Summary Report on Bridge Research

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• Prior to the beginning of the test traffic, studies were made of the probable response of the test bridges to the repeated high overstress and of the probable dynamic magnification of stresses and deflections. The purpose of these preliminary studies was to provide a basis for planning of experiments and to assure safety of traffic operations.

The results of the preliminary investigation of repeated overstress led to the decision to take safety measures, under all test bridges, against catastrophic effects of a sudden collapse. No sudden collapse occurred either during the test traffic or during the accelerated repeated load tests.

The preliminary investigation of dynamic effects utilized an existing analytical solution considered as the best method available at that time. The subsequent research showed that there was no satisfactory agreement between this particular solution and the results of tests with regular vehicles.

These two incidents illustrate the ever-present need for occasional checking of theories and laboratory experiments against the behavior of structures under actual field conditions. Analytical studies and simplified laboratory experiments permit the development of rational theories. However, the accuracy of the assumptions of such theories can be assessed only through realistic tests. The bridge experiment at the AASHO Road Test served this need.

Even the Road Test bridges reproduced only certain aspects of a typical highway bridge. The following major differences should be kept in mind: (a) the test bridges had only three beams and were only one lane wide; (b) the stress levels during the regular test traffic were well above those normally experienced in the actual service life of a highway bridge; and (c) the duration of the tests was only 2½ years.

The major findings of the bridge experiment at the AASHO Road Test may be divided into three groups:

1. Relative over-all performance of different types of bridges;
2. Major effects of repeated high overstress; and
3. Analytical correlations of quantitative test results.

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It has been pointed out that the three major types of test bridges were designed under different criteria and, therefore, direct comparisons of the relative performance of the steel, prestressed concrete and reinforced concrete structures could not be made. However, there were at least four bridges of each major type and comparisons of relative performance were made within all three such groups.

RELATIVE PERFORMANCE

Of the ten bridges with steel beams, two were composite and eight were noncomposite. Although the design of the bridge experiment was such that in no set of two bridges was the composite action the only variable, it was apparent that the behavior of the composite steel bridges was superior to that of the noncomposite steel bridges.

The composite bridges were stiffer than the comparable noncomposite bridges. Composite action increased both the yield and ultimate strength, and the permanent deformations of beams observed in the composite bridges were substantially less than those observed in the noncomposite bridges. Transient deflections increased during the period of test traffic on the order of 5 percent in composite bridges, but on the order of 15 percent in the noncomposite bridges.

The decrease in stiffness of bridges with time, indicated by the increased transient deflections, was caused to a large extent by transverse cracking of slabs. The cracks in composite bridges were fewer in number and remained tight throughout the duration of the traffic. The cracks in the slabs of noncomposite bridges opened up a few months after formation and led to progressive deterioration of the slabs by spalling of edges and seepage of water. At the conclusion of the test traffic, the slabs of composite bridges were in excellent condition; those of noncomposite bridges were in a moderate to advanced state of deterioration.

The noncomposite test bridges had no interaction between the slab and the beams. Formation of natural bond was inhibited by a treatment of the top surfaces of the steel beams prior to casting of the slabs. As this artificial condition is not encountered in bridges on the highway system, the differences in the performance of the two types of structures are likely to be somewhat smaller than indicated by AASHO Road Test data. Nevertheless, the potential superiority of composite bridges was clearly demonstrated.

In two of the four bridges with prestressed concrete beams tensile cracks were present from the beginning of the regular test traffic. There was a marked difference between the cracking of the prestressed concrete beams post-tensioned with parallel wire cables and those pretensioned with 7-wire strands.

In the beams stressed with parallel wire cables, severe tensile cracking changed radically the response of the beams to loading. The stiffness of the beams decreased substantially, progressive loss of bond was indicated and ultimate flexural capacity of the bridge was decreased because of bond failure.

On the other hand, the cracking of beams with 7-wire strand caused practically no change in the response of beams to loading. Deflections during the period of test traffic were essentially the same as those of an uncracked bridge, and excellent bond was preserved at all stages of tests up to the ultimate load.

In the two bridges that remained essentially uncracked throughout the tests with repeated stresses, no basic difference was found in the behavior of the pretensioned and post-tensioned beams. However, it must be remembered that cracking due to overloads is always a possibility so that the behavior of cracked beams must not be overlooked.

Of the four reinforced concrete bridges, two were designed for total dead and live load stresses of 30,000 psi and the other two for 40,000 psi. Actual total stresses during the test traffic were of the order of 33,000 psi and 42,000 psi, respectively.

In both types of bridges the maximum crack width exceeded 0.01 in. in the unloaded condition and the test vehicles caused an additional opening of about 0.002 in. The incidence of the largest cracks was greater in the higher stressed beams. Furthermore, the cracking of the lower stressed beams consisted essentially of vertical cracks only; inclined cracking was quite general in the outer 15 ft of the spans subjected to higher stresses.

Crack width and the number of cracks increased with traffic.

MAJOR EFFECTS OF REPEATED OVERSTRESS

Results of the tests of steel beams with partial length cover plates demonstrated that the sections at the ends of the cover plates can be critical from the standpoint of fatigue cracking. By the end of regular test traffic one or more fatigue cracks were found in all steel bridges with partial length cover plates.

Fatigue cracks first appeared in the bottom surface of the rolled section at the toe of the welds connecting the plates. The cracks then spread toward both ends of the flange and upward into the flange. The cracks were perpendicular to the beam axis.

Fatigue cracks were first discovered after the bridges were subjected to between 480,000 and 540,000 trips of test vehicles. The range of fluctuating stress, caused by the crossing test vehicle, is primarily responsible for cracking. The mean stress range varied between 12.8 and 17.8 ksi in these tests.

In both reinforced concrete bridges designed for a total stress of 40,000 psi, two bars broke in the exterior beam after a total of approximately 730,000 stress cycles. This included approximately 560,000 trips of test vehicles and 170,000 cycles of stress applied by vibration of the bridges.

Although the fracture of the bars was sudden, examination after fatigue failure showed that the fatigue cracks were started at the intersection of the transverse deformations with the longitudinal rib.

Both bridges had a mean stress range of 26 ksi in the bars that failed.

In prestressed concrete beams subjected to tensile stresses lower than the modulus of rupture of concrete, some fatigue cracking of concrete was detected during the period of test traffic. However, the cracking was detectable only with special aids and had no readily observable effects on the behavior of test bridges.

Both of these bridges were subjected to a total of 1,500,000 cycles of stress, including 560,000 vehicle trips and 940,000 stress cycles accumulated by vibration of the bridge.

ANALYTICAL CORRELATIONS

The question of dynamic magnification of stresses in bridge beams, commonly referred to as impact, was investigated on the test bridges both experimentally and analytically. The studies had demonstrated that the dynamic response of a bridge is very sensitive to a number of parameters some of which cannot be controlled or even measured accurately. Thus, it is hopeless to attempt an interpretation of dynamic test data without the aid of a rational theory.

The results of the dynamic tests on the test bridges were found to be in satisfactory agreement with those obtained from the theory used in the final report on this study. This agreement was obtained only by including in the analysis all pertinent characteristics of the bridges and the vehicles, some of which were shown to be significant for the first time by these tests. For example, it was found that the interleaf friction in the suspension system of the trucks had an extremely important effect on the dynamic response of the bridge.

Because of the special conditions of the bridge tests, the relative magnitudes of the dynamic effects for the different bridge types are not indicative of those which would be obtained under more typical conditions. However, the theoretical analyses found in agreement with the test data, can be used to study the relative dynamic effects for various types of bridges and vehicles used on the highway system.

The number of stress cycles at fatigue cracking of steel bridge beams with partial length cover plates and the number of stress cycles at fracture of reinforcing bars in the concrete bridges were compared with laboratory fatigue data obtained for specimens having the same details as those present in the test bridges. As the test bridges were subject to a varying range of live load stresses, the comparisons were based on Miner's hypothesis of cumulative damage.

The comparisons showed that the results of the tests of steel bridges fell within the range of the laboratory test data.

The comparisons for the reinforced concrete bridges suggested that the bars in the bridges were only slightly weaker in fatigue than indicated by the lower limit of the laboratory data.

Thus, where a reasonable estimate of the magnitude and number of repetitions of stress can be made, laboratory fatigue data for the component elements can forecast the life to fatigue failure within reasonable limits.

Out of the original 18 test bridges, 10 were available for further testing after the completion of the regular test traffic and accelerated fatigue tests. These included 4 steel bridges, 4 prestressed concrete bridges and 2 reinforced concrete bridges. The 10 bridges were tested with successively heavier vehicles until failure. The maximum external moment at the time of failure was compared with the computed maximum static resistance of the bridge. The external moment included the weight of the bridge and the test vehicle, but the impact was disregarded.

The agreement between the test and computed moments was satisfactory. In all steel and reinforced concrete bridges, and in all prestressed concrete bridges in which bond was preserved, the test value was in excess of the computed one. Only for the unbonded prestressed concrete bridge was the test value slightly less than the computed moment.

The bridge research at the AASHO Road Test is reported in Special Report 61D of the Highway Research Board. The report was examined by a review subcommittee of the National Advisory Committee and by the Advisory Panel on Bridges.

The Advisory Panel considered also the results of this research in terms of practical application. Its considerations led to dividing the results into two groups: (a) results that may warrant immediate changes in design practices, and (b) results pointing to need for further research.

In the opinion of the Bridge Panel consideration should be given to the following possible changes in the design practices:

1. Allowance of tensile stresses in concrete of prestressed concrete beams pretensioned with strand;
2. Allowance of higher design stresses in the steel beams of composite bridges; and
3. Modification of the criteria for determination of the minimum length of partial length cover plates.

Recommendations on these items were submitted by the Panel to the Highway Research Board for transmittal to the AASHO Committee on Bridges and Structures.

Items in need of further studies selected by the Advisory Panel on Bridges include the following:

1. Systematic collection of strain histories for selected bridges in the highway system. Such data will permit evaluation of the effect of changes in permissible vehicle loads on the life of highway bridges.
2. Further analytical and experimental studies of the dynamic magnification of stresses and deflections. Such studies will lead to reevaluation of the impact factors used in design.
3. Experimental laboratory studies relating to the strength and performance of highway bridges. Fatigue tests of steel beams with various types of weldments, fatigue tests of reinforcing bars, effect of repeated loading on tension cracking of reinforced concrete beams, effect of residual stresses on permanent deformations of steel beams, effect of type of prestressing on cracking of prestressed concrete beams, and relaxation tests of prestressing steel fall into this group.

An outline of these studies was submitted by the Advisory Panel to the Highway Research Board for transmittal to the HRB Bridge Committee.

It is hoped that the AASHO Committee on Bridges and Structures will give an early consideration to the proposed changes in the current design practices and that the HRB Bridge Committee will follow through with execution of the proposed further research.

DISCUSSION

E. L. Erickson.—We are very fortunate in having had the opportunity to include these bridge tests in the Road Test program. They have given information with regard to the effect of overload on highway structures. Certainly it was demonstrated that overloading structures sufficiently is going to wreck them. Furthermore, the information obtained in verification of laboratory work carried on over the years is perhaps the most important achievement. This gives confidence in this work and points the way to needed new work in the bridge and structure field.

A. C. Church, Florida State Road Department.—I know why you are not supposed to weld cover plates across the end, but since you were going to tear down these bridges anyway why not just weld across the end of one cover plate to see what would happen?

Viest.—We had quite a few variables in this program as it was and only 18 test bridges. The planning committee apparently did not consider it desirable to introduce further complications by making the end detail of the cover plates a variable.

C. A. Marmelstein, State Highway Department of Georgia.—In discussing the relative stiffness of the two types of construction, I wonder whether consideration was given to the fact that the same depth beams were used on both the composite and noncomposite. In the AASHO Bridge Specifications there are limiting depth ratios. In noncomposite construction we take the depth of the beam alone, but in composite construction we use the depth of the beam plus the thickness of the slab. The problem in Georgia was that at extreme span lengths allowable from a stress standpoint, the composite bridges became fairly flexible. Did that condition in the design add somewhat to the relative stiffness of the composite over the noncomposite?

Viest.—The comparisons which were made were based on comparable stress levels in the composite and in the noncomposite bridges. Of course, if you increase the depths of the noncomposite steel beam without changing the stress level, the stiffness of the noncomposite beam will be increased. However, the differences between the composite and noncomposite beams were so large that increasing the depths of the noncomposite beams, say one beam size, could not have changed significantly the relative stiffness of composite and noncomposite bridges. Another point that enters into this question of stiffness is the fact that in noncomposite bridges very often bond is preserved so the noncomposite bridge acts as a composite one. In that case, of course, the noncomposite bridge will be stiffer. However, I doubt that you would want to rely on something which may or may not be there.

H. Hewitt, Department of Public Works, Ontario, Canada.—Because the average residual stress in the flanges of some of the beams runs as high as 11,000 psi, should this have an effect on the permissible stress or at least the safety factor which is assumed to be roughly 0.6 times the yield stress?

Viest.—This will depend very much on what you want the factor of safety to do. There is no doubt that residual stress is important in any discussion of overloads. If we want to know the maximum overload that can go over the bridge without producing large permanent deformations, we should pay attention to the magnitude of residual stresses, particularly in noncomposite bridges. Concerning the factor of safety for ordinary loads we may consider this residual stress as covered by the factor of safety. Then the factor of safety is determined either on the basis of the yield point of the steel or on the basis of ultimate strength of the bridge, without considering directly the residual stress. But it should be made certain that allowable stresses for ordinary traffic are well below the discounted yield; that is, below the yield minus the residual stress. In other words, in determining the factor of safety or the allowable stress, we should know approximately what the residual stress is.

W. R. Kahl, Rummel, Klepper & Kahl, Baltimore, Md.—Did the beams that were used have induced camber or rolled-in camber?

Viest.—They had induced camber.

B. F. Friberg, Consulting Engineer, St. Louis, Mo.—The two graphs on amplification factors show for deflection a maximum value approximately 1.4 or 0.4 over static, the amplification for strain approximately 0.2. The question is whether the differences between these two values were because the defliction represents a summation of things and the strain represents something taken in just one particular point, perhaps not the critical point, or is this a logical nature of the problem?

Fenves.—The difference is not because the deflection is a double-integral of the strain. The difference is due to the fact that the quantity we are interested in, the maximum dynamic effect, is the sum of two quantities: a crawl curve (or influence line) and the dynamic component. The dynamic component is almost equal in the two cases. For all practical purposes, it is equal. The largest difference that can be obtained is for a single-axle loading under certain assumed conditions. I think the ratio comes out $\pi^2/12$.

The fact is that the crawl curves are completely different in the two cases. For the 3-axle vehicles shown, the crawl curve for deflection is, in the region of interest, a concave curve varying about 20 percent from the maximum value. For strain (or moment) for vehicles with two essentially equal loads, there is a portion of the crawl curve that is essentially equal to unity for the length of the span equal to the axial spacing. When a dynamic increment is superimposed on this, it is quite critical whether this dynamic increment of 20 or 40 percent adds to 1.00 or whether it adds to 0.90. Thus, by having the same dynamic increment added to different ordinates of the crawl curve, a sizable difference in total effects results. For all practical purposes you cannot divorce the crawl effect from the dynamic effect and deal with dynamic increments alone.

W. E. Thompson, Cornell Aeronautical Laboratory.—First of all, in the graphs of the speed factor versus the amplification factor, was there any indication that a resonance in the vibration of the bridge occurred with the motion of the vehicle across it?

Fenves.—No. There was no resonance observed. Several years ago we investigated this in considerable detail and tried to find a so-called critical alpha for which the speed was such that the time of transit of the two heavy axles over a given point was exactly equal to the natural period of the bridge—without results. Because of the very short duration of transient effect there is no sufficient buildup to produce any resonance whatsoever.

Thompson.—In addition to the parameters already mentioned, is there in your more complete report any account of a damping ratio for the bridge structure which might be imposed on the information here?

Fenves.—Yes. We have done analytical solutions with damping coefficients considered. For these particular comparisons we tried to match the damping coefficient, as computed approximately from the decay of the free vibration curve. This did not have any great effect. However, other studies on the Highway Impact Project go to great length in studying the effect of bridge damping over very large ranges of the damping coefficient.

Thompson.—Would you say these bridges, or the data shown for the several examples here, are under-damped, critically-damped, or highly over-damped?

Fenves.—They are very much under-damped.

Thompson.—You showed on some deflection curves a component due to dynamic effects plus a kind of static effect arising from the crawl condition of loading. Would you advance an opinion on the severity of this dynamic component, versus what one usually gets with a static loading, on fatigue life. We are now engaged in trying to answer the question at the Cornell Aeronautical Laboratory. Is dynamic loading a significant factor in fatigue life? You have worked with it with respect to bridges; we are working with it with respect to highway pavements. There appear to be some common elements in both.

Fenves.—This was discussed by Dr. Viest and Mr. Fisher. It does increase the stress range. If you are willing to do what has been done here in considering the stress range (from the maximum dynamic effect down to the maximum rebound), this does add something of the order of 40 percent to the stress range. The maximum dynamic increment is something of the order of 20 percent and the maximum rebound on the first half-cycle is something of the order of 15 to 20 percent, so 40 percent is added to the stress range. If this is considered, it has the same effect that a 40 percent increase in stress range would have. However, this is a second-order effect, because it is a high-frequency effect superimposed on the extremely low-frequency effect associated with one truck passing over the bridge. This can be answered only when the test data can be compared to compound forcing functions where the laboratory tests themselves include forcing functions superimposed from several components, rather than to the simple sinusoidal forcing-type functions. As of now, it increases the stress range by 40 percent but this figure is probably too high.