VALIDATED ANALYSIS OF WHEELING SUSPENSION BRIDGE

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ABSTRACT: The historic Wheeling suspension bridge over the Ohio River is one of the 57 long-span suspension bridges in operation in the United States. The bridge was designed and constructed in 1849 by Charles Ellet, Jr., who rightfully may be considered the father of the modern American suspension bridge. Despite the fact that the bridge was not designed to carry today's live loads, the bridge has served as a vital link across the Ohio River for nearly a century and a half. This paper represents the first comprehensive structural analysis using modern computer techniques and in situ nondestructive testing evaluation of this bridge. Static analysis showed that deflections and stresses caused by present-day loading conditions are within allowable limits. Rating factors demonstrated that none of the structural members are overstressed by the posted live load. Seismic analysis to the American Association of State Highway and Transportation Officials loads showed little damage, which was confined to floor beams at the east tower. Analysis using historic earthquakes showed localized damage of floor beams and diagonal floor ties at the east tower and top chords of the stiffening truss at midspan. The methodology developed could be applied to a wide range of cable suspension bridges.

INTRODUCTION

Most of the suspension bridges built in the 19th century were designed for live loads quite different from those they have to carry today. Any suspension bridge may have to carry occasional overloads, as well as loads from natural causes such as strong winds and seismic loads that were not anticipated in the original design (Ulstrup 1993). Because many of these bridges are in use today, it is necessary to evaluate their condition and identify necessary stiffening and strengthening to bring them up to modern standards of safety while still respecting the historic fabric of the bridge. The process requires in situ testing of the system coupled with analysis.

The most frequently used classical theories for static analysis of suspension bridges are the elastic theory and the deflection theory (Steinman 1929). In addition to these an approximate method by Tsien (1949) also can be used to get practical answers that are sufficiently accurate for design purposes (Ulstrup 1993). The major distinction between the elastic theory and deflection theory is that the deflection theory does not assume that the ordinates of the cable curve remain unaltered on application of loading (Steinman 1929). The elastic theory is sufficiently accurate for shorter spans. However, the deflection theory is the most accurate and results in more economical and slender bridges.

Determining the earthquake response of historic long-span suspension structures is always a major problem for engineers (Castellani 1987). Research on lateral earthquake response of suspension bridges (Abdel-Ghaffer 1983), vertical seismic behavior of suspension bridges (Abdel-Ghaffer 1982), and suspension bridge response to multiple-support excitations (Abdel-Ghaffer 1982) give in-depth detail about the importance of earthquake response of suspension bridges. The study on natural frequencies and modes (West et al. 1984) using a finite-element (FE) formulation demonstrates the variation of the frequencies and mode shapes of stiffened suspension bridges. Three dimensional models usually are developed with beam and truss elements to model both the superstructure and the substructure (Okamoto 1983; Wilson and Gravelle 1991; Dumanoglu et al. 1992).

West Virginia is listed among low seismic risk areas in the United States. For West Virginia the American Association of State Highway and Transportation Officials (AASHTO) code assigns an acceleration coefficient A of 0.05. However, a recent investigation concluded that earthquake hazards should be of concern to the state. The investigation combined geological and historic data from regional earthquakes that caused structural damage in West Virginia. Broad areas of the sate may have already experienced modified Mercalli intensity (MMI) scale VII excitations during the 1886 Charleston, S.C. and the New Madrid, Mo. earthquakes of 1811 and 1812. Similar intensity earthquakes also were felt in the southern part of the state during the 1897 Giles, Va. earthquake. The seismic analysis of the bridge considers both the AASHTO code assigned acceleration coefficient and the intensities based on recent studies (King et al. 1993).

HISTORIC BACKGROUND AND BRIDGE DESCRIPTION

The long-span suspension bridge together with the metal truss and the high-rise building represent three important contributions by American engineers to structural engineering in the nineteenth century. The Wheeling suspension bridge is particularly important in the development of the long-span wire suspension bridge. When it was opened in 1849 it was the largest span bridge in the world and ushered in a century-long leadership enjoyed by American engineers. As a result, it is one of the most significant antebellum civil engineering structures in the nation.

Despite the fact that the bridge was not designed to carry today's live loads resulting from automobile traffic, the bridge has served as a vital link across the Ohio River for nearly a century and a half. To protect this internationally significant bridge and at the same time insure the safety of the traveling public, it is necessary to understand how the bridge functions under both static and dynamic loads. This paper represents the first comprehensive structural analysis using modern computer techniques and in situ nondestructive measurements of this bridge.

The bridge was designed by Charles Ellet Jr., a leading nineteenth century civil engineer (Kemp 1975). The original bridge consisted of a timber deck suspended from wrought iron suspenders, which in turn were carried by the main cables.

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Figs. 1 and 2 show the elevation of the bridge. In the original design there were 12 independent cables arranged according to the French garland system. In 1854, only 4 years after its completion, the bridge deck was destroyed and the cables dislodged by a tornado that swept up the Ohio valley. It was quickly rebuilt under the supervision of Ellet.

Later the bridge was strengthened and stiffened by the addition of stay cables. This work was undertaken by Washington Roebling who also gathered the 12 cables into two pairs of cables, one pair on each side. This ''Roeblingized'' bridge served until 1956. In 1956 a major rehabilitation was done in which the W24 × 76 floor beams, roadway grating, sidewalk grating, and lateral bracing system were incorporated in the deck system. The most recent rehabilitation was performed in 1983. At that time a neoprene wrapping system was added to the main suspension cables, together with miscellaneous structural steel repairs and replacement of damaged stay cables and hanger rods. Additional work included installing new timber stiffening trusses, cleaning and painting of the superstructure, repointing the masonry towers, installing stay-hanger connectors, and tensioning the hangers and stay cables. In 1989, additional repairs were performed by the State Maintenance Department.

The structure carries the West Virginia state route 251 as it crosses the main channel of the Ohio River connecting the city of Wheeling and Wheeling Island. The bridge span is approximately 307.50 m and carries a 6.10-m roadway. The main suspension cables support the two stiffening trusses of the bridge. Each cable is approximately 19.10 cm in diameter and consists of approximately 1,650 wires of No. 10 gauge (3.42 mm diameter) laid in parallel. The cables are protected with a liquid neoprene paint and an elastomeric cable cover. The cables are supported at each tower by saddles comprised of three cast-iron rollers. The cable ends are connected to anchor bars that are embedded into concrete anchorages (Lichtenstein 1992).

Diagonal stay cables comprised of wire ropes of varying diameter from 2.54 to 4.45 cm are located within the end-quarter spans. The stay cables pass over the saddles at each tower and are anchored into the main cable anchorages.

The framing plan of the existing bridge is shown in Fig. 3. The floor system is suspended from the main cables by pairs of 2.85 or 3.18 cm diameter hanger rods at each end of the floor beams. The hanger rods are connected to the main cables by cable bands and attached to the floor beams, which consist of rolled steel wide flange W24 × 76 sections with a spacing of approximately 2.44 m throughout the structure.

The deck is stiffened by Howe-type trusses in which the top chords, bottom chords, and diagonals are made of timber (Fig. 4). The truss verticals are steel rods with a diameter of 3.18 cm, whereas the junction boxes are cast iron. The trusses are supported by the floor beam extensions at the panel points. The truss bottom chord passes under the extensions and is connected by vertical steel or wrought-iron rods.

The superstructure supports a 12.70-cm-deep open grid steel grating. A 1.22-m sidewalk of 2.54 cm deep open grid steel grating exists along each side of the roadway. The timber truss serves as a guardrail for pedestrians (Fig. 4). A woven-wire protective fence is attached to the stiffening truss to protect pedestrians.

The bridge substructure consists of east and west towers constructed with cut stone masonry. The cables at the top of the tower are protected by galvanized sheet metal housings,

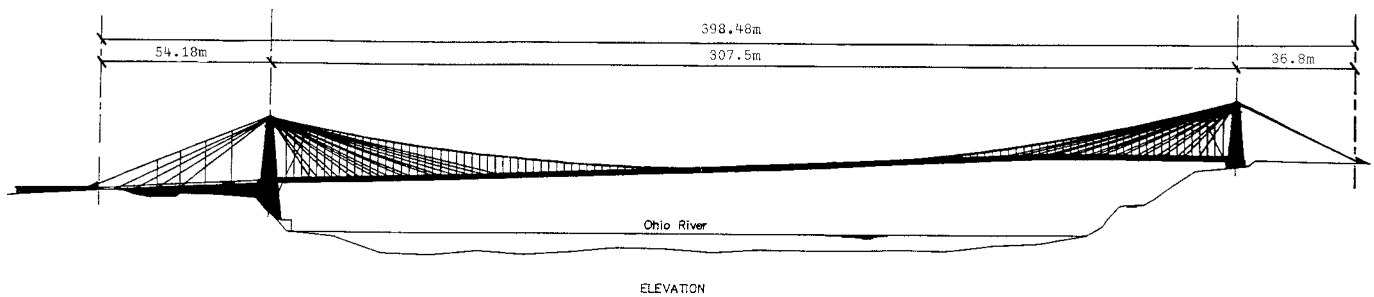

FIG. 1. Elevation of Wheeling Suspension Bridge

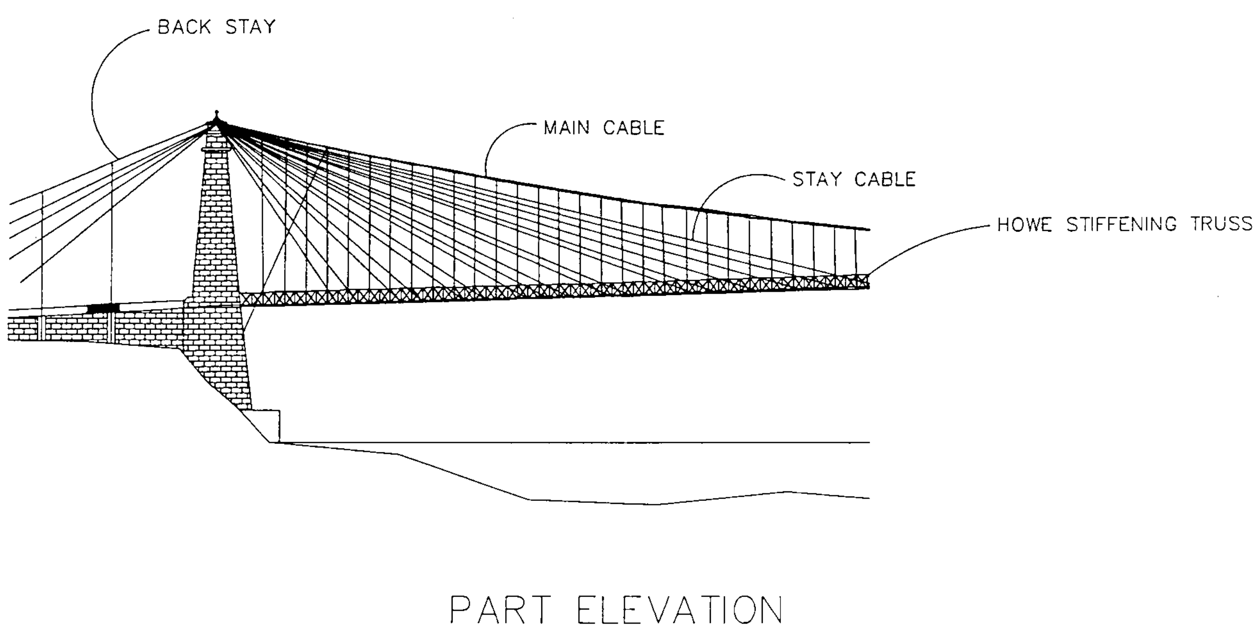

FIG. 2. Part Elevation of Bridge

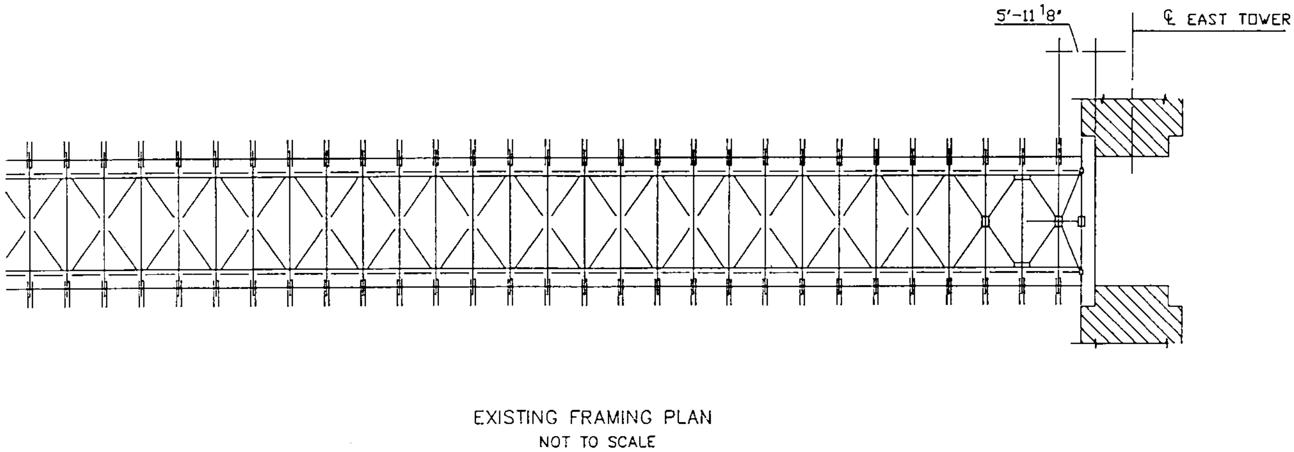

FIG. 3. Framing Plan of Existing Bridge

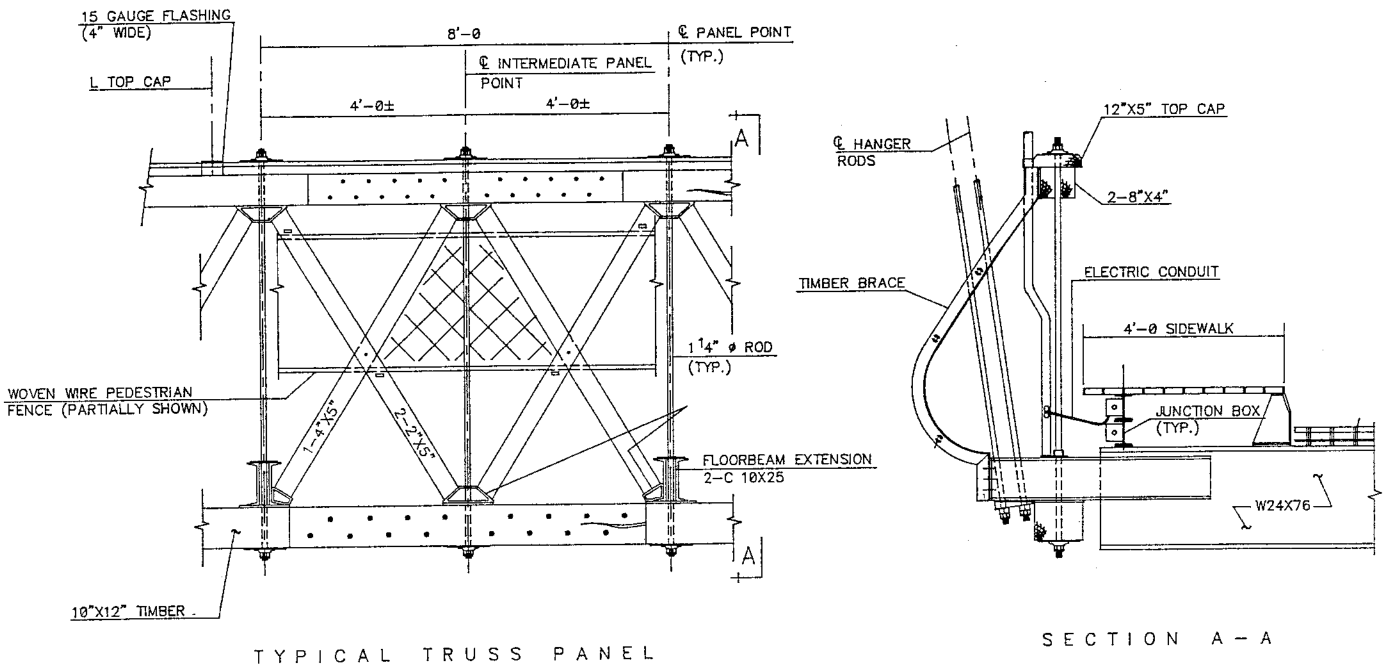

FIG. 4. Typical Stiffening Truss Panel and Detail

whereas the main cable anchorages are protected by cut stone masonry anchorage housings.

Currently, the inventory rating and traffic barriers regulate a clearance 2.44 m, and a weight limit of 2 t. No trucks, buses and trailers, are allowed; and automobiles must maintain 15.24-m intervals.

Based on data provided in the contractor's proposal by the Department of Highways (1981), the material properties of the system components are as follows:

- Timber. Dense select structural Douglas Fir or Southern Pine. Extreme fiber in bending (f_b) = 13,110 kN/m^2; tension parallel to grain (f_t) = 7,590 kN/m^2; compression parallel to grain (f_c) = 8,970 kN/m^2; horizontal shear (f_v) = 586.5 kN/m^2; compression perpendicular to grain (f_{ct}) = 3,139.50 kN/m^2.
- Cable wires. Yield strength = 6.21E5 kN/m^2; ultimate strength = 8.55E5 kN/m^2; elongation in 25.4 cm length 4%; Young's modulus E = 2,000 E5 kN/m^2.
- Structural steel. Steel members of the bridge conform to ASTM standards.

TWO- (2D) AND THREE-DIMENSIONAL (3D) MODELS

Because of the complexity of the structure, an FE analysis was employed to study the behavior of the bridge under static and dynamic loads. Both 2D and 3D models were used to analyze the structure. To develop a proper FE model and to compare the FE results with elastic theory results two 2D models also were developed, one without stay cables and the other with stay cables (Figs. 5 and 6, respectively). In the 2D models the stiffening truss is replaced with an equivalent beam. After comparison of the results from the model without stay cables with elastic theory results, a 2D model with stays and 3D

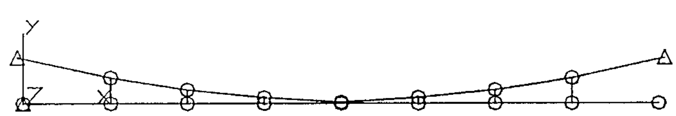

FIG. 5. 2D Model of Wheeling Suspension Bridge without Stays

FIG. 6. 2D Model of Wheeling Suspension Bridge with Stays

models were developed to represent the actual bridge as it exists at the site as of today. The bridge superstructure, comprised of two main cables, suspenders, stay cables, floor beams, a steel grid deck, and a stiffening truss is modeled with 3D beam and truss elements. The suspenders and stay cables are modeled using 3D truss elements. Beam elements are used to model the stiffening truss, floor beams, and deck. The steel grid deck and sidewalk were modeled with beam elements to which equivalent stiffness was assigned to simulate the stiffness of the system (Fig. 2). End moment releases were specified at diagonals and verticals to simulate scissors joint conditions.

Fig. 7 shows the 3D FE model of the western side of the bridge. The parallel wire cables are flattened out over a series of three rollers, which are mounted at the top of the tower. Inspections over a long period of time have indicated that there has been no movement of these cables because of the way they pass over the roller system. Thus, fixity has been assumed in the analysis. The stiffening truss is hinged at the supports. The cables in the 3D model have been placed according to plans of the existing bridge. The suspenders are connected to the channel sections at the bottom of truss joints and the main cable. The spacing between the suspenders is approximately 2.44 m. Only translational displacements at the top of the suspenders are allowed at the junction of the main cable and suspenders. The ordinates of the cable profile are measured from available drawings and used in the FE model of the bridge.

Fig. 8 shows the 3D FE model of the middle portion of the bridge, giving details on the arrangement of the stiffening truss, main cable, suspenders, and floor beams. The stiffening truss is modeled using beam elements, where the moments are released at the end of the diagonals and verticals. The steel grid deck is represented as a beam with equivalent stiffness. The beams supporting the sidewalk also are shown in Fig. 8. The diagonal ties and floor beams are represented using 3D beam elements with six degrees of freedom at each node. The eastern side of the bridge is identical to the western side except that there are 19 stay cables instead of 17. It should be noted that the bridge is not symmetric, the approach level at the eastern side is approximately 9.50 m higher than that of the western side.

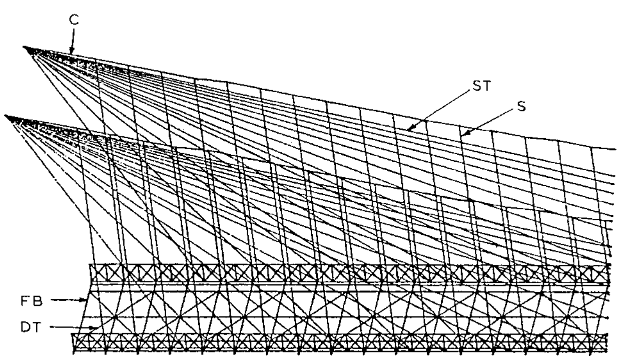

FIG. 7. 3D Model of Wheeling Suspension Bridge

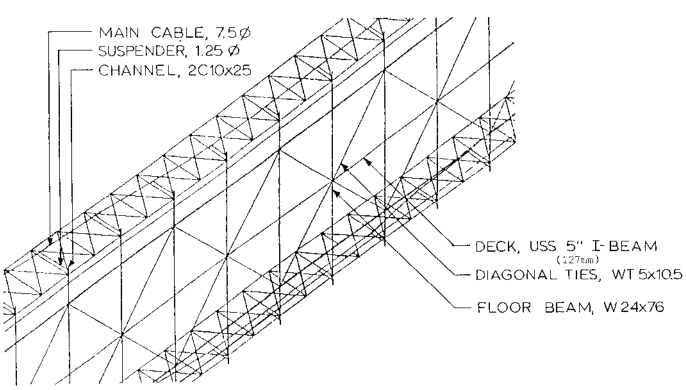

FIG. 8. 3D Model of Central Portion of Bridge

IN SITU EVALUATION OF SUSPENDER FORCES

The main reason for in situ evaluation was to measure the existing axial forces in the suspenders with the axial-load monitor (ALM), a handheld force-measuring instrument developed at West Virginia University and subsequently used in FE analysis.

The operation of the instrument is based on modal testing. It correlates natural frequencies to axial forces. Extensive theoretical and experimental work has been performed (Siros 1992; Nader 1993) in establishing the accuracy of the ALM through laboratory testing. Use of the instrument was extended to field testing of structural members by Nader (1993).

The ALM was used to measure the forces of suspenders that did not intersect with diagonal stay cables. Table 1 shows the actual load carried in representative suspenders. By measuring the actual load carried by the suspenders, the forces in the main cables were calculated using equilibrium equations. After calculating the forces in each element (length between two suspenders) of the main cable, they were applied as initial forces in the FE model to represent the forces developed in the main cable caused by the dead load.

STATIC ANALYSIS

Static analysis of the bridge subjected to present-day loadings as well as a load rating analysis were performed with 2D and 3D models. Details on the rating analysis are given in Venkatareddy (1995).

For static loads the maximum live load stresses at the most heavily loaded members such as the main cable (C), suspender (S), stay (ST), top chord (T) and bottom chord (B), floor beam (FB), and diagonal ties (DT) of truss are shown in Table 2, where the negative sign indicates compressive axial loads. The member locations are shown in Figs. 2 and 7 and the T and B are members of the Howe truss. Notice that the stresses in the members caused by live load plus dead load are much less than the allowable stress. Table 3 lists the deflection at midspan resulting from present-day live loads, i.e., 1,026.90 N/m uniform load, obtained from the theoretical, a 2D model without stays, a 2D model with stays, and a 3D model. The theoretical results have been obtained based on the elastic theory presented in Steinman (1929) and also included in Venkata-

TABLE 1. Measured Suspender Forces

Suspender number (1)	Upstream side (kN) (2)	Downstream side (kN) (3)
37	19.20	18.32
38	18.00	18.32
39	16.41	17.64

TABLE 2. Static Analysis

Member (1)	Live-load maximum stress (kN/m^2) (2)
C	16,456.5
S	21,024.30
T	−577.25
B	−211.48
FB	333.27
DT	−1,240.76
ST	29,939.10

TABLE 3. Midspan Deflections Caused by Live Load

Theoretical (1)	2D without stays (2)	2D with stays (3)	3D model (4)
44.34 cm	42.65 cm	27.48 cm	25.07 cm

reddy (1995) for the 2D model. Static analysis with the FE code included second-order effects for the 3D model. The deflections clearly indicate the additional stiffness contributed by the stays and the steel grid deck in the 2D models with stays and the 3D model, respectively. Results from 3D static analysis showed that live load stresses in the stiffening truss and cables are within allowable limits and the deflections at midspan are acceptable. Specifically, the span length over the midspan deflection ratio is 1,237 for live load. Rating factors of highly stressed structural members were calculated for present-day uniform loading of 1,026.90 N/m. None of them were below unity, which indicates that the present-day live load is not overstressing any structural member (Venkatareddy 1995).

MODAL ANALYSIS

Since the collapse of the Tacoma Narrows Bridge in 1940, considerable amount of research has been conducted to study the dynamic behavior of cable suspension bridges, e.g., Gade et al. (1976), Gimsing (1982), Kumarasena et al. (1991), and Seim et al. (1993). Early contributions by Bleich and Steinman have adopted an analytical approach. This 2D approach is based on several assumptions and empirical rules that could compromise solution accuracy, such as torsional effects. However, use of FEs, allows accommodation of various complexities of the structure. Comparisons between FE results and analytical approaches that yielded comparable results for simple models of cable suspension bridges have been reported by several researchers (West et al. 1984; Dumanoglu et al. 1992).

In this study the natural frequencies and mode shapes of the stiffened suspension bridge are determined using FE analysis. Because of the complexity of the system and consideration of the stiffening effect of the forces in the main cables, the FE procedure was examined through comparison with the well-documented analytical and experimental studies of the Tacoma Narrows Bridge. The results of this research were compared with the results by West et al. (1984) and Bleich et al. (1950). Once comparison was completed the procedure was applied to perform modal analysis of the Wheeling suspension bridge.

First, a 2D model of the Wheeling suspension bridge without stay cables, similar to the one used for the Tacoma Narrows Bridge, was developed to compare the FE modal analysis results with elastic analysis results (Venkatareddy 1995). Transverse deformations were restrained in the 2D models, (Figs. 5 and 6). Beam and truss elements were used to model the system with stiffness and inertia properties corresponding to the deck and stiffening truss. The lowest three natural frequencies are listed together with the theoretical values obtained based on Bleich's elastic theory procedure (Bleich 1950; Venkatareddy 1995) in Table 4. The excellent correlation of the 2D FE model without stays and theoretical results is

TABLE 4. Wheeling Suspension Bridge (2D Model without Stays): Natural Frequencies

Natural frequency (1)	Theoretical (rad/s) (2)	FE analysis (rad/s) (3)
ω1	1.56	1.55
ω2	2.30	2.40
ω3	3.60	3.30

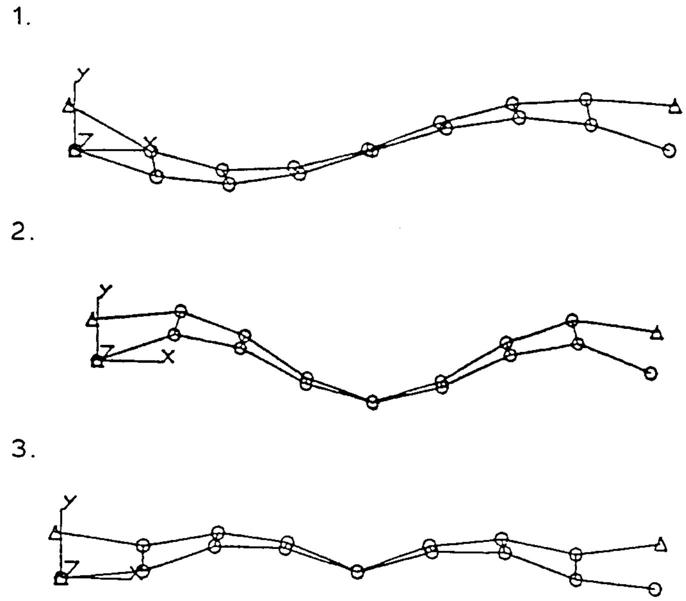

FIG. 9. Mode Shapes of 2D Model without Stays

demonstrated clearly in Table 4. The corresponding mode shapes are presented in Fig. 9.

Modal analysis also was performed on the 2D model with stays for two cases: with and without initial forces in the main cable (Fig. 6). The lowest three natural frequencies and corresponding mode shapes were evaluated. The frequencies are listed in Table 5 and the corresponding mode shapes are shown in Figs. 10 and 11. The results indicated that the effect of initial forces increased the frequencies by approximately 4% compared with the results without initial forces.

Modal analysis was performed on a 3D model of the Wheeling suspension bridge to determine the lowest six natural frequencies and mode shapes. They are listed in Table 6. As ex-

TABLE 5. Wheeling Suspension Bridge (2D Model with Stays): Natural Frequencies

Natural frequency (1)	With initial force (rad/s) (2)	Without initial force (rad/s) (3)
ω1	1.76	1.69
ω2	2.10	1.99
ω3	3.77	3.67

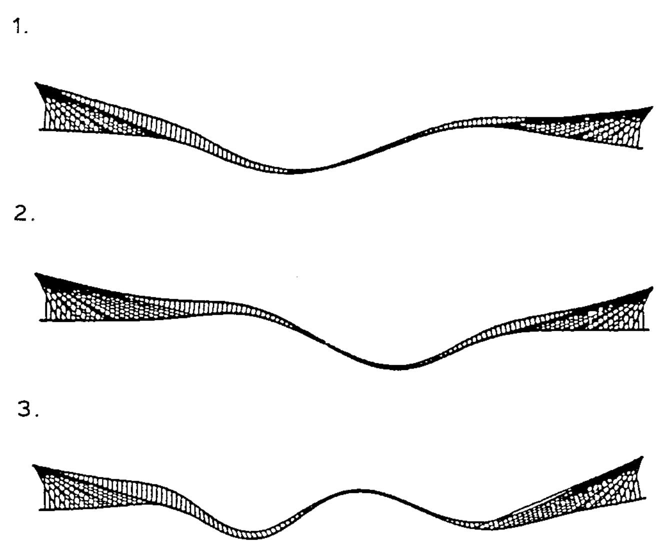

FIG. 10. Mode Shapes of 2D Model with Stays and Initial Forces

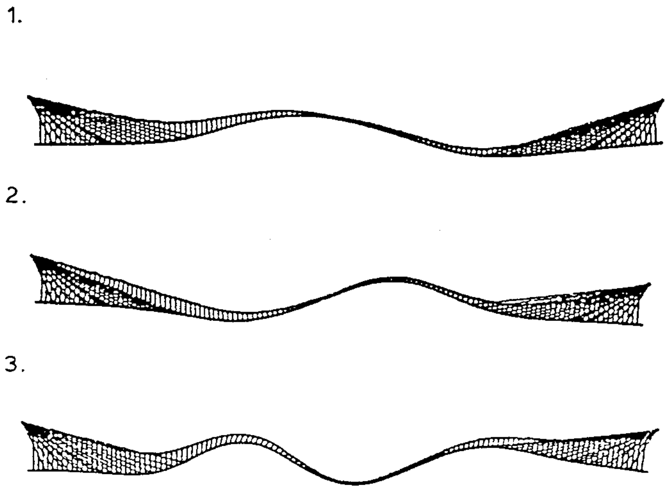

FIG. 11. Mode Shapes of 2D Model with Stays but without Initial Forces

TABLE 6. Modal Analysis Results

Mode number (1)	Frequency (rad/s) (2)
1	1.85
2	1.99
3	2.57
4	2.88
5	3.33
6	4.74

pected the predominant deformation exhibited by the first mode is along the transverse direction. A major difference between 2D and 3D models is the representation of the deck system as well as the inability of the 2D models to capture torsional and coupled flexural-torsional behavior. In the 3D model the bridge is modeled with all load-carrying members as they exist at the site, only the steel grid deck and the sidewalk are replaced with equivalent beams. The 3D model used in the analysis (Figs. 7 and 8) consists of 6,906 members with a total of over 41,000 degrees of freedom.

SEISMIC ANALYSIS

A multimode spectral analysis was applied to obtain the bridge response to three different spectra. The first spectra are developed according to current AASHTO specifications, whereas the second and third are based on historic earthquakes of the region. Specifically, for West Virginia the code assigns a horizontal acceleration coefficient A of 0.05 (AASHTO 1992). The ground acceleration corresponds to a 10% probability of being exceeded in 50 years. Because of the historic significance of the bridge, A was increased to 0.06 to correspond to a 10% probability of being exceeded in 100 years (Department of the Army, the Navy, and the Air Force 1986). Historically, MMI VI and VII earthquakes have been experienced in the area in the early eighteenth century (Federal Highway Administration 1987; King et al. 1993). These historic earthquakes can be characterized with an $0.08 < A < 0.25$ (Department of the Army, the Navy, and the Air Force 1986). In this study $A = 0.15$ was selected as upper bound of an MMI scale of VI earthquake. Modeling of local soil conditions was not considered (Spyrakos 1992), and because of limited soil data information, the spectra were developed for site coefficient $S = 1.2$. Because in the AASHTO code C_{sm} is based on a 5% critical damping, C_{sm} was increased by 25% that corresponds to 2% of critical damping (Department of the Army, the Navy, and the Air Force 1986). The 2% critical damping is considered more realistic for cable suspension bridges (Okamoto 1983; Federal Highway Administration 1987). A seismic analysis also was performed for $A = 0.19$, representative of an MMI scale of VII earthquake, to examine the response of this bridge as a representative suspension bridge of the several historic bridges built in the eastern United States with historic earthquakes higher than those predicted for the Wheeling suspension bridge.

The design loads were obtained by superimposing the dead loads to the seismic forces developed along the longitudinal and transverse directions of the bridge according to AASHTO. The vertical component of the seismic motion also was included in the analysis (FHWA-IP-87-6). Because it is desirable for the structure to remain elastic a response modification factor $R = 1$ was assigned to the superstructure. Because of insufficient data, the behavior of the towers and abutments was not studied.

The most severe seismic load combination corresponds to 100% in the transverse, 66% in the vertical, and 30% in the longitudinal direction. For $A = 0.06$, the total stresses were within the allowable for most of the structure. However, several floor beams would fail at the east tower. They exceeded the allowable stress of 1.38E5 kN/m^2 by 25%. For A/ = 0.15, the displacements at the midspan of the lower chord resulting from dead and seismic loads are given in Table 7. The fourth column of Table 8 lists the stresses developed under dead load and seismic load for $A = 0.15$ of the most heavily loaded members. For the modal combination of seismic stresses the square root of the sum of the squares method was used (Spyrakos 1994). Floor beams and diagonal ties were overstressed near the supports of the east tower and also several chords of the stiffening truss of midspan failed in direct tension. The stresses developed in the most heavily loaded members are almost twice the allowable stresses, especially in floor beams and diagonal ties. Damage also was constrained to the same members for the higher intensity $A = 0.19$. However, inelastic deformation is expected to alter the response of the system as determined by the elastic analysis. Nevertheless, the fact that no overstressing was imposed on the main cables and suspenders indicates that catastrophic failure of the bridge for seismic excitations is highly unlikely. The localized damage of several floor beams and truss members could be avoided through proper stiffening and possible installation of damping devices.

TABLE 7. Displacements Caused by Dead Load and Seismic Loads

Direction (1)	Dead load (cm) (2)	Seismic load (cm) (3)
X	0.00	0.833
Y	0.00	29.44
Z	0.00	43.81

TABLE 8. Dead Load and Seismic Load Stresses

Dead load (1)	Dead load (kN/m^2) (2)	Seismic load (kN/m^2) (3)	Combined (kN/m^2) (4)
C	15,748.69	+17,291.40	+33,040.09
S	12,720.29	+88,803.00	+101,523.29
T	0.00	±8,589.12	±8,589.12
B	0.00	±5,202.60	±5,202.60
FB	0.00	±469,338.00	±469,338.00
DT	0.00	±211,140.00	±211,140.00
ST	30,569.89	+39,854.40	+70,424.29

CONCLUSIONS

The static analysis showed that deflections and stresses caused by present-day loading conditions are within allowable limits. Stay cables carry a significant amount of load that is approximately 10–15% compared with the main cables. Stiffness contributed by the steel grid deck is appreciable. Rating factors demonstrated that none of the structural members are overstressed by a live load of 1,026.90 N/m. Thermal loads were not considered because they do not control the behavior.

Seismic analysis showed that the bridge can withstand an earthquake of intensity V MMI scale with a peak ground acceleration coefficient $A = 0.06$ with little damage. For $A = 0.15$, that corresponds to an MMI scale of VI, several bridge members at the east tower, i.e., floor beams and diagonal floor ties, would fail. For $A = 0.19$, an earthquake intensity magnitude corresponding to an MMI scale of VII, the same members would experience severe damage that could lead to localized failure of the bridge deck. In conclusion, even an intensity MMI scale of VII earthquake will not cause damage of the main cables and the suspenders posing any threat of catastrophic failure to the bridge. Strengthening of floor beams, diagonal ties at the east tower, and truss members at midspan could mitigate localized damage. The methodology could be applied to a wide range of cable suspension bridges.

ACKNOWLEDGMENTS

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