counterparts with an identical or slightly greater major-axis moment of inertia.

2. Spread box-beam bridges exhibit higher overall flexural stiffness than comparable I-beam bridges, although the maximum bridge deflection reached at equivalent beam damage levels is about the same for the two bridge types.

3. In box-beam bridges transverse distribution of load to beams not directly loaded is higher initially and is effectively maintained through the entire load range as compared with I-beam bridges in which the initially poor transverse distribution becomes worse as total applied load is increased into the postelastic range.

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Overloading of Steel Multigirder Bridges

CELAL N. KOSTEM

ABSTRACT

The overloading of steel multigirder highway bridges may have deleterious effects on the structural integrity of the superstructure. The overloading of steel bridges is closely linked with the fatigue-life determination of the connection details. It is observed that the actual structural response of these bridges is different from the assumptions made in the design. Results obtained from a computer-based analytical model and simulation scheme are presented. The method provides a reliable tool to predict the linear-elastic and inelastic response of bridge superstructures up to the collapse load level. The observations from case studies have indicated that (a) interface slip between the girders and the bridge deck can be neglected for any practical overloadings, (b) high stresses due to overloading tend to be more prominent in the vicinity of the details that are prone to fatigue-crack initiation, (c) residual stresses play a nonnegligible role in the inelastic response of primary steel girders, (d) buckling is an important but not a critical phenomenon, and (e) damage initiation due to overloading can initiate in girders or in the deck, depending on the design details. It was also noted that bridges with a high degree of internal and external structural indeterminacy are less prone to damage induced by catastrophic overload.

Most highway bridges are subjected to overloading of varying degrees of severity with varying frequency. It is quite rare that all structural components of a bridge superstructure will not be subjected to stresses and deformations that will be equal to or below the values assumed by the designers. The overloading of a given bridge and its components will not necessarily occur only when a vehicle traversing the bridge is heavier than the design vehicle. Vehicles with close axle spacings, even if they are lighter than the design vehicle, can cause overloading. Thus, the issue of overloading is prevalent for all bridges. The frequency of the overloading cannot be accurately estimated unless the traffic count, including the axle spacing and weights of the axles, is monitored. Because some steel bridge components are known to be susceptible to fatigue, fatigue-crack initiation, and propagation, the overloading of steel bridges is closely related to the fatigue life of the bridges.

The current bridge design specifications (1) and bridge rating provisions (2) do not address the overloading with sufficient specificity. Even with these guidelines, much is still left to engineering judgment. The prudent deployment of engineering judgment requires a firm technical understanding of the structural response of highway bridges when subjected to overloaded vehicles.

A detailed research program on the overloading of prestressed concrete I-beam highway bridges has provided the needed information on the elastic and inelastic response of these bridges (3-6). The pilot research programs on the prestressed concrete spread box-beam bridges have also provided the comparative results between the I- and the box-beam construction (7).

The extensive analytical research and laboratory and field-test comparisons, where possible, have clearly indicated that the actual structural response of highway bridge is three dimensional. This differs substantially from the basic design premise of proportioning each structural component individually with little, if any, consideration for the interaction among these structural components. Thus, in the overload and even in the design-load assessment of the bridges with acceptable accuracy, the three-dimensional interaction among the structural members must be taken into account.

PRELIMINARY CONSIDERATIONS

In view of the difference between the presumed and the actual structural behavior of the bridge superstructure, certain factors with questionable validity have been examined, and their contributions have been identified. In the overloading response of steel multigirder highway bridges it is expected, but not quantified, that the cross framing will provide a more uniform distribution of the vehicular loading among the girders. The contribution of the cross framing in load distribution is not as high as was expected. Furthermore, the effectiveness of the cross framing in distributing the live load is dependent on load location (3,8).

If the cross bracings are to be as effective as expected, another problem surfaces. If the cross bracing is transmitting substantial forces in order to provide a more uniform distribution of the vehicular loads, the forces in these members need to be transmitted to some part of the structure. The cross framings are traditionally connected to the tension flange or to a bracket attached to the web at the vicinity of the tension flange. For increased load levels the forces transmitted by the bracing members cause out-of-plane deformations in the web-to-flange connection. These deformations, however small in magnitude, cause large local stresses. This type of action is known to be the source of the displacement-induced fatigue-crack initiation (8). Thus the possible positive contribution of the cross bracings is offset by adverse structural effects.

ANALYTICAL MODEL FOR OVERLOAD SIMULATION

If information is needed for the stresses, deformation, possible damage to the superstructure, and type and location of the damage, a more sophisticated analytical method needs to be developed. An approach that has been fully successful for prestressed concrete I-beam bridges was modified to simulate the behavior of the types of bridges in question (4-6,9-11). The bridge deck is simulated by a series of plate-bending finite elements with membrane stiffnesses. The girders were also divided into a series of beam finite elements. In order to account for the initiation and the propagation of material nonlinearity and any form of damage, the plate and beam elements were divided into a series of layers (Figure 1). The model developed and the method require computers. The computer program Bridge Overload Analysis--Steel (BOVAS) has been applied to all known field and laboratory test cases to verify its accuracy (11). The complexity of the mathematical derivation of the model prevents its inclusion in this paper; however, the details may be found in other publications (9,10).

One of the case studies conducted was the AASHTO Road Test bridge (12). The characteristics and the loading sequence of this bridge are widely known in bridge engineering. Figure 2 shows the finite-element layering of the bridge deck and the girder. The complexity of the geometry in Figure 2 clearly demonstrates the need for computer-based solution. All the details defined in Figure 2 were automatically generated by the computer program (11).

The experimental tests results and the analytical prediction by program BOVAS may be seen in Figure 3. Good agreement between the test results and the prediction may be noted. The other case studies have also resulted in similar favorable comparisons.

GENERAL OBSERVATIONS

Any investigation as detailed as this but without a detailed parametric investigation would yield observations that are applicable to the types of bridges being studied. However, the lack of a detailed parametric investigation would not permit the development of formulas to quantify the findings.

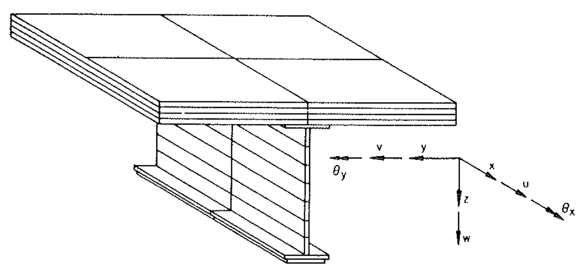

FIGURE 1 Slab and girder layering.

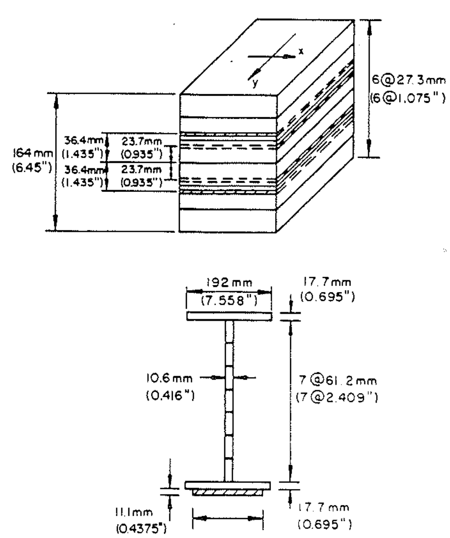

FIGURE 2 Slab and beam layering of test bridge (12).

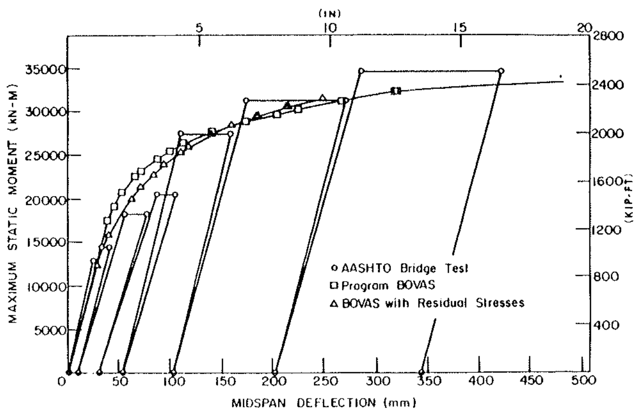

FIGURE 3 Moment versus deflection of test bridge.

Slip

The analytical research combined with the verification of the reported experimental research and field observations have indicated that slip is not a major concern in the structural response of steel multigirder bridge superstructures (13). Because of the friction between the deck and the steel girders, even for noncomposite construction, there exists a composite interaction. For increased load levels intermittent slips occur, but a fully noncomposite response cannot be achieved. In the case of partial or fully composite design until the occurrence of any noticeable slip, the bridge deck slab and the steel girders undergo substantial nonrecoverable damage (10,13).

High Stress Fields

As expected, the highest stresses are observed in the tension flanges and compression flanges (near the support in the case of continuous construction). For design loadings the magnitude of the stresses is within that of the design stresses. However, as the overloading occurs, these stresses increase proportionally to the gross weight of the vehicle. The contributions of the cross bracings do not enter into the lateral live-load distribution until the occurrence of the stress redistribution in the structure because of plastification or limited damage.

RESIDUAL STRESSES

The presence of the residual stresses should be noted in two situations: (a) the determination of the stress fields for the fatigue and fracture analysis of various members and details and (b) its effect on the overall structural response. The former has been well studied and quantified by many researchers and bridge engineers. As far as the latter is concerned, it is interesting to note that in the essentially linear-elastic response regime of the superstructure, the magnitude and the distribution of residual stresses do not play any role (Figure 3). Similarly, the collapse load level of the structure is not greatly affected by the residual stress field in the structural components. However, the variations in the residual stress intensities and their distribution play a predominant role in the structural response after the initiation of the nonlinear behavior and before the collapse (Figure 3).

The magnitude and the distribution of the residual stresses are highly affected by the fabrication procedures. In the absence of more reliable information, the residual stresses need to be considered as a factor that has adverse effects on the integrity of the superstructure.

Connection Details

Unfortunately past and present steel bridge engineering design practice places some critical details in the vicinity of the tension flanges. These details are known to have low fatigue life (1). Usually the stress ranges for these details are computed by using the reverse-design procedure, thereby neglecting the contributions of the out-of-plane deformations. This underestimates the stress range and thus overestimates the fatigue life of the actual connection.

In all case studies that have been conducted and verified with the field test results where available, it was observed that because of overloading the most stressed location in the tension flange is the tip of the cover plate, which is known for its low fatigue strength. Thus, through the visualization of this simple example, it is important to realize that overloading of steel bridges requires consideration of the fatigue provisions. This presupposes that the passage of the overloaded vehicle is not an extreme rarity.

Buckling

It has been observed that in the case of rolled girders, with or without cover plating, web buckling is uncommon. However, in the case of deep built-up girders, and especially in the case of plate girders, the stability of the web becomes a critical issue. This is more notable near the supports. Research has indicated that if the vertical stiffeners are properly designed, the web may buckle as a shear panel defined by the top and bottom flanges and the vertical stiffeners. This buckled web then develops a diagonal tension field and behaves like a truss member (Figure 4). Research has indicated that through the proper use of vertical stiffeners web buckling can be isolated to a few panels and does not initiate progressive spread of buckling. Buckling of the web causes a jog in the load-deformation curve of the structure, indicating a temporary shift in the stiffness. In the case of buckled panels, it would be premature to condemn the load-carrying capacity of the bridge.

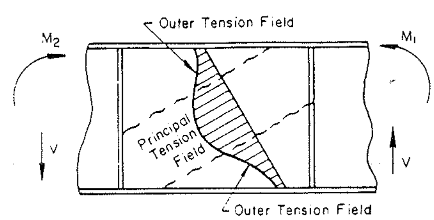

FIGURE 4 Typical transversely stiffened plate-girder web-plate panel under combined moment and shear.

The lateral buckling of the compression flange can occur for high load levels. However, in view of the current detailing practice there are always sufficient supports provided to brace the compression flange. The studies have not revealed the danger of lateral buckling due to the overloading.

Deck Damage

In prestressed concrete I-beam bridges it was noted that the damage was always initiated by the cracking of the reinforced-concrete deck slab (5). These cracks were essentially parallel to the beams. For indiscriminately increased overload levels, the cracking formed partial hinges similar to the formation of the yield lines. The beams did not show any discernible damage until after substantial damage to the bridge deck. In the case of steel multigirder bridges with reinforced-concrete deck, such a generalization cannot be made. Depending on the proportioning of the steel girders, the damage to the superstructure can take place both at the girder as initiation of plastification or web-panel buckling and in the deck slab as cracking of the concrete. In the case of continuous construction, substantial cracking of the concrete over the interior supports takes place before any other damage to the rest of the deck and usually before any damage to the steel components. After the formation of pseudo-yield lines over the support, additional concrete cracking is observed between the girders. It should also be noted that even though such damage to the deck is not desirable, if cracking over the supports is noted and also if no inelastic behavior in the girders has taken place, through the rebound of the superstructure these cracks will close. All the deck concrete cracks, both in steel and prestressed concrete bridges, should be considered working cracks, provided that the girders do not undergo any loss of rebound capability.

Major Girder Damage and Structural Redundancy

In some cases deep cracks were observed, usually by coincidence, in the main girders of the bridges while the bridge was carrying a routine traffic load. An inference should not be drawn that such a bridge can carry overloaded vehicles. Various case studies undertaken by the author have demonstrated that in multigirder steel bridges if the superstructures have a high degree of internal and external structural indeterminacy, major damage to a girder will not result in the immediate loss of the bridge. The redistribution of the stresses permits the structure to hold up, perhaps after undergoing some noticeable deformations, and carry the regular traffic. Through the redistribution of the stresses other members may be highly overstressed. The misleading corollary to this is that if a bridge can carry some overloading and does not exhibit any distress, it should be able to carry some additional overloads. Without a full inspection and engineering computations, additional overloads to the structures should not be permitted without full cognizance of the incipient damage in the structure. The high degree of internal and external indeterminacy built into relatively old steel bridges is, in many cases, a blessing in disguise. The damage, if any, can in many instances go unnoticed for a prolonged period of time; with this probability in mind, the rating of these bridges should not be increased liberally.

RATING OF BRIDGES

If two highway bridges (one prestressed concrete I-beam and the other steel multigirder, designed and built using the same specifications for the same design loading and having equal span lengths and traffic lanes) are to be rated by using the current rating provisions (2), they may not have the same rating. This discrepancy is due to the current AASHTO guidelines for bridge rating (2). The prestressed concrete bridge will probably be rated for heavier loads than the steel bridge. It has been suggested that the rating provisions be revised so that the rating of the steel bridge is increased to that of the prestressed concrete bridge. Regardless of how reasonable this argument may sound, it has major flaws. In the rating of a steel bridge the residual stresses are not taken into account. The reduction of the allowable stresses in part can account for the number of unquantified parameters. Increase in the allowable stresses may result in permission for excessive stresses in some critical members.

CONCLUSIONS

The various findings discussed in this paper are the conclusions, and they will not reiterated. One concept that requires reexamination is overload versus inelastic response versus bridge inspection. It has been observed that depending on the dimensioning of the bridge and especially the detailing, it is possible that portions, and critical portions for that matter, may exhibit material or geometric nonlinearity even under service loads. The issuance of overload permits for such structures, especially if the structure has not been meticulously field inspected, should not be considered. In the case of structures with a high degree of indeterminacy, the possible adverse effects of previous overloadings may go unnoticed. Rating for higher loading requires the uncovering of built-in damages, if any. In the case of bridges with low structural indeterminacy, overloading permits or higher rating factors should be considered with extreme caution.

ACKNOWLEDGMENT

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The Ontario Bridge Code: Second Edition

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ABSTRACT

Based on the limit-state design philosophy, the Ontario Highway Bridge Design Code was first published in 1979. A brief account is given of the implementation of the first edition of the code and the problems associated with the implementation. The second edition of the code was published in late 1983. Major changes in the code provisions are identified, and some details of a computer system that is currently being developed to support the code are given.

Despite the diversity of vehicle weight regulations in various jurisdictions, most highway bridges in North America are designed by the same AASHTO specifications (1) or the Canadian Standards Association (CSA) bridge code (2), which is only a slight variation of the former. The Province of Ontario used the AASHTO specifications until 1979, when the first edition of the Ontario Highway Bridge Design Code (OHBDC) (3) was published. The AASHTO specifications were used by choice, because Ontario, like other Canadian provinces, has full jurisdiction over its highways and related matters, which include the formulation and enforcement of vehicle weight laws and the choice of design codes for its highways and bridges. In 1976 the Ministry of Transportation and Communications (MTC) of Ontario decided to write a highway bridge design code of its own, mainly for the following reasons:

1. The lack of conformity between heavy vehicles in Ontario and the AASHTO design vehicles. It is noted that Ontario permits much heavier vehicles on its highways than do most other jurisdictions in North America.
2. The difficulty and tardiness in the incorporation of latest research findings, however significant, in the AASHTO specifications.
3. A belief that the limit-state philosophy would lead to economy of design and uniform, predictable levels of safety in bridges.
4. The need to have a code in SI units in compliance with the government's commitment to metric conversion.

The first edition was written by 17 technical subcommittees under the steering control of an 11-member Code Development Committee in the relatively short time of about 3 years. This first highway bridge design code with a limit-state design format was written by a team of about 80 engineers from both within and without Ontario. Details of its development have been given elsewhere (4).

Soon after the publication of the first edition, work was started on the revision of the code. This work led to the second edition of OHBDC, which was published in late 1983. The purpose of this paper is to give a brief account of the implementation of the first edition and also to identify major changes that have taken place since the first edition.

IMPLEMENTATION OF FIRST EDITION

Following the limit-state format of the code, designers were required to consider both the ultimate and the serviceability limit states. The former limit state corresponds to the maximum load-carrying capacity, and the latter, which includes cracking, vibration, fatigue, and permanent deformations, is associated with loadings for normal use. The resistance and load factors specified in the code were calibrated to a target safety index value of 3.5 (5). The calibration was carried out for reinforced-concrete, prestressed-concrete, and steel structures from relevant available statistical data. Such data were not available for substructures, wood bridges, and soil-steel structures. Because of the lack of prior knowledge of the limit-state methods for these items, the relevant design equations were calibrated less rigorously: The calibration could only be done with respect to designs obtained from other North American codes.

Most problems in implementation of the code related to sections on foundations, wood bridges, and