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Shakedown Performance of Composite Beams with Partial Interaction

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Summary

This paper reports on the results of one test on a half-scale, two-beam, two-span composite bridge. The test is unique in that varying amounts of composite action (50% and 80%) were used, and in that actual moving loads were used to load the structure. The experimental and theoretical loads compared favorably for the case of 80% interaction when the actual material properties were used in the calculations. The results also indicate that if 50% or less interaction is provided, the structure may not be able to carry cyclic loads into the inelastic range.

1. Introduction

While the high-cycle, low-amplitude fatigue behavior of shear studs has been studied extensively, relatively little is known of the performance of shear studs under reversed cyclic loads (low-cycle, high amplitude regime). This aspect of shear stud performance is of great interest in rating steel composite bridges [Galambos et al. 1993] which are often classified as structurally deficient because of insufficient strength to handle the increase in truck weights that has taken place since they were put in service. If it can be shown that the shear studs have sufficient strength and stiffness to allow the structure to shakedown under large overloads, it may be possible to increase the rating in many of these bridges so that they comply with current loading criteria. This particular aspect of bridge design is the focus of this paper, in which the results of a test on a two-span, half-scale composite bridge will be reported [Flemming 1994]. More generally, the development of knowledge regarding the degradation of shear interaction between steel beams and concrete slabs subjected to cyclic loads is of fundamental importance in understanding the behavior of older steel beam-concrete slab bridges where the interaction may come either from partial encasement or friction and adhesion at the members interface. The behavior of shear studs under cyclic loads is also of great interest in other areas of structural engineering, such as in improving our understanding of composite beam behavior in moment frames subjected to seismic forces, where the floor slab acts as diaphragm in transmitting the inertial forces.

2. Shakedown of Bridges

A recent study on the behavior of straight, continuous composite bridges has suggested that the useful life of many deficient short-span structures could be extended significantly if the structures were allowed to enter into the inelastic range for a low number of cycles [Galambos et al. 1993]. This work developed model rating systems based on the theory of shakedown [Konig 1987], which is a well-developed aspect of plastic design of structures. Shakedown is a term used to describe structural behavior under large cyclic loads. Shakedown implies that after repeated applications of a prescribed load history, which exceeds the elastic limit but not the plastic collapse load of the structure, the residual deflections in the structure will stabilize. Residual deflections are the permanent deformations remaining in the structure after the load has been removed. Because yielding has occurred, there will be additional forces, known as residual moments, locked into the structure when the loads are removed. It is important to note that shakedown implies some damage to the structure, generally in the form of yielding of main members, and thus may result in a serviceability failure. However, a key feature of shakedown is that once the deflections stabilize, the structure will respond elastically to any additional cycles of the prescribed load history.

In the initial work done by the senior author [Galambos et al. 1993] as well as in that by Grundy and Thiru [Grundy and Thiru, 1995], composite beams with different degrees of interactions showed a marked tendency to loose strength and stiffness due to slip at the steel-concrete interface. Slip at the interface could be due, among many other reasons, to damage to the concrete in bearing, cracking of the concrete, damage to the stud due to cyclic plasticity, and/ or propagation of low-cycle fatigue cracks. Other sources of strength and stiffness deterioration of concern in composite beams are local buckling and local bending beneath the shear studs of the steel beam flange. To address the problem of shear connection deterioration an experimental program that included the testing of a half-scale, two-span bridge was developed. This programs is unique in several ways. First, as far as the authors know, this is the first large scale shakedown test to be carried out on a composite bridge in the world. Second, it is the first laboratory bridge test in the U.S. to use an actual rolling load with rubber tires, as opposed to concentrated loads provided by loading jacks, to apply the loads to the structure. Third, it is the first attempt at using partial composite action in this type of structural system, and the first to try to quantify slip at the interface for a partial composite bridge beam.

3. Experimental Work

The design of the model bridge required that compromises be made among scaling laws, structural simplicity, and loading requirements [Flemming 1994]. The model bridge was based on a prototype designed by the current AASHTO LFD Specification. This design was then scaled by following as much as possible similitude laws, recognizing that it was impossible to scale all quantities properly. The most important consideration was the use of shear studs whose behavior closely simulated that of the ones in the prototype. The smallest shear stud commercially available is 10 mm, while actual shear studs in bridge construction range from 19 mm to 25 mm. Thus a half-scale model was deemed to be the smallest that could be tested. The one important subject that was not directly addressed in the scaling process was the quantity and placement of shear studs. The shear stud design for the half scale structure was carried out using the AASHTO strength criteria alone. Thus fatigue criteria, which will usually govern in the design of the shear connection, were not considered. Moreover, since the intent of the research was to study the behavior of the interface, something less than the full interaction as given by code equations was desirable. Thus, it was decided that 80% of the design requirement for shear studs (100% interaction = two 9 mm studs at 200 mm) would be placed in one span and 50% of the design requirement in the adjacent span. This approach was used to maximize the amount of information that could be gathered by this test concerning the behavior of the structure in the shakedown range. Fig. 1 shows a schematic view of the structure and its loading system. The loading system consisted of the tandem axle taken from a real truck. The loads for the shakedown test were increased by adding large lead ingots to the tandem axle.

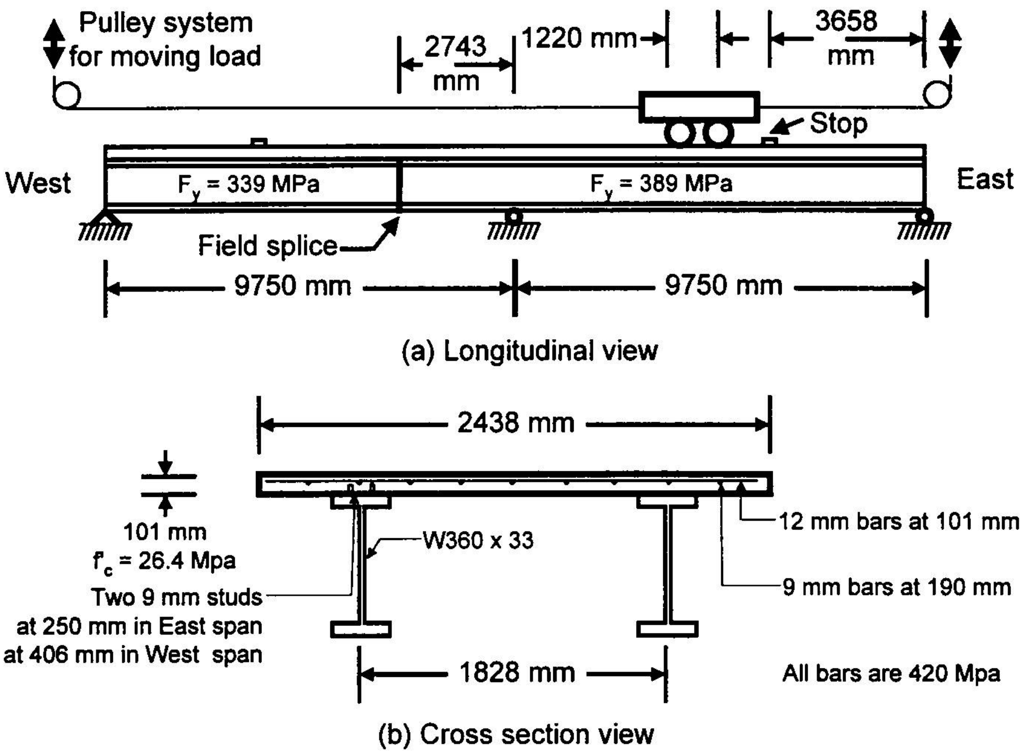

Figure 1 - Details of the test specimen.

4. Results

The results will be summarized first with a plot of the total axle load against the maximum centerline deflection for each span (Figs. 2 and 3). For each beam, the maximum deflection at each cycle at a particular load are shown, i.e., only the envelope of response is shown. Fig. 2 shows this data for the two beams (labeled North and South) for the West span, which had an 80% interaction. In this plot the incremental collapse limit for the composite section, the composite yield capacity, and the non-composite yield capacity are also shown for reference. A cycle of load was defined as one full pass (forward and backwards) of the bogie. Two lines are shown for each beam to indicate the different directions of travel for the axle, since the direction of travel did seem to influence the behavior of the specimen.

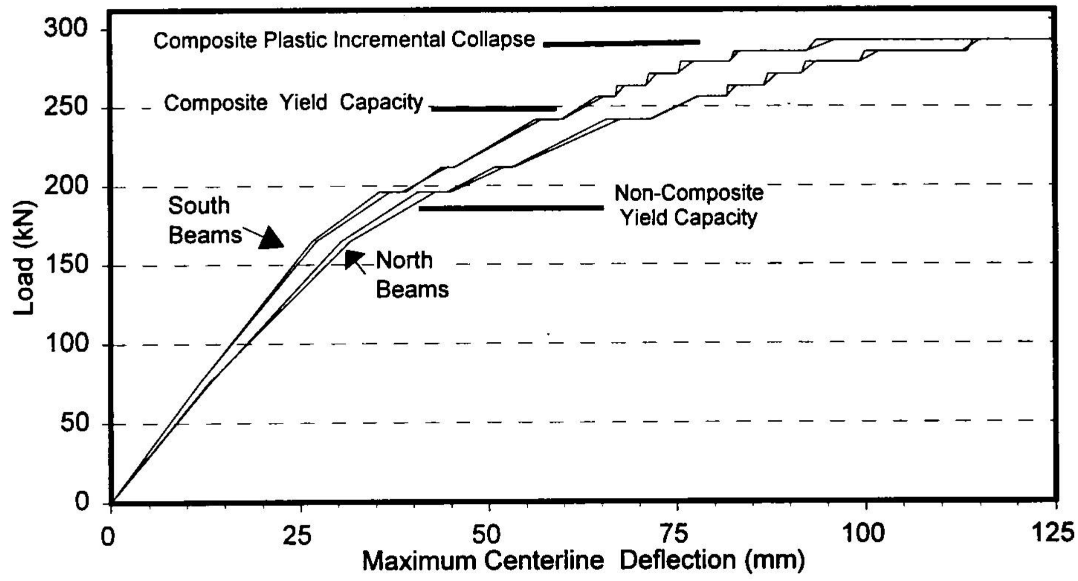

Figure 2 - Load vs. centerline deflection (West Span).

The test was begun by applying to two full cycles of load with the axle self-weight only (75.6 kN). The response of the structure to these cycles was purely elastic. The first lead ingots were then loaded onto the tandem axle to give a total load of 165.1 kN. This value was very close, but lower, than the calculated first yield. The response of the structure was mostly elastic, but some strain gages on the West side of the center support showed slight yielding. The load was then increased to 195.7 kN where the first signs of yield occurred. Four cycles were applied at this level, until the changes in residual deformation from one cycle to the next were less than 1.3 mm. The latter was the criteria used throughout the test to determine whether shakedown had been achieved. As can be seen from Fig. 2, the West beam failed gradually, and reached its nominal shakedown capacity. A hinge formed at the centerline support first, with clear indications of a plastic hinge behavior occurring at a load of 241.8 kN. At this level it took 10 cycles to reach shakedown. The loads were increased in five increments up to 285.1 kN, where it took 14 cycles to shakedown. A slightly increase of the load to 292.7 kN resulted in the formation of a full plastic hinge at the center of the West span and the collapse of the structure.

Fig. 3 shows similar data to Figs. 2, but for the North Beam on the East span, which had 50% interaction. The behavior of this span was quite different. After the hinge formed at the center support at a load of 241.8 kN, successive passes on the East span resulted in the immediate formation of a plastic hinge and large deflections. With a slight increase of the load to 256.7 kN, the East span collapsed. The collapse was quite rapid after the second pass, with the deflection taking only a few seconds to reach the temporary support jacks placed about 150 mm below the undeformed bridge to prevent a complete collapse of the structure. This span only reached the plastic moment capacity of the bare steel beam. Much of the composite action was lost in the East span due to a progressive failure of the shear studs beginning at the East span centerline and propagating to the center support.

Fig. 4 shows similar deflection data but plots it versus the cycle number. Fig. 4 also shows the bounds provided by a simple analysis assuming that the section remained either fully composite or non-composite throughout the test. It is evident from Fig. 4 that the beam did not behave as fully composite past the first few cycles. However, it was able to mobilize considerable strength and ductility such that for purposes of strength calculations the West beam can be assumed to have acted compositely. From the degree of yielding observed it is clear that substantial strain hardening occurred and this is probably the best explanation of the strength performance.

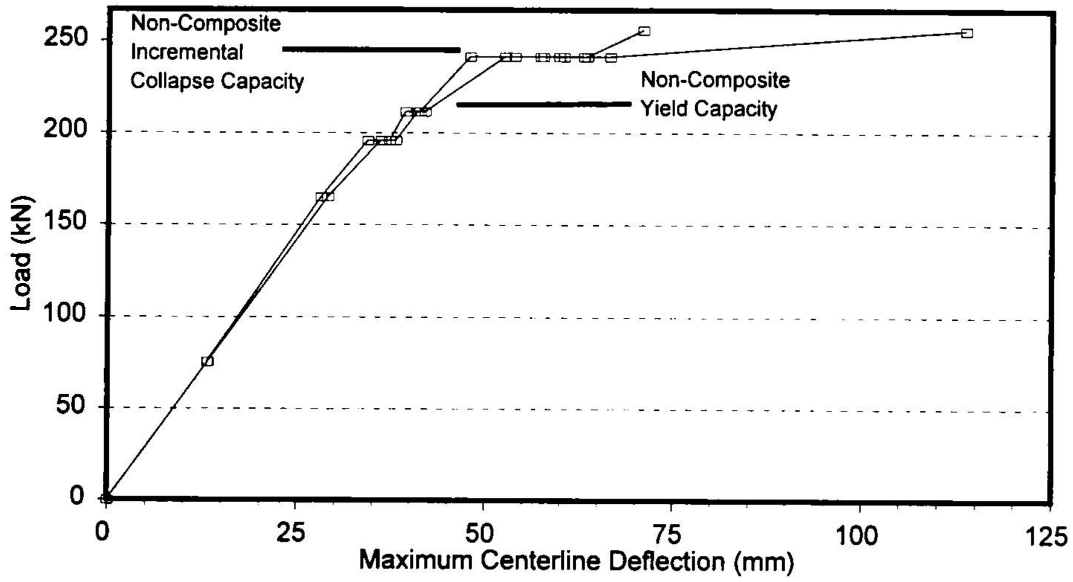

Figure 3 - Load vs. centerline deflection (East span).

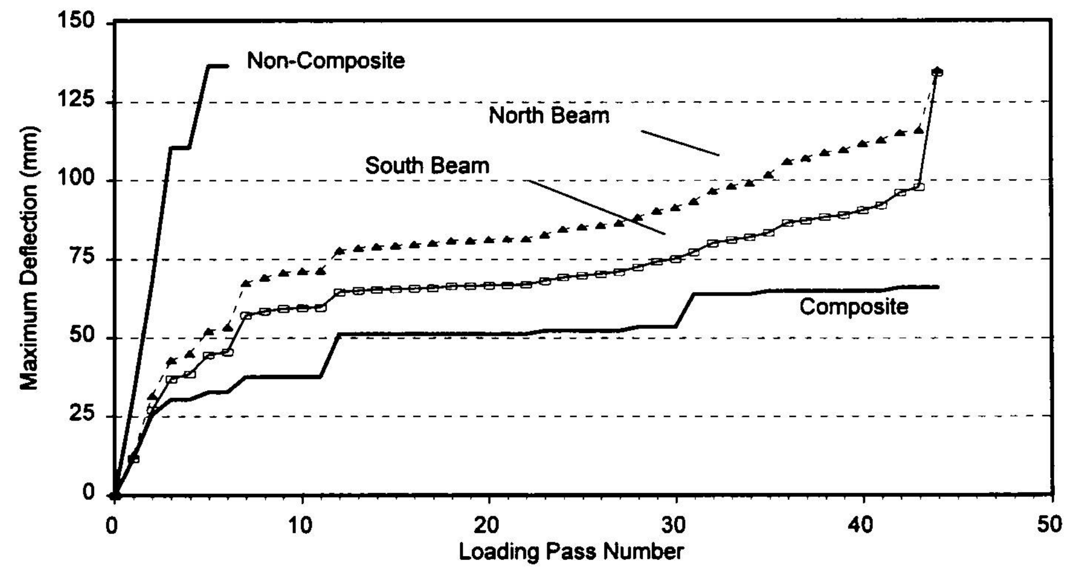

Figure 4 - Deflection vs. cycle number (East span).

Perhaps the most surprising data obtained in this test concerns the slip at critical locations in the positive moment regions of the East and West beams respectively. The data from eight different slip and strain measurements indicate that the slip did not stabilize under any given load cycle. For the East beam, which behaved mostly as non-composite, the slip reached 0.8 mm just before the structure collapsed. For the West beam, which acted compositely, the slips at the same load level were roughly half those of the East beam. At collapse, the slips in the West beam were close to 1.1 mm. It is interesting to compare this range to typical monotonic tests on shear studs, where 0.8 to 1.0 mm of slip is considered to be the service limit and 4 to 5 mm of slip at ultimate is considered desirable. Although at lower levels of load (less than 241.8 kN) the slip seemed to be beginning to stabilize after a few cycles of load, near the end of the test the slips seemed to increase almost constantly with every cycle of load.

5. Conclusions

The experimental and theoretical load calculations for the test compared favorably after the actual material properties were used in the section capacity calculations. The theoretical static collapse load limit in the West span was within 2.8% of the experimental load at which failure was observed. In the East span, however, the actual incremental collapse load limit was just slightly above the theoretical non-composite incremental collapse load limit. The theoretical composite static collapse load should have been 35.8% higher than the observed failure load. This seems to imply that if 50% or less of the required shear connection is provided one should not expect the structure to carry cyclic loads into the inelastic range. On the other hand if at least 80% of the required connection is provided the structure may achieve its fully composite capacity assuming elastic-perfectly plastic section capacities. The most important experimental observation from this test is that the slip did not stabilize in the case of extreme overloads.

6. Acknowledgments

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7. References

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