AN INVESTIGATION OF THE BEHAVIOR OF A THREE-SPAN COMPOSITE HIGHWAY BRIDGE.

MISSOURI STATE HIGHWAY DEPARTMENT
UNIVERSITY OF MISSOURI, COLUMBIA
FEDERAL HIGHWAY ADMINISTRATION
An Investigation of the Behavior of a Three-Span Composite Highway Bridge

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Study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

This investigation considered the feasibility of conducting a fatigue test and a subsequent ultimate load test on a three-span continuous composite highway bridge in Butler County, Missouri. An analytical study, in which the bridge was treated as a multi-degree of freedom system, showed that critical fatigue zones occur in the flanges within the end spans. The base metal under the top-flange shear connectors between a field splice and the end of a cover plate and the base metal at the end of a cover plate in the top and bottom flanges are susceptible to fatigue failure. This anticipated test behavior simulates the concern of the current AASHO Specifications. Fatigue failure is probable within 600,000 cycles, at a stress range of approximately 14 ksi. A shakedown analysis of the bridge revealed that an ultimate load test could be performed if the bridge deck is cut longitudinally, to include only the two interior girders. The test vehicle recommended for this test is a special overload vehicle such as a military tank carrier. This vehicle should cause incremental collapse of the reduced bridge. A series of steady-state resonance tests were conducted on a single-span composite model to assess the practicability of the proposed fatigue testing technique. These laboratory tests verified that a "moving mass" - electrohydraulic actuator combination attached to the model would perform satisfactorily as a loading technique to induce resonant vibrations. The performance of this system established the capability of this technique to produce the required stress levels for the fatigue test on the prototype bridge. This study has demonstrated that the proposed fatigue and ultimate load tests of the bridge are feasible and meaningful.

Key Words:
continuous-composite bridge, fatigue test, ultimate load test, electrohydraulic actuators
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MISSOURI STATE HIGHWAY DEPARTMENT

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The opinions, findings, and conclusions expressed in this publication are not necessarily those of the Federal Highway Administration.
ABSTRACT

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CHAPTER 1
INTRODUCTION

1.1 PREFACE

As a result of flood control work on the St. Francis River, a relatively new (1963) three-span continuous composite girder highway bridge in Butler County, Missouri, is scheduled for demolition, see Figs. 1-1 and 1-2. A rare opportunity for full-scale destructive testing of a modern structure has therefore arisen.

Professors Salane, Duffield, McBean, and Baldwin of the College of Engineering at the University of Missouri-Columbia were awarded a contract by the Missouri State Highway Department and the Federal Highway Administration to study the feasibility of a field investigation of the dynamic properties, fatigue behavior, and the ultimate load capacity of the superstructure. Dr. T. V. Galambos of Washington University served as a consultant for the ultimate load study. The results and conclusions of this feasibility study are documented in this final report.

1.2 BACKGROUND ON BRIDGE DYNAMICS AND FATIGUE

Recent design innovations and improvements in construction materials and techniques have combined to achieve significant economies and aesthetically pleasing profiles in bridge structures. These advances, however, may be accompanied by an increased susceptibility to undesirable vibrations and structural fatigue.

Although few fatigue failures have been reported, concern is mounting because of the increasing use of welds and high-strength
FIGURE 1-1. AERIAL PHOTO OF TEST BRIDGE
steels, and the likelihood of future heavy traffic inducing repeated high stress levels. The importance of periodic inspections to guarantee safe performance and adequate maintenance was recently emphasized by the ASCE Subcommittee on Inspection of Steel Bridges for Fatigue.(2) It has been suggested that cumulative damage due to fatigue under service conditions may change the dynamic properties of a bridge sufficiently that occasional determination of natural frequencies, damping characteristics, and other properties could serve as a practical inspection tool.(3)

According to current (1971) AASHO Specifications,(4,5,6) design for fatigue requires consideration of three factors:

1. Required fatigue life of the bridge.
2. Regions where fatigue may be a problem.
3. The allowable stress as a function of the ratio of minimum stress to maximum stress.

An extensive review of the state-of-the-art on structural fatigue and its relation to the AASHO Specifications was prepared by the ASCE Task Committee on Structural Fatigue.(7 to 14) A comprehensive treatise on fatigue in welded structures was written by Gurney.(15) Reemsnyder(16) reviewed a practical approach to the analysis and presentation of fatigue data. Pertinent to the design of composite girder bridges is a report by Daniels and Fisher(17) in which the researchers recommended fatigue specifications for shear connectors.

Fatigue failure is characterized by the initiation of a microscopic crack that propagates under repeated application of stress until eventually the effective cross-section can no longer sustain the load.

Ideal conditions for fatigue failure occur in the base metal
adjacent to the fillet weld at the end of a cover plate. Consequently, considerable attention has recently been focused on this zone which is of great concern to the bridge designer. The AASHO Road Tests\textsuperscript{(18)} at Ottawa, Illinois, demonstrated the significance of the problem in a major series of field tests. In 1967 the ASCE Subcommittee on Cover Plates\textsuperscript{(19)} reviewed pertinent tests to that date and recommended an extensive research study of the fatigue life of cover-plated beams with various configurations. A substantial test program was subsequently undertaken by Fisher et al.,\textsuperscript{(20)} the outcome of which was a set of design recommendations for consideration by AASHO. Stress range was determined to be the most significant stress variable. The AASHO 1971 Interim Specifications reflected Fisher's findings but retained stress ratio as the design parameter until ongoing research has been completed and a comprehensive design criteria can be established.

1.3 BACKGROUND ON ULTIMATE STRENGTH OF BRIDGES

Current design of continuous girder highway bridges is based on elastic theory consistent with the AASHO specifications. Members are proportioned to ensure elastic behavior in resisting the most severe combinations of dead and live loading. The allowable steel stresses are established as an appropriate fraction of the yield stress. Wherever buckling and fatigue criteria govern, the allowable elastic stresses are subject to modification.

Although inelastic behavior serves as the basis of an acceptable design procedure for concrete and steel members in buildings, the adequacy of a comparable method for designing members in a bridge has not yet been recognized. Nonetheless, a shakedown analysis, which was
originally developed for steel buildings, can be modified to account for the nonproportional and repetitive characteristics of bridge loading.

A state of "shakedown" is achieved in a redundant structure in the following manner. A prescribed loading is applied which causes inelastic strains in some portions of the bridge but which does not precipitate a failure. Upon removal of the loading, residual deflections will remain. If the load is reapplied and removed, a further increase in the residual deflections will occur. Eventually, subsequent reapplications of the prescribed loading will no longer produce an increase in residual deflections. In fact, the loading is now resisted in an elastic manner.

Two modes of failure are considered in a shakedown analysis. If the loading is too heavy, the residual deflections may continue to increase with repeated application of the loading until a member fails. Alternatively, a member may fail from inelastic fatigue brought on by the loading and unloading cycles. The smaller loading from either mode is termed the shakedown load.

Considerable theoretical research on shakedown is reported in the literature. However, these theoretical concepts require substantial verification by experimental tests before they can be applied in the design of composite bridges and consequently incorporated into the specifications. The existing bridge in Butler County, Missouri, presents itself as a unique experimental full-scale test specimen.

1.4 SCOPE OF THE RESEARCH

In this report, the results of a comprehensive feasibility study are presented. The findings are summarized in Chapter 2. Throughout
the report, the terminology "Phase I" refers to this now-completed study. The proposed field study is denoted by "Phase II."

Analytical studies were undertaken to predict both the fatigue and ultimate load characteristics of the bridge. The bridge is described in Chapter 3, and Appendices III and IV. In Chapter 4 an overview of the proposed fatigue test is reported. The details of the instrumentation and experimental procedures are included in Chapter 5. A summary of the proposed ultimate load test comprises Chapter 6. The details are left to Appendix VII.

The feasibility of the experimental program from mechanical and electronic points of view was substantiated by an extensive series of tests conducted on the simple-span model pictured in Fig. 1-3. The findings of the model test are summarized in Appendix II, whereas the experimental procedures, equipment, and results are reported in detail in Appendix VIII.
CHAPTER 2
SUMMARY AND CONCLUSIONS

2.1 GENERAL SUMMARY AND CONCLUSIONS

The overall study in Phase I of this project was directed toward evaluating the feasibility of performing a fatigue test and a subsequent ultimate load test on a three-span composite highway bridge in Butler County, Missouri. The study comprised

1. A series of analytical investigations of the bridge and experimental tests of a simple laboratory model which led to a fatigue test procedure, and
2. A shakedown analysis of the bridge from which a procedure evolved for conducting the ultimate load test.

The results of the Phase I study have demonstrated that the proposed fatigue and ultimate load tests of the bridge are feasible. A summary and conclusions related to various aspects of this study are presented in the following articles.

2.2 FATIGUE TEST

2.2.1 Model Test

In order to assess the practicability of performing a fatigue test on the full-scale bridge, a laboratory model consisting of two wide-flange steel beams composite with a concrete deck was subjected to a series of steady-state resonance tests. Experimental data consisted of strains, deflections and accelerations at locations which would serve to corroborate the analyses. For the majority of the tests, the source of
excitation was a 5 kip electrohydraulic actuator mounted at mid-span. Attached to the rod of the actuator was a 40 pound steel plate. The prescribed movement of the plate under closed-loop control at the resonant frequency of the model produced dynamic deflections in the model about the static equilibrium position. In all tests it was found that the peak amplitude of the model deflections could be maintained at a constant level with a fixed setting of the stroke of the actuator. In two tests the 5 kip actuator was replaced with two 80 kip actuators mounted side-by-side at mid-span.

The electrohydraulic actuators, both small and large, performed very satisfactorily as a shaker device to induce resonant vibrations in the model. No unusual difficulties were encountered in using the actuators, hydraulic power supply, and other mechanical fixtures which were in the test.

By and large, all instrumentation worked satisfactorily. No serious problems were experienced in measuring steel strains, deflections at frequencies under 10 cps, and accelerations. It was found that some improvements should be made in the data acquisition system. These improvements would facilitate the reduction of data. Details of the model and a description of the instrumentation and test procedures used in the tests are presented in Appendix VIII.

The observed behavior of the model substantially agreed with that predicted from analytical models. Analyses were based on both one-degree-of-freedom and multi-degree-of-freedom systems. The one-degree system was used to predict the overall performance characteristics of the actuators, structure (model or bridge) and hydraulic power supply.
The proposed fatigue test is based on studies made from analytical models. The correspondence between experimental and predicted values in the tests verified the reliability of the analytical models. Experimental results and analyses of selected data for the model tests are contained in References 22 and 23. A review of the results of the model test and a prognosis of the behavior of the bridge during the proposed fatigue test are presented in App. II.

2.2.2 Fatigue Study

The bridge was analyzed for maximum and minimum combinations of dead, live and impact stresses in accordance with the AASHO Code. Allowable fatigue stresses were based on the 1969 AASHO Specifications, and the 1970 and 1971 Interim Specifications. These analyses indicated that a critical zone in fatigue occurs in the end spans between the end of a cover plate and the field splice. In view of the fact that the bridge has four girders and two end spans, the field test would involve 16 critical zones including both top and bottom flanges. On the basis of the results of a fatigue study by Fisher (20), the estimated fatigue life at the planned stress range for the critical zone in the bottom flange of an exterior girder is about 140,000 cycles; whereas, the top flange of an interior girder has a projected fatigue life of about 600,000 cycles. The stress range required in the fatigue test can be attained with one 20 kip actuator situated at midspan on the bridge with a weight of 7 kips attached to the actuator rod. The bridge deflection at midspan will be ±1.73 inches about the static equilibrium position. Dead load ballast of 40 kips will be placed in each span to minimize the maximum tensile strains in the concrete deck at the center of the three spans when these spans are at their uppermost position.
during a fatigue cycle. Some cracking of the concrete deck is anticipated, although the ballast was designed to restrict the dynamic tensile strains to approximately the level encountered from the service loads. The model tests showed that deterioration of the concrete deck reduced the bending stiffness of the cross-section to that approaching a semi-composite state. The semi-composite state represents a lower bound stiffness which assumes that only the steel reinforcing bars in the slab act compositely with the steel girders in the positive moment regions. However, it should be noted that no ballast was used in the model tests and the model was subjected to accelerations of 20 g's in a third mode test conducted to check the performance of the small actuator. For the fatigue test, the maximum acceleration level will be less than 1 g. Most of the decrease in bridge stiffness would probably occur in the first few thousand cycles. In any event, the vibration excitation system can be adjusted to compensate for any changes in stiffness so that the required stress range for the fatigue test can be maintained. The analyses related to the fatigue study are presented in Chapter 4.

2.2.3 Bridge Foundation

A study was undertaken to ascertain if the vibratory loading required for the fatigue test would cause excessive settlement of the bridge piles. The results of the study indicate that

1. The allowable pile bearing capacity as determined from the Engineering News Record formula exceeds the maximum load (static + dynamic) to be imposed on each pile, and

2. The resonant frequency of a pile is approximately 10 cps whereas the fatigue test will be conducted at 2.5 cps.
Consequently, pile resonance should not be a contributing factor to settlement. Nonetheless, some settlement is anticipated. The margin between the allowable pile capacity, which contains a factor of safety, and the actual loading will tend to minimize the settlement. The dynamic behavior of the bridge foundation is discussed in App. I.

2.2.4 Field Tests

The initiation and subsequent growth of fatigue cracks will be detected by the following procedures: (Ultrasonics may also be used.)

1. Monitoring of strain gages located in critical fatigue zones during the test.
2. Changes in dynamic behavior during the test.
3. Visual inspection and photographs after every 25,000 cycles during a stoppage of the test.

When a crack has propagated through 50 to 100 percent of one flange, the fatigue test will be terminated. The fatigue behavior of structural elements and connections, such as diaphragms, field splices and the deck itself, will also be subject to visual inspection. As the test progresses, field judgment may dictate that the fatigue test be terminated prior to achieving a 50 percent crack in a flange.

One of the specific aims of this research study is to determine whether the bridge properties are significantly altered by cumulative damage during the service life of the bridge and whether a forced vibration test could be used to detect this damage. During the series of model tests, progressive deterioration of the concrete slab caused significant changes in stiffness and corresponding changes in the resonant frequencies. Damping was also affected. The primary intent of the laboratory tests was to establish the feasibility of the experimental procedure rather than to simulate service loading on a bridge.
In order to determine if cumulative damage during the service life is detectable, low level resonance tests will be conducted when the fatigue test is interrupted for inspection purposes. The ballast will be removed for these tests. Chapter 5 contains a detailed explanation of the experimental procedures and data acquisition systems for the fatigue and dynamic properties tests.

2.3 ULTIMATE LOAD TEST

An ultimate load test is scheduled upon completion of the fatigue test. The ultimate strength of the bridge was determined from a shake-down analysis. Consideration was given to the details of the bridge in order to ensure that secondary elements would not fail prior to application of the loading which would initiate incremental collapse. Fatigue damage to the girders will be repaired prior to the initiation of the ultimate load test.

A rubber-tired, special overload test vehicle similar to that used in the AASHO Road Test will be moved slowly across the bridge. The vehicle load will be incrementally increased. Because the strength of the existing bridge far exceeds the loading capability of the test vehicle, the bridge deck will be cut longitudinally between the interior and exterior girders. With the bridge modified in this manner, the test vehicle will be adequate to cause the remaining two interior girders and the slab to fail. The test is described in Chapter 6. Appendix VII comprises the complete analytical investigation which established the feasibility of performing this test.
CHAPTER 3
THE TEST BRIDGE

3.1 DESCRIPTION OF THE TEST BRIDGE

The two-lane, three span (72'-93'-72'), continuous, composite girder bridge to be tested crosses a levee ditch parallel to the St. Francis River, 19.1 miles S.E. of Poplar Bluff, Missouri, on Route WW. The bridge was designed in early 1962, according to 1961 AASHO specifications (24), for one lane loaded with H15-44.

The principal features of this bridge are shown in Fig. 3-1. The section details are described in App. III. Four 30-inch wide flange girders of ASTM A36-60T, cover-plate over the main piers (13'-6" each side of piers), carry a six-inch slab acting compositely with the girders. Bolted field splices for the girders are located 18 ft. from the main piers in the end spans and 22 ft. from these piers in the center span.

Unshored construction was used during the placing of the slab. The 22 ft. wide roadway has a parabolic crown. Over the girders a varying concrete haunch was used to ensure a roadway parallel to grade. Slab reinforcing consists of transverse #5 bars at 6½" top and bottom, and longitudinal #4 and #5 bars at various spacings top and bottom. An independently cast reinforced concrete curb anchors a steel guard rail. There are no sidewalks. Shear connectors are C4x5.4, 6½" long, fillet welded to the top flange of the girders at varying spacing except in the region 12'-6" from the main pier in the end span to 14'-9" from the pier in the center span.
FIGURE 3-1. ELEVATION AND SECTION VIEWS OF TEST BRIDGE
The abutments and piers are supported by precast concrete piles driven to minimum 32 ton capacity in sand.

3.2 APPLICABLE FATIGUE REQUIREMENTS OF THE AASHO CODE

The test bridge was designed prior to the introduction of fatigue constraints in the AASHO specifications. It is important, however, to review the latest applicable requirements.

The allowable design stress at points subjected to repeated variations of stress is the lower of 20 ksi and the applicable allowable fatigue stress, $F_r$. The latter is primarily a function of $R$, the algebraic ratio of the minimum stress to the maximum stress at the point on the cross-section under review, and the number of cycles of maximum stress to be expected during the service life of the bridge.

Appendix IV summarizes the fatigue formulas for $F_r$ that are applicable to this bridge, as specified by the latest of the 1969 AASHO Standard Specifications\(^4\), the 1970 Interim Specifications\(^5\), and the 1971 Interim Specifications\(^6\). The categories A, F, G, and J are those used by AASHO.

3.3 SERVICE STRESSES ACCORDING TO THE AASHO CODE

The dead load of the girders and deck was carried by the girders alone due to the unshored construction, whereas the dead load of the curbs and railings is carried by the composite action of the exterior girders. Figure 3-2 shows the dead load stresses in the exterior girders. Behavior of the interior girder is similar. In all calculations, account has been taken of the varying moment of inertia.

The maximum and minimum combinations of dead, live, and impact stresses generated by two lanes of H15-44 truck or lane loading\(^4\) are
FIGURE 3-2: DEAD LOAD STRESSES IN EXTERIOR GIRDER AT EXTREME FIBER
illustrated for the exterior girder in Fig. 3-3. These envelopes together with the fatigue specifications noted in App. IV define the variation of allowable stress along the bridge. These allowable stresses for 500,000 cycles are also shown in Fig. 3-3. A similar figure can be constructed for the interior girder. The static allowable stress governs everywhere except in the region of the field splices. The most critical zones exist at the end of the cover plates.
Figure 3-3. Stress envelopes in exterior girder due to AASHO load.
CHAPTER 4
THE FATIGUE STUDY

4.1 TEST VARIABLES

During a search for the fatigue test procedure that would be most meaningful, an extensive series of analytical investigations was undertaken. The bridge was modelled as a continuous beam of variable rigidity. Linearly elastic, small deflection theory was assumed. A more refined idealization could not be justified, in view of the uncertain geometric and material properties of the structure. The analytical technique is explained in Appendix V.

The variables considered in the study were:
1. Mode shape to be excited.
2. Number and position of the actuators.
4. Weight attached to the moving piston.
5. Position and magnitude of the ballast.
6. Equivalent damping in the bridge.

4.2 TEST CRITERIA AND DEVELOPMENT OF TEST SCHEME

In order to achieve the most meaningful data, a set of test criteria was formulated:
1. The level of stress near the end of the cover plates should exceed the allowable stress at 500,000 cycles, as calculated from the AASHO formulas (see Arts. 3.2 and 3.3) for combined dead, ballast, and dynamic loading.
2. In the critical region near the end of the cover plates, the envelopes of minimum and maximum stress for the test loading should resemble the envelopes for service loading.

3. Tensile strains in the concrete should be of the same order of magnitude as the service strains produced by two lanes of H15-44 truck or lane loading.

4. The feasible scheme must provide a reserve capability to meet contingencies.

Furthermore, equipment capabilities suggested the following experimental constraints:

1. Piston stroke should be limited to ±3.5 inches.
2. Attached weight should be kept as small as practicable.
3. No uplift should occur on any supports.
4. Vertical accelerations of the deck should not exceed 1 g. (32.2 ft/sec²).

Excitation in both the first and third modes, shown in normalized form in Fig. 4-1, was carefully examined. The high curvatures at midspans and over the piers in the third mode appear promising, but a detailed study indicates that the power input (in terms of oil flow capacity, primarily) required to obtain the desired stress levels at the points of interest is too great for practical consideration. The curvatures near the ends of the cover plates are simply not adequate to provide a useful stress range. Furthermore, uplift at the end abutments and high vertical accelerations of the deck raise the problems of securely anchoring all equipment and perhaps adding ballast over the abutments. Excitation in the second mode would create essentially the same problems because of low curvatures near the field.
FIGURE 4-1. MODE SHAPES FOR THREE LOWEST NATURAL FREQUENCIES

<table>
<thead>
<tr>
<th>MODE</th>
<th>BALLASTED NATURAL FREQUENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.27 CPS</td>
</tr>
<tr>
<td>2</td>
<td>3.57 CPS</td>
</tr>
<tr>
<td>3</td>
<td>4.14 CPS</td>
</tr>
</tbody>
</table>
splices. The above problems do not occur for first mode excitation, so it was decided to pursue only this mode in further studies.

Actuators were positioned either at the center of the mainspan or at the centers of both endspans (actuators in phase) for this analytical study. Either scheme could be used but fewer test problems are likely to occur if the center of the mainspan is used. With the available electronics, one or two actuators could be used at this position. Two actuators necessitate careful electronic control but offer versatility (torsion testing is facilitated, for example) and the desirable features of a reduced moving weight on the piston and a smaller mounting system than is required for one actuator. On the other hand, a system with a single actuator is more economical and easier to control. A single actuator has therefore been selected for the test program. In order to minimize transverse bending of the slab, a stiff spreader beam is needed to distribute the load directly to the bridge girders.

The piston stroke and attached weight are coupled parameters which, in combination with the excitation frequency, determine the dynamic force imparted to the structure. This force is directly proportional to the stroke and weight, and proportional to the square of the excitation frequency (see App. V). The actuator selected for the field test is discussed in Chapter 5.

Concrete tensile strains calculated on the basis of a linear variation of strain through a cross-section, for two lanes of H15-44 lane loading, are of the order of 40 micro-inches per inch at the center of the mainspan and 360 micro-inches per inch over the main piers. It was decided that a meaningful test program should keep the experimental strains in this range. This objective is readily achieved
by adding ballast to the deck in the form of concrete weights. To minimize the effect of the ballast on dampening the vibrations, these weights should be supported so that the ballast does not act as a part of the structure in any way. A suitable support system is described in Chapter 5. The effect of ballast in the center span only, and alternatively in all three spans, was examined for several weights of ballast. Loading all three spans leads to a low ballast stress, and hence high stress range, in the critical region between the field splice and the end of the cover plate in the end span. The three-span ballast scheme has therefore been adopted. Of course, the addition of ballast reduces the natural frequencies of vibration.

Laboratory tests on a model (see Apps. II and VIII) and reports by others indicate that damping should not exceed 2.0 percent. During the AASHO Road Tests (18) damping was determined, by the log decrement method, to be 1.1 percent on one composite bridge, and 0.8 percent on another. Because the response of the bridge is inversely proportional to this assumed damping, unexpectedly high damping must be countered by a reserve capability in the test scheme.

Furthermore, in order to maintain the desired stress levels in the steel girders as progressive degradation of the concrete in the bridge deck occurs, it will be necessary to increase the weight attached to the actuator and/or the stroke. This point is discussed further in Arts. 4.4 and II.2.

4.3 FATIGUE TEST SCHEME

The test philosophy and practical considerations outlined in the previous article have led to the test scheme illustrated in Fig. 4-2, the features of which are as follows:
FIGURE 4-2. FIRST MODE FATIGUE TEST SCHEME
1. Excitation in the first mode at about 2.3 Hz. This mode shape is shown in Fig. 4-1.

2. 40 kips ballast near the center of each span; each span to have four concrete modules, the placement and support of which is described in Chapter 5. This portable ballast can be used for the ultimate load tests as well.

3. An actuator at the center of the main span, with 2.15 inches stroke (single amplitude) and 7 kips weight attached to the piston.

Chapter 5 is devoted to a detailed explanation of the experimental setup.

4.4 FATIGUE TEST DEFLECTIONS AND STRESSES

This test system should provide dynamic deflections about the dead load plus ballast position as indicated in Fig. 4-3. The dynamic load was presumed to be equally shared by the girders. In the figures, the term "inertial" refers to the dynamic effect. Under the actuator the deflections are 1.37 inches due to the dead load, 0.17 inches due to the ballast, and ±1.73 inches due to the excitation. That is, the peak-to-peak oscillation of the deck near the actuator should be almost 3½ inches! In fact, the oscillation will be even greater as probable degradation of the slab occurs.

The vertical accelerations of the deck are 13.1 ft/sec² and 29.4 ft/sec² at the center of the end and main spans, respectively. Therefore, only a nominal "tie-down" of the ballast in the center span should be required.

The bending moment in the bridge (total 4 girders) will vary as shown in Fig. 4-4. It should be noted that the inertial moments near
FIGURE 4-3. DEFLECTIONS OF GIRDER DURING FATIGUE TEST
Figure 4.4. Bending moments in the bridge during fatigue test.
the end of the cover plate in the center span are not as high as those in the corresponding regions of the end spans.

No support problems due to uplift are anticipated. The total reaction on the abutment will vary from 38 kips to 125 kips during each cycle while that on the main piers will vary from 235 kips to 370 kips. The design loads are 150 kips and 395 kips for the abutments and main piers, respectively.

Subsurface settlement of the piers due to the vibration should not be a problem. This point is discussed in App. I.

Figure 4-5 summarizes the range of steel stresses and concrete strains expected at several points in the flanges of interior and exterior girders. The stresses at midspan exceed the allowable, but this is of no significance as far as fatigue is concerned. The stresses at the main piers approach the design levels. Most important is the high stress range in the flange where the cover plate ends. This is a critical zone. The concrete tensile strains exceed the service strains at midspans, but they are nonetheless low level strains (and the slab has considerable reinforcement).

For the exterior girder, the complete range of steel stresses between field splices, for fully composite action, is shown in Fig. 4-6. The solid lines denote the dead plus ballast, plus or minus inertial stress. Superimposed on these curves are the allowable fatigue stresses for 500,000 cycles of stress reversal, according to the formulas in App. IV. The critical zones are in the base metal adjacent to the fillet welds at the end of the cover plate (top and bottom flange) and at the shear connectors between the field splice and the cover plate (top flange), in the end spans. Since it is these zones that are usually found by design engineers to be important, the test
FIGURE 4-5. RANGE OF STEEL STRESSES AND CONCRETE STRAINS DURING FATIGUE TEST
FIGURE 4-6. TEST STRESS RANGE IN EXTERIOR GIRDERS

MAX, MIN STRESS

ALLOWABLE

500,000 CYCLES

FATIGUE STRESS

ALLOWABLE

500,000 CYCLES

FATIGUE STRESS

MAX, MIN STRESS

ALLOWABLE

TOP FLANGE

BOTTOM FLANGE

STRESS

KSI

STRESS

KSI

32
should provide considerable meaningful data. No other zones are critical in the proposed test.

Test data are not available for estimating the fatigue life of the base metal under the shear connectors. Therefore, the emphasis in the following discussion is centered on the anticipated behavior at the end of the cover plate.

The allowable stress does not directly provide a means of predicting the number of cycles to failure of the base metal adjacent to the end of the cover plate. Because the current (1971) AASHO code requirements for cover plate design are based on an extensive study by Fisher et al. (20), it is appropriate to employ their regression curve developed from experimental data to estimate the fatigue life. The formula, based on fourteen tests denoted by Fisher as the CRA series, is

\[ \log N = 8.9754 - 2.8768 \log S_r \]  

(4.1)

where \( N \) is the number of cycles to failure, and \( S_r \) is the stress range in ksi. Failure was defined by Fisher in terms of an increase in beam deflection that corresponded to a cracked area equal to about 75 percent of the flange area. For most experiments, in common with the majority of other laboratory studies (7), failure occurred in the tension flange, accompanied by minor cracking at the toe of the fillet weld in the compression range. Three tests, however, produced cracks which propagated significantly through the compression flange. It will therefore be important to monitor the strains in both flanges of the bridge. Equation 4.1 indicates that failure should be expected as early as 140,000 cycles after the test begins. A complete set of predictions for failure, as defined by this equation, is given in Table 4-1. The allowable stresses are given for comparison. The
### Table 4-1. Estimated Fatigue Life of Base Metal Adjacent To Fillet Weld at End of Cover Plate in End Span

<table>
<thead>
<tr>
<th>Stress Level</th>
<th>Composite Action</th>
<th>Allowable Stress</th>
<th>Maximum Stress</th>
<th>Minimum Stress</th>
<th>Fatigue Life</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior Girder Top Flange</td>
<td>100,000</td>
<td>13.6</td>
<td>7.3</td>
<td>315,000</td>
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<tr>
<td></td>
<td>Exterior Girder Top Flange</td>
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<td>13.5</td>
<td>7.2</td>
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<tr>
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<td>Interior Girder Bottom Flange</td>
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<td>7.2</td>
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<td>315,000</td>
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**Minimum Maximum Stress N**

**Stress Stress Range cycles**

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<th>Kilobars (ksi)</th>
<th>Kilobars (ksi)</th>
<th>Kilobars (ksi)</th>
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</thead>
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<tr>
<td>1.15</td>
<td>13.95</td>
<td>12.8</td>
</tr>
<tr>
<td>0.57</td>
<td>15.3</td>
<td>14.5</td>
</tr>
<tr>
<td>-17.48</td>
<td>1.82</td>
<td>19.3</td>
</tr>
<tr>
<td>-19.20</td>
<td>2.24</td>
<td>21.4</td>
</tr>
</tbody>
</table>

---

**TABLE 4-1. ESTIMATED FATIGUE LIFE OF BASE METAL ADJACENT TO FILLET WELD AT END OF COVER PLATE IN END SPAN**
values in the table clearly indicate the feasibility of the bridge test. Even if fatigue failure should occur in the top flange, the test duration would be relatively short. The anticipated fatigue life of the base metal under the shear connectors is about the same as that at the end of the top flange cover plate. Therefore, it will be necessary to monitor the flange strains directly below the connectors, in the region between the field splice and the end of the cover plate, in the end spans. No attempt has been made to estimate the number of cycles that the bridge may already have experienced due to traffic.

In the foregoing discussion, composite behavior was assumed. As the fatigue test progresses, however, it is inevitable that the concrete in the bridge deck will crack. The reduced stiffness will cause the fundamental frequency to drop, and it will be necessary to increase the stroke of the actuator and the attached weight in order to maintain the initial stress range. Laboratory model studies (see App. II) have warned that the bridge could approach a semi-composite state. That is, the deck concrete could eventually fail to provide a significant flexural stiffness contribution, although the effort of the reinforcement in the slab would not be much impaired. Although it is doubtful that the bridge could degrade as extensively as did the unballasted model, the nature of the phenomenon is so complex and unpredictable that a conservative approach must be observed. An analysis of the bridge for semi-composite action should provide a bound on the requirements of the experimental equipment. If the stress range at the end of the cover plate is maintained for the duration of the test, then the fatigue life should not be affected. Table 4-1 reflects this fact. A comparison of the requirements and anticipated
results for both composite and semi-composite behavior is documented in Table 4-2. The stress range is given in the table only for the exterior girder. For the interior girder the behavior is similar. The table indicates a dramatic change in the top flange stress range at midspans, but minor changes elsewhere.

Figure 4-6 should be compared with Fig. 3-3 which indicates the minimum and maximum service stresses calculated for 2 lanes of H15-44 loading, together with the allowable fatigue stresses for 500,000 cycles. The base metal at the end of the cover plate is of most significance for service conditions, in common with the proposed test.

It must be emphasized that the deflections and stress ranges predicted for the bridge test are based on an assumed two percent damping and an assumed lateral distribution of load. The results must be interpreted in the correct perspective—they are careful engineering estimates that demonstrated the feasibility of the field test. The predictions may have to be adjusted to compensate for the true conditions encountered in the test.
<table>
<thead>
<tr>
<th>Item</th>
<th>Composite</th>
<th>Semi-Composite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actuator Stroke</td>
<td>2.15&quot;</td>
<td>3.00&quot;</td>
</tr>
<tr>
<td>Attached Weight</td>
<td>7 kips</td>
<td>10.5 kips</td>
</tr>
<tr>
<td>Fundamental Frequency</td>
<td>2.27 cps</td>
<td>1.53 cps</td>
</tr>
<tr>
<td>Maximum Acceleration</td>
<td>29.4 ft/sec²</td>
<td>24.6 ft/sec²</td>
</tr>
<tr>
<td>Abutment Reaction</td>
<td>38k to 125k</td>
<td>41k to 123k</td>
</tr>
<tr>
<td>Pier Reaction</td>
<td>235k to 370k</td>
<td>254k to 350k</td>
</tr>
<tr>
<td>Maximum Dynamic Deflection</td>
<td>±1.73&quot;</td>
<td>±3.20&quot;</td>
</tr>
<tr>
<td>Stress Range in Exterior Girder</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Bottom Flange</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mid End Span</td>
<td>19.9 ksi</td>
<td>24.6 ksi</td>
</tr>
<tr>
<td>End Cover Plate, End Span</td>
<td>21.4</td>
<td>21.4</td>
</tr>
<tr>
<td>Main Pier</td>
<td>10.3</td>
<td>10.1</td>
</tr>
<tr>
<td>Mid Center Span</td>
<td>30.4</td>
<td>31.4</td>
</tr>
<tr>
<td>b) Top Flange</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mid End Span</td>
<td>1.5</td>
<td>16.7</td>
</tr>
<tr>
<td>End Cover Plate, End Span</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>Main Pier</td>
<td>6.6</td>
<td>6.4</td>
</tr>
<tr>
<td>Mid Center Span</td>
<td>2.0</td>
<td>21.3</td>
</tr>
</tbody>
</table>

**TABLE 4-2** COMPARISON OF COMPOSITE AND SEMI-COMPOSITE ACTION, REQUIREMENTS AND RESULTS, 2% DAMPING
CHAPTER 5
EXPERIMENTAL SETUP AND PROCEDURES FOR BRIDGE TESTS

5.1 INTRODUCTION

This chapter is concerned with the experimental setup and procedures for the fatigue test and the dynamic tests of the bridge. The dynamic loading system, the transducers and the data acquisition systems presented in this chapter have been selected on the basis of the results of the experimental model study and the bridge fatigue analysis.

The results of the experimental model study show that a vibration excitation system composed of a "moving mass" - hydraulic actuator combination performs quite satisfactorily as a dynamic loading technique to induce resonant vibrations in a structure. The performance of this system leaves no doubt as to the ability of this dynamic loading technique to excite the test bridge at resonance to the required stress levels for the fatigue test.

The transducers selected for the field study are very similar to those used in the model study of Phase I. However, some changes are required in the design or type of transducer in order to meet the response conditions of the bridge and environmental conditions at the field site.

A study of the instrumentation needs for the field study shows that two data acquisition systems are required: one for the fatigue test and one for the dynamic properties tests. The data acquisition system for the dynamic tests is similar to the system used during the Phase I investigation. However, a different system is required for the fatigue
test to handle the large number of channels of data associated with this test.

The test procedures for the fatigue test, which are outlined in Art. 5.6.1, are based on the results of the bridge fatigue analysis and to some extent on the information published in Fisher's (20) report. The set of dynamic tests selected for the bridge investigation is the same as that used during the Phase I model study.

5.2 VIBRATION EXCITATION SYSTEM

The schematic of the University of Missouri-Columbia structural vibration system to be used for the field test is shown in Fig. 5-1. The basic system essentially consists of two sections: the hydraulic section and the electronic section.

5.2.1 Hydraulic Section

The hydraulic section consists of a 20 kip hydraulic actuator, which is described in Art. 5.2.3, a 90 gpm high performance servo-valve, two 60 gpm hydraulic control manifolds and two trailer mounted 35 gpm, 3000 psi hydraulic power supplies developed by the University of Missouri-Columbia. The hydraulic power supplies use oil-to-air cooling which eliminates the need for an adequate water supply at the field site.

5.2.2 Electronic Control Section

The electronic section of the vibration system is basically a closed-loop (feedback) control system. The main part of the control section is a servo-controller which operates the servo-valve which in turn regulates the flow of hydraulic fluid into the actuator. Independent static and dynamic positioning of the actuator rod is provided by the servo-controller. The feedback signal from the LVDT located in the
FIGURE 5-1. SCHEMATIC FOR VIBRATION EXCITATION SYSTEM
base of the actuator, as illustrated in Fig. 5-1, is fed into an A-C transducer conditioner before it reaches the servo-controller.

The command signal for the control section comes from an MTS 410 function generator for the fatigue test, as shown in Fig. 5-1, and from a Spectral Dynamics SD 104 sweep oscillator for the dynamic test.

The purpose of the function generator and the sweep oscillator is to establish the driving frequency of the hydraulic actuator. It was found during the Phase I study that the sweep oscillator had frequency drift problems which make it unsuitable for use in the sustained fatigue test. However, the versatility of the sweep oscillator makes it ideally suited for short term dynamic tests.

The stroke of the hydraulic actuator, in terms of the LVDT voltage, is the primary control parameter for this experimental setup. Manual adjustment of the servo-controller is required to establish the control level of acceleration for the dynamic tests, or of strain for the fatigue test. The controlled acceleration or strain is then observed on an oscilloscope, as described in Arts. 5.4.2 and 5.4.3.

5.2.3 Hydraulic Actuator: Description and Location

The data from the fatigue study of the bridge, presented in Chapter 4, indicates that a hydraulic actuator with the following specifications will be required for the field tests:

1. 20 kip dynamic load rating,
2. 10 inch double amplitude stroke,
3. separate nitrogen gas system for statically positioning the "moving mass", and
4. pedestal base.

This actuator will provide approximately 100 percent reserve force capacity over the force required to achieve the strain levels needed for
the fatigue test at the resonant frequency for either the composite or semi-composite case.

The actuator will be positioned at the center of the main-span, as illustrated in Fig. 5-2, throughout the field test. To obtain uniform distribution of force to the four girders, a spreader beam, as shown in Fig. 5-3, will be placed between the actuator and the bridge. The spreader beam will be anchored to the bridge by rods passing through the concrete deck and attached to the four girders as illustrated in Fig. 5-3.

5.2.4 Moving Mass

To achieve the force needed for the fatigue test, a "moving mass" ranging between approximately 7 kips for the composite case and 10.5 kips for the semi-composite case will be employed. The weight will be approximately 3' square by 1'-7" to 2'-5" in height and will consist of individual steel plates bolted to an adapter plate attached to the actuator rod as shown in Fig. VIII-14 in App. VIII. The individual plates will be of such thickness as to facilitate handling during field assembly. This type of design will permit adjustment of the magnitude of the weight to meet field conditions.

5.2.5 Ballast

Each span of the bridge will contain 40 kips of ballast (see Chapter 4) consisting of four 10 kip concrete modules arranged on the spans as indicated in Fig. 5-2. The concrete modules will be composed of four 2.4 kips reinforced concrete slabs which are tied together and attached to two wide-flange beams as illustrated in Fig. 5-4. The concrete module was designed to be moved in four sections with a 2 ton fork-lift truck. The modules will be secured to the bridge, as shown in
FIGURE 5-2. FATIGUE TEST SETUP
FIGURE 5-3. ACTUATOR ATTACHMENT TO BRIDGE

SPREADER BEAM (TWO WF)

20 KIP HYDRAULIC ACTUATOR

"MOVING MASS"

7'

3'

3'
FIGURE 5-4. BALLAST DIAGRAM
Fig. 5-4, to prevent their movement during the fatigue test. The method of support for the ballast was chosen to minimize the effect of the modules on the damping characteristics of the bridge.

5.3 TRANSDUCERS

5.3.1 Strain Gages

The fatigue analysis indicates that one of the critical fatigue zones is in the base metal adjacent to the transverse fillet welds of the shear connectors shown in Fig. 5-5. There are eight of these zones in the test bridge, but only four zones will be strain-gaged as shown in Fig. 5-5. The four zones will consist of one exterior and one interior girder in each end-span in an asymmetric arrangement. They will coincide with enlargement A in Fig. 5-7.

The analysis in Chapter 4 indicates that the other critical zones in fatigue occur in the end-span adjacent to the fillet welds at the ends of the top and bottom flange cover plates. In the test bridge, there are sixteen of these potentially critical fatigue zones that will require strain gaging. Figure 5-6 shows the strain gage locations on the bottom flange of the girders for both the fatigue and the dynamic tests.

Each of the four critical fatigue locations, as defined by enlargement A in Figs. 5-6 and 5-7, will have four strain gages located 18" fore and aft of the fillet welds: two on the bottom flange of the girder and two on the underside of the top flange as illustrated in Fig. 5-7. The two gages on the top and bottom flanges respectively will be used to establish the strain gradient through their respective critical fatigue zones. Also, the top flange strain gages in conjunction with their mating strain gages on the bottom flange will be
FIGURE 5-5. STRAIN GAGE LOCATIONS FOR END-SPAN SHEAR CONNECTORS
Strain gage locations are symmetrical about mid-span except for gages associated with enlargements, which are asymmetric.

Figure 5-6: Strain gage locations.
FIGURE 5-7. STRAIN GAGE LOCATIONS (ENLARGEMENT A)
FIGURE 5-8. STRAIN GAGE LOCATIONS (ENLARGEMENT B)
used to determine the strain distribution through the girder.

It was reported by Fisher et al.\(^{(20)}\) that, for all specimens they tested, a fatigue crack originated in the flange at the toe of the cover plate end weld directly over the web. To detect the formation of a fatigue crack, a strain gage will be located near the center of the end weld at each of the sixteen potential fatigue zones, as illustrated in Fig. 5-7. In this figure the strain gage on the underside of the top flange, directly below the end of the cover plate, is the same gage as shown at this location in Fig. 5-5.

The neutral axis of the bridge may change during the field test due to possible deterioration of the concrete deck. To measure a representative change in the neutral axis location, a set of strain gages will be placed on one exterior and one interior girder at the center of the main-span as shown in Figs. 5-6 and 5-8.

To check the symmetry of the applied load, strain gages will be placed on the bottom flange of each girder at the center of the main-span and on the bottom flange of one exterior and one interior girder at the center of each end-span as indicated in Fig. 5-6.

5.3.2 Deflection Gages

Four deflection gages will be used during the field test and they will be mounted longitudinally on the bottom flange of one interior and one exterior girder at the center-span locations in one end-span and in the main-span as shown in Fig. 5-9. These deflection gages will be similar to, but will have a greater range than, those used in the Phase I tests and described in Reference 23.

5.3.3 Accelerometers

The fatigue and dynamic tests will require the use of two and
Figure 5-9. Accelerometer and Deflection Gage Locations

- Typical accelerometer attachment studs located on longitudinal axis of bridge.
- Bridge and accelerometer attachment studs are symmetrical about center.
- Location of center-span deflection gage.
- Location of end-span deflection gage.
- Field splice cover plate.
- Interior girder.
five accelerometers, respectively. These accelerometers will be of the piezoresistive or servo type.

Two of the accelerometers will maintain fixed positions throughout the field test. As shown in Fig. 5-13, one accelerometer (called the control accelerometer) will be located on top of the "moving mass" and the other accelerometer (called the reference accelerometer) will be located on the spreader beam at the base of the actuator. These accelerometers will measure the absolute accelerations of the "moving mass" and the actuator base and will thus indirectly measure the exciting force acting on the bridge. This force is equal to the moving mass times the relative acceleration of the "moving mass" with respect to the actuator base.

During the dynamic tests the response of the bridge will be measured by sequentially moving three accelerometers along the length of the bridge, while maintaining the control and reference accelerometers at their designated positions. These three accelerometers will be attached to the bottom surface of the bridge deck at almost equally-spaced intervals as illustrated in Fig. 5-9. The bottom surface of the bridge deck was chosen for the purpose of protecting the accelerometer attachment studs from damage due to movement of equipment on the top surface.

5.4 ELECTRONIC DATA ACQUISITION SYSTEM

5.4.1 Introduction

Described in this section are the data acquisition systems which will be required for the field test. The test program will require instrumentation for the detection and monitoring of resonance and for the monitoring and recording of fatigue and dynamic test data.
The University of Missouri electronic control and data acquisi-
tion equipment and associated hardware required for the field test
will be housed in an eight foot wide by thirty-six foot long air-condi-
tioned trailer. A controlled temperature environment is desirable
for the purpose of limiting data errors due to the effects of thermal
changes on the electronic equipment. It will be necessary to acquire
the trailer several months in advance of the field tests in order to
install the electronic equipment and to conduct simulated field
tests to check out and debug the installation.

5.4.2 Data Acquisition System - Fatigue Test

Sixty-five channels of data will be recorded during the fatigue
test. The channels of data to be recorded and/or monitored are as
follows:

- Strain Gages - 60 channels
- Deflection Gages - 2 channels
- Accelerometers - 2 channels
- LVDT - 1 channel

The transducers listed above, with the exception of the LVDT in
the base of the actuator, are described in Art. 5.3. Temperature com-
pensated strain gages will be used to minimize the effect of thermal
drift on the signals from the transducers.

The data acquisition system used during Phase I of this investi-
gation and described in App. VIII does not contain a sufficient num-
ber of channels of signal conditioning for data acquisition during the
fatigue test. Hence, it will be necessary to use the Federal Highway
Administration's instrumentation van described in Reference (25).

A supplemental system will be supplied by the University of
Missouri. However, this system will require modification of an ex-
isting one-hundred channel digital data-acquisition system which was
designed for static testing. This system contains sixty channels of strain gage conditioning. The remaining forty channels can be used to scan and record signals from any external transducer. The system can be modified to record the amplitudes of time-dependent data by the addition of a positive and negative peak amplitude detector. The recording of the maximum and minimum values of a periodic data signal will be sufficient for the fatigue test. One advantage of this system is that the data are directly recorded in digital form on a standard data card from which it can be directly processed by a digital computer. A schematic of the data acquisition system for the fatigue test using the modified one hundred-channel system discussed above is shown in Fig. 5-10. The peripheral equipment shown in Fig. 5-10 is part of the data acquisition system developed during Phase I of this project. The data acquisition system described above is not presently in operation. The system would be made operational during the early phases of the bridge investigation.

A second alternate or backup system using just the signal conditioning portion of the one-hundred channel system is shown in Fig. 5-11. The basic concept of this system is the same as that used in Phase I except that the oscillographs are replaced with a seven-track tape recorder. In order to record the sixty-five channels of data on the seven-track tape recorder a manual switching system would be employed.

5.4.3 Data Acquisition System - Dynamic Properties Tests

During the dynamic tests eleven channels of data will be recorded as follows:

- Accelerometers: 5 channels
- Deflection Gages: 4 channels
- Strain Gages: 1 channel
- LVDT: 1 channel
FIGURE 5-10. DATA ACQUISITION SYSTEM FOR FATIGUE TEST
FIGURE 5-11. ALTERNATE DATA ACQUISITION SYSTEM FOR FATIGUE TEST
One strain gage will be required in the dynamic tests to establish the strain level on the bottom flange of a girder at the center of the main-span. The transducers listed above are described in Art. 5.3.

The data acquisition system to be used for the dynamic tests is shown in Fig. 5-12. It will be essentially the same as the system used in Phase I, the difference being in the type of recording device. For the field tests, a seven-track tape recorder will be used instead of the oscillographs, but oscillographs will be employed as backup units. The use of the tape recorder in the field tests will permit conversion of the experimental analog data to digital form by means of the analog-to-digital conversion capability of the University of Missouri's SEL-840A computer. The digitized data will then be processed through the University's IBM 360/50 digital computer for the purpose of data reduction and analysis.

5.4.4 Resonance Monitoring System

In order to achieve the strain levels required for the fatigue test, it will be necessary that the bridge be at, or very near to, resonance in the first mode. This will require not only monitoring of selected strain gages but it will also require monitoring of the actuator stroke and the bridge deflection in order to ensure that the bridge remains at resonance. Furthermore, for establishing the strain level for the dynamic tests, it is necessary that the bridge be at resonance.

Resonance, for both the fatigue and dynamic tests, will be detected by observing the phase relationship between the stroke of the actuator, which is measured by the LVDT, and the bridge deflection at the center of the main-span. A Lissajous diagram, as viewed on an
FIGURE 5-12. DATA ACQUISITION SYSTEM FOR DYNAMIC TESTS
oscilloscope, will be used to measure this phase relationship; Fig. 5-13 shows the mechanical and electronic systems that will be employed to obtain this diagram. The use of the Lissajous diagram during the experimental model study is discussed in Art. VIII.8.2.2.

5.5 TEST SITE

Figure 5-14 shows the physical layout of the equipment at the test site. The objectives of the dynamic test require that the ballast be removed from the bridge, so it will be necessary to keep clear the pavement areas on both ends of the bridge. The two hydraulic power supplies and the instrumentation trailer will be located at the east and west ends of the bridge, respectively, just south of the pavement areas. Presently there are not adequate level areas south of the bridge to locate this equipment. Hence, it will be necessary to perform some fill work and surface preparation at these locations, subject to the condition of the site after the by-pass road is built.

The area under the bridge will be back-filled to within approximately five feet of the bottom of the four girders in order to provide an adequate working area. Tentative arrangements have been made with the Corps of Engineers Office in Memphis, Tennessee, for the contractor building the by-pass road to do the backfill work under the bridge.

A study of the power requirements for the field test shows that a 35 KVA transformer will be required for the two hydraulic power supplies and a 10 KVA transformer will be required for the two instrumentation trailers. Adequate power capacity is available at the test site since Ozark Border Electric Cooperative has a 7000 volt distribution line within several hundred feet of the bridge. The power
FIGURE 5-13. MONITORING SYSTEM
FIGURE 5-14. FIELD SITE
requirements and the arrangements for the electrical facilities at the test site were discussed in a meeting with representatives of Ozark Border Electric Cooperative at Poplar Bluff, Missouri.

Figure 5-14 shows gasoline storage tanks near the hydraulic power supplies. These tanks will have approximately 1000 gallon storage capacity in order to provide enough fuel for 48 hours of continuous operation.

An additional piece of equipment needed at the test site is a 2 ton capacity fork-lift truck for moving the ballast on and off the bridge.

5.6 TEST PROCEDURES

5.6.1 Fatigue Test

The fatigue test will commence after:

1. the initial visual inspection of the bridge, including the taking of photographs, and
2. the performance of the first set of dynamic tests on the bridge, see Art. 5.6.2

The fatigue test will be conducted by:

1. exciting the bridge at its first bending mode frequency, and
2. adjusting the magnitude of the exciting force (using the stroke of the actuator) until the stress ranges, at the ends of the cover plates in the end spans, are equal to the values given in Table 4-2, with consideration given for actual load distribution in the bridge.

The fatigue test will be conducted in intervals of 25,000 cycles of loading until:

1. detection of a fatigue crack, or
2. two million cycles of loading have been applied to the bridge.
The following three methods will be used to detect the formation of a fatigue crack. (Ultrasonic methods may also be used.)

1. Visual inspection after every 25,000 cycles of loading.
2. Noticeable changes in the reading of the strain gages located at the ends of the cover plates in the end-spans, see Art. 5.4.1. This method is suitable only for detection of cracks at ends of the cover plates.
3. Noticeable changes in the dynamic properties of the bridge.

The fatigue test will be temporarily stopped if a fatigue crack is detected by one of the latter two methods. The visual inspection after each 25,000 cycles of loading will include photographing of key sections of the bridge such as the cover plates, the field splices and the diaphragm, or any other areas which show fatigue cracks. The size of the fatigue cracks will be recorded so that their growth rate can be documented.

After detection of a fatigue crack, the fatigue test will be continued in intervals of 25,000 cycles of loading or less, depending on field judgment, until a fatigue failure has occurred. A fatigue failure will be defined as 50 to 100 percent failure of a girder flange, subject to field judgment. Only one fatigue failure of a flange will be allowed to occur in the bridge. The definition of fatigue failure for secondary components, such as diaphragms, will be based on field conditions. Fisher\(^{(20)}\) reported that the number of cycles for a visible crack to form at the toe of a cover plate end weld was approximately 75\% of the fatigue life of the specimen.

If the fatigue crack does not occur at the end of a cover plate on the compression flange, or if a fatigue crack in the compression flange stops growing, then the fatigue test will be continued again
in 25,000 cycle intervals or less until a fatigue failure occurs in the tension flange at the end of a cover plate. A fatigue failure of a secondary component, such as a diaphragm, will be repaired, if possible, before the test continues.

Upon completion of the fatigue test, the fatigue damage will be repaired as much as possible, to restore the structural integrity of the bridge for the ultimate load test.

5.6.2 Dynamic Properties Tests

The following set of dynamic tests will be performed on the bridge to determine the effects of fatigue damage on its dynamic properties:

1. Resonant-Dwell Test,
2. Log-Decrement Test,
3. Sweep-Sine Test and
4. Profile Test.

These are the same dynamic tests which were conducted on the model during the Phase I investigation. Their purpose is discussed in App. II and in App. VIII which also contains the procedures for conducting these tests.

The complete set of dynamic tests will be conducted on the bridge before commencement of the fatigue test, to determine the initial dynamic properties of the bridge. Additional tests will be conducted, after specified intervals of cyclic loading, according to the Log of Dynamic Tests listed in Table 5-1. The dynamic tests will be terminated after a final set of tests at $2 \times 10^6$ cycles of loading or after fatigue failure of the bridge, whichever comes first. The dynamic tests after fatigue failure will be conducted only if it is
judged in the field that these tests can be meaningfully performed without further damage to the bridge.

All tests indicated in Table 5-1 will be conducted with the ballast removed from the bridge in order to eliminate the influence of the ballast on the dynamic properties of the bridge. In addition, a log-decrement test will be conducted every 25,000 cycles, when the fatigue test is stopped for visual inspection of the bridge. When the dynamic tests are conducted on the unballasted bridge, the concrete strains in tension will not be allowed to exceed the service strains given by the AASHO code.

<table>
<thead>
<tr>
<th>No. of Cycles of Loading</th>
<th>Resonant Dwell Test</th>
<th>Log Decrement Test</th>
<th>Sweep Sine Test</th>
<th>Profile Test</th>
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<tbody>
<tr>
<td>0</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>(0.05 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(0.075 \times 10^6)</td>
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<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(0.10 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(0.25 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(0.35 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(0.50 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(1.0 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>(2.0 \times 10^6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>FATIGUE FAILURE</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

**TABLE 5-1. LOG OF DYNAMIC TESTS**

X - Denotes Test to be Conducted
CHAPTER 6
ULTIMATE LOAD TEST OF BRIDGE

6.1 INTRODUCTION

Information on the ultimate load capacity of real bridges is not especially abundant for the following reasons:

1. It is seldom possible to obtain a bridge for this purpose since bridges, once they are constructed, remain in use until they are retired due to old age when they are usually in a state of deterioration.

2. Such tests are quite expensive and complicated.

3. It is difficult to devise realistic loading schemes.

Since there is very little known about the ultimate capacity of continuous composite bridges under moving vehicular loading, a study was undertaken at Washington University to determine the feasibility of subjecting this bridge to an ultimate load test. This study was performed by Mr. Charles W. Kilper, under the direction of Dr. T. V. Galambos, and his report is included as App. VII. This report is self-contained, and only a very brief summary will be given here.

The ultimate load test is feasible only if the bridge remains essentially intact after the fatigue experiments described in earlier portions of this report have been completed. This means that no major deterioration of the slab shall have occurred, composite action between slab and girders is still intact, and fatigue cracks in the girders are either not present or they have been adequately repaired. Since it is not entirely certain at this time that those conditions will exist, it
is recommended that the decision on whether to proceed with the ultimate load testing should be made after the fatigue test is completed.

6.2 ULTIMATE LOAD TEST SCHEME

In principle the ultimate load test scheme is simple: a heavily loaded rubber-tired tractor-trailer vehicle should be moved slowly back and forth on the bridge. Failure under such loading is by the mechanism of incremental collapse, where the deflections of the bridge continue to grow with each pass of the vehicle until the residual inelastic deformations are so large that the bridge must be considered to have failed. The critical condition is reached by increasing the loads on the vehicle gradually until the deformations no longer shake down. That is, the deflections no longer stabilize after several passes of the vehicle.

Kilper's report in App. VII considers a variety of failure mechanisms and it explores several types of rolling load. The study concludes that it would not be feasible to test the bridge as a whole because of the impractical large loads which would be needed. Therefore, it is recommended that only the two center girders be involved in the testing program and that the two outer girders be excluded from participation by making two longitudinal cuts in the slab along the whole length of the bridge. The test vehicle recommended for the experiment is Vehicle 97 (military tank carrier) used in the AASHO Road Test at Ottawa, Illinois or a similar special overload vehicle. This vehicle, when fully loaded with concrete blocks, should just cause incremental collapse of the reduced bridge.

The testing scheme is envisioned to have the following steps:

1. Survey of the complete bridge to decide whether it is advisable to proceed with the ultimate load testing.
2. If the ultimate load test is to proceed, then the preliminary steps of
   a) arranging rental of the vehicle
   b) casting concrete blocks for loading and
   c) checking out the required instrumentation,
are to be performed.

3. The entire bridge is to be subjected to repeated passes of the fully-loaded vehicle to test the response of the bridge, especially to verify whether it still behaves as a composite bridge, and to check out the test procedures and the instrumentation. It is not expected that the bridge will fail or even be substantially yielded under this loading.

4. Cutting of the slab to reduce the size of the bridge.

5. Repeated passes of the vehicle with increased loads until incremental collapse occurs.
REFERENCES


APPENDIX I
DYNAMICS OF THE BRIDGE FOUNDATION

The soil under the bridge consists of a thin layer of clay over a deep layer of sand, as shown in Fig. I-1. Since saturated sands have a tendency to compact and to liquefy when subjected to vibratory loading a study of the pile behavior was undertaken.

The piles are precast concrete, cast with an 8 inch tip diameter and a 16 inch butt diameter. The average length of the piles under the end abutments is about 37 feet, and for those under the main piers about 20 feet. They were driven to the coarse sand where their bearing capacity was reached. The bearing is obviously not firm bedrock so settlement can be anticipated during testing.

Prior to the testing program the channel will be backfilled to about El. 320. This fill will add a substantial load to the piles under the main piers. The position of the water table will probably not change significantly.

The test conditions will produce the footing loads given in Table I-1. At a pile under the end abutments the axial force will vary between 23k and 47k, at about 2.3 cps. The pile force under the main piers will oscillate between 57k and 71k. Pile driving data furnished by the Missouri Highway Commission(1) indicates that the average allowable bearing capacity per pile in the end abutments is 114 kips and the corresponding capacity of a pile in the main piers is 83 kips. These values are based on the Engineering News Record formula which presupposes a factor of safety of six. Table I-2 lists the
FIGURE I-1
SOIL LOG DATA

ABUTMENTS - CORE LOG

1 BATTER

PIER

PILE

PILE CUT-OFF EL. 321.45

GRADE EL. 327

PIERS - AUGER LOG

1 BATTER

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

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SILTY CLAY LOAM

HIGH WATER

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EL 320

EL 321.45

EL 327

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EL 321.45

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HIGH WATER

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EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA

SANDY SILT

LOW WATER

SILTY CLAY LOAM

HIGH WATER

PILE CUT-OFF EL. 293.

EL 320

EL 321.45

EL 327

FIGURE 1-1. SOIL LOG DATA
<table>
<thead>
<tr>
<th></th>
<th>Abutment</th>
<th>Main Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of piles in footing</td>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>Static Load on Footing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL Superstructure</td>
<td>$67^k$</td>
<td>$257^k$</td>
</tr>
<tr>
<td>DL Substructure</td>
<td>$59^k$</td>
<td>$108^k$</td>
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<tr>
<td>DL Earth Fill</td>
<td>0</td>
<td>$225^k$</td>
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<td>DL Ballast</td>
<td>$14^k$</td>
<td>$46^k$</td>
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<td>Total Static Load</td>
<td>$140^k$</td>
<td>$636^k$</td>
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<tr>
<td>Dynamic Load on Footing</td>
<td>$\pm 46^k$</td>
<td>$\pm 70^k$</td>
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<tr>
<td>Maximum Load</td>
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<td>$706^k$</td>
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<tr>
<td>Minimum Load</td>
<td>$94^k$</td>
<td>$566^k$</td>
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<tr>
<td>Resultant Load per Pile</td>
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<td></td>
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<tr>
<td>Static Load</td>
<td>$35^k$</td>
<td>$64^k$</td>
</tr>
<tr>
<td>Maximum Load</td>
<td>$47^k$</td>
<td>$71^k$</td>
</tr>
<tr>
<td>Minimum Load</td>
<td>$23^k$</td>
<td>$57^k$</td>
</tr>
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</table>

**TABLE I-1. PILE LOADS DURING FATIGUE TEST**

<table>
<thead>
<tr>
<th>End Abutment</th>
<th>Avg. Allowable Bearing Capacity per Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>West (No.1)</td>
<td>$134.0^k$</td>
</tr>
<tr>
<td>East (No.4)</td>
<td>$94.2^k$</td>
</tr>
<tr>
<td>Main Pier</td>
<td></td>
</tr>
<tr>
<td>West (No.2)</td>
<td>$83.4^k$</td>
</tr>
<tr>
<td>East (No.3)</td>
<td>$82.6^k$</td>
</tr>
</tbody>
</table>

**TABLE I-2. TEST BRIDGE PILE CAPACITY ACCORDING TO ENGINEERING NEWS FORMULA.**
average bearing capacity per pile for each bent in the test bridge.

Few full-scale tests on piles subjected to static and dynamic loads have been reported. Tests at high static and dynamic loads on pipe piles at the NASA Mississippi Test Facility\(^2\) yielded settlements of the order of 0.5 inch in a few instances. The natural frequencies in these tests ranged between 5 and 10 cps. One plunging failure (3.3 inch sudden settlement) occurred during these tests. Whether this failure was due to liquefaction of the saturated sand or a load combination that exceeded the ultimate load capacity of the pile was not determined. It is therefore likely that some settlement of the bridge piles will be found in the proposed test. Frequent monitoring of the pier elevations will be required, and shimming under the bearings will probably be needed periodically to reduce the effects of differential settlement. Since the area beneath the bridge will be filled, this surcharge will help to prevent liquefaction. Nonetheless, Richart\(^3\) notes that a state of liquefaction may progress downward during the tests, especially if the sand is very permeable. A well-point system could be established to lower the water table in the vicinity of the bridge.

The results of the pile tests at the NASA facility indicate that soil support of the pile caps greatly reduces settlement under vibratory loading. This procedure could be used on the test bridge if settlement becomes excessive.

The NASA tests also established that the effects of vibratory loading were significant only when the vibration frequency was near the resonant frequency of the pile-soil system. If it is assumed that the piles in the test bridge are in point-bearing then Reference 3 [Richart et.al., p.237] indicates a lowest resonant frequency for
these piles exists at about 10 cps. First mode excitation below 2.5 cps should then not cause the soil to compact to a serious extent.

REFERENCES


APPENDIX II
REVIEW OF RESULTS OF THE MODEL TESTS

II.1 INTRODUCTION

In evaluating the feasibility of performing the fatigue test on the bridge, it is believed that all important aspects of the test have been considered. One consideration was the model tests. At this juncture a review of the results of the model tests and a prognosis of the behavior of the bridge during the fatigue test would be useful.

The model tests served three distinct purposes. Qualitatively, these tests verified that the actuator as a loading device performs satisfactorily. Quantitatively, these tests provided a means of comparing experimental and predicted values to confirm the analyses. Of equal importance, the experience has provided considerable insight for planning of the proposed fatigue test.

A discussion of the tests can best be made on the basis of the following characteristics related to the behavior of the model:

a. Stiffness
b. Frequency
c. Deflection
d. Stresses
e. Actuator force
f. Damping
g. Nonlinear behavior

It is recognized that these characteristics are interrelated. However, a discussion of the individual items may serve to clearly
demonstrate wherever possible the comparison between predicted and observed values. The model consisted of two steel wide-flange beams composite with a reinforced concrete deck. Figures II.1 and II.2 show longitudinal and transverse views of the model. Details of the shear connectors, reinforcing bars and end supports are contained in App. VIII.

II.2 STIFFNESS

In predicting the stiffness of the model, two cases were considered. For the first case, full composite action was assumed throughout the span. This is designated as an upper bound stiffness. Although the end supports provided some rotational constraint, this is in fact very small, being approximately 5% of that provided by a fixed support. As a lower bound stiffness, it is assumed that only the wide-flange beams and the reinforcing bars were effective. For this semi-composite case, the concrete deck has cracked but it is able to maintain continuity between the reinforcement and the steel beams. On these premises, the stiffness at mid-span was calculated to be

\[ 33.4 \text{ kips per inch (composite)} \]
\[ 16.1 \text{ kips per inch (semi-composite)} \]

The dead load of the deck was supported by the steel beams. Hence, at the static equilibrium position, the concrete has no dead load bending stresses. From the very outset of the dynamic testing, cracks developed in the deck. Its continuing deterioration was visibly evident, particularly after the third mode test. However, the reinforcement maintained the integrity of the concrete and the only spalling which occurred consisted of small minute particles of concrete.
Figure II-1. Longitudinal View of Model and Loading System

- Moving Mass
- Actuator
- Concrete Deck
- W12x27
- S15x42.9
FIGURE II-2: TRANSVERSE VIEW OF MODEL

CONCRETE SLAB

FLEX-SUPPORTS

S15x42.9

M12x27

4"

2'-11"

5'-0"
The laboratory tests disclosed that the model exhibited two stiffnesses in each cycle of vibration. With upward motion, the cracks opened and this resulted in a lower stiffness. Whereas, for downward motion, the cracks closed and the resulting stiffness was higher. Evidence of this was noted from oscillograph recordings which showed that the upward deflections were larger than those downward. From experimental data, the stiffness was calculated at the steady-state resonant frequency for each test\(^{(2)}\). Account was taken of the added mass of the actuator(s) and attached weight. The experimental stiffness ranged from

32.2 kips per inch (Test 1) to
17.1 kips per inch (Test 13).

Table II-1 contains a summary of the results of the model tests. A complete tabulation is presented in Reference 2. It should be noted that the predicted composite and semi-composite stiffnesses closely resemble the upper and lower values of experimental stiffnesses.

With regard to the bridge, a static analysis reveals that the bounds on the stiffness are

101 kips per inch (composite)
50.7 kips per inch (semi-composite)

However, it is not anticipated that the bridge deck will deteriorate as appreciably as did the model during the fatigue test because the added dead load ballast will alleviate the concrete tensile strains when the span is at its uppermost position. The predicted concrete tensile strain at midspan during the fatigue test is 130 micro in./in. Whereas, during the model tests, concrete strains for the composite section would have been 260 micro in./in. had cracking not occurred. In the event that cracking of the bridge deck occurs, the lower
<table>
<thead>
<tr>
<th>Test</th>
<th>Type</th>
<th>Flange Stiffness</th>
<th>Bottom Stiffness</th>
<th>Average Actuator Stroke</th>
<th>Actuator Acceleration</th>
<th>Deflection Inch</th>
<th>Midspan Acceleration</th>
<th>Midspan Deflection</th>
<th>Deflection Inch</th>
<th>Kips/In.</th>
<th>Kips/In.</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td></td>
<td>18.35</td>
<td>45.13</td>
<td>0.698</td>
<td>2.846</td>
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<td>205</td>
<td>6.93</td>
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<td>15</td>
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<td>2</td>
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<td>0.488</td>
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<td>0.87</td>
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<td>5.96</td>
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<td>130.6</td>
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<td>0.837</td>
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<td>5.96</td>
<td>169</td>
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<td>300</td>
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<td>1.03</td>
<td>0.96</td>
<td>0.27</td>
<td>205</td>
<td>4.60</td>
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<td>8</td>
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<td>18.39</td>
<td>261.8</td>
<td>0.08</td>
<td>1.19</td>
<td>0.40</td>
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<tr>
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<td>0.10</td>
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<td>6.10</td>
<td>2</td>
<td>2</td>
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<tr>
<td>15</td>
<td></td>
<td>32.22</td>
<td>3.2</td>
<td>0.08</td>
<td>0.12</td>
<td>0.27</td>
<td>0.10</td>
<td>205</td>
<td>6.10</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

TABLE II-1: TABULATION OF EXPERIMENTALY OBTAINED RESULTS - MODEL TESTS
stiffness should not invalidate the feasibility of performing the fatigue test. The vibration system has been designed with an adequate reserve capacity.

II.3 FREQUENCY

For the model, including the small actuator and attached weight, the predicted fundamental bending frequencies were

- 8.28 cps (composite)
- 5.76 cps (semi-composite)

During the tests the observed frequencies ranged from

- 8.07 cps to
- 5.87 cps

These latter values are bounded by the predicted values. Seventy percent of the frequency reduction occurred within Tests 1 through 4 in which the dynamic stress in the bottom flange of the steel beam reached 17 ksi.

For the bridge, the estimated first mode frequencies are

- 2.30 cps (composite)
- 1.53 (semi-composite)

Consideration should be given to the fact that the bridge has been open to traffic for seven years, and consequently the frequency will probably be somewhat lower than the predicted 2.30 cps. In the AASHO Road Tests\(^1\), the first mode frequency reduction for the composite bridge after two years of test traffic was approximately 10\%. If this is the situation in the Missouri bridge, then the early decrease in resonant frequency as the fatigue test is initiated will be somewhat less than that observed in the model tests. The ballast on the bridge should also lessen the frequency decay during the fatigue test.
In any event, the lowest possible frequency should be the value predicted for the semi-composite case.

II.4 DEFLECTION

For a prescribed attached weight and stroke, the deflections are inversely proportional to the damping in the structure. Figure II.3 illustrates the variation in midspan bridge deflections at resonance versus the attached weight on actuator located at midspan for several damping ratios for the composite and semi-composite cases. The analyses are based on an equivalent one degree system. It can be seen that changes in stiffness have a small effect on deflections at resonance.

II.5 STRESSES

Bending stresses are of paramount importance in the fatigue test of the bridge. Although it has been shown that the deflections are not appreciably altered by changes in stiffness, the stresses are notably influenced by variations in stiffness at resonance. This is a result of a change in stiffness which affects the curvature and the location of the neutral axis in the cross section. As a consequence, moments and stresses are affected.

In the model tests, the predicted bending stresses per inch of deflection at midspan (small actuator) were

- 24.9 ksi per inch (composite)
- 15.6 ksi per inch (semi-composite)

and the corresponding observed stresses were

- 23.4 ksi/inch (Test 1)
- 17.3 ksi/inch (Test 13)

Tests 1 and 13 are used for comparison because these tests have the
FIGURE II-3. BRIDGE MIDSSPAN DEFLECTIONS VS. ATTACHED WEIGHT ON ACTUATOR
largest and smallest experimentally observed stiffnesses. In large amplitude tests, notably Test 4, the stresses did not decrease with decreasing stiffness. This may be due to the nonlinearity of the model. A discussion of the nonlinear behavior appears in Art. II.8.

II.6 ACTUATOR FORCE

For the model tests, and in a similar fashion for the proposed fatigue test of the bridge, the externally applied force stems from the electrohydraulic actuator(s) attached to the test specimen. The arrangement of the actuators during the model tests is shown in App. VIII. The actuator used in the majority of the first mode tests and the third mode tests was an MTS Model 205.31 actuator (dynamic rating 5.5 kips) with a 40 pound plate attached to the rod end of the actuator. For two of the first mode tests (Tests 10 and 11) and the torsion test, two MTS Model 202.01 actuators (dynamic rating 80 kips) were mounted side-by-side and weights totaling 1300 pounds were attached to each actuator. Details of the experimental setup are given in App. VIII. The latter arrangement was used primarily to disclose the problems which might arise in synchronizing the motion of two actuators. No difficulties were encountered in using dual actuators.

The actuator forces\(^{(2)}\) were not measured experimentally but they have been calculated on the basis of the observed accelerations and the damping ratios calculated from the experimental data (peak amplitude method). For example, in Test 4, the actuator force was approximately 0.4 kips, whereas, in Test 11, the actuator force was 3.8 kips.

In order to achieve the desired stress level in the proposed fatigue test of the bridge, the calculated actuator forces required are 10.6 kips for the fully composite case and 11.7 kips for the
semi-composite case. These calculations are based on 2% damping and attached weights of 7 kips and 10.5 kips respectively.

II.7 DAMPING

The evaluation of damping in Phase I has proceeded along the lines of the established tests which are described in App. VI. In applying any of these methods, the gross behavior of the structure is analyzed for the damping property. Several mechanisms contributed to the overall damping of the model. These can generally be classified as

1. internal friction within the materials which, for the model, are steel and concrete, and
2. coulomb friction at the interfaces of the connections.

The interfacing effects occur at supports and at the joint between the composite deck and steel beams.

For purposes of this investigation the primary concern is with damping in the total structure rather than a rheological study of the damping in the materials. This approach is consistent with comparable studies(3) involving damping in Civil Engineering structures. To keep the analyses tractable, equivalent viscous damping and linear equations are used, with one exception which is described in App. VI.

The results of the calculations for the damping ratio based on the five methods discussed in App. VI are shown in Table II.2. An examination of Table II.2 shows that for any particular test there is some discrepancy between damping ratios as calculated from the several methods. This is to be expected because of (1) experimental error and (2) the approximations introduced in deriving the equations for each method. All methods referred to in App. VI, which are a) bandwidth, b) linear peak amplitude, c) nonlinear peak amplitude, d) log decrement,
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Log Decrement</th>
<th>Linear Peak Amplitude</th>
<th>Nonlinear Peak Amplitude</th>
<th>Normal Mode</th>
<th>Bandwidth</th>
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<tr>
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<td>.715</td>
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<tr>
<td>2</td>
<td>.880</td>
<td>.887</td>
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<td>5</td>
<td>1.180</td>
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<td>8</td>
<td>1.464</td>
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<td>.69 (U)</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>.995 (D)</td>
</tr>
<tr>
<td>9</td>
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</tr>
<tr>
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</tr>
<tr>
<td>13</td>
<td>1.135</td>
<td>1.004</td>
<td>1.012</td>
<td>1.17</td>
<td>.98 (U)</td>
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<tr>
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<td>.827</td>
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<td>15</td>
<td>.894</td>
<td>.654</td>
<td>.730</td>
<td>.633</td>
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</table>

(U) Determined from upward frequency sweep
(D) Determined from downward frequency sweep

TABLE II-2. DAMPING RATIO $\zeta$, %
and e) normal mode, have been used in evaluating damping.

In comparing the methods, the bandwidth method was found to be the least useful because of the difficulty in many instances of obtaining an adequate number of data points in the vicinity of the resonance peak. Additionally, the nonlinearity of the model at large amplitudes produced a sudden discontinuous jump in the response. Figures II-4, II-5, and II-6 represent bandwidth curves.

The peak amplitude method is satisfactory from the viewpoint that the damping is obtained at resonance. However, there is always the possibility of missing the peak response when recording data because the transition in frequency from one side of resonance to the other is very small.

The simplest form of measurement is provided by the log decrement method. Here the damping ratio is based on the decay rate in a free vibration state. This method is adequate if the emphasis is on determining changes in damping rather than forecasting the amplitude of resonant vibrations due to dynamic loading. A semi-log plot of the decay curves from several tests is shown in Fig. II.7. If the damping were truly viscous, the semi-log plot of the decay curves would consist of straight lines. However, in some tests the curves are convex upward. This indicates that the damping is a mixture of viscous and coulomb friction. The most likely place for friction to arise is at the interface between the concrete slab and steel beams. The values of damping ratio reported in Table II-2 for the log decrement method are based on a least squares solution for the straight line through the data points in Fig. II.7

The normal mode method is the only one which takes into account
FIGURE II-4. FREQUENCY RESPONSE - TEST 8
FIGURE II-5. FREQUENCY RESPONSE - TEST 13
FIGURE II-6. FREQUENCY RESPONSE - TEST 14
Figure II-7. Logarithmic decay curves

Test no. shown adjacent to curve.
the multiple degrees of freedom in the structure. The use of this method entails more measurements than the log decrement method but the computations are more straightforward. In the latter method, there is considerable arbitrariness in choosing the number of cycles over which to measure decay. The normal mode method also provides a profile of the structure which may serve as an inspection tool. It seems plausible to assume that any degeneration in structural stiffness will be accompanied by a change in resonant frequency and mode shape. However, in a highly redundant structure wherein several resonant frequencies are nearly equal, the use of mode shapes may serve to better identify the deterioration within the structure. This concept was not explored in the model tests.

The variation in damping from one test to another can be partially attributed to the continuing deterioration of the concrete deck with successive tests. Figures II-8 and II-9 relate the damping ratio to the measured average strain in the wide flange beams for first mode bending tests using the small actuator. Figure II-8 shows an increasing trend for damping ratios in Tests 1 through 4, which were conducted at successively higher amplitudes. However, in Tests 5, 7, and 8, the damping trend was downward with increasing strain. Test 9 was a third mode test in which the model was subjected to extremely high accelerations. The remaining first mode bending tests (13, 14, 15) also show a downward trend in damping ratios with increasing strain. It can be seen from Table II-2 that damping from the normal mode method in Tests 13, 14 and 15 also indicates a downward trend as the strain increases.

No consideration has been given to stress history nor any of the nonlinear rate-dependent damping dissipators such as the curing or
TEST NO. SHOWN ADJACENT TO DATA POINT

△ PEAK AMPLITUDE METHOD
● LOG DECREMENT METHOD

FIGURE II-8. DAMPING RATIO VS. STRAIN, TESTS 1-4
FIGURE II-9. DAMPING RATIO VS. STRAIN, TESTS 5-15
aging effects in concrete.

In rheological studies of material damping, the energy absorbed per cycle is often used as a criterion to rate materials. The standard test procedure involves the use of a rotating cantilever beam and small test specimens of negligible mass. The energy dissipated $E$ versus strain $\varepsilon$ relationship can be expressed as

$$ E = C \varepsilon^n $$

where

- $C = \text{constant}$
- $n = \text{exponent}$

For linear damping, characterized by a viscous dashpot, the exponent $n = 2$. From the first mode tests in the model study, a log-log plot of the energy dissipated versus the average strain in the steel beams is shown in Fig. II-10. A least squares analysis for a straight line through the data points reveals that the exponent $n = 1.89$. The approximate agreement between the exponent based on experimental data and the exponent for a linear system is an indication that the assumptions of linearity and equivalent viscous damping are reasonably valid.

II.8 NONLINEAR RESPONSE

A sudden jump in the response of the model, which is illustrated in Figs. II-6 and II-11, was observed when the amplitude of the response was relatively large. The jump phenomenon occurred at the first-mode frequency for both a continuous increase and decrease in the excitation frequency.

Figure II-11, which shows an arbitrary nonlinear response curve, is used as an aid in the following discussion of the jump phenomenon. As the frequency of excitation increases, the amplitude of the model
TEST NO. SHOWN ADJACENT TO DATA POINT

FIGURE II-10. LOG-LOG PLOT OF ENERGY DISSIPATED VS. STEEL STRAIN

LOG OF STEEL STRAIN, MICRO IN./IN.

LOG OF ENERGY DISSIPATED/CYCLE IN.-LB.
FIGURE II-11. THE JUMP PHENOMENON FOR THE HARDENING SPRING SYSTEM
gradually increases until point A is reached. Then the response suddenly jumps to the smaller amplitude indicated by point B and continues along the curve to the right. With a decreasing frequency of excitation, moving from right to left on the curve in Fig. II-11, the amplitude continues to increase beyond point B to point C. Then the response jumps to the larger amplitude at point D, which is not as large as the amplitude at point A. Even if the frequency sweep were to change direction at either point A or C, a jump in the response of the model would occur at these points. A stability analysis shows that the response from point A to point C is dynamically unstable.

II.9 TORSION AND THIRD-MODE TESTS

In concluding the summary of the model tests, some mention should be made of the torsional test and third mode bending test. Neither of these tests was considered to be essential in establishing the feasibility of the bridge test but it was thought that the general information gained from these tests would be useful.

A torsional frequency of 7.17 cps was recorded with the two large actuators mounted side-by-side at midspan. With this arrangement, one actuator, or two actuators operating 180 degrees out of phase, provided the eccentric loading to excite the fundamental torsional mode. No corroborating frequency computation was made for this specific case. Such computations would require a finite element analysis. Although a finite-element program was developed, the time required to run the program was excessive to the point of excluding this type of analysis.

The third-mode bending test was accomplished with the small actuator located at midspan. Deflections were not measured because the gages were approaching a state of resonance. However, the acceleration
level was recorded at 19.6 g's at a frequency of 45.6 cps. Computa-
tions based on the semi-composite section show the frequency to be
51.0 cps. Because there was some concern that the high acceleration
level would cause the actuator to puncture the concrete deck, extended
third-mode testing was discontinued.

REFERENCES

1. Highway Research Board, "The AASHO Road Test, Report 4, Bridge
Research," Special Report No. 61 D, National Academy of Sciences-


3. Rea, D., Bouwkamp, J., Clough, R., "The Dynamic Behavior of
Steel Frame and Truss Buildings," Bulletin No. 9, American Iron
and Steel Institute, Berkeley, California, 1968.

APPENDIX III
BRIDGE SECTION PROPERTIES

The dead weight of the structural steel and slab was carried by the girders alone. For this noncomposite action the section properties are as indicated in Fig. III-1 and Table III-1.

For this study, as-built data was interpreted for computing the concrete strength. Cylinders cast from the concrete used in the deck slab failed at an average 28-day compressive strength of 5,270 psi\(^1\). Due to aging, the present strength should be about 5,800 psi. The average cylinder weight was 145 pcf. The current (1971) ACI Code\(^2\) indicates that Young's modulus should therefore now be about 4,400 ksi, and the modular ratio, \(n\), should be about 7.

For \(n\) equal to 7, the section properties, for composite action, at various stations along the bridge were found to be as indicated in Fig. III-2 and Table III-2. The concrete has been assumed to participate in composite action only in the regions of positive dead load moment. Where this moment is negative, the slab reinforcement has been included in the composite section. Since shear connectors are not present over the supports, this latter assumption is not strictly true. The longitudinal reinforcement in the slab is only partly effective.\(^3\) However, the conclusions reached in this study would not be affected should a more rigorous analysis be attempted.

REFERENCES

FIGURE III-1. SECTION DETAILS FOR NONCOMPOSITE ACTION
### TABLE III-1. SECTION PROPERTIES FOR NONCOMPOSITE ACTION

<table>
<thead>
<tr>
<th>LOCATION AND SECTION</th>
<th>I in. 4</th>
<th>c in.</th>
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<td>14.82</td>
</tr>
<tr>
<td>B</td>
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<td>14.91</td>
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<tr>
<td>C</td>
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<td>15.66</td>
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<td><strong>EXTERIOR GIRDER</strong></td>
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<td></td>
</tr>
<tr>
<td>A</td>
<td>3989</td>
<td>14.82</td>
</tr>
<tr>
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<tr>
<td>C</td>
<td>7429</td>
<td>15.44</td>
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A B

---

REINFORCEMENT

---

C D E

---

SLAB

---

HAUNCH

---

1.86 IN²

4.08 IN²

3.43" 1.14"

W = 65.5 IN. EXTERIOR
W = 80.0 IN. INTERIOR
W30X99

B W30X99 EXTERIOR, INTERIOR
C W30X99 EXTERIOR
D W30X108 INTERIOR

E SAME AS SECTION 'A' BUT HAUNCH = 2 INCH

---

FIGURE III-2. SECTION DETAILS FOR COMPOSITE ACTION
### TABLE III-2. SECTION PROPERTIES FOR COMPOSITE ACTION

<table>
<thead>
<tr>
<th>LOCATION AND SECTION</th>
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<th>C TOP in.</th>
<th>C CONCRETE in.</th>
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<td>1.77</td>
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REFERENCES (Contd.)

2. ACI Committee 318, *Building Code Requirements for Reinforced Concrete* (ACI 318-71), American Concrete Institute, Detroit, 1971.

### APPENDIX IV
**AASHO ALLOWABLE FATIGUE STRESS FORMULAS**

<table>
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<th>Cycles</th>
<th>Allowable Stress $F_{r}$ ksi</th>
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<td>500,000</td>
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<td></td>
<td></td>
<td>2,000,000</td>
<td>24 1-R</td>
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<td>F Base Metal Adjacent to Fillet Welds</td>
<td>Tension or Compression</td>
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<td>21 1-R</td>
</tr>
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<td></td>
<td></td>
<td>500,000</td>
<td>12.5 1-R</td>
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<td></td>
<td></td>
<td>2,000,000</td>
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<tr>
<td>G Weld Metal</td>
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<td></td>
<td>500,000</td>
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<td>2,000,000</td>
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<td>27.5 1-R</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2,000,000</td>
<td>18 1-R</td>
</tr>
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</table>

**Note:** $R = \text{Minimum stress} \over \text{Maximum stress}$. 
APPENDIX V
MULTI-DEGREE-OF-FREEDOM ANALYSIS

V.1 INTRODUCTION

Analyses for deflections, shears, and moments in the model and bridge, for static and dynamic loading, were performed by the matrix displacement method. This method has been documented by Weaver, (1) Rubinstein, (2) and others, but a brief description of the technique is included here for completeness.

Each structure was idealized as an assemblage of discrete beam elements of finite length, connected at nodes as illustrated in Fig. V-1. At each of these nodes, vertical and horizontal displacements and a rotation were defined.

![Structure Idealization Diagram]

FIGURE V-1. STRUCTURE IDEALIZATION

V-2. STATIC ANALYSIS

For static analyses, an equilibrium relationship between the set of nodal displacements \( \{u\} \), and loads \( \{F\} \) applied at these nodes was developed.
The key to this relationship is the matrix of stiffness coefficients \([k]\), or nodal forces generated by imposing unit nodal displacements. When the stiffness matrix and load vector had been assembled, then the nodal displacements were determined by solving Eq. \(V.1\). For each element the shears and moments were next computed from the element stiffness matrix and the associated nodal displacements. For the analysis of the model and the bridge, 18 and 28 elements were used, respectively.

**V.3 DYNAMIC ANALYSIS**

The dynamic analysis involved a determination of natural frequencies and associated mode shapes as a prelude to solving for the deflections, shears, and moments. Although rotatory inertial forces exist, they are not significant compared to the translational inertial forces. The analysis was therefore expedited by the procedure outlined in this article.

By the application of Eq. \(V.1\), the influence coefficients, or vertical deflections due to a unit vertical load at an unconstrained node, for all points on the structure were found. The influence coefficients for each position of the load constitute one column of a flexibility matrix \([a]\). The reduced stiffness matrix \([k]\), involving only vertical nodal forces generated by unit vertical deflections, is the inverse of \([a]\).

\[
[k] = [a]^{-1}
\]  

The mass of the structure was "lumped" at the nodes. Each nodal quantity consisted of one-half the mass of each adjacent element plus the mass of any applied ballast or actuator. The resulting diagonal
lumped mass matrix \([M]\), together with the associated reduced stiffness matrix \([k]\), was assembled to formulate the differential equation of motion for free vibrations.

\[
[k] \{u\} = -[M] \{\ddot{U}\} \quad \text{(V.3)}
\]

where \(\{\ddot{U}\}\) is the vector of nodal accelerations. For free vibrations the motion is harmonic.

\[
\{u\} = \{U\} \sin(\omega t + \alpha) \quad \text{(V.4)}
\]

Equation V.3, after substitution of Eq. V.4 and rearrangement, becomes

\[
([k] - \omega^2 [M])\{U\} = \{0\} \quad \text{(V.5)}
\]

where \(\{0\}\) is the null vector. Equation V.5 constitutes a standard eigenvalue problem, the solution for which is the set of natural frequencies (eigenvalues) and their associated mode shapes or deflection configurations (eigenvectors). A modified Jacobi method(3) was used to accomplish this step. The mode shapes were arranged in columnar fashion to build a matrix \([V]\).

The dynamic response was next determined by the normal mode method.(4) In matrix form, the equation of motion for the model or bridge is

\[
[M] \{\ddot{u}\} + [c] \{\dot{u}\} + [k] \{u\} = \{F\} \quad \text{(V.6)}
\]

where

- \(\{\ddot{u}\}\) = a vector of nodal velocities
- \([c]\) = a matrix of damping coefficients
- \(\{F\}\) = a time-dependent forcing function vector

All terms in \(\{F\}\) are zero except those associated with the displacement of the structure beneath the actuators, which has the value

\[
m\omega^2 z \sin \omega t
\]

where \(\omega\) = the frequency of operation of the actuator
where $Z$ is the piston stroke (single amplitude) of the actuator and $m$ is the moving mass on the actuator.

In the normal mode method a coordinate transformation is employed to uncouple the Equations in Eq. V.6. The substitution required is

$$
\{u\} = [V] \{\eta\} \tag{V.7}
$$

where $\{\eta\}$ is a vector of normal coordinates. Equation V.6 is first premultiplied by $[V]^t$. Then the displacements $\{u\}$ and their derivatives are replaced with the corresponding normal coordinates. The resulting equation is

$$
[M] \{\ddot{\eta}\} + [C] \{\dot{\eta}\} + [K] \{\eta\} = [V]^t \{F\} \tag{V.8}
$$

where

- $[M] = [V]^t [M] [V]$, a diagonal matrix
- $[C] = [V]^t [C] [V]$, a diagonal matrix
- $[K] = [V]^t [K] [V]$, a diagonal matrix

The validity of the assumptions involved in reaching Eq. V.8 was summarized by Neville.\textsuperscript{(5)} Equation V-8 represents $n$ independent equations. The $r^{th}$ equation is

$$
\dddot{\eta}_r + 2\xi_r \omega_r \dot{\eta}_r + \omega_r^2 \eta_r = \frac{F_r}{M_r} \tag{V.9}
$$

where

- $\xi_r$ = modal damping factor in $r^{th}$ mode
- $F_r = \{V_r\}^t \{F\}$ \tag{V.10}

Equation V.9 was solved, with initial conditions $\eta_r = \dot{\eta}_r = 0$, for the first five values of $r$. It is well-known that only the first few modes are important in this kind of a study. The deflections in the real coordinate system were next established from Eq. V.7.

In order to minimize the errors in computing element shears and
moments from the deflections, where discrete coordinates have been used, the procedure outlined by Hurty and Rubinstein\(^{(4)}\) was employed. The inertia forces \(\{P\}\), given by
\[
\{P\} = [M] [V] [\omega^2] \{\eta\}
\quad (V.11)
\]
where \([\omega^2]\) is a matrix with the squares of the natural frequencies on the diagonal, were then treated as if they were static loads, and Eq. V.1 was used to determine the deflections, and hence the shears and moments.

Two computer programs were used in the study. The first, a conventional program for static analyses of framed structures, was used to determine the flexibility matrix \([a]\). This result was punched by the program for immediate input to the second program. The complete dynamic analysis was accomplished by the second program. Algorithms for these FORTRAN programs are given in Tables V-1 and V-2.

REFERENCES


TABLE V-1 ALGORITHM FOR STATIC ANALYSIS PROGRAM

1. Input structure data
   a) Number of nodes
   b) Number of elements
   c) Number of load cases
   d) Modulus of elasticity
   e) Node coordinates and support conditions
   f) Element properties

2. Generate structure stiffness matrix

3. Calculate displacements at nodes due to applied loads, and punch
   for input to dynamic analysis program

4. Calculate shears and moments
TABLE V-2 ALGORITHM FOR DYNAMIC ANALYSIS PROGRAM

1. Input structure data
2. Input flexibility matrix
3. Invert flexibility matrix to find reduced stiffness matrix
4. Input mass matrix
5. Find frequencies and mode shapes
6. Input force parameters and damping
   a) attached weight
   b) stroke
   c) percentage of critical damping
7. For each of the first five modes calculate
   a) Phase angle
   b) Normal force
   c) Normal displacement
8. Calculate vertical deflections in real coordinates
9. Calculate inertia loads
10. Determine element shears and moments in manner of static analysis program.
VI.1 INTRODUCTION

The methods used to evaluate damping in the model study are
1. Logarithmic decrement
2. Bandwidth
3. Linear peak amplitude
4. Normal mode
5. Nonlinear peak amplitude

In the following discussion a brief description is given for each method, the pertinent equations are displayed, and the experimental data required in the computations for damping are identified. The objective is to determine the equivalent viscous damping ratio \( \zeta \) which is commonly used in computing response from a linear analysis. These methods are valid for a multi-degree-of-freedom system provided that the equivalent mass and stiffness are known and the system is vibrating in a single mode.

VI.2 LOGARITHMIC DECREMENT METHOD

For a single-degree-of-freedom system, the equation of motion for free vibrations is

\[
\ddot{x} + 2\zeta \omega_n \dot{x} + \omega_n^2 x = 0 \tag{VI.1}
\]

where \( x \) = displacement of structure at midspan
\( \dot{x} \) = velocity
\( \ddot{x} \) = acceleration
\[ \omega_n^2 = \frac{k}{M} \]

\( k \) = equivalent stiffness of structure

\( M \) = equivalent mass of structure

\( \zeta = \frac{c}{c_c} \), damping ratio

\( c \) = damping coefficient

\( c_c \) = critical damping = \( 2M \omega_n \)

Critical damping is defined, for a spring-mass system, to be that amount of viscous resistance provided by a dashpot that will prevent free oscillation of the mass, but will permit it to return to its original position.

The solution to Eq. VI.1 is

\[ x = X e^{-\zeta \omega_n t} \sin \left( \sqrt{1-\zeta^2} \omega_n t + \beta \right) \]  

(VI.2)

in which \( X \) and \( \beta \) are constants to be evaluated from initial conditions at time \( t=0 \). If, as in the model tests, accelerations are used to evaluate damping, Eq. VI.2 becomes

\[ x = X \omega_n^2 e^{-\zeta \omega_n t} \sin \left( \sqrt{1-\zeta^2} \omega_n t + \beta \right) - 2X \omega_n^2 \zeta e^{-\zeta \omega_n t} \left( \sqrt{1-\zeta^2} \right) \cos \left( \sqrt{1-\zeta^2} \omega_n t + \beta \right) + X \omega_n^2 \left( 1-\zeta^2 \right) e^{-\zeta \omega_n t} \sin \left( \sqrt{1-\zeta^2} \omega_n t + \beta \right) \]  

(VI.3)

For \( \zeta \ll 1 \), the first two terms in Eq. VI.3 are insignificant compared to the third term which contains \( (1-\zeta^2) \).

With reference to Fig. VI-1, the ratio of amplitudes of acceleration can be shown to be
FIGURE VI-1. ACCELERATION DURING FREE VIBRATION

\[
\frac{x_1}{x_{n+1}} = e^{2\pi\zeta n} \quad (VI.4)
\]

where

- \( x_1 \) = initial amplitude of acceleration
- \( x_{n+1} \) = amplitude of acceleration after \( n \) cycles
- \( n \) = number of elapsed cycles of vibration

A measure of damping is given by the logarithmic decrement \( \delta \) which is defined as

\[
\delta = \ln\left(\frac{x_1}{x_{n+1}}\right) = 2\pi\zeta n \quad (VI.5)
\]

From Eq. VI.5,

\[
\zeta = \frac{\delta}{2\pi n} \quad (VI.6)
\]

It should be noted that in using various \( x_{n+1} \) values, Eq. VI.5 will usually not yield a unique value for \( \delta \). A better evaluation of damping can be obtained from a semi-log plot of the relative amplitudes of acceleration versus the cycle number \( n \). This is shown in Fig. VI-2.
VI.3 BANDWIDTH METHOD

For the case of forced vibrations produced by a mass attached to a hydraulic actuator, the equation of motion is

\[ x + 2\zeta \omega_n \dot{x} + \omega_n^2 x = \frac{m}{M+m} \omega^2 Z \sin \omega t \]  

(VI.8)
where m = moving mass attached to actuator
M = equivalent mass of structure and actuator
Z = actuator stroke with respect to its base
mω^2Z = amplitude of force

\[ \omega_n = \sqrt{\frac{k}{M+m}} \]

k = stiffness

The steady-state solution to Eq. VI.8 is

\[ x = \frac{m}{M+m} \frac{\omega^2 Z}{\omega_n^2 \left(1-(\frac{\omega}{\omega_n})^2\right)^2 + \left(2\zeta \frac{\omega}{\omega_n}\right)^2} \frac{\sin(\omega t-\phi)}{\left(1-(\frac{\omega}{\omega_n})^2\right)^2 + \left(2\zeta \frac{\omega}{\omega_n}\right)^2}^{1/2} \] (VI.9)

where \( \phi = \) phase angle between force and response.

Let

\[ X = \frac{m}{M+m} \frac{\omega^2 Z}{\omega_n^2 \left(1-(\frac{\omega}{\omega_n})^2\right)^2 + \left(2\zeta \frac{\omega}{\omega_n}\right)^2}^{1/2} \] (VI.10)

In the model study, accelerations were used in the bandwidth calculations for damping. Hence, from Eq. VI.9

\[ \ddot{x} = -\omega^2 x \] (VI.11)

and by definition let

\[ \ddot{X} = \omega^2 X \quad \text{and} \quad \ddot{Z} = \omega^2 Z \] (VI.12)

A plot of \( X \) versus \( \omega \) is shown in Fig. VI-3.
FIGURE VI-3. ACCELERATION VS. FORCING FREQUENCY

At \( \omega = \omega_n \),

\[
\ddot{X}_{\text{max}} = \frac{m}{M+m} \frac{Z_n \omega_n^2}{2\zeta}
\]  \hspace{1cm} (VI.13)

To determine the frequencies \( \omega_1 \) and \( \omega_2 \) for which \( \ddot{X} = \frac{X_{\text{max}}}{\sqrt{2}} \), Eqs. VI.12 and VI.13 are related in the following manner:

\[
\frac{1}{\sqrt{2}} \frac{m}{M+m} \frac{Z_n \omega_n^2}{2\zeta} \omega_n^2 = \frac{m}{M+m} \frac{Z_n \left(\frac{\omega}{\omega_n}\right)^2 \omega^2}{\{(1-(\frac{\omega}{\omega_n})^2)^2 + (2\zeta \frac{\omega}{\omega_n})^2\}^{1/2}}
\]  \hspace{1cm} (VI.14)

For a constant force, that is,

\[
m \omega_n Z_n = m \omega_n Z
\]  \hspace{1cm} (VI.15)

and neglecting higher order terms in \( \zeta \), the solution of Eq. VI.14 is

\[
(\frac{\omega}{\omega_n})^2 = 1 + 2\zeta
\]  \hspace{1cm} (VI.16)

Upon substituting VI.16 into

\[
\frac{\omega_2^2 - \omega_1^2}{\omega_n^2}
\]  \hspace{1cm} (VI.17)

and noting that \( \omega_n = \frac{\omega_1 + \omega_2}{2} \)

\hspace{1cm} (VI.18)
the damping ratio $\zeta$ becomes

$$\zeta = \frac{\omega_2 - \omega_1}{2\omega_n} \quad (VI.19)$$

During model tests 1 through 12 for bandwidth data, the magnitude of $\ddot{y}$ was held constant. The acceleration $\ddot{y}$ is the absolute acceleration of the moving mass and is defined as a vector summation:

$$\ddot{y} = \ddot{x} + \ddot{z} \quad (VI.20)$$

in which

$$\ddot{z} = -\omega^2 z \quad (VI.21)$$

Holding $\ddot{y}$ constant does not satisfy the criterion for bandwidth damp­

*ing as established in VI.19. However, if all observed magnitudes of

accelerations $\ddot{x}_i$ are modified in the following manner,

$$\ddot{x}_i = \ddot{z}(Z_n) \quad (VI.22)$$

the resulting accelerations would be those obtained from a constant

force. This procedure does not require a consideration of phase

angle $\phi$,

$$\phi = \frac{2\zeta \omega_n \omega}{\omega_n^2 - \omega^2} \quad (VI.23)$$

because changing $Z$, with the frequency $\omega$ a constant, is accomplished

by altering the stroke $Z$. However, the assumption is made that there

is no appreciable change in the damping ratio for different amplitudes

of force during a particular test. In Model Tests 13, 14, and 15,

an electronic summing junction was installed to monitor $\ddot{z}$ and conse­

quently the force $m\omega^2 Z$ was held constant for all readings of accelera­

tion $\ddot{x}$. 
VI.4 LINEAR PEAK AMPLITUDE METHOD

This method is undoubtedly the simplest method that can be employed to establish the damping ratio from steady-state motion. Equation VI.10 at resonance reduces to

\[ X = \frac{m}{M+m} \frac{Z}{2\zeta} \]  

(VI.24)

The damping ratio is

\[ \zeta = \frac{m}{M+m} \frac{Z}{2X} \]  

(VI.25)

From Eq. VI.25 with measured deflections \( X \) at midspan and the stroke \( Z \), the damping ratios are determined.

VI.5 NORMAL MODE METHOD

For a system in which well-defined modes of vibration can be established, damping may be determined on the basis of normal mode theory. This procedure is the only method which directly accounts for the multiple degrees of freedom.

Experimentally, the structure is excited in a resonant mode. At selected points on the structure, deflections or accelerations are measured to establish the mode shape. The number of points is quite arbitrary. All that is required is that these points, commonly referred to as joints, be convenient for lumping the mass in discrete parameter calculations.

For a known mode shape and exciting force, the damping ratio \( \zeta \) is computed from the equation

\[ \zeta_r = \frac{V_k(r)}{V_m(r)} \frac{F_m}{\ddot{u}_k} \]  

(VI.26)

where \( \zeta_r \) = damping coefficient for the \( r^{th} \) mode

\( V_k(r) \) = mode shape at joint \( k \)

\( V_m(r) \) = mode shape at joint \( m \)
\[ \ddot{u}_k = \text{measured acceleration at joint } k \]
\[ F_m = \text{exciting force at joint } m \]
\[ F_m = m \omega_r^2 Z \]
\[ \omega_r = \text{frequency of } r^{th} \text{ mode} \]
\[ Z = \text{actuator stroke} \]

A derivation of Eq. VI.26 is given in Neville's work(1).

In Eq. VI.26 the mode shapes must be scaled such that
\[ \{V(r)\}^T [M] \{V(r)\} = 1 \]

where \([M]\) = lumped mass at each joint, diagonal matrix.

For the model tests, the actuator was located at midspan and the
mode shapes were determined from acceleration measurements. Accordingly, from Eq. VI.26, one value of \( \zeta_r \) is computed from each test.

VI. 6 NONLINEAR PEAK AMPLITUDE METHOD

The nonlinear peak amplitude method is the only damping measurement technique used in this study which accounts for the nonlinearity in the response of the model. The method is based on an equivalent single-degree-of-freedom-system which has nonlinear stiffness and linear viscous damping.

Based on steady-state motion at resonance, the equation for the
damping ratio is
\[ \zeta = \frac{m \beta Z}{2(M+m)X} \]

where \( m \) = moving mass attached to actuator
\( M \) = equivalent mass of structure and actuator
\( Z \) = relative acceleration of "moving mass"
\( X \) = absolute acceleration at mid-span of model
\[ g^* = \frac{\omega}{\omega_n} \]

\[ \omega = \text{excitation frequency at resonance (rad./sec.)} \]

\[
\omega_n = \left[ \frac{(\omega_n^*)^2 - \left( \frac{A_2}{A_1} \right)^2 (\omega_n^*)^2}{1 - \left( \frac{A_2}{A_1} \right)^2} \right]^{\frac{1}{2}} \tag{VI.29}
\]

\[ A_1 = \frac{\omega_n^*}{(\omega_n^*)^2} \]

\[ A_2 = \frac{\omega_n^*}{(\omega_n^*)^2} \]

\[ \omega_n^* = \text{nonlinear natural frequency (rad./sec.)} = \omega \text{ at resonance} \]

The quantities \( X_1, \omega_n^* \) and \( X_2, \omega_n^* \) must be experimentally measured at two different force levels respectively. The quantities \( X \) and \( Z \) are not restricted to any particular excitation levels. The derivation of Eq. VI.28 is given in reference(2) with a discussion of the limitations of this method.

REFERENCES


APPENDIX VII

SHAKEDOWN OF A COMPOSITE BRIDGE

by

CHARLES W. KILPER

under the supervision

of

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SHAKEDOWN OF A COMPOSITE BRIDGE

1. INTRODUCTION

There are two considerations which prompt the testing of bridges for their ultimate capacities. The more obvious consideration is the practical value of verifying that analytical procedures can predict the maximum loads which can be placed on a bridge. Clearly a knowledge of bridge behavior under accidental or emergency overloads or an increase in the loads carried by normal traffic is desirable. The second consideration is the applicability of the test data to the formulation of design procedures. Currently highway bridges in the United States are designed according to the AASHO(1)* specifications, which do not recognize the relevance of ultimate capacities in design. Instead members are proportioned to respond to the most severe combination of loads in an elastic manner. Elastic response requires that no permanent deflections or residual interior forces are caused by the applied loads. Stresses are limited to a specified fraction of the stress which is the nominal demarcation between elastic and inelastic behavior. This represents a more rigid philosophy of design than that found in the

* The numbers in parentheses in the text indicate references in the Bibliography.
AISC\(^{(2)}\) and ACI\(^{(3)}\) specifications for building structures. These latter codes permit, to some degree, the consideration of inelastic behavior in design, and tolerate some incursions into the inelastic range at working loads.

The reason for greater conservatism in the design of highway bridges seems to be the nature of the loadings, which are inherently dynamic and highly variable. The ultimate strength analytical procedures developed to date do not consider dynamic effects or elastic fatigue damage, both of which are important in the working load range. In addition, the validity of rigid plastic analysis, a particular ultimate strength analytical procedure, has been questioned on the grounds that the assumption of proportional loading, while reasonable for buildings, is unrealistic for bridges.

The nonproportional and variable repeated nature of bridge loads can be accounted for by shakedown analysis. The theoretical bases for this method as applied to steel structures are found in a sizeable body of literature\(^{(4)}\) but since the method considers inelastic behavior, present practice would not permit its use in the design of highway bridges. Shakedown analysis considers two possible modes of failure. A structure shakes down if, for some series of loadings, it avoids both of these failure modes. One mode, termed incremental collapse, considers the onset of indefinitely large deflections. For repeated applications of a given load, residual deflections and residual internal moments will be produced if the load causes inelastic action. If the increments in residual deflection tend to decrease with each application of load, shakedown is said to occur. Once deflections have stabilized, the response to additional loadings at the same level will be purely elastic.
For some critical load level, the deflections do not stabilize, but continue to increase with each load application, and incremental collapse is said to occur. Another failure mode associated with variable repeated loading is termed alternating plasticity or low cycle fatigue. This is a fracture phenomenon, and results when some point in the structure is subjected to repeated and reversed bending moments of sufficient magnitude in the inelastic range. Hence the shakedown load is actually the more critical of two values associated with incremental collapse and alternating plasticity.

The shakedown load is a lower bound to the rigid plastic collapse load, although the two values are typically found to be relatively close. A statistical argument(5) has been used to show that, for buildings, the occurrence of one load at the rigid plastic collapse level is much more likely than several occurrences of load at the shakedown level. Hence shakedown is commonly ignored in favor of the simpler rigid plastic collapse method in the design of buildings. For statically indeterminate beam type bridges, the variable repeated nature of the loads together with their inherent nonproportionality lends considerable importance to shakedown considerations. As far as strength is concerned, shakedown analysis seems to be appropriate. Dynamic and elastic fatigue effects must still be considered separately. It remains for researchers to demonstrate that highway bridges designed by shakedown methods can perform adequately at working loads and compete economically with those designed by traditional elastic methods.

The present work is a study of the feasibility of testing an existing highway bridge for its shakedown characteristics. The bridge, designed according to 1961 AASHO specifications, is a three span
continuous composite beam. Four steel stringers and a reinforced concrete deck carry a two lane road over a levee ditch in Butler County, Missouri. Obsolescence of the ditch has caused the Missouri Highway Department to make the bridge available for testing, pending the formulation of a suitable test proposal. The bridge will first become available to the University of Missouri at Columbia for testing of the applicability of current (1971) AASHO fatigue design requirements under nominally elastic levels of response. If the bridge survives the first series of tests, shakedown tests could be conducted.

The bridge presents an unusual opportunity to gather shakedown data on a full scale structure. That AASHO places considerable importance on full scale testing was demonstrated by the extensive bridge studies incorporated into the AASHO Road Test. The studies, which were completed in 1961, involved eighteen simply supported, single lane bridges. Types of construction were: steel beam (noncomposite and composite), prestressed concrete, and reinforced concrete. One composite span (designated 3B) was subjected to a test to failure with monotonically increasing loads. The vehicles used for this particular test to failure (designated at vehicles 97, 98, and 99) were military tractor-trailers loaded with specially cast concrete blocks. The largest load, applied by vehicle 97, was not sufficient to collapse the bridge, but the test was terminated due to a large permanent set at midspan. Since the bridge was statically determinate, the test did not investigate shakedown as discussed above, but was concerned with the verification of ultimate strength theory and the response of a simple composite span to extremely heavy loads.
During the summer of 1970, the University of Tennessee along with cooperating governmental agencies conducted tests on four existing bridges in Tennessee. The bridges were designed according to AASHO specifications and the types of construction were: four span continuous composite, simple span prestressed concrete, simple span reinforced concrete T-beam, and three span continuous steel beam with reinforced concrete approach spans. The overall objective of these tests was to correlate full scale bridge behavior with design criteria given in the AASHO code. A portion of the tests involved investigation of the ultimate strengths of selected segments of each bridge. The load was applied by a stationary system of prestressing jacks along with concrete blocks which simulated HS-20 truck loadings. The specific objective of the tests to determine ultimate strength was to provide data on the strength of bridges designed according to existing AASHO specifications, this being considered a necessary step in the formulation of ultimate strength design methods. Since the loading was static, the results must be extrapolated to the realistic case of a rolling load.

The proposed tests of the Butler County bridge offer valuable additions to the data already gathered on the ultimate capacity of full scale composite highway bridges. The most important contributions would be:

1. Verification that classical shakedown analysis, which was developed for steel structures, can be applied (possibly with some modification) to composite bridges,

2. Empirical evidence of the beneficial effects of indeterminacy of composite bridges with respect to ultimate strength,
(3) data based on a realistic moving load provided by a rubber
tired vehicle,

(4) data on the behavior of a full scale bridge designed according
to (1961) AASHO specifications, and

(5) a logical extension of the AASHO and Tennessee tests providing
additional information which is required for the formulation
of ultimate strength design methods for highway bridges.

The present work focuses on demonstrating that shakedown tests of
the Butler County bridge are feasible, and suggests a general test
procedure. The primary objectives are to show that a vehicle can be
provided which will produce the desired shakedown phenomena, and to
discover any inherently weak elements of the bridge which might force
premature discontinuation of the tests. A cost estimate of the proposed
tests is included.
2. DESCRIPTION OF THE BRIDGE

The bridge, shown in Figure 1, is located in Butler County, Missouri. It is a three span continuous composite beam composed of four steel stringers supporting a six inch thick reinforced concrete slab. Shear connection is provided by channel shear connectors which are not present over the intermediate supports. The approach spans are 72 feet long and the main span is 93 feet long. The bridge width accommodates two traffic lanes. Concrete curbs and steel guard rails are present but there are no pedestrian walkways. The foundation consists of precast concrete piles. Each of the two intermediate supports is a monolithic reinforced concrete frame consisting of two columns and a cross beam. The bridge was designed for M15-44 loadings. Details of the bridge design are discussed below. Member sizes and dimensions and other design information were obtained from engineering drawings provided by the Missouri State Highway Department.

2.1 STEEL BEAMS

Figure 2 shows sizes and dimensional information for the steel beams. Splice locations are also noted on Figure 2. The beam size is W30x99 everywhere except for the interior beams over the intermediate supports. At these locations a W30x108 was used.

Figure 3 shows the locations of cover plates, diaphragms, and cross frames. There are both top and bottom cover plates in the vicinity of the intermediate supports. Diaphragms are provided within each span and cross frames, similar in construction to the diaphragms, are located at the supports.

During construction of the bridge (1962), tests were conducted on rolled beam coupons and steel plate coupons. A portion of the
test results is listed in Table 1. The 12"x5/8" cover plates were placed on the top and bottom of the W30x99 beams. Similarly the 12"x3/4" cover plates were placed on the top and bottom of the W30x108 beams. The last column in Table 1 lists assumed upper bound values for the yield stresses of steel sections based on the test data. These upper bounds, and the corresponding lower bounds represented by the specified minimum yield points, set reasonable limits on the strength of steel sections for use in calculations. The same procedure will be used for other components of the bridge, and so the results of calculations will be in terms of upper and lower bounds, rather than single deterministic values.

2.2 REINFORCED SLAB

The design specifications called for Class B1 air entrained concrete to be used in the bridge deck. The nominal 28 day compressive strength of this type concrete is 4 ksi. Cylinder tests(11) conducted at 28 days indicated compressive strengths in the range 5.14 to 5.46 ksi. For calculations, the lower and upper bounds on compressive strength of the slab concrete will be taken as 4.0 and 5.5 ksi, respectively.

The minimum specified slab thickness was 6 inches. Slab haunches were provided at the steel stringers to compensate for dead load deflections and to accommodate the curvature of the roadway. Details of the slab haunch are shown in Figure 4.

The slab is orthotropically reinforced. The transverse steel is uniformly distributed over the length of the bridge, and consists of #5 deformed bars on 6.5 inch centers, top and bottom. The longitudinal steel is also uniformly distributed over the bridge length. The top
Table 1. Yield Strengths of Steel Sections

<table>
<thead>
<tr>
<th>Steel Section</th>
<th>ASTM Designation</th>
<th>Specified Minimum Yield Stress, ksi</th>
<th>Range of Test Yield Stress, ksi*</th>
<th>Assumed Upper Bound Yield Stress, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam W30x99</td>
<td>A36</td>
<td>36</td>
<td>41.6--44.8</td>
<td>45</td>
</tr>
<tr>
<td>Cover Plate 12x5/8</td>
<td>A7</td>
<td>33</td>
<td>44.2--44.4</td>
<td>44</td>
</tr>
<tr>
<td>Beam W30x108</td>
<td>A36</td>
<td>36</td>
<td>39.9</td>
<td>40</td>
</tr>
<tr>
<td>Cover Plate 12x3/4</td>
<td>A7</td>
<td>33</td>
<td>38.2--43.7</td>
<td>44</td>
</tr>
</tbody>
</table>

*Refer to text for sources of test data.
longitudinal steel consists of alternating #4 and #5 bars with 4.5 inches between centerlines of adjacent bars. The bottom longitudinal steel consists of #5 bars on 10 inch centers. Bottom longitudinal steel was omitted in the vicinity of the stringers. Figure 5 shows reinforcing information for the slab.

During construction, tests were conducted on reinforcing bar specimens. A portion of the test data for bar sizes used in the deck and intermediate supports is presented in Table 2. For computation, the specified yield stresses will be taken as lower bounds. The assumed upper bounds, based on test data, are listed in the last column of Table 2.

2.3 SHEAR CONNECTION

Channel shear connectors were used. Connectors were made from 6.25 inch lengths of C4x5.4, and were welded to the top flanges of the stringers with 3/16 inch fillet welds. Each weld extended around the entire perimeter of the joint area. Each stringer was provided with 141 shear connectors with a nonuniform spacing. Connectors were not placed over the intermediate supports. Shear connector information is shown on Figure 6.

2.4 ABUTMENTS

The shape and dimensions of the reinforced concrete abutments are shown on Figure 7; reinforcing details are omitted. Each abutment is a simple combination of backwall and wingwalls. The wingwalls are parallel to, and a continuation of, the backwall. Foundation support is provided by precast concrete piles, four piles beneath each abutment. The exterior piles are battered as shown in Figure 7.
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Grade of Reinforcing Steel</th>
<th>Specified Yield Stress, ksi</th>
<th>Range of Test* Yield Stress, ksi</th>
<th>Assumed Upper Bound Yield Stress, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4</td>
<td>Rail</td>
<td>50</td>
<td>69.2--69.4</td>
<td>70</td>
</tr>
<tr>
<td>#5</td>
<td>Rail</td>
<td>50</td>
<td>60.8--71.5</td>
<td>70</td>
</tr>
<tr>
<td>#5</td>
<td>Hard</td>
<td>50</td>
<td>67.2</td>
<td>70</td>
</tr>
<tr>
<td>#6</td>
<td>Rail</td>
<td>50</td>
<td>69.0--70.9</td>
<td>70</td>
</tr>
<tr>
<td>#7</td>
<td>Rail</td>
<td>50</td>
<td>68.9</td>
<td>70</td>
</tr>
<tr>
<td>#9</td>
<td>Intermediate</td>
<td>40</td>
<td>44.1</td>
<td>44</td>
</tr>
<tr>
<td>#10</td>
<td>Intermediate</td>
<td>40</td>
<td>43.8</td>
<td>44</td>
</tr>
</tbody>
</table>

*Refer to text for source of test data.
2.5 INTERMEDIATE SUPPORTS

The intermediate support structures are reinforced concrete portal frames. Dimensional and selected reinforcing information is given on Figure 8. The frames are of monolithic construction and rest on precast concrete pile foundations. Beneath each column are five piles and a footing. Four of the five piles are battered as indicated in Figure 8.

2.6 BEARINGS

Expansion (rocker) bearings are located at bents 1, 2, and 4. (Refer to Figure 2 for bent numbering scheme.) Details of the expansion bearings are shown on Figure 9(a). Fixed bearings are located at bent 3. Details of the fixed bearings are shown on Figure 9(b).

2.7 DIAPHRAGMS AND CROSS FRAMES

The stringers are connected by transverse diaphragms within the spans and cross frames at the supports. Details of these members are shown on Figure 10 and their locations are given on Figure 3. Also shown on Figure 10 are the bearing stiffener plates which are located at the reaction points.

2.8 CURBS AND GUARD RAILS

The curbs and guard rails are of interest here only because they contribute to the bridge dead weight. It will be assumed that these elements make no significant contribution to the strength of the bridge.
3. STRENGTH OF THE BRIDGE

This study is primarily concerned with the ultimate capacity of the bridge as indicated by shakedown analysis. Such an analysis assumes that localized failures of components will not occur. Clearly it is possible that the true ultimate strength of the bridge is not determined by its flexural capacity, but by the strength of its components. Any of the following failure modes might occur.

(1) Localized failure of the slab
(2) Loss of shear connection
(3) Failure of intermediate support
(4) Foundation failure
(5) Bearing failure
(6) Local buckling of stringer flange and/or web

In the computations which follow, these modes will be considered as well as major axis bending failure. The object of the calculations will be to assure that the bridge, when subjected to heavy vehicular loading, will survive long enough to provide meaningful data on shakedown behavior. Since the present work is concerned with the feasibility of testing a particular bridge for a particular kind of behavior, rather than the development of new or more refined analytical techniques, maximum use will be made of existing analytical schemes. In cases where an analytical procedure is not yet fully developed, simplifying assumptions will be made to arrive at a reasonable estimate of, or bound on, a critical load. Uncertainties will be clearly noted in the expectation that future investigators, with the aid of test data, will be able to propose needed analytical revisions.
3.1 COMPOSITE T-BEAMS

To determine ultimate positive moments, the bridge is imagined as being composed of four parallel T-beams. Associated with each steel stringer is a portion of slab which lies within the "effective flange width." Rules for determining the effective flange width are contained in the AASHO code. In computing the ultimate positive moment of a composite T-beam, the entire steel portion of the cross section is assumed to be at the yield stress level. An equivalent rectangular stress distribution, shown in Figure 10.1.1, is assumed for the concrete. A sample calculation is presented in Appendix 10.1. Results for the Butler County bridge are summarized in Table 3. Variations in capacity from beam to beam and span to span are due to different haunch heights and flange widths. Midspan haunch heights were used in the calculations and the curvature (crown) of the roadway was neglected. The curbs were assumed to be structurally ineffective. The lower bound capacities are based on the lower bound material strengths listed in Tables 1 and 2. Similarly, the upper bound capacities are based on upper bound material strengths.

Table 3. Positive Ultimate Moment Capacities, $M_u^+$, of Composite T-Beams

<table>
<thead>
<tr>
<th>Span</th>
<th>Beam</th>
<th>Lower Bound $M_u^+$, ft.-kip</th>
<th>Upper Bound $M_u^+$, ft.-kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approach</td>
<td>Interior</td>
<td>1747</td>
<td>2183</td>
</tr>
<tr>
<td>Approach</td>
<td>Exterior</td>
<td>1676</td>
<td>2097</td>
</tr>
<tr>
<td>Main</td>
<td>Interior</td>
<td>1779</td>
<td>2224</td>
</tr>
<tr>
<td>Main</td>
<td>Exterior</td>
<td>1714</td>
<td>2145</td>
</tr>
</tbody>
</table>
At the intermediate supports, the ultimate negative moment capacities must be determined. The concrete, assumed to be incapable of resisting tensile stress, does not contribute to the negative ultimate capacity.\(^{(14,15)}\) The cross section then consists of the steel beam, cover plates, and longitudinal reinforcing bars within the effective flange width. The ultimate moment is based on yielding of all steel elements of the cross section,\(^{(14,15)}\) with due regard given to differing yield strengths. Upper and lower bound moment values are based on the yield stresses given in Tables 1 and 2. A sample calculation is presented in Appendix 10.2 and the results are summarized in Table 4.

**Table 4. Negative Ultimate Moment Capacities, \(M_u\), of Composite Beams**

<table>
<thead>
<tr>
<th>Location</th>
<th>Beam Size and Cover Plates</th>
<th>Lower Bound (M_u), ft.-kip</th>
<th>Upper Bound (M_u), ft.-kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>W30x108 12&quot;x3/4&quot;</td>
<td>2138</td>
<td>2613</td>
</tr>
<tr>
<td>Exterior</td>
<td>W30x99 12&quot;x5/8&quot;</td>
<td>1896</td>
<td>2457</td>
</tr>
</tbody>
</table>

The strength of composite T-beams in the negative moment region is affected by the local stability of the steel beam web and compression flange. The problem has been discussed\(^{(14)}\) with reference to static tests of two span composite beams. It was reported that even where severe local buckling occurred at the support, the ultimate load carrying capacity was not greatly affected. Since only four specimens were tested, no general conclusions could be drawn.
The minimum thickness provisions in the AISC code(2) can be applied in the present case, keeping in mind that these provisions were actually derived for symmetrical wide flange shapes.(16) Sample calculations are presented in Appendix 10.3 and the results are summarized in Table 5.

Table 5. Local Stability at Intermediate Support

<table>
<thead>
<tr>
<th>Width-Thickness Ratios</th>
<th>Beam</th>
<th>Flange Actual</th>
<th>Allowable</th>
<th>Cover Plate Actual</th>
<th>Allowable</th>
<th>Web Actual</th>
<th>Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior</td>
<td>6.90</td>
<td>8.5</td>
<td>14.0</td>
<td>33.1</td>
<td>51.6</td>
<td>52.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.0</td>
<td></td>
<td></td>
<td>28.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Exterior</td>
<td>7.30</td>
<td>8.5</td>
<td>16.7</td>
<td>33.1</td>
<td>54.1</td>
<td>50.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td>28.6</td>
<td></td>
<td>43.3</td>
</tr>
</tbody>
</table>

In Table 5, there are double entries for maximum allowable width-thickness ratios. In each case, the top value is based on lower bound yield stresses and the bottom value is based on upper bound yield stresses. Inspection of the table reveals that the flanges and cover plates appear to be stable under ultimate loading. The web of the exterior beam appears unstable, and the web of the interior beam is adequate only for lower bound yield stresses. Web buckling is likely under ultimate loading.

3.2 SLAB PANELS

In order to consider possible localized slab failures, the moment capacities per unit length of the slab panels between the steel stringers must be computed. The slab ultimate moments are computed using the same method as for rectangular beams,(17) except that the effects of
compression reinforcement are customarily neglected for yield line analysis. The compression reinforcement can be the top or bottom layer, depending upon the sense of the moment being computed. Sample computations are presented in Appendix 10.4 and results are summarized in Table 6.

Table 6. Slab Ultimate Moments

<table>
<thead>
<tr>
<th>Direction of Reinforcement</th>
<th>Sense of Moment</th>
<th>Lower Bound $M_u$ ft.-kip/ft.</th>
<th>Upper Bound $M_u$ ft.-kip/ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>Positive</td>
<td>6.01</td>
<td>8.50</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>Negative</td>
<td>11.9</td>
<td>16.5</td>
</tr>
<tr>
<td>Transverse</td>
<td>Positive</td>
<td>10.5</td>
<td>14.5</td>
</tr>
<tr>
<td>Transverse</td>
<td>Negative</td>
<td>9.25</td>
<td>13.1</td>
</tr>
</tbody>
</table>

3.3 SHEAR CONNECTORS

A formula for computing the ultimate strength of channel shear connectors is

$$q_u = 550 (h + 0.5t) w \sqrt{f'_c}$$

where

- $q_u = \text{ultimate strength of one connector in pounds}$
- $h = \text{average flange thickness}$
- $t = \text{web thickness}$
- $w = \text{length of connector in inches}$
- $f'_c = \text{concrete compressive strength in psi}$
If the concrete compressive strength is taken as 4000 psi, the above formula gives a connector capacity of 84 kips.

Each connector is attached to the rolled beam flange by a rectangular weld group. The perimeter of the weld is 15.75 inches. The allowable stress for a 3/16 inch fillet weld according to the 1963 AISC specification\(^{(19)}\) is 1.80 kips per inch of weld. This indicates an allowable load on the weld group of 28.3 kips. Allowable fillet weld stresses in the 1963 AISC specification were based on a factor of safety against failure of 3 to 5.\(^{(20)}\) Then the ultimate strength of the weld group should be at least three times the allowable load, or 84.9 kips.

The strength of each connector assembly will be taken as 84 kips. An evaluation of adequacy of shear connection will be postponed until the locations of the points of ultimate moment have been discussed.

3.4 ABUTMENTS

The abutments will be investigated only with respect to the strength of the pile foundations. The precast concrete piles have a nominal capacity of 32 tons each. All piles were driven to at least the nominal capacity. Presumably the capacity at driving was determined by one of the dynamic formulas in the AASHO code. These formulas have a built-in safety factor of 6. Since each abutment is supported by four piles, bounds on the foundation strength are 128 and 768 tons. There is no need to rely on the nominal safety factor of 6. It will be shown that for the structure and test loadings being considered here, a safety factor of 2 is sufficient.

3.5 INTERMEDIATE SUPPORTS

Each intermediate support is a reinforced concrete portal frame. During tests, the ditch beneath the bridge will be backfilled with soil
to within a few feet of the bottom stringer flanges. Hence the columns will be stiffened by loosely packed soil. This stiffening effect will not be accounted for in the calculations; its neglect is conservative.

Lower bound capacities of the frame components are listed in Table 7. The computations are contained in Appendix 10.5. The precise capacity of the columns depends on the interaction of bending moment and axial load. Further discussion of the intermediate supports will be postponed until the test loadings have been discussed.

Table 7. Lower Bound Capacities of Frame Components

<table>
<thead>
<tr>
<th>Component</th>
<th>Ultimate Moment ft.-kip</th>
<th>Ultimate Axial Load, kip</th>
<th>Balanced Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>746</td>
<td>2540</td>
<td>747 ft.-kip</td>
</tr>
<tr>
<td>Column</td>
<td>202</td>
<td>1150</td>
<td></td>
</tr>
</tbody>
</table>

3.6 BEARINGS

There are two possibilities of bearing failure. Extremely heavy loads could cause yielding in the vicinity of the pin or failure of the entire bearing assembly. Codes typically account for the former by specifying a limiting bearing stress in the area contacting the pin. A corresponding upper bound is simply the contact area times the smaller yield stress of the materials in contact. The proximity of this upper bound to the true bearing strength depends upon the amount of rotation which the pin must undergo. This aspect is considered to prevent a
gross overestimation of bearing capacity. Clearly local yielding near the pin would not render the bridge incapable of carrying load.

Also to be considered is the strength of the bearing assembly as a whole. The hinged bearing presumably will fail if the web is subjected to its yield load. For the rocker bearing, the ultimate strength should lie between the yield load of the web and an allowable load given in a design code. The bearing reference loads discussed above are summarized in Table 8. The allowable loads listed in the table are based on provisions in the 1965 AASHO code. Details of the computations are presented in Appendix 10.6.

Table 8. Bounds on Bearing Strength

<table>
<thead>
<tr>
<th>Bent No.</th>
<th>Bearing Type</th>
<th>Allowable Load on Pin, kips</th>
<th>Allowable Load on Rocker, kips</th>
<th>Yield Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 &amp; 4</td>
<td>Rocker</td>
<td>269</td>
<td>158</td>
<td>1010</td>
</tr>
<tr>
<td>2</td>
<td>Hinge</td>
<td>294</td>
<td>---</td>
<td>1100</td>
</tr>
<tr>
<td>3</td>
<td>Rocker</td>
<td>294</td>
<td>167</td>
<td>1100</td>
</tr>
</tbody>
</table>

3.7 RIGID PLASTIC COLLAPSE FOR SINGLE CONCENTRATED LOAD

An approximation of the required vehicle weight can be made by performing a rigid plastic analysis for a single concentrated load. The load is imagined as being uniformly distributed along a line segment across the width of the bridge. It is assumed that the full ultimate moments, given in Tables 3 and 4, can develop at the hinge locations. The location of the positive hinge in the approach spans is adjusted to
achieve the minimum collapse load. Due to symmetry of loading and structure, the minimum collapse load for the main span must correspond to a midspan loading. The quantities of interest in the calculations are shown in Figure II.

For either approach span, the collapse load is computed as follows.

\[
\text{Internal work} = W_i = M_p l (a + \frac{ax}{72-x}) + aM_p 2
\]

\[
\text{External work} = W_e = Pax + (2.53)(72)(\frac{P}{ax})
\]

By equating internal and external work and rearranging terms, an expression for the collapse load is obtained.

\[
P + 91.0 = M_p l (\frac{1}{x} + \frac{1}{72-x}) + M_p 2 (\frac{1}{x}) \tag{1}
\]

The collapse load is minimized by taking the derivative of equation (1) with respect to x. Performance of the differentiation and rearrangement of terms gives

\[
-(M_p l + M_p 2) (72-x)^2 + M_p l x^2 = 0 \tag{2}
\]

Equation (2) gives the location of the positive plastic hinge corresponding to the minimum collapse load. Equation (1) gives the magnitude of the collapse load. For the lower bound case, the values of x and P are 42.8 feet and 493 kips, respectively. For the upper bound case, the values are 42.6 feet and 638 kip.
The computations are simpler for the main span because the location of the positive hinge is known from symmetry.

\[ W_1 = 2a (M_{p2} + M_{p3}) \]

\[ W_e = 46.5aP + (2.53)(93)(\frac{a}{2})(46.5) \]

Equating internal and external work, the collapse load is found to be 531 kips (lower bound) and 694 kips (upper bound).

The results of the above computations show that the approach spans are critical with a collapse load (lower bound) of 493 kip.
4. TEST VEHICLE

4.1 CRITERIA FOR VEHICLE SELECTION

Preliminary computations indicate a minimum required gross weight of approximately 500 kips. Since testing involves a gradual increase in load, the bed of the vehicle must offer easy accessibility for addition of load increments. These increments could be parcelized, such as the concrete blocks used in the AASHO tests, or measured quantities of loose ore or other dense material. The shortest possible vehicle length is desired, with a large proportion of the weight being supported by a few closely spaced axles.

Logistics is an important factor. The bridge is located on a two lane county road in a somewhat remote rural area. The test vehicle must be able to reach the site under its own power or be transported on another vehicle. If the vehicle must be disassembled for transport, it must be relatively easy to reassemble at the site. Similarly, the material used to load the vehicle must be easily obtainable and be reasonably easy to transport to the site.

4.2 CAPACITIES OF SOME VEHICLES

The preliminary analysis indicates an extremely heavy vehicle. There are certainly no commercial vehicles with the required capacity. American military tanks (M48 and M60) have a loaded weight of 94 kips. Off-the-road dump trucks (e.g. Euclid R-75 and Mack Series M rear dumps) used in earth moving operations typically have a rated payload capacity of 150 kips. The military vehicle (vehicle 97) used for ultimate strength bridge loadings in the AASHO tests provided a gross weight of 318 kips.
Assuming that railroad tracks could be erected on the bridge deck, railroad cars (both flatcars and open type hopper cars) are designed for a maximum gross weight of about 260 kips. Even if the cars could be substantially overloaded, the distance between truck centerlines (66 feet for flatcars and 42 feet for hopper cars) makes them poorly suited for use on this bridge. It would be possible to utilize only the trucks, which are rated for a maximum of 200 kips each (50 kips per wheel), and design and build a special car. But the cost of a custom built car along with the cost of erecting the track on the bridge make this alternative unattractive.

4.3 PROPOSED BRIDGE MODIFICATIONS

It appears that even among unusually heavy vehicles, there is none whose capacity is sufficient to fail the bridge. It would be possible to build a special vehicle which would be winched or towed across the bridge. A simpler alternative would be saw through the bridge deck longitudinally leaving only the two interior composite beams. In the latter case the modified bridge would probably be vulnerable to vehicle 97 used in the AASHO tests. In addition to greater simplicity and smaller cost, the second alternative is more appealing because it utilizes an existing rubber tired vehicle. Clearly it is desirable that the test loading be as realistic as possible. This feature could not be easily incorporated into the design of a special vehicle with a capacity of 500 kips or more.

The best course seems to be to concede that the bridge is invulnerable to any sort of realistic vehicle which could be provided. The strength of the bridge can be reduced by means of longitudinal cuts so that it can be failed by an existing, albeit unusually heavily
loaded, vehicle. The need to substantially reduce the strength of the bridge emphasizes the conservatism of the design with respect to strength. Given that design criteria other than strength require a minimum stiffness, it may still be that the design of this bridge was quite conservative. Preliminary testing of the bridge in its unmodified state should demonstrate that gross overloads will not produce serious damage, much less collapse.

4.4 CHARACTERISTICS OF PROPOSED VEHICLE

The vehicle proposed above, referred to hereafter as vehicle 97, is a military tractor-trailer with seven axles. When the vehicle is fully loaded, the rearmost three axles support about 72 percent of the total weight. Figure 12 shows the axle spacing and axle loads. The loads were measured during the AASHO tests, and represent the practical capacity of the vehicle.

For purposes of computations, the axle loads given in Figure 12 will be considered as representative of the relative proportions of axle loads at all stages of loading. Hence the axle loads will be assumed to increase in the relative proportions given by the maximum loads. Computations based on this assumption will not be in serious error. It will be shown that the phenomena which are of primary interest, namely yielding of the stringers, shakedown at loads near the collapse load, and collapse, occur only after the vehicle has been substantially or completely loaded. It should be noted that the assumption of proportional axle loads is in no way required for the validity of the shakedown analysis. As far as shakedown is concerned, the assumption is merely a computational expedient. This represents an important distinction between shakedown and plastic analyses.
5. ANALYSIS OF MODIFIED BRIDGE FOR PROPOSED TEST VEHICLE

A vehicle has been proposed based on the theoretical rigid plastic collapse load of an approach span. Several possible failure modes were listed in section 3. Shakedown is the preferred mode of failure for the test but no attempt will be made to eliminate other failure modes. The bridge will be analyzed for the suggested vehicle, and conclusions will be drawn as to the likelihood of each mode and the values of critical loads.

5.1 ELASTIC FLEXURAL RIGIDITY OF MODIFIED BRIDGE

An elastic analysis is of interest because it provides a prediction of the load which causes first yielding, and because the shakedown analysis is based on elastic response. Consideration of the elastic behavior of the bridge raises the question of the proper way to evaluate its flexural rigidity. The slab is customarily assumed to contribute to the moment of inertia only when the curvature at the point being considered is positive. The portion of the continuous beam which experiences positive curvature, especially for loads near the ultimate capacity, is highly dependent upon the location of the vehicle. The elastic stiffness is then a function of vehicle location. A precise elastic analysis would involve a tedious iterative process in which the elastic flexural rigidity of segments of the beam would be adjusted according to a previously computed moment diagram.

Figure 13 shows elastic moments of inertia for the modified bridge for both positive and negative curvatures at each point. Minor variations due to changes in the slab haunch (see Figure 4) are neglected. Elastic section moduli for the bottom steel fiber at the middle of the spans and at the intermediate supports are also listed on
Figure 13. It is seen from the figure that the magnitude of the moment of inertia for the coverplated region under negative curvature is very nearly the same as for the remainder of the beam under positive curvature. The beam would be prismatic for all practical purposes if the coverplated regions always experienced negative bending and the remainder of the beam positive bending. This is not the case. Figure 14 shows the elastic moment envelope for the fully loaded vehicle 97 assuming a prismatic beam. There is no clear definition of positive and negative curvature regions.

A precise elastic analysis, taking into account the load dependency of the beam stiffness, will not be presented here. Instead the results of the computations for an assumed prismatic beam will be accepted as a reasonable approximation to true behavior. Dead load will also be assumed to act on a prismatic beam, and live and dead load effects will be superposed in the usual way. A more refined elastic analysis is a worthwhile topic for future study, and should be performed as part of the research if the bridge is actually tested as proposed in this report.

5.2 ASSUMPTIONS AND DEFINITIONS FOR SHAKEDOWN ANALYSIS

The procedure which will be used to compute the shakedown load(21) of the modified bridge is based on the assumption that each composite beam exhibits an idealized elastic-plastic moment-curvature relation shown in Figure 15. The ultimate or plastic moments in positive and negative bending are denoted by $M_u^+$ and $M_u^-$, respectively. The assumption implies that behavior is elastic until the ultimate moment is reached. A comparison of some experimental moment-curvature relations for positive and negative bending of composite beams with the idealization of Figure
15 is shown in reference 14. A more extensive investigation of moment-curvature relations for composite beams in both positive and negative bending is presented in reference 15. The resemblance of experimental curves to the idealized behavior of Figure 15 is apparent, although the authors of that work employed a more complex analytical scheme which did not result in a bilinear moment-curvature relation.

In a strict sense, the shakedown analysis used here also assumes that the beam cross section has two axes of symmetry and that the yield stress is the same in tension and compression. Clearly a composite T-beam violates both of these assumptions. However if these restrictions are conveniently ignored, the shakedown analysis yields reasonable results. "Reasonableness" here is indicated by the fact that the computed shakedown load is close to, but slightly less than, the plastic collapse load. This has been found to be a common result for steel structures, and it is reasonable to expect that a similar behavior would be exhibited by composite beams.

It is emphasized that no assumption is made regarding the loading history. In particular, the loading need not be proportional. The only information required is the location of, and extreme values of, each load. Any fluctuation of a load within its specified extreme values is permitted.

Shakedown analysis considers three types of internal bending moments which will be referred to as actual moment, purely elastic moment, and residual moment. These three moments will be referred to by the symbols $M_a$, $M_e$, and $M_r$, respectively. The actual moment, $M_a$, as the name implies, is that internal moment which actually exists at some cross section. The purely elastic moment, $M_e$, is an internal moment
which is computed by assuming that the entire structure responds to the load in a purely elastic fashion, regardless of the magnitude of the load. The residual moment, \( m \), is defined by the relation

\[
m = M_a - M_e
\]

(3)

where all three quantities in equation (3) refer to the same cross section. The residual moment as defined by equation (3) is not necessarily the actual residual moment which would be present in the structure after unloading. It is a hypothetical quantity which, as is explained in the next section, is used in shakedown analysis.

Another term which requires explanation is "incremental collapse mechanism." Incremental collapse can occur only if during each loading cycle there are inelastic rotations at certain cross sections. If plastic hinges were to form simultaneously at each of these same cross sections, plastic collapse would occur.\(^{(21)}\) Hence an incremental collapse load is associated with a particular incremental collapse mechanism and the associated set of hinges. The utility of this concept in the calculations is explained in the next section.

5.3 SHAKEDOWN ANALYSIS PROCEDURE

The shakedown analysis is performed according to the following steps. The analysis for the modified bridge is contained in Appendix 10.7.

(1) Formulate equations of equilibrium for the residual moments, \( m \), the external loads being zero. This is done the same way as in plastic analysis, that is, by applying virtual work to assumed mechanisms.
(2) Assuming purely elastic response, compute and tabulate the moments, $M_e$, at all possible hinge locations corresponding to the extreme values of each external load.

(3) Using the results of step 2, compute and tabulate the maximum and minimum elastic moments, $(M_e)_{\text{max}}$ and $(M_e)_{\text{min}}$, at each possible hinge location, considering all possible combinations of external loads. Tabulate $(M_e)_{\text{max}} - (M_e)_{\text{min}}$ for each possible hinge location.

(4) Select a trial incremental collapse mechanism for a given span by assuming the location of the positive hinge. The negative hinge(s) will form at the intermediate support(s).

(5) Formulate expressions for the residual moments at each of the hinge locations.

(6) Substitute the expressions for the residual moments from step 5 into the equilibrium equation from step 1 for the selected mechanism and solve for the incremental collapse load. The critical location of the positive hinge corresponds to the smallest computed incremental collapse load.

(7) Compute the alternating plasticity load by equating for each possible hinge location the quantities $(M_e)_{\text{max}} - (M_e)_{\text{min}}$ and $(M_{u+} - M_{u-})$. The smallest computed alternating plasticity load is critical.

(8) The shakedown load is the smaller of the computed critical shakedown and alternating plasticity loads.

The procedure outlined above is applicable for continuous beams for which the correct incremental collapse mechanisms are known, the locations of positive hinges being determined by investigation of likely
locations. The correct location of the positive hinge corresponds to the smallest computed incremental collapse load. Further discussion of this approach for continuous beams may be found in reference 22.

For the case of a general framed structure, the incremental collapse may not be obvious, and additional steps are required in the analysis. The more general case has been discussed by Neal. (21)

Steps 5 and 7 are extremely important because they make use of the so called "shakedown theorems," which are discussed in detail and proven rigorously in theoretical works on shakedown. The formulation of expressions for the residual moments in step 5 is based on the reasoning that if the incremental collapse load is repeatedly applied to a structure, the moments at a plastic hinge location of the incremental collapse mechanism will eventually be related by the following equations.

\[ m + (M_e)_{\text{max}} = M_u^+ \] (positive hinge) \hspace{1cm} (4a)

\[ m + (M_e)_{\text{min}} = M_u^- \] (negative hinge) \hspace{1cm} (4b)

Equations (4) state that at each hinge the residual moment plus the extreme elastic moment must just equal the ultimate moment.

The alternating plasticity load is computed in step 7 according to the equation

\[ (M_e)_{\text{max}} - (M_e)_{\text{min}} = M_u^+ - M_u^- \] \hspace{1cm} (5)

Equation (5) states that the range of applied elastic moments is just equal to the elastic range of the internal moment. The elastic range
of internal moment depends upon the moment-curvature relation for the member being considered. In the present case, the moment-curvature relation shown in Figure 15 indicates that the elastic range is the difference between the positive and negative ultimate moments.

It is not intended that the above paragraphs serve as proofs of equations (4) and (5). For reasons noted in section 5.2, the present application of these equations is outside the realm of rigorously proven shakedown theorems. Nevertheless the equations are used here in the expectation that they represent a good approximation to the behavior of composite beams.

5.4 ANALYSIS FOR BRIDGE STRENGTH ASSUMING UNIFORM ELASTIC STIFFNESS

The results of an analysis of the modified bridge for vehicle 97 are summarized in Table 9. The calculations, which are presented in Appendix 10.7, assume uniform elastic stiffness, as well as the idealized elastic-plastic moment-curvature relationship shown in Figure 15. The analysis considers moments at the tenth points of each span and increments in vehicle location of 7.2 feet, or one-tenth of the length of an approach span. Consideration of moments only at tenth points is an approximation. In view of the rather broad limits on material strengths and the assumption of uniform elastic stiffness, a more precise location of maximum moments is unwarranted.

Of particular interest is the fact that hinges form first within the spans rather than at the intermediate supports. This condition has been the topic of considerable discussion. Barnard and Johnson discussed in detail the differences between positive and negative hinges in composite beams. The internal moment of a negative hinge tends to increase with inelastic rotation; strain hardening occurs. The internal
Table 9. Analysis Based on Uniform Elastic Stiffness

\[ P = \text{Weight in Kips of Front Axle} \]

Vehicle Capacity: \( P = 13.8 \text{ Kips} \)

See Figure 10.7.1.2 for notation

<table>
<thead>
<tr>
<th>Value of P</th>
<th>Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower Bound</td>
<td>Upper Bound</td>
</tr>
<tr>
<td>7.57</td>
<td>9.98</td>
</tr>
<tr>
<td>8.01</td>
<td>10.5</td>
</tr>
<tr>
<td>11.8</td>
<td>15.3</td>
</tr>
<tr>
<td>12.3</td>
<td>15.8</td>
</tr>
<tr>
<td>13.8</td>
<td>17.8</td>
</tr>
<tr>
<td>14.9</td>
<td>19.1</td>
</tr>
<tr>
<td>14.9</td>
<td>19.2</td>
</tr>
<tr>
<td>15.4</td>
<td>19.8</td>
</tr>
</tbody>
</table>
moment of a positive hinge begins to decrease after a certain amount of inelastic rotation has occurred. This has been termed strain softening. For a statically indeterminate structure, strain softening can result in instability and instantaneous collapse. For a continuous beam, a positive hinge may begin unloading before an adjacent negative hinge has fully formed. This would amount to a reduction in the ultimate strength of the beam. A further discussion of this phenomenon will not be presented here. It is noted that formation of positive hinges before negative hinges may result in reduced strength. This possibility should be considered in the analysis of the test data.

5.5 OTHER POSSIBLE MODES OF FAILURE

Several secondary modes of failure were discussed briefly in section 3. While the tests are intended primarily to investigate the ultimate strength of the bridge in terms of major axis bending failure, there is no intention to artificially prevent other modes of failure. The only stiffening of the structure will be the earth fill around the intermediate support frames. This is obviously necessary for reasons of safety. Various secondary failure modes will be further investigated below.

5.5.1 Local Buckling of Steel Beam

Based on the information in Table 5, local buckling of the beam webs can be expected at the intermediate supports for loads near the ultimate capacity. The single bearing stiffener plate on each side of the beam web, shown in Figure 10(a), should not be effective in preventing buckling, since buckling has been observed to occur in test beams with several stiffeners at the support. The effects of local buckling on strength, if any, should be carefully observed and interpreted during and after the actual test.
5.5.2 Loss of Shear Connection

The ultimate strength of a single shear connector as computed in section 3.3 is 84 kips. The analysis summarized in Table 9 reveals the location of positive hinges at collapse to the nearest one-tenth of the span length. For each approach span, the positive hinge at collapse forms at approximately four-tenths of the span length from the abutment. The critical region for shear connection is between the hinge and the point of inflection which occurs at about 17.5 feet from the intermediate support. The number of connectors in this region for one beam is 17, providing a potential resisting force of 1430 kips. The force in the effective flange width of the slab at the hinge is 1310 kips. The shear connection is thus adequate in the approach spans.

For the main span, the hinge forms at midspan. The location of the points of inflection on either side of the hinge depend on the stage of loading. If the structure is entirely elastic except at the midspan hinge, the points of inflection are on the approach spans, the vehicle being entirely on the main span. Then one-half of the main span shear connectors are effective in each direction. A more severe case would be when the full plastic hinges have formed at the intermediate supports as well as at midspan. Presumably this will never happen since the approach spans will collapse first. But for purposes of evaluating the shear connection, it may be assumed that a mechanism can form in the main span. Then, from statics, the inflection point which is closer to midspan is between the second and third axles of the vehicle, about 22 feet from the support. The number of connectors in this region is 17 (for one beam), with a potential resisting force of 1430 kips. The slab force is 1310 kips, indicating adequate shear connection.
5.5.3 Yield Line Failure Within Slab Panels

Yield line analysis allows consideration of additional collapse modes. Figure 16 shows a possible collapse mode for an approach of the modified bridge. A single concentrated load simulating an axle is shown distributed along a line segment. The load rests only on the two interior composite beams. The dashed line indicates the yield line pattern. The two interior composite beams fail in major axis bending, but the exterior beams do not experience ultimate moments. Instead yield lines form in the slab along diagonals and in the vicinity of the exterior stringers as the middle portion of the bridge sags under the load. The precise location of the longitudinal yield lines may be on either side of the steel beams, depending upon stiffness considerations. It will be assumed that these yield lines are just inside the stringer flange tips. The lower bound collapse load of an approach span for this mode is 1100 kips, as compared to 493 kips for the rigid collapse case evaluated in section 3.7. Details of the yield line analysis are presented in Appendix 10.8. The considerably higher load required for the yield line mode as compared to rigid collapse may be attributed to the relative dimensions of the bridge. If the bridge deck were wider, the two loads would more nearly coincide. These results are further indication that modification of the bridge is necessary.

For the modified bridge, there is the possibility that wheel loads might cause a cantilever type failure of the overhanging T-beam flanges. Figure 17(a) shows the modified bridge with vehicle 97 centered. The precise shape of the yield line pattern corresponding to the smallest possible collapse load is not obvious. Figure 17(b) shows a possible yield line pattern in the slab near an abutment. A single concentrated
load, simulating one pair of dual wheels, is considered. The lower bound collapse load is 82.6 kips. Assuming the rear axle loads of vehicle 97 to be evenly distributed among the four sets of dual wheels, the maximum force on one pair of dual wheels is 20.6 kips. Considering the outboard duals of the three heavy rear axles to act together as a concentrated load, their maximum combined magnitude is about 60 kips. From this limited investigation, a cantilever type collapse does not appear likely.

Another possibility which will not be investigated here concerns possible vehicle damage. When the heavy axles are near midspan, the flexibility of the cantilevers of the modified bridge might cause a transfer of load from the exterior to the interior sets of dual wheels. Depending on how the vehicle load is applied to the axles, overstressing of an axle might result. Another danger is overloading of the interior dual tires. These possibilities are worthy of further study if the bridge is actually tested as proposed in this report.

5.5.4 Liftoff

Since the bearings were designed to resist only a downward force, there is a danger that the bridge might lift off of the supports at the abutments when the vehicle is on the main span. In such a case, the uplift force at an abutment is proportional to the negative moment at the adjacent intermediate support. Considering an approach span as a cantilever, the moment at the intermediate support due to dead load is 3550 ft.-kips. This is the maximum support moment which can be sustained without liftoff.

Examination of Table 9 reveals that the fully loaded vehicle (P = 13.8) may or may not cause a hinge to form within the main span.
If no hinge forms, the response is elastic and the larger support moment is 2340 ft.-kip. If a hinge does form, there is some redistribution of moment and the larger support moment is 3260 ft.-kip. Since the limiting value of 3550 is not exceeded in either case, no liftoff is anticipated.

It is interesting to note that the main span could not support its ultimate load unless ballast were placed on the approach spans. If the full ultimate moment (lower bound) developed at the intermediate supports, a concentrated force of 10.1 kips would be required at each abutment to hold down the approach spans. This is of course not a practical problem since the approach spans will collapse before uplift occurs.

5.5.5 Intermediate Support Collapse

The intermediate support frames will receive an unspecified amount of bracing from the earth fill. Of primary concern here is whether the frames are likely to fail in a symmetric mode. Since the amount of fixity at the column bases is not known, pinned and fully fixed bases will be considered. Lower bound ultimate strengths of the frame components are listed in Table 7.

To examine the adequacy of the intermediate supports under test loadings, the distribution of internal moments based on elastic analysis will be compared to member ultimate moments. This is a customary approach in the design of reinforced concrete structures, and provides a conservative estimate\(^\text{(17)}\) of member strength. No reliance is placed on the ability of the member to undergo inelastic rotations, and so there is assumed to be no internal moment redistribution.
The frame, which is symmetrically loaded and is itself symmetrical, is shown in Figure 18. For purposes of analysis, the stiffening effect of the corner brackets is neglected. An elastic analysis according to the slope-deflection method is contained in Appendix 10.9. Table 10 summarizes the results. The lower bound on column strength is based on an assumed linear failure interaction curve between ultimate moment and the balance point. This is a conservative assumption. The results listed in the table indicate that both beam and column members have sufficient strength to withstand test loadings.

Table 10. Portal Frame Analysis

<table>
<thead>
<tr>
<th>Member</th>
<th>Maximum Applied Moment, Axial Load</th>
<th>Lower Bound on Strength of Member</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>521 ft.-kip</td>
<td>746 ft.-kip</td>
</tr>
<tr>
<td>Column</td>
<td>254 ft.-kip 275 kip</td>
<td>332 ft.-kip 275 kip</td>
</tr>
</tbody>
</table>

5.5.6 Foundation Failure

The precast concrete piles were discussed in section 3.4. A comparison of maximum vertical load on each pile group and the rated capacity is presented in Table 11. For this comparison it is assumed that each pile group is required to support the entire vehicle load, plus dead load as determined by elastic analysis. It is seen that the intermediate pile groups are essentially adequate without reliance on a safety factor, whereas the abutment pile groups are adequate if the
Table 11. Loads on Precast Concrete Piles

<table>
<thead>
<tr>
<th>Location of Pile Group</th>
<th>Number of Piles in Group</th>
<th>Rated Capacity of Group, kip</th>
<th>Maximum Load, kip*</th>
<th>Required Actnal Safety Factor Against Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>End Bent</td>
<td>4</td>
<td>256</td>
<td>443</td>
<td>1.73</td>
</tr>
<tr>
<td>Intermediate Bent</td>
<td>10</td>
<td>640</td>
<td>658</td>
<td>1.03</td>
</tr>
</tbody>
</table>

*Bent weight, elastic dead load, reaction, and full vehicle load.

*Includes bent weight, elastic dead load, reaction, and full vehicle load.
actual factor of safety against failure is about 2. Considering that the nominal design safety factor was 6, the foundations appear to be adequate.

5.5.7 Bearing Failure

Bounds on the strengths of the bearings are listed in Table 8. To evaluate the bearings for test loadings, it is assumed that each bent must support the elastic dead load reaction plus the entire vehicle weight. The critical location is bent 3 where each of two rockers of the modified bridge supports a total of 217 kips (load $P_1$ in Appendix 10.9). The allowable rocker load from Table 8 is 167 kips, indicating an overload of about 30 percent. Since the 167 kip figure refers to working load (not ultimate), there does not seem to be any danger of bearing failure during the tests.
6. TEST PROJECT

6.1 OPERATIONS AT TEST SITE

Table 12 is a timetable for the test project. The unmodified bridge will first be instrumented and subjected to a limited amount of testing. The application of several load increments up to the full vehicle capacity should not cause significant damage. The bridge strength will then be reduced by sawing through the slab between the interior and exterior stringers on both sides of the bridge.

The main body of tests will be performed on the modified bridge. The vehicle will be slowly driven back and forth with the occasional addition of load increments. Enough passes will be made with each load increment to assure that deflections have stabilized, that is, that measured deflections are changing by a negligibly small amount. Eventually a load will be reached for which deflections will not stabilize, but will continue to increase with each pass of the vehicle. The resulting large permanent deflections will mark the end of the test. A test procedure similar to that described above has been used in laboratory shakedown tests of steel wide flange beams.\(^{(23)}\)

6.2 COST ESTIMATE

Table 13 contains a cost estimate for the test project. About 45 percent of the total cost involves the technical staff. Ultimate responsibility for the project will rest with the chairman of the Department of Civil and Environmental Engineering of Washington University. Details of the project will be carried out by a graduate student, with student assistance during instrumentation and testing operations, and as required during test preparations and reduction of test data.
Nonuniversity personnel will be required for operation and maintenance of the test vehicle and crane, performance of the sawing operation, and fabrication of concrete loading blocks for the test vehicle. The relatively large estimate for the test vehicle reflects the anticipated high cost of transporting the vehicle to and from the test site, as well as operation and maintenance costs. The rental periods for the cutting unit and crane are based on the schedule shown in Table 12.
<table>
<thead>
<tr>
<th>Operation at Test Site</th>
<th>Operation at End of Operation</th>
<th>Cumulative Weeks Required</th>
<th>Weeks Required at Test Site</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test vehicle, concrete blocks, power units</td>
<td>Tests of modified bridge</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>4.5</td>
<td>Instrumentation of modified bridge</td>
<td>4.5</td>
<td>4.5</td>
</tr>
<tr>
<td>3.5</td>
<td>Modification (sawing) of bridge</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td>2.5</td>
<td>Instrumentation of unmodified bridge</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>2</td>
<td>Tests of unmodified bridge</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>and recording equipment in readiness</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 12. Test Project Timetable
Table 13. Cost Estimate

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Technical Staff</strong></td>
<td></td>
</tr>
<tr>
<td>Administration and Supervision Engineer (Graduate Student)</td>
<td>2000</td>
</tr>
<tr>
<td>3 months full time at $700/month</td>
<td>2100</td>
</tr>
<tr>
<td>9 months half time at $300/month</td>
<td>2700</td>
</tr>
<tr>
<td>Tuition at $1100 per semester</td>
<td>2200, 7000</td>
</tr>
<tr>
<td>Student Help</td>
<td>4000</td>
</tr>
<tr>
<td>2 people for 3 months</td>
<td></td>
</tr>
<tr>
<td><strong>Overhead</strong></td>
<td></td>
</tr>
<tr>
<td>60 percent of above staff cost excluding tuition</td>
<td>6500</td>
</tr>
<tr>
<td><strong>Travel Between St. Louis and Test Site</strong></td>
<td>1000, 20500</td>
</tr>
<tr>
<td><strong>Reproduction of Reports</strong></td>
<td>500</td>
</tr>
<tr>
<td><strong>Bridge Modification</strong></td>
<td></td>
</tr>
<tr>
<td>Cutting Unit Rental at $100/week</td>
<td>100</td>
</tr>
<tr>
<td>Cutter Blades at $10.50 each</td>
<td>500</td>
</tr>
<tr>
<td>Labor</td>
<td>200, 800</td>
</tr>
<tr>
<td><strong>Test Vehicle Plus Driver Including Transport and Maintenance</strong></td>
<td>15000</td>
</tr>
<tr>
<td><strong>Concrete Loading Blocks</strong></td>
<td></td>
</tr>
<tr>
<td>300 kips at $50/cubic yard</td>
<td>3700</td>
</tr>
<tr>
<td><strong>Rental of Power Units and Data Recorders</strong></td>
<td>2000</td>
</tr>
<tr>
<td><strong>Electrical Supplies and Gages</strong></td>
<td>200</td>
</tr>
<tr>
<td><strong>Crane Rental at $1250/month Plus Operator and Insurance</strong></td>
<td>3300, $46000</td>
</tr>
</tbody>
</table>
7. SUMMARY AND CONCLUSIONS

The feasibility of testing a continuous composite highway bridge for ultimate strength under rolling vehicular load has been investigated. The strength of the bridge was determined by shakedown analysis, which is appropriate for bridges in that it does not require the assumption of proportional loading. Details of the bridge were examined to assure that the failure of a weak element would not cause premature termination of tests. A modification of the bridge to expedite testing, and a test procedure were proposed. A cost estimate for the test project was presented.

The analyses presented in this work lead to the following conclusions.

(1) The strength of the Butler County bridge in its present form is too great to allow tests for ultimate strength with a realistic vehicular load.

(2) Tests are feasible if the bridge strength is reduced by approximately half by making longitudinal cuts through the slab between the interior and exterior stringers on both sides. The cost of such a modification is not excessive in comparison to the cost of the overall project.

(3) A vehicle used in the AASHO Road Test (designated as vehicle 97) has sufficient capacity to produce inelastic response in the modified bridge, although it should not cause instantaneous collapse. Collapse is not essential to the success of the tests.
(4) Structural elements of the bridge, including foundation, intermediate supports, shear connection, and bearings, have sufficient strength to withstand test loadings.

(5) The unmodified bridge should be subjected to the full capacity of the test vehicle to verify that no significant damage will occur.

(6) Shakedown tests of the Butler County bridge would provide a valuable contribution to the understanding of full scale bridge behavior. A thorough understanding of shakedown in continuous composite bridges is essential to the development of ultimate strength design procedures for this type of bridge.
8. NOMENCLATURE

The following alphabetically arranged symbols and abbreviations were used in this report. They are summarized here for convenience.

- \(a\)  
  (1) Depth of rectangular stress block.  
  (2) Angle of virtual rotation.  
  (3) Refers to approach span when used as a subscript.

- \(A\)  
  Area of cross sectional element (used with a subscript).

- AASHO  
  American Association of State Highway Officials.

- ACI  
  American Concrete Institute.

- AISC  
  American Institute of Steel Construction.

- \(A_s\)  
  Area of tensile reinforcement.

- \(A'_s\)  
  Area of compression reinforcement.

- \((A_s)_{\text{total}}\)  
  Total area of reinforcement.

- \(b\)  
  (1) Angle of virtual rotation.  
  (2) Width of cross sectional element.

- \(c\)  
  (1) Angle of virtual rotation.  
  (2) Distance of plastic centroid of composite T-beam above horizontal centerline of steel beam.

- \(d\)  
  (1) Effective depth of reinforced concrete section.  
  (2) Depth of steel beam.  
  (3) Diameter of bridge rocker.

- \(e_b\)  
  Balanced eccentricity for reinforced concrete beam-column.

- \(F\)  
  Force in cross sectional element (used with a subscript).

- \(f'_c\)  
  Concrete compressive cylinder strength.

- \(F_c\)  
  Internal compressive force resultant.

- \(F_t\)  
  Internal tensile force resultant.

- \(f_y, F_y\)  
  Yield stress.
h  (1) Height of concrete haunch.
     (2) Average flange thickness of channel shear connector.

J  Distance between lines of action of internal compressive and tensile resultants.

k  A constant defined in Appendix 10.1.

m  (1) Residual moment as defined in section 5.2.
     (2) Refers to main span when used as a subscript.

M_a  Actual internal moment.

M_e  Purely elastic moment as defined in section 5.2.

M_p  Plastic moment (used interchangeably with ultimate moment).

M_u  Ultimate moment.

M_u  Ultimate moment under positive curvature.

M_u  Ultimate moment under negative curvature.

p  (1) Allowable load on bridge rocker.
     (2) Steel ratio for reinforced concrete section.

P  (1) Vehicle front axle load.
     (2) Applied concentrated load.
     (3) Axial load on cover plated beam section.

P_{1,P_2}  Reactions at intermediate support, defined in Appendix 10.9.

P_A  Alternating plasticity load.

P_b  Balanced axial load for reinforced concrete beam-column.

P_h  Load causing first formation of a plastic hinge.

P_{IC}  Incremental collapse load.

P_u  Ultimate axial load for reinforced concrete column.

P_y  (1) Load causing first yield of a beam flange or cover plate.
     (2) Axial yield load of cover plated beam section.
\( Q_u \) Ultimate strength of a channel shear connector.

\( S \) Refers to cross section at intermediate support.

\( t \)
(1) Web thickness of channel shear connector.
(2) Thickness of cross sectional element of steel beam.
(3) Total depth of reinforced concrete section.

\( w \)
(1) Length of channel shear connector.
(2) Width of cross sectional element.

\( W_e \) External virtual work.

\( W_i \) Internal virtual work.

\( x \) Dimension which locates the positive hinge in a rigid plastic collapse mechanism.
FIGURE 2 BEAM SIZES AND DIMENSIONS
FIGURE 3 LOCATIONS OF COVER PLATES, DIAPHRAGMS, AND CROSS FRAMES
REINFORCED SLAB WITH 6" MINIMUM THICKNESS

\[ h = \text{HAUNCH HEIGHT} \]

W30x99 OR W30x108

(a) TYPICAL HAUNCHING DETAIL

SYMmetrical about \( \ell \)

APPROACH SPAN   MAIN

\[
\begin{array}{cccccccc}
4 \text{@18'} & & & & & & & 2 \text{@23.25'} \\
\hline
\end{array}
\]

INTERIOR BEAM

\[
\begin{array}{cccccccc}
15'' & 11'' & 7'' & 15'' & 19'' & 2'' & 4'' \\
16 & 16 & 16 & 16 & 16 & 16 & 16 \\
\end{array}
\]

EXTERIOR BEAM

\[
\begin{array}{cccccccc}
13'' & 7'' & 9'' & 13'' & 5'' & 2'' \\
16 & 16 & 16 & 16 & 16 & 16 \\
\end{array}
\]

(b) VALUES OF HAUNCH HEIGHT \( h \)

FIGURE 4 DETAILS OF SLAB HAUNCH
DIRECTION OF TRAFFIC

TRANSVERSE STEEL
TOP: #5’s AT 6.5"

LONGITUDINAL STEEL
TOP: ALTERNATING # 4’s AND # 5’s, 4.5" CENTER TO CENTER OF ADJACENT BARS

BOTTOM: # 5’s AT 6.5"

BOTTOM: # 5’s AT 10”, OMITTED NEAR BEAMS

(a) REINFORCING PATTERNS

(b) ARRANGEMENT OF LAYERS

FIGURE 5 SLAB REINFORCEMENT
A number of spaces between connectors in interval $B = \text{length of space in inches}$

Note: Same spacing applies for all four beams.

- Denotes no connectors, splice within interval
- Denotes no connectors, intermediate support region

**Total Connectors**

<table>
<thead>
<tr>
<th>Span 1-2</th>
<th>49</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span 2-3</td>
<td>49</td>
</tr>
<tr>
<td>Span 3-4</td>
<td>46</td>
</tr>
<tr>
<td>Each Beam</td>
<td>141</td>
</tr>
</tbody>
</table>

**Figure 6: Shear Connector Information**

- (a) Shear connector spacing for one beam
- (b) Channel shear connector

**Legend**

- $A =$ number of spaces between connectors in interval
- $B =$ length of space in inches

**Note:**

- Symmetrical about main span
- Bearing
NOTE: DIMENSIONS SHOWN ARE COMMON TO END BENTS 1 AND 4. SOME DIMENSIONS NOT SHOWN ARE DIFFERENT FOR THE TWO BENTS.
NOTE: NOT ALL REINFORCEMENT SHOWN

SYMMETRICAL ABOUT $t$

BATTER 2'/FT.
PRECAST CONCRETE PILE

FIGURE 8 INTERMEDIATE SUPPORT
DIMENSION "A" 10 $\frac{1}{2}$ BTS. 1, 4
11 $\frac{1}{2}$ BT. 2

(a) EXPANSION BEARING

(b) FIXED BEARING

FIGURE 9 BEARINGS
SYMMETRICAL ABOUT €

(a) CROSS FRAME AT INTERMEDIATE BENT

(b) CROSS FRAME AT END BENT

(c) DIAPHRAGM WITHIN SPAN

FIGURE 10 DIAPHRAGMS AND CROSS FRAMES
Figure II Rigid plastic collapse for single concentrated load
AXLE LOADS (KIPS) | 13.8 | 13.9 | 31.5 | 30.6 | 72.3 | 73.5 | 82.3
LOAD | 1 | 1.01 | 2.28 | 2.22 | 5.25 | 5.33 | 5.98

LOADS, IN KIPS, WERE MEASURED DURING AASHO TESTS\(^{(6)}\)

FIGURE 12 VEHICLE 97 AXLE SPACINGS AND MAXIMUM LOADS
FIGURE 13 ELASTIC SECTION PROPERTIES OF MODIFIED BRIDGE
FIGURE 14 MOMENT ENVELOPE FOR FULLY LOADED VEHICLE

SYMMETRICAL ABOUT ℓ.

MOMENT, FT·KIP

4040 2760 1380 0 -1380 -2760
FIGURE 15 IDEALIZED MOMENT-CURVATURE RELATION
OF UNMODIFIED BRIDGE

FIGURE 16 YIELD LINE ANALYSIS OF APPROACH SPAN
(a) REAR AXLE ON MODIFIED BRIDGE

DASHED LINES DENOTE ASSUMED YIELD LINE PATTERN. VEHICLE ONE FOOT OFF CENTER.

(b) YIELD LINE PATTERN

FIGURE 17 CANTILEVER SLAB FAILURE
SYMMETRICAL ABOUT

P_2
6.67'

3.33'

P_1

P_1

P_2

2.58'

4.08'

14.83'

2.74'

(a) CENTERLINE DIMENSIONS FOR PORTAL FRAME

PINNED

FIXED

(b) LIMITING CASES OF COLUMN BASE FIXITY

FIGURE 18 PORTAL FRAME ANALYSIS
9. ACKNOWLEDGEMENTS

The author wishes to thank the Missouri State Highway Department for providing engineering drawings and other detailed information on the bridge which was investigated in this report. He also wishes to express his appreciation for the helpful cooperation of the Civil Engineering Department of the University of Missouri at Columbia.
APPENDIX 10.1

Example of Computation of Positive Ultimate Moment

The procedure will be demonstrated for the interior composite T-beam of an approach span. The curvature (crown) of the roadway will be neglected. The elements of the beam have the following properties.

\( f_y \) for W30x99 = 36 ksi (lower bound), 45 ksi (upper bound)

\( f'_c \) = 4.0 ksi (lower bound), 5.5 ksi (upper bound)

\( b \) = minimum of \( \frac{t}{4} \) of span length (216")

Stringer spacing (80")

12 times slab thickness (72")

\( h \) = 1.88 in. (near midspan)

The cross section and assumed stress distribution at ultimate are shown in Figure 10.1.1.

![Figure 10.1.1 STRESSES IN COMPOSITE SECTION AT ULTIMATE](image)

\[ \text{(a) CROSS SECTION} \]

\[ \text{(b) STRESSES AT ULTIMATE} \]
In Figure 10.1.1(b), $k_1$ takes the value 0.85 if $f'_c$ is less than or equal to 4 ksi. For larger values of $f'_c$, $k_1$ is determined by the following formula.(3)

$$k_1 = 0.85 - (f'_c - 4)(0.05)$$

For the assumed upper bound on concrete strength of 5.5 ksi, $k_1$ takes a value of 0.775.

 Bounds on the ultimate moment, $M_u$, are calculated as follows.

<table>
<thead>
<tr>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c = 4$ ksi</td>
<td>$f'_c = 5.5$ ksi</td>
</tr>
<tr>
<td>$k_1 = .85$</td>
<td>$k_1 = .775$</td>
</tr>
<tr>
<td>$F_t = A f_y = (29.1)(36) = 1048$ kip</td>
<td>$F_t = (29.1)(45) = 1310$</td>
</tr>
<tr>
<td>$a = F_t/(.85f'_c b) = \frac{1048}{(.85)(4)(72)} = 4.28$ in.</td>
<td>$a = \frac{1310}{(.775)(5.5)(72)} = 4.27$</td>
</tr>
<tr>
<td>Neutral axis is in slab since $a$ is less than 6.</td>
<td>Neutral axis is in slab.</td>
</tr>
<tr>
<td>$j = 36.9 - \frac{1}{2}d - \frac{1}{2}a$</td>
<td>$j = 36.9 - 14.8 - 2.14 = 20.0$ in.</td>
</tr>
<tr>
<td>$j = 36.9 - 14.8 - 2.14 = 20.0$ in.</td>
<td>$j = 20.0$</td>
</tr>
<tr>
<td>$M_u = F_t j = (1048)(20.0) = 20,960$ in.-kip</td>
<td>$M_u = (1310)(20.0) = 26,200$</td>
</tr>
<tr>
<td>$= 1747$ ft.-kip</td>
<td>$M_u = 2183$ ft.-kip</td>
</tr>
</tbody>
</table>

Note that the assumption of the neutral axis within the slab in Figure 10.1.1 was verified by the calculations. It can happen that the neutral axis falls below the bottom of the slab, in which case the stress distribution shown in Figure 10.1.1(b) is not correct.
APPENDIX 10.2

Example of Computation of Negative Ultimate Moment

The cross section consists of the steel beam and cover plates plus the longitudinal slab reinforcement within the effective flange width.* As shown in Figure 5, there are two layers of longitudinal reinforcement. The bottom layer, consisting of six #5 bars within the effective width, has its centroid 1.875 inches above the bottom face of the slab (not including the haunch). The top layer, consisting of eight #5 and eight #4 bars within the effective flange width, has its centroid 3.625 inches above the bottom face of the slab. Since all of the bars have the same upper and lower bound yield stresses (Table 2), the reinforcement can be conveniently represented as a single element of area in the cross section by a simple computation.

<table>
<thead>
<tr>
<th>Bars</th>
<th>Total Area</th>
<th>Centroidal Distance Above Bottom Face of Slab</th>
<th>Area Times Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 #5</td>
<td>1.86 in²</td>
<td>1.875 in.</td>
<td>3.49</td>
</tr>
<tr>
<td>8 #5</td>
<td>2.48</td>
<td>3.625</td>
<td>9.00</td>
</tr>
<tr>
<td>8 #4</td>
<td>1.60/5.94</td>
<td>3.625</td>
<td>5.80/18.29</td>
</tr>
</tbody>
</table>

The centroid of the reinforcement is \((18.29)/(5.94) = 3.08\) inches above the bottom face of the slab.

For the interior beam at the intermediate support the cross section is shown in Figure 10.2.1.

* This is based on the assumption that the negative steel will participate in the composite behavior. This may not be justified and more detailed analysis will need to be performed later.
FIGURE 10.2.1 CROSS SECTION FOR COMPUTING NEGATIVE ULTIMATE MOMENT

In Figure 10.2.1 the numbers in parentheses refer to rectangular elements of cross sectional area. The plastic centroid is the demarcation between tensile and compressive zones. If the reinforcement were not included, the plastic centroid would coincide with the horizontal axis of symmetry of the cover plated beam. Due to the tensile contribution of the reinforcement, the plastic centroid is above the steel beam centerline. The moment corresponding to full yield of all rectangular elements may be computed about any convenient axis. The steel beam centerline will be used here. The distance \( c \) is determined by internal force equilibrium.

The lower and upper bound negative ultimate moments are computed as follows.
Case 1. Lower Bound

Yield Stresses: Steel Beam 36 ksi
Cover Plates 33
Reinforcement 50

Width of beam web = w = .548 in.

Force in reinforcement = \( F_1 = A_1 f_y = (5.94)(50) = 297 \text{ kip} \)

Internal force equilibrium requires that

\[ (.548)(36)c = \frac{1}{2}(297) \]

so that \( c = 7.53 \text{ in.} \)

<table>
<thead>
<tr>
<th>Element</th>
<th>Area</th>
<th>Stress</th>
<th>Force</th>
<th>Arm About Centerline</th>
<th>Moment (in.-kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.94</td>
<td>50</td>
<td>297</td>
<td>18.17</td>
<td>5396</td>
</tr>
<tr>
<td>2</td>
<td>9.00</td>
<td>33</td>
<td>297</td>
<td>15.29</td>
<td>4541</td>
</tr>
<tr>
<td>3</td>
<td>7.97</td>
<td>36</td>
<td>287</td>
<td>14.53</td>
<td>4170</td>
</tr>
<tr>
<td>4</td>
<td>3.63</td>
<td>36</td>
<td>131</td>
<td>10.84</td>
<td>1420</td>
</tr>
<tr>
<td>5</td>
<td>11.88</td>
<td>36</td>
<td>-428</td>
<td>-3.31</td>
<td>1417</td>
</tr>
<tr>
<td>6</td>
<td>7.97</td>
<td>36</td>
<td>-287</td>
<td>-14.53</td>
<td>4170</td>
</tr>
<tr>
<td>7</td>
<td>9.00</td>
<td>33</td>
<td>-297</td>
<td>-15.29</td>
<td>25655</td>
</tr>
</tbody>
</table>

\( M_u = \frac{(25655)}{(12)} = 2138 \text{ ft.-kip} \)

Case 2. Upper Bound

Yield Stresses: Steel Beam 40 ksi
Cover Plates 44
Reinforcement 70

\( w = .548 \)

\( F_1 = 416 \) and \( c = 9.49 \)

\( \frac{1}{12} M_u = \frac{25655}{12} = 2138 \text{ ft.-kip} \)
<table>
<thead>
<tr>
<th>Element</th>
<th>Area</th>
<th>Stress</th>
<th>Force</th>
<th>Arm About Centerline</th>
<th>Moment (in.-kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.94</td>
<td>70</td>
<td>416</td>
<td>18.17</td>
<td>7559</td>
</tr>
<tr>
<td>2</td>
<td>9.00</td>
<td>44</td>
<td>396</td>
<td>15.29</td>
<td>6055</td>
</tr>
<tr>
<td>3</td>
<td>7.97</td>
<td>40</td>
<td>319</td>
<td>14.53</td>
<td>4635</td>
</tr>
<tr>
<td>4</td>
<td>2.55</td>
<td>40</td>
<td>102</td>
<td>11.82</td>
<td>1206</td>
</tr>
<tr>
<td>5</td>
<td>12.95</td>
<td>40</td>
<td>-518</td>
<td>-2.33</td>
<td>1207</td>
</tr>
<tr>
<td>6</td>
<td>9.00</td>
<td>44</td>
<td>-319</td>
<td>-14.53</td>
<td>4635</td>
</tr>
<tr>
<td>7</td>
<td>9.00</td>
<td>44</td>
<td>-396</td>
<td>-15.29</td>
<td>6055</td>
</tr>
</tbody>
</table>

\[ M_u = \frac{(31352)}{(12)} = 2613 \text{ ft.-kip} \]
APPENDIX 10.3

AISC Local Buckling Provisions Applied at Intermediate Support

Consider the interior beam. The cross section is shown in Figure 10.3.1.

![Figure 10.3.1](image)

The critical width-thickness ratio for the flange is

\[ \frac{b}{2t} = \frac{10.48}{(2)(.760)} = 6.90 \]

and the corresponding allowable values are

- lower bound; \( F_y = 36 \) ksi; \( \frac{b}{2t}_{\text{max}} = 8.5 \)
- upper bound; \( F_y = 40 \) ksi; \( \frac{b}{2t}_{\text{max}} = 8.0 \)

The allowable values are tabulated in the AISC specifications. (2)

For the cover plate, the critical width is the distance between longitudinal welds, which are at the beam flange tips. The actual width thickness ratio for the cover plate is \( \frac{10.48}{.75} = 14.0 \).
The corresponding maximum allowable ratio is \(190/\sqrt{F_y}\). The numerical values are

- **lower bound;** \(F_y = 33\) ksi; \((w/t)_{\text{max}} = 33.1\)
- **upper bound;** \(F_y = 44\) ksi; \((w/t)_{\text{max}} = 28.6\)

The allowable depth-thickness ratio for the beam web is given by the formula

\[
(d/w)_{\text{max}} = (412/\sqrt{F_y})(1-1.4\ P/P_y) \quad \text{if } P/P_y \leq .27
\]

Where \(P\) is the axial load on the cover plated beam section. In the present case, \(P\) is taken as the force in the longitudinal reinforcement. \(P_y\) is the force required to yield the cover plated beam section. Applying the above formula

<table>
<thead>
<tr>
<th>Case</th>
<th>(F_y,\text{ksi})</th>
<th>(P,\text{kip})</th>
<th>(P_y,\text{kip})</th>
<th>((d/w)_{\text{max}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>lower bound</td>
<td>36</td>
<td>297</td>
<td>1740</td>
<td>52.2</td>
</tr>
<tr>
<td>upper bound</td>
<td>40</td>
<td>419</td>
<td>2060</td>
<td>46.8</td>
</tr>
</tbody>
</table>

The actual \(d/w\) ratio is \((28.30)/(.548)\) or 51.6.
APPENDIX 10.4

Slab Ultimate Moments

A set of formulas for computing the ultimate moment of singly reinforced rectangular sections (3, 17) is:

\[ a = \frac{(p f_y d) / (k_1 f'_c)}{M_u = A_s f_y (d - \frac{1}{2}a)} \]

where

- \( a \) = depth of assumed rectangular compression stress block
- \( p \) = reinforcement ratio = \( A_s / (bd) \)
- \( b \) = width of section
- \( f_y \) = yield stress of reinforcement
- \( d \) = depth of section from compression face to centroid of tensile reinforcement

\( k_1 = .85 \) if \( f'_c \) is less than or equal to 4 ksi and decreasing by .05 for each 1 ksi increase in \( f'_c \)

\( f'_c \) = compressive concrete cylinder strength

\( A_s \) = area of tensile reinforcement

\( M_u \) = ultimate moment

Figure 10.4.1 shows a typical section. Only one layer of steel is shown since the compression reinforcement is customarily neglected in yield line analysis. (18) A numerical example will be presented for the ultimate moments in the bridge slab corresponding to the longitudinal layers of reinforcement.

Case 1. Lower Bound

\( f'_c = 4 \) ksi; \( f_y = 50 \) ksi; \( k_1 = .85 \)

Positive Moment (Bottom layer effective)

#5 bars at 10 inches; \( A_s = .37 \text{ in}^2/\text{ft.}; \) \( d = 4.125 \text{ in.} \)
FIGURE 10.4.1 SINGLY REINFORCED RECTANGULAR SECTION

\[ P = \frac{A_s}{bd} = \frac{.00747}{(12)(4.125)} = .00747 \]

\[ a = \frac{p f_y d}{k_1 f'_{c}} = \frac{(.00747)(50)(4.125)}{(.85)(4)} = .453 \text{ in.} \]

\[ M_u = A_s f_y (d - \frac{1}{2} a) = (.37)(50)(3.90) = 72.1 \text{ in.-kip/ft.} \]

\[ M_u = 6.01 \text{ ft.-kip/ft.} \]

**Negative Moment** (Top layer effective)

#5 bars at 9 inches; \( A_s = .41 \text{ in}^2/\text{ft.} \)

#4 bars at 9 inches; \( A_s = .27 \)

\[ A_s = .68 \text{ in}^2/\text{ft.} \]

\[ d = 4.625 \text{ in.} \]

\[ P = \frac{.68}{(12)(4.625)} = .01225 \]

\[ a = \frac{(.01225)(50)(4.625)}{(.85)(4)} = .832 \text{ in.} \]

\[ M_u = 11.9 \text{ ft.-kip/ft.} \]

**Case 2 Upper Bound**

\( f'_c = 5.5 \text{ ksi}; f_y = 70 \text{ ksi}; k_1 = .775 \)

**Positive Moment**

\[ P = .00747 \text{ (same as above)} \]
\[
a = \frac{(0.00747)(70)(4.125)}{(0.775)(5.5)} = 0.505
\]

\[
M_u = (0.37)(70)(3.872) = 100 \text{ in.-kip/ft.}
\]

\[
M_u = 8.50 \text{ ft.-kip/ft.}
\]

**Negative Moment**

\[
p = 0.01225
\]

\[
a = \frac{(0.01225)(70)(4.625)}{(0.775)(5.5)} = 0.931
\]

\[
M_u = (0.68)(70)(4.16) = 198 \text{ in.-kip/ft.}
\]

\[
M_u = 16.5 \text{ ft.-kip/ft.}
\]
APPENDIX 10.5

Strength of Portal Frame Components

Refer to Figure 8 for reinforcing details. The concrete is class B with a nominal compressive strength of 3 ksi. Only the lower bound case is considered here.

To compute the ultimate moment of the cross beam, the compression reinforcement and two #6 bars at middepth will be conservatively neglected.

\[ A_s = 8.89 \text{ in}^2 \quad (7 \#10 \text{ bars}) \]
\[ f_y = 40 \text{ ksi} \quad (\text{Table 2}) \]
\[ d = 27.5 \text{ in.}; \ b = 30 \text{ in.} \]
\[ p = \frac{A_s f_y}{bd} = \frac{8.89}{(30)(27.5)} = .0108 \]
\[ a = \frac{A_s f_y}{.85 f_y} \cdot \frac{8.89}{(.85)(3)(30)} = 4.65 \text{ in.} \]
\[ M_u = A_s f_y (d - \frac{1}{3}a) = (8.89)(40)(25.2) = 8950 \text{ in.-kip} \]
\[ M_u = 746 \text{ ft.-kip} \]

To compute the ultimate moment of the column, the compression reinforcement and bars at middepth will again be conservatively neglected.

\[ A_s = 1.80 \text{ in}^2 \quad (3 \#7 \text{ bars}) \]
\[ f_y = 50 \text{ ksi} \quad (\text{Table 2}) \]
\[ d = 27.5 \text{ in.}; \ b = 30 \text{ in.} \]
\[ p = \frac{1.80}{(30)(27.5)} = .00218 \]
\[ a = \frac{(1.80)(50)}{(.85)(3)(30)} = 1.18 \]
\[ M_u = (1.80)(50)(26.9) = 2420 \text{ in.-kip} = 202 \text{ ft.-kip} \]
The ultimate axial load of a column is computed as follows.\(^{(17)}\)

\[
P_u = 0.85 f'_c A_c + (A_s)_{\text{total}} f_y
\]

\((A_s)_{\text{total}} = 4.80 \text{ in}^2 \quad (8 \#7 \text{ bars})\)

\[
P_u = (0.85)(3)(900) + (4.80)(50) = 2300 + 244 = 2544 \text{ kips}
\]

The axial load corresponding to a balanced failure (simultaneous crushing of concrete and yielding of tension steel) is computed as follows.\(^{(17)}\)

\[
P_b = 0.85 k_1 f'_c b d \frac{0.003}{(f_y/E_s) + 0.003}
\]

\[
= (0.85)(0.85)(3)(30)(27.5) \frac{0.003}{(50/30x10^3) + 0.003}
\]

\[
= 1150 \text{ kips}
\]

The balanced eccentricity is approximately\(^{(17)}\)

\[
e_b = 0.20 t + 0.77(A_s + A_k) f_y
\]

\[
= 0.20t + \frac{0.77(A_s + A_k) f_y}{0.85b f'_c}
\]

where

- \(t = \text{total depth of section}\)
- \(A'_c = \text{area of compression reinforcement}\)

Substituting,

\[
e_b = (0.20)(30) + \frac{(0.77)(3.60)(50)}{(30)(0.85)(3)} = 7.80 \text{ in.}
\]

The balanced moment is

\[
P_b e_b = (1150)(7.80) = 8960 \text{ in.-kip} = 747 \text{ ft.-kip}
\]
APPENDIX 10.6

Bounds on Bearing Strength

Referring to Figure 9, the web areas and yield loads can be readily evaluated. The webs are made of A373 steel with a minimum specified yield stress of 32 ksi.

<table>
<thead>
<tr>
<th>Bent</th>
<th>Web Dimensions</th>
<th>Web Area</th>
<th>Yield Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3x10.5</td>
<td>31.5</td>
<td>1010 kips</td>
</tr>
<tr>
<td>2</td>
<td>3x11.5</td>
<td>34.5</td>
<td>1100</td>
</tr>
<tr>
<td>3</td>
<td>3x11.5</td>
<td>34.5</td>
<td>1100</td>
</tr>
<tr>
<td>4</td>
<td>3x10.5</td>
<td>31.5</td>
<td>1010</td>
</tr>
</tbody>
</table>

The bearing area for a pin is the pin diameter times the length of the web. For pins subject to rotation, an allowable bearing stress is \(0.40 F_y\), where \(F_y\) is the smaller yield stress of the materials in bearing. Here \(F_y\) is 32 ksi so that \(0.40 F_y\) is 12.8 ksi. The corresponding allowable bearing loads are listed below.

<table>
<thead>
<tr>
<th>Bent</th>
<th>Bearing Area</th>
<th>Allowable Bearing Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.0</td>
<td>269 kips</td>
</tr>
<tr>
<td>2</td>
<td>23.0</td>
<td>294</td>
</tr>
<tr>
<td>3</td>
<td>23.0</td>
<td>294</td>
</tr>
<tr>
<td>4</td>
<td>21.0</td>
<td>269</td>
</tr>
</tbody>
</table>

The rockers have a diameter of 15 inches. An allowable load in pounds per inch is given by the formula
\[ p = \frac{F_y - 13,000 \times 600d}{20,000} \]

where

\( F_y \) = smaller yield stress of materials in bearing, psi
\( d \) = diameter of rocker in inches

Substituting,

\[ p = \frac{32,000 - 13,000 \times 600 \times 15}{20,000} = 8550 \text{ lbs./in.} \]

<table>
<thead>
<tr>
<th>B Hartford Length of Rocker</th>
<th>Allowable Rocker Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&amp;4</td>
<td>18.5</td>
</tr>
<tr>
<td>2</td>
<td>19.5</td>
</tr>
</tbody>
</table>
**APPENDIX 10.7**

**Analysis of Modified Bridge Assuming Uniform Elastic Stiffness**

10.7.1 Shakedown Analysis

The shakedown analysis will be presented first. The numbered steps correspond to the outline in section 5.3.

(1) Equations of Equilibrium

Each span is considered separately. The incremental collapse mechanism for each span involves a positive hinge within the span and negative hinge(s) at the intermediate support(s). The incremental collapse mechanisms are shown in Figure 10.7.1.1.

![Figure 10.7.1.1 Incremental Collapse Mechanisms](image)

**FIGURE 10.7.1.1 INCREMENTAL COLLAPSE MECHANISMS**

In Figure 10.7.1.1, $a$ refers to the residual moment, as defined in section 5.2, at each hinge location. By applying the principle of virtual work (19) the following equilibrium equations are obtained.

\[(a-b)m_1 - bm_2 = 0\]  \[-(c-d)m_2 + (c-d)m_3 = 0\]  \[(10.7.1.1)\]

Equations (10.7.1.1) can be rewritten as

\[\frac{(a+b)}{b} m_1 - m_2 = 0\]  \[m_2 + m_3 = 0\]  \[(10.7.1.2)\]
(2) and (3) Compute and Tabulate Elastic Moments

The prismatic beam supports a dead load plus the vehicle (Figure 12) which moves back and forth. Since the bridge is symmetric about the centerline, the maximum and minimum moments need only be tabulated for either half. In anticipation of the trial mechanisms to be investigated in step (4) of the calculations, moments will be tabulated for the possible hinge locations labeled in Figure 10.7.1.2. The values of elastic moments are listed in Table 10.7.1.1. \( P \) is the value of the front axle load. The remaining axle loads are taken as multiples of \( P \) (see section 4.4).

![Symmetrical ABOUT Diagram](attachment:diagram.png)

**FIGURE 10.7.1.2 POSSIBLE HINGE LOCATIONS**

(4) and (5) Select Trial Mechanisms and Formulate Residual Moments

For the approach span, three trial mechanisms will be investigated with positive hinges at \( .3_a, .4_a, \) and \( .5_a \). For the main span, the trial positive hinge locations will be \( .4_m, \) and \( .5_m \). To obtain expressions for residual moments, equations \( (4) \) in section 5.3 are rearranged as follows.
\[
m = M_u^+ - (M_e)_{\text{max}} \quad \text{(positive hinge)}
\]
\[
m = M_u^- - (M_e)_{\text{min}} \quad \text{(negative hinge)}
\]

The results of equations (10.7.1.3) for the trial hinge locations are contained in Table 10.7.1.2.

(6) Substitute Values for \( m \) Into Equilibrium Equations (10.7.1.2)

This step results in a value for the incremental collapse load since \( P \) is the only unknown when expressions for \( m \) are substituted into equations (10.7.1.2). The computations are listed in Table 10.7.1.3. From the table, the approach span is seen to be critical with an incremental collapse load of \( P = 13.8 \) kips (lower bound) to 17.8 kips (upper bound).

(7) Compute Alternating Plasticity Load

The alternating plasticity load is computed by first referring to the column labeled \( (M_e)_{\text{max}} - (M_e)_{\text{min}} \) in Table 10.7.1.1. For points .3a, .4a, and .5a, which are assumed to have the same cross sectional properties, the maximum entry in the column is \( 318P \) for .5a. For the main span, the maximum entry is \( 309P \) for .4m. The alternating plasticity load is then the minimum of the values obtained for points .5a, .4, and .4m. The computations are summarized in Table 10.7.1.4. The point .5a is critical with an alternating plasticity load of \( P = 19.1 \) kips (lower bound) to 24.2 kips (upper bound).

(8) Shakedown Load

The shakedown load is the incremental collapse load computed in step (6).
### Table 10.7.1.1 Elastic Moments

<table>
<thead>
<tr>
<th>Point On Structure</th>
<th>Dead Load Moment</th>
<th>Maximum Live Load Moment</th>
<th>Minimum Live Load Moment</th>
<th>$(M_e)_{\text{max}}$</th>
<th>$(M_e)_{\text{min}}$</th>
<th>$(M_e)<em>{\text{max}} - (M_e)</em>{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>457</td>
<td>229P</td>
<td>- 50.8P</td>
<td>229P+457</td>
<td>- 50.8P+457</td>
<td>280P</td>
</tr>
<tr>
<td>a</td>
<td>467</td>
<td>246P</td>
<td>- 67.8P</td>
<td>246P+467</td>
<td>- 67.8P+467</td>
<td>314P</td>
</tr>
<tr>
<td>a</td>
<td>407</td>
<td>233P</td>
<td>- 84.7P</td>
<td>233P+407</td>
<td>- 84.7P+407</td>
<td>318P</td>
</tr>
<tr>
<td>3.5a</td>
<td>-962</td>
<td>35.7P</td>
<td>-169.5P</td>
<td>35.7P-962</td>
<td>-169.5P-962</td>
<td>205P</td>
</tr>
<tr>
<td>4.5m</td>
<td>478</td>
<td>247P</td>
<td>- 61.7P</td>
<td>247P+478</td>
<td>- 61.7P+478</td>
<td>309P</td>
</tr>
<tr>
<td>5.5m</td>
<td>538</td>
<td>256P</td>
<td>- 45.5P</td>
<td>256P+538</td>
<td>- 45.5P+538</td>
<td>302P</td>
</tr>
<tr>
<td>Point On Structure Type of Hinge</td>
<td>Moment in Ft-k</td>
<td>Moment in Ft-k</td>
<td>Lower Bound (Me)_{max}</td>
<td>Lower Bound (Me)_{min}</td>
<td>Lower Bound (Me)_{max}</td>
<td>Lower Bound (Me)_{min}</td>
</tr>
<tr>
<td>---------------------------------</td>
<td>---------------</td>
<td>---------------</td>
<td>------------------------</td>
<td>------------------------</td>
<td>------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>170P-4264</td>
<td>8448</td>
<td>3558</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
</tr>
<tr>
<td>170P-3314</td>
<td>8448</td>
<td>3558</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
</tr>
<tr>
<td>24EP+3027</td>
<td>4366</td>
<td>794</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
</tr>
<tr>
<td>23EP+3909</td>
<td>4366</td>
<td>794</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
<td>Positive</td>
</tr>
</tbody>
</table>
Table 10.7.1.3 Incremental Collapse Load, $P_{IC}$

All Moments in Ft.-Kips
Double Entries Correspond to Lower Bound (Top Value) and Upper Bound

APPROACH SPAN

<table>
<thead>
<tr>
<th>Positive Hinge At</th>
<th>a</th>
<th>b</th>
<th>$\frac{a+b}{b}$</th>
<th>$m_1$</th>
<th>$(\frac{a+b}{b})m_1$</th>
<th>$m_2$</th>
<th>$(\frac{a+b}{b})m_{1-2}$</th>
<th>$P_{IC}$</th>
<th>Minimum $P_{IC}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.3_a$</td>
<td>0.0463</td>
<td>0.0198</td>
<td>3.34</td>
<td>-229P+3037</td>
<td>-765P+10140</td>
<td>170P-3314</td>
<td>-935P+13450</td>
<td>14.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-229P+3909</td>
<td>-765P+13060</td>
<td>170P-4264</td>
<td>-935P+17320</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td>$0.4_a$</td>
<td>0.0347</td>
<td>0.0231</td>
<td>2.50</td>
<td>-246P+3027</td>
<td>-615P+7570</td>
<td>170P-3314</td>
<td>-785P+10870</td>
<td>13.8</td>
<td>13.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-246P+3899</td>
<td>-615P+9750</td>
<td>170P-4264</td>
<td>-785P+14010</td>
<td>17.8</td>
<td>17.8</td>
</tr>
<tr>
<td>$0.5_a$</td>
<td>0.0278</td>
<td>0.0278</td>
<td>2.00</td>
<td>-233P+3087</td>
<td>-466P+6170</td>
<td>170P-3314</td>
<td>-636P+9480</td>
<td>14.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-233P+3959</td>
<td>-466P+7920</td>
<td>170P-4264</td>
<td>-636P+12180</td>
<td>19.2</td>
<td></td>
</tr>
</tbody>
</table>

MAIN SPAN

<table>
<thead>
<tr>
<th>Positive Hinge At</th>
<th>$m_2$</th>
<th>$m_3$</th>
<th>$-m_2m_3$</th>
<th>$P_{IC}$</th>
<th>Minimum $P_{IC}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.4_m$</td>
<td>170P-3314</td>
<td>-247P+3080</td>
<td>-417P+6394</td>
<td>15.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>170P-4264</td>
<td>-247P+3970</td>
<td>-417P+8234</td>
<td>19.7</td>
<td></td>
</tr>
<tr>
<td>$0.5_m$</td>
<td>170P-3314</td>
<td>-256P+3020</td>
<td>-426P+6334</td>
<td>14.9</td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td>170P-4264</td>
<td>-256P+3910</td>
<td>-426P+8174</td>
<td>19.2</td>
<td>19.2</td>
</tr>
<tr>
<td>Point On Minimum</td>
<td>Structure Moment</td>
<td>Maximum Moment</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------</td>
<td>------------------</td>
<td>----------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25°2 20°0</td>
<td>309F</td>
<td>355F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>55°3 45°8</td>
<td>205F</td>
<td>318P 19.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24.2</td>
<td>24.2</td>
<td>34.94 20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 10.7.1.4 Alternating Plasticity Load, Pa

All Moments in Ft.-Kip

Double Entries Correspond to Lower Bound (Top Value) and Upper Bound

Entries Correspond to Lower Bound (Top Value) and Upper Bound.
10.7.2 First Yield Loads and First Hinge Loads

The loads which cause first yielding in the bottom steel fiber of a flange or cover plate depend upon the residual stresses due to rolling and welding. An estimate of the yield loads can be obtained by neglecting residual stresses and considering the maximum elastic moments (Table 10.7.1) and elastic section moduli (Figure 13). The information is summarized in Table 10.7.2.1.

The load which first causes formation of a hinge at each critical section can be estimated by assuming that the structure responds elastically except at the point where the hinge forms. The computations are summarized in Table 10.7.2.2.

10.7.3 Plastic Collapse Loads

The plastic collapse load for each span can be readily computed by locating the positive hinge at the point of maximum positive elastic moment, as indicated in Table 10.7.1.1. It is noteworthy that these locations correspond closely to the locations obtained for a single concentrated load in section 3.7. The three closely spaced heavy rear axles of the vehicle cause structural response to be much the same as it would be for a single axle load. The computation of plastic collapse loads is summarized in Figure 10.7.3.1 and Tables 10.7.3.1 and 10.7.3.2.
Table 10.7.2.1 First Yield Loads

<table>
<thead>
<tr>
<th>Yield Section</th>
<th>Yield Stress</th>
<th>Modulus of Elasticity</th>
<th>Moment</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange 36</td>
<td>36 ksi</td>
<td>24.76</td>
<td>2476</td>
<td>246 Pt.</td>
</tr>
<tr>
<td>Cover 33</td>
<td>33 ksi</td>
<td>21.84</td>
<td>1184</td>
<td>94 Pt.</td>
</tr>
<tr>
<td>Plate 44</td>
<td>44 ksi</td>
<td>24.48</td>
<td>2448</td>
<td>467 Pt.</td>
</tr>
<tr>
<td>Flange 45</td>
<td>45 ksi</td>
<td>24.92</td>
<td>2492</td>
<td>538 Pt.</td>
</tr>
</tbody>
</table>

*The yield load obtained for point S is not meaningful since substantial inelastic action will occur before this load is reached. The yield load obtained for point S is not valid.*
Table 10.7.2.2 First Hinge Loads

\( P_h \) is the Weight in Kips of the First Axle

*Double Entries Correspond to Lower Bound (Top Value) and Upper Bound*

<table>
<thead>
<tr>
<th>Point On Structure</th>
<th>Maximum Elastic Moment ( \mu_{UM}^+ ) or ( \mu_{UM}^- ) ft.-kip</th>
<th>First Hinge Load, ( P_h ) kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>( .4_a )</td>
<td>( 246P+467 )</td>
<td>3494</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4366</td>
</tr>
<tr>
<td>( S )</td>
<td>( -170P-962 )</td>
<td>-4276</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5226</td>
</tr>
<tr>
<td>( .5_m )</td>
<td>( 256P+538 )</td>
<td>3558</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4448</td>
</tr>
</tbody>
</table>

*The first hinge load obtained for point \( S \) is not meaningful since substantial inelastic action will occur before this load is reached.*
FIGURE 10.7.3.1 PLASTIC COLLAPSE OF MODIFIED BRIDGE
Table 10.7.3.1 Plastic Collapse of Approach Span

<table>
<thead>
<tr>
<th>Axle</th>
<th>Load, kip</th>
<th>Displacement</th>
<th>Work</th>
<th>Moment</th>
<th>Rotation</th>
<th>Work</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.01P</td>
<td>.126</td>
<td>.127P</td>
<td>3494</td>
<td>.0578</td>
<td>202</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.28P</td>
<td>.338</td>
<td>.770P</td>
<td>4366</td>
<td>.0578</td>
<td>252</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2.22P</td>
<td>.466</td>
<td>1.03 P</td>
<td>4276</td>
<td>.0231</td>
<td>98.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5.25P</td>
<td>.906</td>
<td>4.76 P</td>
<td>5226</td>
<td>.0231</td>
<td>121</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>5.33P</td>
<td>.990</td>
<td>5.27 P</td>
<td>5.98P</td>
<td>.838</td>
<td>5.01 P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Dead Load</td>
<td>(.37)(72)</td>
<td>.5</td>
<td>49.3</td>
<td>16.97P</td>
<td>49.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Equate External and Internal Work

Collapse Load = 14.9 Lower Bound

19.1 Upper Bound
Table 10.7.3.2 Plastic Collapse of Main Span

<table>
<thead>
<tr>
<th>Location</th>
<th>External Virtual Work</th>
<th>Internal Virtual Work</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle 1</td>
<td>1.22p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
<tr>
<td>Axle 2</td>
<td>2.26p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
<tr>
<td>Axle 3</td>
<td>3.29p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
<tr>
<td>Axle 4</td>
<td>4.32p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
<tr>
<td>Axle 5</td>
<td>5.36p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
<tr>
<td>Axle 6</td>
<td>6.40p</td>
<td>64.2T</td>
<td>63.64</td>
<td>63.69</td>
</tr>
</tbody>
</table>

Collapse Load: 15.4 kip

Dead Load Load, Kip Displacement Work

Axle 1 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
Axle 2 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
Axle 3 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
Axle 4 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
Axle 5 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
Axle 6 496 9.5 6.9 9.9 2.7 2.4 1.7 1.4 1.1 0.8 0.5 0.2 0.0
\textbf{APPENDIX 10.8}

Yield Line Analysis of Approach Span of Unmodified Bridge

The internal and external virtual work terms are evaluated in Tables 10.8.1 and 10.8.2, respectively. The slab capacities are listed in Table 6. Notation and geometry for the yield line analysis are shown in Figure 16. By equating the total internal work and total external work from the tables, the lower bound collapse load is found to be 1100 kips. This is considerably larger than the lower bound collapse load (493 kips) obtained in section 3.7 for a beam type collapse produced by a single concentrated load. The loads would more nearly coincide if the bridge deck were wider in comparison to the span length.
Table 10.8.1 Internal Virtual Work

<table>
<thead>
<tr>
<th>Comments</th>
<th>Work Lower Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lines CD and EF, representing two interior composite beam hinges, contribute the same amount as they did in the beam analysis in Appendix 10.7.3.</td>
<td>301</td>
</tr>
<tr>
<td>For line AE or BF, the work is simply the length times the slab capacity times the rotation. Taking these lines together, the work is ((2)(72)(9.25)(.300)).</td>
<td>400</td>
</tr>
<tr>
<td>For a yield line not parallel to a direction of reinforcement, the work is (l_xm_xa_x l_ym_ya_y) where (l_x) is the projected length of the yield line along the x-axis, (m_x) is the ultimate slab moment for rotation about the x-axis, and (a_x) is the component of rotation of the yield line about the x-axis. Taking lines AC and BD together, the work is ((2)(28.8)(10.5)(.300)+(2)(3.33)(6.01)(.0347)).</td>
<td>183</td>
</tr>
<tr>
<td>Taking lines CE and DF together, the work is ((2)(43.2)(10.5)(.300)+(2)(3.33)(6.01)(.0231)).</td>
<td>272</td>
</tr>
<tr>
<td></td>
<td>1156</td>
</tr>
</tbody>
</table>
Table 10.8.2 External Work

<table>
<thead>
<tr>
<th>Comments</th>
<th>Work</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied load times virtual deflection</td>
<td>P</td>
</tr>
</tbody>
</table>

Slab area ACE

Slab weight = 0.075 kip/ft²
Triangle area = \( \frac{1}{2} \)(72)(3.33) = 120 ft²
Triangle weight = (0.075)(120) = 9.00 kip
Deflection of centroid = 1/3
Work = \( \frac{1}{3} \)(9.00) = 3.00

Slab area BDF

Slab area ACDB

Area = \( \frac{1}{2} \)(20.0+13.33)(28.8) = 479
Weight = (0.075)(479) = 36.0
Distance of centroid from CD = \( \frac{(28.8)(53.3)}{3(33.3)} \)
Deflection of centroid = \( \frac{(13.5)}{(28.8)} \)
Work = \( \frac{(36)(13.5)}{(28.8)} \) = 16.9

Slab area CEFD

Area = \( \frac{1}{2} \)(13.33+20.0)(43.2) = 719
Weight = (0.075)(719) = 53.9
Distance of centroid from CD = \( \frac{(43.2)(53.3)}{3(33.3)} \)
Deflection of centroid = \( \frac{(20.2)}{(43.2)} \)
Work = \( \frac{(53.9)(20.2)(43.2)}{25.2} \) = 25.2

Two interior stringers

Total weight = (0.2)(72) = 14.4 kips
Average deflection = \( \frac{1}{2} \); Work = 7.2

P 55.3
APPENDIX 10.9

Analysis of Intermediate Support

The elastic analysis of the portal frame can be performed by any of the methods found in texts on structural analysis. The results presented below were obtained using the slope-deflection method.\(^{(24)}\)

Figure 18 shows the notation, loads, and dimensions used in the analysis, and the two limiting cases of column base fixity which were considered. The moments of inertia were based on the gross concrete sections,\(^{(17)}\) which were the same for the beam and columns. The stiffening effect of the brackets at the beam-to-column joints was ignored.

Table 10.9.1 lists critical moment values in terms of the bearing loads \(P_1\) and \(P_2\). \(M_{AB}\) refers to the moment at end A of member AB. The load \(P_2\) is due to dead load alone for the modified bridge and equals about 58 kips. The load \(P_1\) equals \(P_2\) plus (conservatively) half the weight of the fully loaded vehicle. Hence \(P_1\) equals 58 plus 159, or 217 kips. Table 10.9.2 contains numerical values for critical moments based on the above values of the bearing loads. The corresponding axial load in each column is the sum of \(P_1\) and \(P_2\), or 275 kips.

Table 10.9.1 Summary of Maximum Moments

<table>
<thead>
<tr>
<th>Moment</th>
<th>Pinned Column Bases</th>
<th>Fixed Column Bases</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{AB})</td>
<td>0</td>
<td>(.766P_1-.670P_2)</td>
</tr>
<tr>
<td>(M_{BA})</td>
<td>(1.31P_1-1.15P_2)</td>
<td>(1.53P_1-1.34P_2)</td>
</tr>
<tr>
<td>(M_{BC})</td>
<td>(-1.31P_1-1.42P_2)</td>
<td>(-1.53P_1-1.24P_2)</td>
</tr>
<tr>
<td>(M_{EB})</td>
<td>(-2.78P_1+1.42P_2)</td>
<td>(-2.57P_1+1.24P_2)</td>
</tr>
</tbody>
</table>
### Table 10.9.2 Numerical Values for Moments in Table 10.9.1

<table>
<thead>
<tr>
<th>Moment</th>
<th>Pinned Column Bases</th>
<th>Fixed Column Bases</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{AB}$</td>
<td>0</td>
<td>127</td>
</tr>
<tr>
<td>$M_{BA}$</td>
<td>217</td>
<td>254</td>
</tr>
<tr>
<td>$M_{BC}$</td>
<td>201</td>
<td>404</td>
</tr>
<tr>
<td>$M_{EB}$</td>
<td>521</td>
<td>486</td>
</tr>
</tbody>
</table>
11. BIBLIOGRAPHY


(8). Missouri State Highway Department Drawing R-317, Sheets 1A and 2 through 8, Bridge Over Levee Ditch, Butler County, Drawing Approved 3/15/62.


APPENDIX VIII
SUMMARY OF THE MODEL STUDY

VIII.1. INTRODUCTION

The Phase I study included an investigation of the dynamic behavior of a composite steel-and-concrete model. This appendix contains a discussion of the following items related to the model study:
1. A description of the model and its structural properties.
2. The experimental setup which was developed to test the model.
3. The experimental methods and procedures.
4. Conclusions and recommendations concerning the excitation system, instrumentation, and test procedures and methods.

VIII.2. EXPERIMENTAL MODEL

VIII.2.1 Design Procedures and Considerations

The model that was adopted has a 30 foot clear span and consists of two wide-flange steel beams composite with a concrete slab. The end supports, which were made from a standard beam section, were positioned transverse to the longitudinal axis of the model. The concrete slab has a depth of 4 inches and a width of 60 inches. The dimensions of the model are shown in Figs. VIII-1 and VIII-2. The adequacy of the model was determined from a static analysis for moments, stresses, and deflections. In these calculations the indeterminancy of the supports was considered. On the basis of a multidegree of freedom analysis, the approximate fundamental resonant frequency of the system (model plus small actuator) was found to be 8.28 cps.
FIGURE VIII-1. TRANSVERSE VIEW OF MODEL

W12 x 27

4 - \( \frac{3}{8} \)" DIA. BOLTS
EACH END OF
EACH WIDE FLANGE

4"

REINFORCED
CONCRETE SLAB

FLEX-SUPPORTS
S15 x 42.9

1" DIA.
THREADED RODS

LOADING FRAME WALL

3'-8"

LABORATORY JOIST
FLOOR STRUCTURE

7'-0"
5'-0"
2'-11"

VIII 3
FIGURE VIII-2. LONGITUDINAL VIEW OF MODEL
The spacing of the wide flange beams was given consideration in an attempt to minimize the likelihood of the slab developing serious longitudinal cracks. From a transverse section, it can be seen that the slab is continuous over the two beams with a cantilevered portion at each edge. The beams were spaced such that the negative dead load moments in the slab over the beams and the positive dead load moment midway between the beams were equal in magnitude.

Transformed areas were used in all calculations to represent the cross section. In the design calculations, the modulus of elasticity for the steel was 29,000 ksi and the assumed modulus of the concrete was 3,200 ksi. This results in a ratio of $E_s/E_c = 9$. The neutral surface of the composite section was found to be 1.0 inch below the top surface of the top flange, but it should be remembered that this is for the uncracked section. During testing, cracks developed and the neutral surface was continually changing.

To realize composite action, shear connectors were placed on both beams at the required spacing. These shear connectors were designed in accordance with AISC Specifications. Calculations resulted in the placement of shear connectors at intervals of 20 inches. Individual shear connectors consisted of 4 inch long sections of 3 inch channel weighing 4.1 pounds per linear foot. These channels were welded to the top flange of the beams. The placement of the shear connectors is illustrated in Fig. VIII-3.

Longitudinal reinforcing steel accounted for approximately one percent of the transverse cross sectional area of the slab. This is well in excess of the one quarter of one percent required for temperature steel. It was recognized that the dynamic loading would impose
FIGURE VIII-3. SHEAR CONNECTOR DETAILS
tensile stresses in the slab. The extra steel was added as a safeguard to maintain the integrity of the deck. Two layers of #4 bars, one near the top and one near the bottom of the slab, were provided. This is illustrated in Fig. VIII-4.

It was decided that the transverse reinforcement should be greater than the longitudinal reinforcement since it would be required to resist bending and twisting effects. The transverse steel was also placed in two layers. The total reinforcement accounted for approximately four percent of the gross cross sectional area of the slab. This required the placement of #6 bars at 10 inches center to center, top and bottom. This is illustrated in a longitudinal section in Fig. VIII-5. The concrete cover was 3/8 in.

Because of the nature of the tests, the supports at each end of the model would have to withstand upward as well as downward forces. Each support consisted of an American Standard beam securely attached to the model and the floor of the laboratory. The supports provide translational and rotational constraint. A major concern in the design of this component was for the buckling of the web. Analysis found a 15 inch Standard beam with a web thickness of 0.49 inches to be adequate.

VIII.2.2 Fabrication

The model was constructed in the Civil Engineering Laboratory. Supports were placed in position and rigidly connected to the laboratory floor. This was accomplished by bolting through the Standard beam flanges and concrete floor.

After the shear connectors had been welded to the wide-flange
FIGURE VIII-4. REINFORCING STEEL DETAILS, TRANSVERSE SECTION
BARS USED:

76–#6 BARS 4′-10″ LONG
10–#4 BARS 30′-10″ LONG

SYMMETRIC ABOUT

LONGITUDINAL SECTION

FIGURE VIII-5. REINFORCING STEEL DETAILS, LONGITUDINAL SECTION
beams, the bottom flanges of these beams were bolted to the supports. The forming for the concrete slab was supported entirely by the wide-flange beams. This type of construction results in a structure with bending stresses and shear stresses in the steel beams but no dead load stresses in the concrete. All steel is A36 except rebars, which are intermediate grade. A 3.6 ksi compressive strength concrete mix was specified. Required curing was provided for both the slab and the 24 test cylinders. Ultimate compression tests and splitting tests were performed on the test cylinders. The results showed the ultimate compressive stress to be 4.29 ksi and the splitting stress to be 0.396 ksi. Also, the modulus of elasticity was found to be 3,570 ksi; whereas, the assumed value had been 3,200 ksi.

VIII.2.3 Structural and Material Properties

Table VIII-1 lists the structural and material properties associated with the model.

The composite and semicomposite values for the moments of inertia, stiffnesses, and natural frequencies shown in the table establish an upper and lower bound for the structural properties of the model. The upper bound, which is the composite case, represents full composite action by the concrete, reinforcing bars, and wide-flange beams. The lower bound, which is the semicomposite case, represents action only by the reinforcing bars and wide-flange beams in resisting deformation of the model.

The stiffness given in Table VIII-1 represents the equivalent one-degree-of-freedom stiffness of the model and is the ratio of the load to displacement at midspan. The natural frequencies are those of the
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity, Concrete, $E_c$</td>
<td>$3,570$ ksi</td>
</tr>
<tr>
<td>Assumed Modulus of Elasticity, Steel, $E_s$</td>
<td>$29,000$ ksi</td>
</tr>
<tr>
<td>Modular Ratio, $E_s/E_c$</td>
<td>$8.15$</td>
</tr>
<tr>
<td>Moment of Inertia, Composite</td>
<td>$1103$ in$^4$</td>
</tr>
<tr>
<td>Moment of Inertia, Semicomposite</td>
<td>$523$ in$^4$</td>
</tr>
<tr>
<td>Unit Weight of Concrete</td>
<td>$135$ pcf</td>
</tr>
<tr>
<td>Total Weight of Model</td>
<td>$8.9$ kips</td>
</tr>
<tr>
<td>Stiffness, Composite</td>
<td>$33.4$ kips/in.</td>
</tr>
<tr>
<td>Stiffness, Semicomposite</td>
<td>$16.1$ kips/in.</td>
</tr>
<tr>
<td>Calculated Natural Frequency, Composite</td>
<td>$8.57$ cps</td>
</tr>
<tr>
<td>Calculated Natural Frequency, Semicomposite</td>
<td>$5.95$ cps</td>
</tr>
<tr>
<td>Neutral Axis (Below Bottom of Slab)</td>
<td></td>
</tr>
<tr>
<td>Composite</td>
<td>$0.80$ in.</td>
</tr>
<tr>
<td>Semicomposite</td>
<td>$5.08$ in.</td>
</tr>
</tbody>
</table>

**Table VIII-I. Model Properties**
model only and they were determined by using an equivalent single-degree-of-freedom system.

**VIII.3  VIBRATION EXCITATION SYSTEM I**

Fifteen separate tests were performed on the experimental model, and these tests were categorized for presentation here into three different sets corresponding to the nature of the excitation system employed for a particular set of tests. All of these excitation systems made use of an MTS closed-loop electro-hydraulic structural vibration system, and the vibrators were constructed by attaching weights to servo-controlled hydraulic actuators. The MTS closed-loop system provided complete control of frequency and stroke of the hydraulic actuators which is required for this type of testing.

**VIII.3.1 Actuator Location and Supports**

For the first nine tests, (Setup I), an MTS 205.31, 5 kip hydraulic actuator was bolted to the center of the midspan of the experimental model as shown in Figs. VIII-6 and VIII-7. The base of the hydraulic actuator was attached to the experimental model by the use of two steel plates, one located on top of the concrete deck and one below. The top plate was set in hydra-stone to ensure its levelness. The two plates were bolted together with 1" diameter bolts which passed through 4" x 1-1/2" diameter pipes embedded in the concrete deck as illustrated in Fig. VIII-7. The 2-inch angle bracing the pipes were not attached to the flange of the girder; the actuator was similarly connected to the deck.

**VIII.3.2 Moving Mass**

The weight of the "moving mass" for all of the small ram tests (Tests 1-9 and Tests 13-15) included 38 pounds for the piston and rod of the actuator plus 40 pounds for the attached steel plate, for a
FIGURE VIII-6. FIVE KIP ACTUATOR LOADING SYSTEM
FIGURE VIII-7. SUPPORT ARRANGEMENT FOR 5 KIP ACTUATOR
VIII.3.3 Hydraulic and Electronic Control System

The MTS Systems Corporation Vibration System, shown in Fig. VIII-9, consisted of a hydraulic section and an electronic control section. The hydraulic section consisted of a 35 gpm, 3000 psi hydraulic power supply, a 60 gpm hydraulic control manifold, a 90 gpm high performance servo-valve and a 5 kip hydraulic actuator.

The electronic section was basically a closed-loop (feedback) electronic control system. The main part of the control system was an MTS 401 servo-controller which controlled the MTS 251.32 servo-valve and the flow of hydraulic fluid into the actuator. The servo-controller was capable of independent control of the static and dynamic positions of the actuator rod. The feedback signal was from an LVDT located in the base of the actuator as illustrated in Fig. VIII-9. The LVDT signal was conditioned with a MTS 425.31 A-C transducer conditioner before it reached the servo-controller.

The command signal for the control system was from a Spectral Dynamics SD104-5 sweep oscillator, and it was used to establish the driving frequency of the actuator. The oscillator was capable of automatic frequency sweep between two fixed frequency limits.

The stroke of the hydraulic actuator, in terms of the LVDT signal, was the only control parameter that was permissible with this setup. Thus, dynamic tests listed under Setup I had to be controlled with stroke. If the desired control parameter was acceleration or strain, manual adjustment of the servo-controller was required to maintain the desired level. The acceleration or strain was observed on an oscilloscope.
FIGURE VIII-8. "MOVING MASS" FOR 5 KIP ACTUATOR
FIGURE VIII-9. SCHEMATIC FOR VIBRATION EXCITATION SYSTEM NO. 1

DECK OF MODEL

MTS 203.3I 5 KIP ACTUATOR

MTS 284 MANIFOLD

MTS 251.32 90gpm SERVO-VALVE

MTS 251.32 90gpm SERVO-VALVE

MTS 284 HOST PRESSURE SUPPLY

MTS 303.0A HYDRAULIC POWER SUPPLY

PUMP

7HP ELECTRIC MOTOR

MTS 303.0A MAIN CONTROL LOOP

SERVO CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

MTS 413 CONTROL PANEL

MTS 425 A-C TRANSDUCER

MTS 425 FREQUENCY/AMPLITUDE COUNTER

MTS 425 FREQUENCY/AMPLITUDE COUNTER

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL

MTS 413 CONTROL PANEL

SERVO-CONTROLLER CONTROL SIGNAL

MTS 422 SERVO-CONTROLLER

SERVO-CONTROLLER CONTROL SIGNAL
VIII.4 VIBRATION EXCITATION SYSTEM II

VIII.4.1 Actuator Supports

The setup of Tests 13-15 (Setup II) again used the MTS 205.31, 5 kip actuator, with a slightly different base attachment than Setup I. Because of deterioration of the deck about the base of the actuator, two C6x10.5 channels were attached between the wide-flange beams at the midspan to strengthen the concrete deck, as shown in Fig. VIII-10. Otherwise, the supports were the same as described in Art. VIII.3.1.

VIII.4.2 Hydraulic and Electronic Control System

The hydraulic and electronic control systems used during Setup II were the same as the systems used in Setup I except that additional electronic control equipment was added to the vibration system. The additional control equipment was used to establish an automatic closed-loop control of the relative acceleration. Setup II contained main and secondary control loops in which the secondary control loop was composed of the MTS 401 servo-controller and an LVDT.

The main control loop shown in Fig VIII-11 consisted of a Spectral Dynamics SD105 servo/monitor whose function was to give a controlled command signal to the MTS 401 servo-controller. The command signal to the SD105 was from a Spectral Dynamics SD104-5 sweep oscillator. The feedback signal was from an accelerometer mounted at the base of the hydraulic actuator (called the reference accelerometer) and from an accelerometer mounted atop the "moving mass" (called the control accelerometer, see Art. VIII.6.6). The two signals from the accelerometers were conditioned (see Art. VIII.7.2)
FIGURE VIII-10. SECOND SUPPORT ARRANGEMENT FOR 5 KIP ACTUATOR
FIGURE VIII-11. SCHEMATIC FOR VIBRATION EXCITATION SYSTEM NO. II
and then sent through a summing junction whose output signal was the relative acceleration. This signal was filtered by a Spectral Dynamics SD101B tracking filter, and the filtered signal was passed on to the SD105 servo/monitor completing the closed-loop control system.

This control technique proved to be very effective in maintaining a constant exciting force on the experimental model and was judged to be the best control system used during the test program.

VIII.5 VIBRATION EXCITATION SYSTEM III

VIII.5.1 Actuator Location and Supports

The third experimental setup, (Setup III) shown in Fig. VIII-12, for Tests 10-12, was employed for the purpose of disclosing problems which might have arisen in coordinating the motion of two hydraulic actuators. Two MTS 202.01, 80 kip hydraulic actuators were bolted to the concrete deck over the two wide-flange beams. The two plates were set in hydra­stone to ensure their levelness and then were bolted through the concrete to the two girders as shown in Fig. VIII-13.

VIII.5.2 Moving Mass

The weight of the "moving mass" for all of the tests using the 80 kip actuators included 70 pounds for the piston and rod of the actuator plus 1280 pounds of attached weight for a total weight of 1350 pounds. The attached weight is illustrated in Fig. VIII-14.

To achieve the exciting force levels needed for Tests 10-12 the large attached weights were needed because of the relatively small amplitude response that could be obtained from the 80 kip actuators.

VIII.5.3 Hydraulic and Electronic Control Systems

The hydraulic and electronic control system used for each hydraulic actuator in Setup III, see Fig. VIII-15, was almost identical
FIGURE VIII-12. EIGHTY KIP ACTUATOR LOADING SYSTEM
FIGURE VIII-13. SUPPORT ARRANGEMENT FOR 80 KIP ACTUATOR
FIGURE VIII-14. "MOVING MASS" FOR 80 KIP ACTUATOR
FIGURE VIII-15. SCHEMATIC FOR VIBRATION EXCITATION SYSTEM NO. III
to the system of Setup I except the hydraulic power supply was shared by the two actuators. Also, an MTS 410 function generator was used for the command signal instead of the Spectral Dynamics SD104 sweep oscillator.

VIII.6 LOCATION OF TRANSDUCERS

VIII.6.1 Introduction

Five different types of electronic transducers were utilized throughout all of the fifteen experimental tests. They included a linear variable differential transformer (LVDT), steel strain gages, concrete strain gages, deflection gages, and piezoresistive accelerometers. Four of the five transducers used the basic Wheatstone bridge as the signal sensing element.

The following four sections describe the type, function, and location of each of the electronic transducers. Figures VIII-16 and VIII-17 show the positions of all the main transducers used throughout the experimental program. Transducers were added and deleted during the series of tests, as more insight into the experimental program was gained. Additional information on the transducers and their calibration data is contained in Reference (2).

VIII.6.2 Linear Variable Differential Transformer

Each hydraulic actuator had a LVDT located in its base which sensed not only the static position of the actuator rod but also the dynamic displacement. The two 80 kip actuators had Collins Model LMT-689VOL LVDT's with Serial Nos. 82787 and 82788. The 5 kip actuator had a Collins LVDT. A description of the function of the LVDT in the feedback loop of the control system is contained in Art. VIII.3.3.
<table>
<thead>
<tr>
<th>DISTANCE FROM NORTH END</th>
<th>POINT</th>
<th>FT.</th>
<th>IN.</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 - 9</td>
<td></td>
<td>1 8</td>
<td></td>
</tr>
<tr>
<td>7 - 6</td>
<td></td>
<td>1 4</td>
<td></td>
</tr>
<tr>
<td>11 - 3</td>
<td></td>
<td>3 8</td>
<td></td>
</tr>
<tr>
<td>15 - 0</td>
<td></td>
<td>1 2</td>
<td></td>
</tr>
<tr>
<td>18 - 9</td>
<td></td>
<td>5 8</td>
<td></td>
</tr>
<tr>
<td>22 - 6</td>
<td></td>
<td>3 4</td>
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<tr>
<td>26 - 3</td>
<td></td>
<td>7 8</td>
<td></td>
</tr>
<tr>
<td>30 - 0</td>
<td></td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

**FIGURE VIII-16. TRANSDUCER LOCATION DIAGRAM**
FIGURE VIII-17. TRANSDUCER LAYOUT
VIII.6.3 Steel Strain Gages

Three Micro-Measurements EA-06-500BH 120 ohm, temperature-compensated, steel strain gages were glued to strategic locations on the girders of the experimental model as illustrated in Figs. VIII-16 and VIII-17. Two of the gages were placed so that the maximum bending strains of the wide-flange beams could be monitored and recorded. They were glued, using Eastman 910, lengthwise beneath the bottom flange at the midpoint of each beam.

After the first test, a third steel strain gage was added to the underside of the top flange at the midpoint of the west wide-flange beam. This enabled the determination of the neutral surface by recording the strain due to bending at some point, other than the outermost fiber of the wide-flange beam.

Four Micro-Measurements EA-06-250BG 120 ohm, temperature-compensated, steel strain gages were attached to the outside and inside surfaces of the web in the middle of the north flex-support as shown in Fig. VIII-18. These gages were used to determine the magnitude of the reactions in the flex-support.

VIII.6.4 Concrete Strain Gages

Initially three specially designed\(^{(3)}\) concrete strain gages were used to record the strains on the top surface of the concrete deck. The concrete strain gages, as shown in Fig. VIII-19, each used a four active arm Wheatstone bridge as the sensing element. This permitted use of conventional strain gage instrumentation. The effective gage length of this gage was six inches. Because of the nature of the design of this transducer, it cannot be loaded in compression. Thus, in use, the
FIGURE VIII-18. STRAIN GAGE LOCATIONS ON FLEX-SUPPORT
FIGURE VIII-19. CONCRETE STRAIN GAGE

LEGEND

A  STRAIN GAGES
B  PRETENSIONING SCREW
C  CLIP
D  FLEXURE ELEMENTS
E  CENTER HOLES
concrete strain gage must be pretensioned by means of a pretensioning hinge and screw.

Two concrete strain gages (Serial Nos. 6036 and 6035) were set longitudinally over each wide-flange beam at the midspan, as illustrated in Fig. VIII-16. The third gage (Serial No. 6040) was set transverse at the approximate midspan (see Fig. VIII-16) to record the strain due to Poisson's effect on the concrete deck.

After the first experimental test another steel strain gage was placed on the model and because of a shortage of instrumentation channels one of the concrete strain gages (Serial No. 6040) had to be deleted. To compensate for the loss of the concrete strain gage, the gage with Serial No. 6036 was moved to the transverse gage location and the gage with Serial No. 6035 was moved to a longitudinal position at the approximate center of the midspan, as illustrated in Fig. VIII-16, for the series of tests two through twelve.

VIII.6.5 Deflection Gages

Two deflection gages were designed\(^{(2)}\) to record the dynamic deflections at the midspan location of the experimental model. These two gages were bolted to the underside of the outer top flange of the girder, as illustrated in Figs. VIII-16 and VIII-17.

As illustrated in Fig. VIII-20, the deflection gage was prestressed by deflecting the free end of the gage downward two inches from its straight position and then securing the free end of the gage at this position with a tie-down device. The electronic sensing element was a Wheatstone bridge composed of four Micro-Measurements EA-060250BG
FIGURE VIII-20. DEFLECTION GAGE ATTACHMENT
120 ohm strain gages arranged in such a manner that only bending strains could be detected as the wiring diagram shows in Fig. VIII-20.

VIII.6.6 Piezoresistive Accelerometers

Four Endevco piezoresistive accelerometers were used during the fifteen experimental tests. Two of the accelerometers, Model No. 2262, had a maximum range of ± 25 g's and the other accelerometers, Model No. 2260, had a maximum range of ± 250 g's. One of the two 25-g accelerometers, Serial No. WH24, was attached at the approximate midspan location of the experimental model during all fifteen tests, as shown in Fig. VIII-16. This particular accelerometer (the reference accelerometer) recorded the accelerations of the concrete deck at midspan.

A 250-g accelerometer (control accelerometer) was attached to the "moving mass" on the actuators, regardless of which vibration excitation system was used. Its function was to record the absolute acceleration sensed atop the "moving mass". However, the control function of this accelerometer varied with each set of tests and those control functions will be discussed in detail in Arts. VIII.8.2 and VIII.8.4. For the first test, an Endevco Model 2260 accelerometer, Serial No. A741, was used for the control accelerometer. However, after the first test it was discovered that the sensitivity of the accelerometer as supplied by the manufacturer was incorrect. This accelerometer was then replaced with another Model 2260 accelerometer, Serial No. A730 for the remainder of the tests.

The 25-g accelerometers were attached to 1/2" x 1/2" diameter plexiglas rods glued to the concrete deck with epoxy cement. A detailed drawing of the accelerometer "pad" is shown in Fig. VIII-21.
FIGURE VIII-21. ACCELEROMETER ATTACHMENT
Excluding the accelerometer pads at the midspan, eighteen similar pads were attached to specific locations along the model deck. The locations of all accelerometer pads are shown in Fig. VIII-16. Numbering from the north end of the model to the south end, three pads forming a transverse line across the model's deck were named Station 1, Station 2, ..., Station 7 indicating at what fraction of the total length of the experimental model a particular acceleration was being recorded. For example, Station 3 referred to the three pads--one over the west beam, one in the center, and one over the east beam--at the three-eighths point along the length of the experimental model. Thus Station 3 corresponded to the position 11.25 feet from the north end of the model. It was at Station 3 Center that the other 25-g accelerometer, Serial No. WE53, was attached for a majority of the tests. This Station 3 accelerometer was also used for measuring accelerations at the other points, by temporarily attaching it to other stations.

VIII.7 ELECTRONIC DATA ACQUISITION SYSTEMS

VIII.7.1 Introduction

This section presents the data acquisition systems and the monitoring and recording system which were developed for the experimental study. These systems were continuously improved during the series of fifteen tests. Except for minor problems the data acquisition, monitoring and recording system functioned quite satisfactorily.

VIII.7.2 Data Acquisition System Schematics

The schematic diagram for the data acquisition system used for Tests 1-9 (Setup I) and Tests 10-12 (Setup III) is shown in Fig. VIII-22.
FIGURE VIII-22. DATA ACQUISITION SYSTEM SCHEMATIC FOR TESTS 1-12
The dotted lines represent the additional LVDT feedback of the second MTS 202.01, 80 kip actuator used for Tests 10-12 (Setup III). Fig. VIII-23 shows the instrumentation setup for Tests 1-9. For Setup III, the concrete gages were deleted because of severe cracks in the concrete deck under the gages.

Figure VIII-24 illustrates the schematic diagram for the data acquisition system used for Tests 13-15 (Setup II). An important addition to Setup II was the summing junction whose function was discussed in Art. VIII.4.2.

VIII.7.3 Signal Conditioning Equipment

Two 5-channel transducer conditioner units, three 6-channel amplifiers, one low-pass filter, and a tracking filter were used during the fifteen experimental tests.

The five Honeywell 105 transducer conditioners were used specifically with transducers which had one or four active arm (strain gage) Wheatstone bridges as shown in Figs. VIII-22 and VIII-24. The Honeywell 105 system functioned as the power supply, bridge balance, and calibration unit for the transducers interfaced with it.

Two of the five channels of the Endevco 4470 universal transducer conditioner, with Model 4471.1 plug-in mode cards, were used with strain gages or concrete gages; while the remaining three channels, with Model 4476.1 plug-in mode cards, were used with piezoresistive accelerometers. The basic Endevco 4470 unit functioned as a power supply, bridge balance, and calibration unit. The 4471.1 plug-in mode card was used only for completing Wheatstone bridge circuits and when used with the Endevco 4470 unit caused the combined unit to behave as a passive transducer.
FIGURE VIII-24. DATA ACQUISITION SYSTEM SCHEMATIC FOR TESTS 13-15
conditioner like the Honeywell 105's. The 4476.1 plug-in mode card was designed specifically for use with piezoresistive accelerometers, and it served as single-ended D-C amplifier which could be adjusted to an accelerometer so that the maximum output voltage of ± 2.5 volts was a fixed percentage of the maximum range of the accelerometer.

The Honeywell 120 six-channel amplifier system shown in Figs. VIII-22 and VIII-24 was composed of differential D-C amplifiers, with fixed gain steps between 10 and 1000. The Honeywell 117 six-channel amplifier system was composed of single-ended D-C amplifiers with fixed gain steps between .01 and 10. The Honeywell 117 amplifiers were used as the power amplifier stage for the Endevco 4470-4476.1 D-C amplifiers for the purpose of conditioning the accelerometer signals for recording on an oscillograph. Finally, the Honeywell A-20 six-channel amplifier system contained two differential D-C amplifiers and four single ended D-C amplifiers both with fixed gain steps between .01 and 1000.

The General Radio Model 952 universal filter shown in Figs. VIII-22 and VIII-24 is a low-pass, high-pass, band-pass or band-reject single channel filter with adjustable cut-off frequencies between 4 cps and 60,000 cps. For all of the experimental tests, except Test 9 which was a third mode test, the General Radio filter was used as a low-pass filter with a cut-off frequency of approximately 10 cps. For Test 9, the General Radio filter was used as a band-pass filter with cut-off frequencies of 30 cps and 70 cps.

The Spectral Dynamics SD 1018 tracking filter shown in Fig. VIII-24 was used as 2 cps band-pass filter which was tuned or tracked with the Spectral Dynamics SD104 sweep oscillator described in Art. VIII.3.3.
VIII.7.4 Data Recording and Monitoring Equipment

All data was recorded on two 12-channel Honeywell 1508 Visicorder oscillographs. The printouts of these oscillographs resulted in tracings of time versus the amplitude response of the individual transducers.

Three Tektronic oscilloscopes were used during the fifteen tests for monitoring certain transducer signals. The three oscilloscopes were a Tektronic 502A dual-beam oscilloscope, a Tektronic 565 dual-beam oscilloscope with two Tektronic 3A3 differential dual-trace D-C amplifiers and a Tektronic 564 storage oscilloscope with a Tektronic 3Ae amplifier and a 2B67 time base. The function of the three oscilloscopes is illustrated in Fig. VIII-25. The 565 oscilloscope was used to monitor one of the strain gages located on the bottom flange of a girder and the control accelerometer.

For the first several model tests, the method used to determine the resonant frequency was to make use of the phase relationship between the LVDT signal and control accelerometer signal with the aid of the 564 storage oscilloscope as shown in Fig. VIII-25. For the remaining tests, Lissajous figures, using the LVDT signal and a deflection gage signal were employed to determine resonance as shown in Fig. VIII-25. The 502A oscilloscope was used for displaying the Lissajous figures.

VIII.8 EXPERIMENTAL METHODS AND PROCEDURES

VIII.8.1 Model Tests

A series of fifteen tests were performed on the experimental model. These model tests served two distinct purposes:

1. The verification and evaluation of the hydraulic actuator-"moving mass" combination as a satisfactory dynamic loading technique.
FIGURE VIII-25. MONITORING SYSTEM
2. The determination of the dynamic properties of the experimental model at different strain levels and states of deterioration due to cumulative damage from the cyclic loading.

The series of Tests 1-5, 7-8, 10-11, and 13-15 were conducted at or near the first fundamental bending natural frequency of the experimental model and they included the following set of tests: a resonant-dwell test, a log-decay test and a sweep-sine test. Each of the numbered tests was conducted at a preselected strain level, at resonance of the experimental model. Three preselected strain levels of approximately 100, 300, and 500 $\mu$in/in were used for the series of Tests 1-5, 7-8, 10-11 and 13-15. One of the strain gages on the bottom flange was used to monitor the preselected strain level. The set of dynamic tests listed above was performed approximately four times at each of the three preselected strain levels to assess the effect of damage, due to the cyclic loading, on the dynamic properties of the experimental model. Table VIII-2 gives a tabulation of the preselected strain levels and the dynamic tests conducted for each of the numbered tests.

Test 9, which was conducted at the third mode bending frequency of the model, consisted only of a resonant-dwell test because of a failure in the servo-valve during the test. Because of extensive damage to the concrete deck during the resonant-dwell test, it was decided that a log-decay test and a sweep-sine response test should not be conducted for Test 9 after the servo-valve was repaired. Test 12, which was conducted at the first torsional mode natural frequency of the model, consisted only of a resonant-dwell test and a log-decay
<table>
<thead>
<tr>
<th>TEST NUMBER</th>
<th>TEST TYPE</th>
<th>SELECTED STRAIN LEVEL µ IN/IN.</th>
<th>RESONANT DWELL</th>
<th>LOG DECAY TEST</th>
<th>SWEEP SINE TEST</th>
<th>PROFILE TEST</th>
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<tr>
<td>1</td>
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</tr>
<tr>
<td>4</td>
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<td>X</td>
</tr>
<tr>
<td>6</td>
<td>1st MODE</td>
<td>N.A.</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>7</td>
<td>1st MODE</td>
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<td>X</td>
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<td></td>
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<td>X</td>
<td>X</td>
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</tr>
<tr>
<td>11</td>
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<td>X</td>
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<td>X</td>
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<td></td>
<td>X</td>
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<td>1st MODE</td>
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<td>X</td>
</tr>
</tbody>
</table>

X-Denotes Test Conducted

TABLE VIII-2. LOG OF EXPERIMENTAL TESTS
test. Finally, a profile test was conducted for the purpose of determining the mode shape associated with the fundamental natural bending frequency in Test 6 and Tests 13-15.

The series of fifteen tests also served as a proving ground for the evaluation of the three vibration systems and the data acquisition, monitoring and recording system described in Art. VIII.7.

VIII.8.2 Initial Setup Procedures for Tests 1-15

VIII.8.2.1 Introduction

The resonant-dwell test was used in all fifteen tests to set the preselected strain levels mentioned in the last section. The resonant-dwell test was also used as an experimental method to determine dynamic properties of the system such as stiffness, natural frequency, and damping (2). The procedures using the resonant-dwell test to initially set the preselected steel strain levels differed slightly with each of the vibration excitation systems, and they will be discussed separately in the following sub-articles. However, the monitoring system for observing strain on the bottom flange remained the same for all fifteen tests.

VIII.8.2.2 Detection of Resonance

Two experimental techniques were employed for the detection of resonance. For Tests 1 and 2, the experimental technique used to determine the resonant frequency was to make use of the phase relationship between the exciting force and the response of the model. As stated earlier, the response of the model lagged the exciting force by 90° at resonance. However, for the purposes of phase detection, the stroke of the actuator, which was measured by the LVDT, was used in place of
the exciting force since both quantities were always in phase. The Tektronix 564 storage oscilloscope was used to visually detect the phase between the LVDT and deflection gage as illustrated in Fig. VIII-25. Because of the difficulty encountered with visually detecting the 90° phase shift on the storage oscilloscope, a Lissajous diagram was employed to determine the resonant frequency for the remaining tests. The Lissajous diagram was displayed on the Tektronix 502A oscilloscope by connecting the LVDT signal to the vertical plate and a deflection gage signal to the horizontal plate of the oscilloscope as shown in Fig. VIII-25. At resonance, the Lissajous diagram in general corresponded to an ellipse with vertical and horizontal axes.

VIII.8.2.3 Setup Procedures for Tests 1-12

The procedures used to setup this series of tests using the resonant-dwell technique were as follows:

1. Adjust the output frequency (driving frequency) of the Spectral Dynamics SD 104 sweep oscillator until resonance of the experimental model is detected by observing a 90° phase difference between the LVDT signal and a deflection gage signal.

2. Adjust stroke of actuator(s) to obtain the preselected strain level at the resonant condition. Monitor the strain gage and record the final value of the strain.

3. Monitor and record the period of the resonant frequency using the General Radio Model 1191 period counter.

4. Monitor and record the value of the acceleration given by the control accelerometer.

5. Record the response of all transducers on the oscillographs.
Because of rapid loss in the stiffness of the experimental model due to deterioration of the concrete deck under the cyclic loading, it was difficult in some tests to maintain steps one and two for a long enough period of time to execute steps three, four and five. Sometimes several attempts were required to completely carry out the procedures listed above.

**VIII.8.2.4 Setup Procedures for Tests 13-15**

Since a more refined control system was used during Tests 13-15, a different set of procedures had to be adopted and they were as follows:

1. Set the control acceleration level on the Spectral Dynamics SD105 servo/monitor at a preselected value.

2. Adjust output frequency of the Spectral Dynamics SD104 sweep oscillator until resonance of the experimental model is detected by use of the Lissajous diagram.

3. Monitor strain gage for the strain level. If the strain level is not at the preselected value of resonance, repeat steps one and two until the preselected strain level at resonance is obtained.

4. Monitor and record the period of the resonant frequency of the model using the period counter.

5. Record the magnitude of the control acceleration.

6. Record the response of all transducers on the oscillographs.

**VIII.8.3 Log Decay Test**

One method chosen to determine the damping properties of the experimental model was the log decrement method. The log decrement method was chosen because it is one of the most commonly used techniques to measure damping in structures. Log decay tests were performed during
all of the experimental tests except for Tests 6 and 9. The experimental procedure was as follows:

1. Take zero readings of all transducers on the oscillographs with the model in its static position.
2. Determine resonance for a preselected strain level as outlined in Art. VIII.8.2.
3. Record the magnitude of the strain.
4. Record the period of the resonant frequency.
5. Record the magnitude of the control acceleration.
6. Shut off the hydraulics to the actuator, leaving all recording instrumentation running until the response of the model decays to zero.
7. Again take zero readings of all transducers.

VIII.8.4 Sweep-Sine Test

VIII.8.4.1 Introduction

The sweep-sine test was originally conducted for the purpose of determining the damping properties of the experimental model by the bandwidth and peak amplitude methods. However, early in the testing program it was determined that because of the over-hang characteristic of the nonlinear response of the model that the bandwidth method could not be used to find damping ratios. The half-power point was simply not definable. Thus, the sweep-sine test was used to provide experimental data for the peak amplitude method only.

The sweep-sine test was conducted using two different control accelerations during the experimental study. For Tests 1-5, 7-8, 10-11 and 13-15, the sweep-sine test was performed while maintaining the absolute acceleration of the "moving mass" as a constant.
This acceleration was sensed by the control accelerometer located on top of the "moving mass" as illustrated in Figs. VIII-7 and VIII-13. The sweep-sine test was carried out while maintaining the relative acceleration of the moving mass as a constant, for Tests 13-15.

Control on the relative acceleration during the sweep-sine test proved to be the better experimental technique. This was due in part to the fact that the absolute acceleration recorded by the control accelerometer atop the "moving mass" was dependent on the motion of the experimental model as well as the motion of the actuator rod, whereas the relative acceleration was dependent only on the stroke of the actuator rod which was an independent controllable parameter.

The experimental data recorded during a sweep-sine test for Tests 10 and 11 were not useable because the model acceleration was the dominant part of the absolute acceleration measured by the control accelerometer. For this reason resonance of the experimental model could not be detected.

VIII.8.4.2 Sweep-Sine Test Procedures

The procedures followed during the sweep-sine test, when control of the experiment was with absolute acceleration, were as follows for a numbered test:

1. Reestablish control acceleration level recorded during the initial setup procedure at a preselected frequency below the resonant frequency. Record stroke of actuator. Record the LVDT range setting on the MTS 425 A-C transducer conditioner.
2. Repeat step one for a frequency above the resonant frequency.
3. Set range setting on the MTS 425 unit at highest stroke range recorded in steps one and two.
4. Reestablish control acceleration level at a preselected frequency below the resonant frequency. Record accelerations at stations three and four center, see Fig. VIII-16, on an oscillograph. Record period of frequency manually on oscillograph chart.

5. Increment frequency upward and maintain the control level of the acceleration. Record data listed in step four.

6. Repeat step five at least twenty times until frequency of model is above the resonant frequency at a preselected level.

7. Increment frequency downward by essentially repeating steps six and seven. (This step normally was conducted only for high amplitude response of the model at resonance.)

The procedures followed during the sweep-sine test, when control of the experiment was with relative acceleration, were as follows:

1. Maintain control acceleration level on the Spectral Dynamics SD105 servo/monitor as determined during step 3 of the initial setup procedures outlined in Art. VIII.8.2.4. Set frequency of the Spectral Dynamics SD104 sweep oscillator at a preselected value below the resonant frequency. Record the stroke of actuator and the LVDT range setting on the MTS 425 unit.

2. Repeat steps two through seven listed in the above procedures contained in this section. It is not necessary to adjust control acceleration since this is done automatically by the SD105 unit.

VIII.8.5 Profile Test

The profile test was used to obtain experimental data for
determining the damping ratio of the model by the normal mode method
which is discussed in Reference 4. The profile test, which was con­
ducted during Tests 6, 13, 14 and 15, is basically a procedure where
the experimental model was excited at resonance and the response of
the model (in terms of acceleration) was measured at enough points on
the concrete deck to ensure accurate representation of the mode shape.

The experimental procedures for the profile test were as follows:

1. Use the resonant-dwell test, as outlined in Art. VIII.8.4.2
to establish resonance of the experimental model at a pre-
selected strain level.

2. Record stroke, absolute acceleration or relative acceleration
of "moving mass" and the acceleration at station three center
on an oscillograph chart.

3. Move a 25-g accelerometer sequentially from station one to
station seven on the concrete deck and record the resulting
accelerations on an oscillograph chart.

VIII.8.6 Data Reduction

At the beginning of each sequence of numbered tests a calibrated
signal for each transducer was recorded on the charts of the oscil-
lographs. This calibration data was then used to obtain physical data
from the records of the transducers as recorded on the oscillograph
charts.

VIII.9 CONCLUSIONS AND RECOMMENDATIONS - EXPERIMENTAL METHODS

VIII.9.1 Introduction

A study was conducted to determine the feasibility of performing
full-scale fatigue and dynamic tests on a three-span composite highway
bridge. As part of the feasibility study, an experimental investigation was performed on a composite model to evaluate the "moving mass" - hydraulic actuator combination as a dynamic loading technique to induce a resonant vibration in the bridge during the fatigue and dynamic tests. The dynamic test procedures and a data acquisition system proposed for the bridge tests were also included in the evaluation. This article contains an evaluation of the vibration excitation systems, the test procedures and the instrumentation used in the model test along with recommendations for their use in the bridge tests or in future investigations.

**VIII.9.2 Vibration Excitation Systems**

The vibration excitation systems, which consisted of either the 5 kip or 80 kip hydraulic actuator and their respective "moving mass" and control systems, performed most satisfactorily as a dynamic loading technique to induce resonant vibrations in the experimental model. Also, the simultaneous control of two 80 kip hydraulic actuators presented no problems. The performance of these vibration excitation systems leaves little doubt as to the ability of this loading technique to excite the test bridge at resonance to the desired stress levels, provided an appropriate sized system is used for the fatigue test.

Minor problems were encountered with Vibration Excitation System No. III and with the hydraulic actuator supports for systems I and II. The response characteristics of the 80 kip hydraulic actuators near the resonant frequencies of Tests 10 and 11 were such that only small strokes of the actuator rods were attainable, and as a consequence of this the relative accelerations of the "moving masses" were small in comparison to their absolute accelerations. Under this condition the
major components of the absolute accelerations of the "moving masses" were the accelerations of the model deck, at the base of the hydraulic actuators. Thus for Tests 10 and 11, control on constant absolute acceleration during the sweep-sine test resulted in a constant acceleration level of the model deck at the reference accelerometer location over much of the excitation frequency range of the sweep-sine test. Because of this situation, the sharp peak at resonance could not be produced for Tests 10 and 11.

The experimental data shows that the support systems for the 5 kip hydraulic actuator may have influenced the vibratory response of the experimental model, although the evidence is not conclusive. It appears that the unsymmetrical nature of the angle brace support caused the two wide-flange beams to have different maximum deflections at resonance during most of the experimental tests. A symmetrical brace support was not possible with the 5 kip actuator because the location for the second brace support was taken up by the servo-valve and the accumulators.

Improvement in the experimental data for Tests 13-15, such as reduction in scatter of the data, and experience from conducting these model tests indicates that the Spectral Dynamics' control system for Vibration Excitation System II provides the best method of control for keeping the magnitude of the exciting force constant. Also, it was observed that the test procedures for a given test could be carried out in a shorter period of time than was possible with the other type of control system.

Spectral Dynamics' control method is ideally suited for the field tests because of its ability to automatically maintain constant strain
or exciting force levels on the bridge. Unfortunately, the Spectral Dynamics' control system or similar commercial systems can not be used below 5 cps because of electronic instability problems. This lower frequency limit eliminates the use of this control system for the field tests since the bridge has a natural frequency below 2.3 cps.

The recommendations with regard to the vibration excitation systems are as follows:

1. The "moving mass" - hydraulic actuator combination should be used as the dynamic loading system for the fatigue and dynamic tests of the composite bridge.

2. The hydraulic actuator support system for the dynamic tests portion of the bridge study should be designed in such a way that its influence on the dynamic response of the bridge is minimized.

3. For the field tests, control of the magnitude of the applied force to the bridge should be established on the relative acceleration instead of the absolute acceleration of the "moving mass".

VIII.9.3 Instrumentation and Transducers

The data acquisition system developed during the course of this investigation to record the experimental data generally functioned quite satisfactorily and no serious problems were encountered with the system except for some difficulties with less than 0.1 mv unamplified noise signals. A few problems were encountered with the transducers.

The servo-valve attached to the 5 kip hydraulic actuator contained a small electro-dynamic shaker in the pilot stage of the servo-valve.
which produced a high-frequency excitation on the experimental model. The excitation was sensed by the accelerometers and this resulted in an equivalent noise signal superimposed on the response signal from the accelerometers. The only way accurate data could be obtained from the accelerometer signal was to filter the signal before it was recorded by the oscillograph. This produced a minor data reduction problem since there were not enough filters available for each accelerometer. The accelerometer signals or accelerations which were filtered are shown in Fig. VIII-22.

The concrete strain gages were originally designed for creep and static tests, and they required pretensioning by means of an adjustment screw in order for them to function properly. Considerable difficulty was encountered with drift in the concrete strain gage signal, and this was due to a loss in the pretension setting as a result of the vibratory environment to which the gages were subjected.

The deflection gages performed satisfactorily for the first mode bending tests but during the one, third mode bending test the deflection gages went into resonance. However, this was consistent with the results of the vibration analysis of the deflection gage\(^{(2)}\). There is some question about the accuracy of these gages as used in this study when the model has a non-constant transverse deflection at midspan. Under this condition, the girder and the fixed end of the deflection gage will rotate which results in an error in the vertical deflection reading.

The recommendations with regard to the instrumentation system are as follows:

1. The type of concrete strain gage used during the model study
should be replaced with standard commercial strain gages for the field tests.

2. Filters should be used to a greater extent in order to remove noise from the transducer signals. This will result in a more reliable signal from which to take measurements.

3. An AM-FM magnetic tape recorder or an analog-to-digital system should be used in place of oscillographs during the field test to record the experimental data. The use of a magnetic tape recorder would permit conversion of the experimental analog data to digital form by means of the analog-to-digital conversion capability of the College of Engineering's SEL-840A computer. The digitized data could then be processed through a digital computer for the purpose of reducing and analyzing the experimental data.

VIII.9.4 Experimental Methods and Procedures

The experimental methods and procedures as outlined in Art. VIII.8 accomplished quite satisfactorily the experimental objectives of the investigation. However, an examination of the experimental data reveals that certain minor improvements should be made in the experimental procedures and these improvements are as follows:

1. The log decay test should immediately follow the recording of data at the end of the initial setup test in order to insure that the maximum amplitude of the test structure at resonance is the same for both tests. The correlation of the experimental data and results for these two tests will be made easier by this change in procedure.
2. The transducer signals should be recorded at the beginning and end of each dynamic test, with no exciting force acting on the model, to establish the zero magnitudes of the signals.

3. For the sweep-sine test, the number of peak amplitude data points on either side of the resonant peak should be increased in order to better define the response curve in this region. A further improvement would be to record the peak amplitude and plot it versus frequency with the aid of a X-Y plotter. This would permit a continuous record of the model response as a sweep-sine test was conducted over a selected frequency range which included at least the first natural bending frequency of the model.

An experimental technique, which should be explored for possible application to future tests where dynamic properties of structures are studied, is mechanical impedance testing. Mechanical impedance is defined as the ratio of the exciting force to a resulting velocity of the structure at the same frequency. Mechanical impedance is measured as either point impedance or transfer impedance. Point impedance is a measurement of force divided by velocity where both are measured at the point of excitation. Transfer impedance is the same measurement of force divided by the velocity at some other point on the structure.

Normally mechanical impedance is plotted on a log scale versus the log of the excitation frequency. It is calculated from experimental data obtained from a sweep-sine test over a selected frequency range which includes one or more natural frequencies of a structure. Dynamic properties which can be obtained directly from an experimental mechanical impedance plot are: dynamic mass, dynamic stiffness, natural frequencies, and damping at resonance.
A possible useful application of mechanical impedance plots would be in the detection of changes in dynamic properties of a structure due to damage from cyclic loading. To apply the technique, mechanical impedance plots would be obtained at different times during the load history of a structure. Then the plots would be compared to detect changes in the dynamic properties.

REFERENCES


