIDGES

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SUMMARY

This paper discusses the results of a series of static and dynamic analyses of cable-stayed bridge structures. It is shown that, if a nonlinear analysis is used to obtain the structure stiffness in the dead load deformed position, a linear static or dynamic analysis may be used for normal design loads. This indicates that linear dynamic analysis procedures, such as the Response Spectrum Method, are applicable to this type of structure.

INTRODUCTION

In a cable-stayed bridge, the roadway is supported elastically at specific points by inclined cable stays which are attached directly to tall towers. One of the main differences encountered in the analysis of a cable-stayed bridge, compared to more conventional structures such as continuous girder bridges or rectangular framed buildings, is the possiblility of significant nonlinear behavior. Most design engineers lack experience with nonlinear systems, therefore, they might be hesitant to undertake the design of a structure of this type. In order to compensate for this inexperience and to facilitate the expanded use of cable-stayed bridges by designers, information must be made available concerning their behavior under various types of design loads.

NONLINEAR STATIC ANALYSIS

Under normal design loads, the material in a cable-stayed bridge can be considered to remain elastic, however, the overall load-deformation relationship can still be nonlinear. Three primary sources of nonlinear behavior have been proposed by previous investigators. These are: the nonlinear axial force-elongation relationship for the inclined cable stays; the interaction of the bending deformations and high axial forces in the towers and longitudinal deck members; and the geometry changes caused by the large displacements which can occur in this type of structure under normal design loads.

In order to investigate the importance of each of these possible sources of nonlinear behavior, a number of static analyses were performed, at the University of Pittsburgh, on mathematical models which represented several

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different actual or proposed bridges. The analyses were performed by the Stiffness Method using a combined incremental and iterative approach, in which the stiffness of the structure was recomputed after each load increment was applied. Iterations were continued until the unbalanced joint loads were within an acceptable limit. A general three dimensional static analysis computer program has been developed which uses this approach. The program is written in FORTRAN and is presently running on the University of Pittsburgh DEC PDP-10 Computer System.

Due to lack of space, it will not be possible to present the results of all of the various static analyses which have been performed, however, the overall results were similar for each different mathematical model which was considered. As a typical example, selected results for the mathematical model shown in Figure 1 will be discussed. This particular mathematical model has the cables situated in two planes and has structural properties similar to the Luling Bridge in Louisiana.

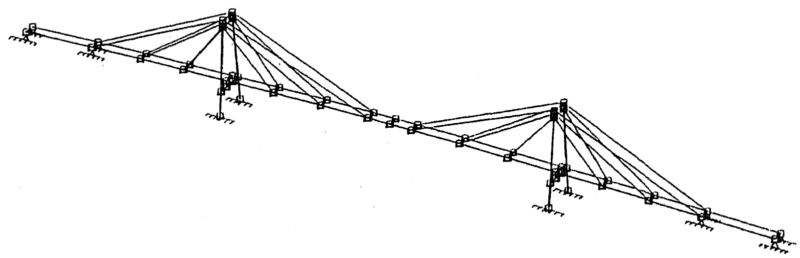


Figure 1 - Mathematical Model

The effect of the change in geometry, as the structure deformed under the applied loads, was incorporated in the analysis by revising the geometry of the mathematical model after each load increment was applied. The structural stiffness was then recomputed using the revised geometry. The structure was considered to be subjected to a uniform deck load and a set of initial cable tensions. A wide range of initial cable tensions and deck loads were investigated. The results of these analyses, and similar results for other quantities such as deflection of the towers or the moments in the deck, show that the effect of the change in geometry of the structure is small, for normal design loads, and can be neglected without significantly affecting the computed behavior of the structure (Ref. 1).

The effects of the interaction of the bending deformations and high axial forces, in the deck and tower members, were incorporated in the analysis by introducing stability functions as multipliers to modify the terms in the individual member stiffness matrices (Ref. 2). The stability functions were recomputed at the beginning of each load increment to correspond to the actual axial forces and bending moments in the members. For all cases considered, over a wide range of conditions, the computed stability functions varied by less than three percent from a value of 1.0, and for most cases the variation was less than one percent (Ref. 3). Since the stability functions are used as multipliers, a value of 1.0 corresponds to no change in the structure stiffness. Therefore, it can be concluded that, eventhough the longitudinal deck members and towers are subjected simultaneously to high axial forces and bending moments, the effect of the interaction of these quantities upon the overall stiffness of the structure is small.

The final nonlinear effect to be considered is the overall change in the stiffness of the structure due to the variation of the axial stiffness of the cables as the tensions in the cables change under the applied load. The change in the cable stiffness was incorporated in the analysis by using an equivalent cable modulus of elasticity, which combines the effect of both the deformation resulting from material strain and the deformation resulting from the change in sag in the cables, as suggested by Ernst (Ref. 4). By using the equivalent modulus, the cables can be treated as normal tension members. The change in the structure stiffness, due to the nonlinearity in the cables, can be considered by recomputing the equivalent modulus for each cable to correspond to the tension in the cable at the beginning of each load increment.

Figure 2 shows the variation of the normalized vertical deflection at center span with the uniform deck load for the mathematical model shown in Figure 1. The individual curves correspond to different values of the initial

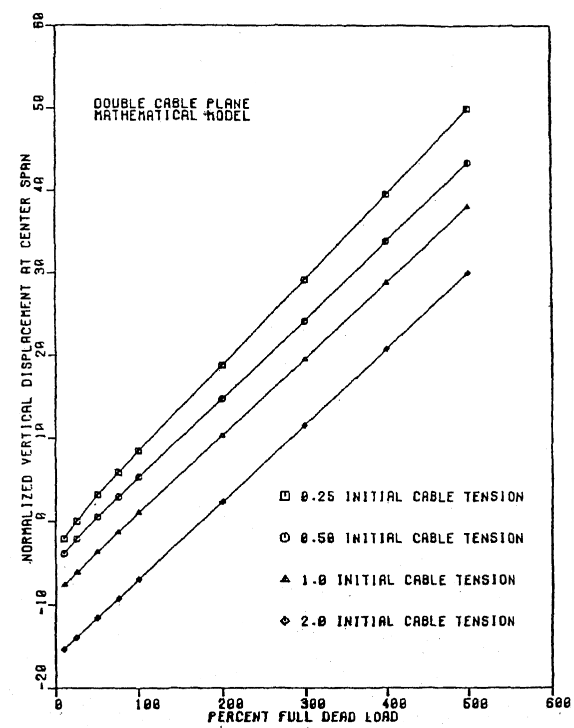


Figure 2 - Effect of Cable Nonlinearity

cable tensions, ranging from 0.25 to 2.0 times the full design values for each cable. These curves show that the load-displacement relationship is nonlinear for low values of the uniform deck load, however, as the load is increased, the relationship becomes more linear. For loads equal to the full dead load or greater, the relationship is essentially linear for all initial cable tensions considered.

The results of these static analyses lead to the conclusion that a cable stayed bridge structure does behave in a nonlinear manner for low loads, however, after the full dead load deformed position has been reached, the structure can be considered to behave linearly. Therefore, the stiffness of the structure, which is represented by the slope of the curves shown in Figure 2, can be considered to be constant during the application of the live load. This indicates that linear static analysis techniques are applicable to this type of structure, starting at the dead load deformed position.

NONLINEAR DYNAMIC ANALYSIS

Many of the cable-stayed bridges which have been constructed, or are presently in the design or proposal stage, are in active seismic regions. This introduces some serious problems to the designer, since very little information is available in the engineering literature concerning the response of cable-stayed bridges to seismic or other types of dynamic loads.

In a study, which was conducted at the University of Pittsburgh, a number of time history analyses were performed for a simplified mathematical model of a cable-stayed bridge (Ref. 5). Several different loadings were considered, which consisted of: the vertical component of the May 18, 1940 El Centro, California earthquake; a simulated wind loading; and a moving traffic load. The lumped mass mathematical model which was considered was a single load bearing plane of a bridge with a geometry similar to the bridge shown in Figure 1. A finite time step numerical integration procedure was used to solve the dynamic equations of motion of the mathematical model starting at the dead load deformed position.

Three distinct types of analyses were performed, consisting of the following combinations of static analysis and dynamic analysis: linear static analysis, to compute the structure stiffness in the static dead load deformed position, and linear dynamic analysis, in which the stiffness was assumed to remain constant as the structure deformed due to the dynamic loads, hereafter denoted as a Linear-Linear analysis; nonlinear static analysis, using the combined incremental and iterative analysis procedure described previously, and linear dynamic analysis, hereafter denoted as Nonlinear-Linear; and nonlinear static analysis and nonlinear dynamic analysis, in which the stiffness of the structure was changed corresponding to the cable tensions and member loads at the end of each dynamic time step, hereafter denoted as Nonlinear-Nonlinear.

Due to lack of space, only one set of results will be presented here. Figure 3 shows the computed variation of the undamped vertical displacement of the deck at the center of the middle span, due to the El Centro earthquake ground motion. It can be seen that the Nonlinear-Linear and Nonlinear-Nonlinear analyses give almost identical results, which vary considerably from the Linear-Linear analysis. The results of all of the analyses, which have been performed, indicate that although a nonlinear static analysis is necessary to obtain the stiffness of the structure in the dead load deformed position, a linear dynamic analysis will suffice starting at this position. This is an important conclusion since a linear time history dynamic analysis is much simpler and more economical to perform than a nonlinear analysis. Also, this suggests that linear dynamic analysis techniques, such as the Response Spectrum Method, are applicable to this type of structure.

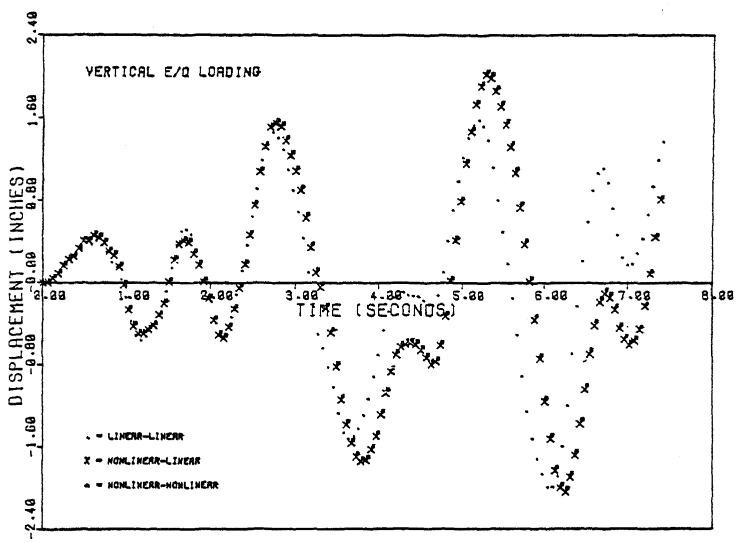


Figure 3 - Time History Analysis

RESPONSE SPECTRUM METHOD

The Response Spectrum Method is a very popular tool for predicting the response of complex structural systems to earthquake ground motions. The general procedure is to compute the response of each of the structures individual modes and then to combine these responses to obtain the overall response. In many cases, only a few of the modes must be included in computing any particular response of the system. The specific modes which must be considered will depend upon the properties of the structure and the particular quantity which is being computed.

The natural frequencies and mode shapes, of the mathematical model shown in Figure 1, were computed by a standard eigenvalue analysis. Figure 4 shows the mode shape for the fundamental frequency. In this mode, the movement consists primarily of vertical translation of the bridge deck, at a frequency of 0.321 cycles per second. At any instant, all points on the deck are translating in the same direction, and the shape is symmetric about the center of the middle span. The movements of the tops of the towers are very small in this mode.

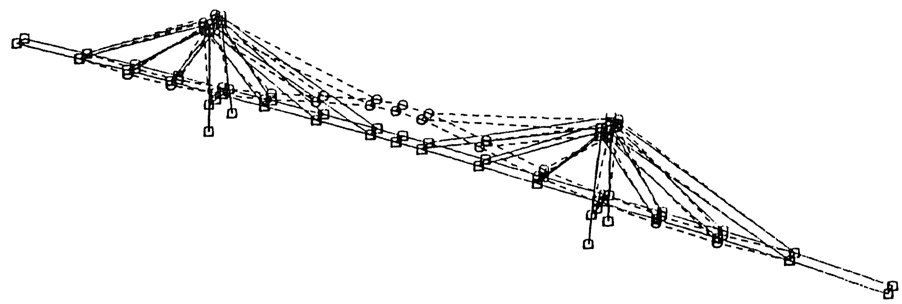


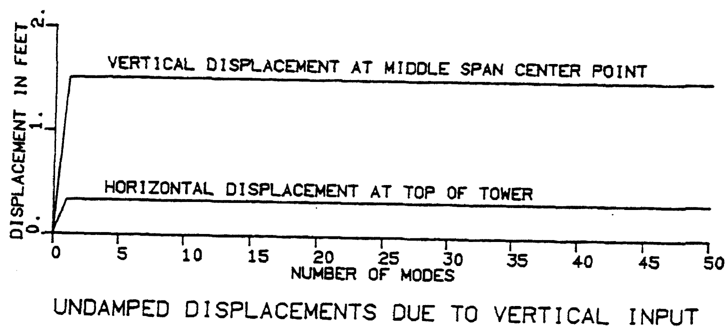
Figure 4 - Fundamental Mode Shape

In the first seven modes, the primary movement consists of various forms of vertical and transverse displacements of the bridge deck. Very little movement can be observed at the tops of the towers, except for that caused by the rotation of the connection between the tower and the deck. The first mode to exhibit any significant bending in the towers is the eighth mode. After approximately the fifteenth mode, the movements become localized and are usually due to axial deformation in a particular member.

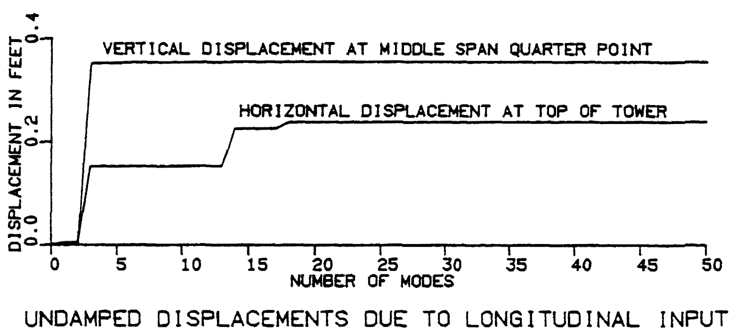
It appears, from the mode shapes of this bridge, that the first few modes will be very important in predicting the displacements and bending moments in the bridge deck, however, they will not be sufficient to predict the displacements and moments due to bending in the towers. The majority of the tower bending will not be picked up unless at least the eighth mode is included in the analysis.

The next step in this investigation was to use the Response Spectrum Method to analyze this bridge, utilizing the computed frequencies and mode shapes. The specific ground motion which was used was that for the May 18, 1940 El Centro, California earthquake. This earthquake was used since it has been used by many other investigators for other types of structures. The response spectra for the three measured components of this earthquake have been developed by the Earthquake Engineering Research Laboratory of the California Institute of Technology (Ref. 6).

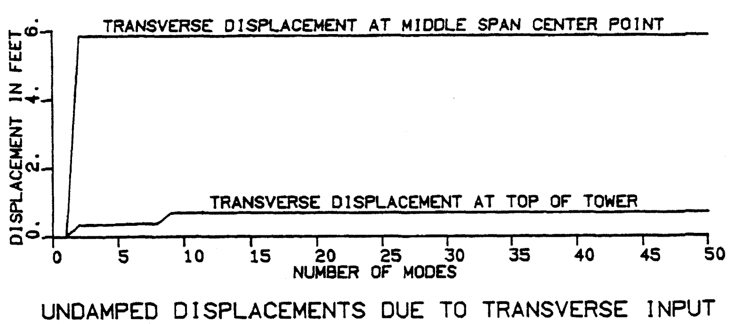
Figures 5a, 5b and 5c show the translation at the top of one of the towers and the translation of a point on the middle span deck, which were obtained from the response spectrum analyses, for the three earthquake components. Figure 5a shows the effect of the vertical earthquake component, while Figures 5b and 5c show the effect of the two horizontal components acting in the longitudinal and transverse directions on the bridge. In each figure, the vertical axis is the displacement, which was obtained by using the square root of the sum of the squares combination procedure for each mode, while the horizontal axis is the number of modes considered. The authors realize that various procedures have been recommended in the literature for



(a)



(b)



(c)

Figure 5 - Response Spectrum Analysis

combining the individual modes. This procedure was used here since it has been used by a number of different investigators for a variety of different structures. The primary purpose of the analyses presented here was not to investigate the actual level of stress in a cable-stayed bridge, but rather to study the contribution of the various modes to the overall response.

It can be seen from these plots that the major contribution to translation of the bridge deck comes from the first few modes, while the translation of the tops of the towers is significantly affected by the higher modes. These analyses show that extreme care must be exercised when applying the Response Spectrum Method to cable-stayed bridge structures since various components can be excited by different modes. For some responses, only the first few modes must be considered, while for other responses, the higher modes make the greatest contribution.

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