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Final Report—Part I

Strengthening of Existing Single Span Steel Beam and Concrete Deck Bridges

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5. SUMMARY AND CONCLUSIONS

5.1 Summary

The literature review indicated that behavior of skewed, orthotropic bridges was reasonably close to that for right-angle bridges, if the angle of skew did not exceed 45°. At a 45° skew, however, load distribution was affected somewhat, in that exterior beams tended to carry more load and interior beams less load. Several of the skew effects noted in the literature, such as increased moments and reactions near obtuse corners, were not checked in Phase II.

The literature review also established the validity of push-out tests for determining strength of shear connectors. Previous research had indicated that high strength steel bolts might be substituted for welded stud connectors of equal diameter with no loss in fatigue or ultimate capacity.

Laboratory testing of shear connectors established the capacity of existing angle-plus-bar connectors. Although the angle-plus-bar connectors were stiffer and exhibited less slip under load than comparable channel shear connectors did, the angle-plus-bar connectors did not have an ultimate capacity significantly larger than a comparable channel. On the basis of testing of Phase II, it was determined that the ultimate capacity of the angle-plus-bar connector could be determined from a modified AASHTO channel connector formula, provided that the weld capacity between the angle and bridge beam was not exceeded.

Two methods of adding connectors to existing bridge beams were tested, both of which involved high strength bolts. The double-nutted

bolt method (Series 6) and the epoxied bolt method (Series 7) of attaching bolt connectors gave load-slip characteristics similar to those for welded studs (Series 5). Both methods for attaching bolts provided connectors which gave a higher ultimate strength than a welded stud of the same diameter. Consequently, the AASHTO formula for ultimate strength of welded studs was conservative for high strength bolt connectors installed by either method. The double-nutted method was judged easier to install in the laboratory and consequently was used in the field with no difficulty.

The composite beams, which were cut from the half-scale bridge model of Phase I and tested to failure, gave an indication of the overall performance of post-tensioned composite beams. Although the beams deformed in the region of the brackets and the post-tensioning tendons deformed at the brackets at high loads, the post-tensioning system did not fracture. Instead, the observed beam failures occurred due to failure of the shear connectors or crushing of the slab concrete. In all cases the experimental ultimate moments were within 10% of computed ultimate moments.

The tests demonstrated that the addition of shear connectors to the model beams did increase ultimate capacity by an amount up to approximately 9%. In the case of the exterior beams, addition of shear connectors also changed the failure mode from shear connector failure to a flexural, slab/curb concrete crushing failure. Computations for the model bridge beams indicated that the addition of post-tensioning could increase ultimate capacity by up to 17%.

The plexiglas skewed bridge model duplicated the behavior of Bridge 2 very closely in terms of load distribution. Application of post-tensioning to the model caused beam moment fractions which deviated from moment fractions computed by orthotropic plate theory in the following manner: A greater fraction of the post-tensioning moment was shifted to interior beams than expected. The measured and computed moment fractions for post-tensioning of Bridge 2 deviated in the same manner. Moment fractions measured for model truck loads fell closer to moment fractions computed by orthotropic plate theory for a right angle bridge than those for post-tensioning. The model behavior again was duplicated in Bridge 2. Post-tensioning of the plexiglas model essentially did not affect model truck load distribution. Post-tensioning also did not affect truck load distribution in Bridge 2.

Somewhat unexpectedly, field-measured strains and deflections for Bridge 1 were less than those computed on the basis of orthotropic plate theory and simple span beam end conditions. The field results, however, were bracketed by simple span and fixed end beam conditions. All of the data indicated that end restraint at bridge abutments was greater than might be expected.

For post-tensioning only, measured strains and deflection at midspan of Bridge 1 indicated considerable end restraint for exterior beams, but almost no restraint for interior beams. For truck loading, with or without post-tensioning, measured strains and deflections indicated significant restraint at both interior and exterior beam ends. The difference in restraint from post-tensioning to truck loading might be explained by the fact that post-tensioning applied a

negative moment to the bridge, whereas truck loading applied a positive moment to the bridge. Abutment and support details for Bridge 1 most likely caused the difference in end restraint from negative to positive moment.

Because strains as a result of post-tensioning were only two-thirds of those computed to be required for strengthening the bridge, there could be concern that the strengthening was ineffective. However, strains measured for truck loading were also only two-thirds of those computed. The unexpected post-tensioning strain loss essentially was compensated by the also smaller than expected truck strains.

Testing of deck concrete cores from Bridges 1 and 2 gave strengths greater than 6000 psi vs. the 3000 psi assumed for analytical purposes. The higher deck strength had a very minor effect on the need for strengthening and the required post-tensioning force. The higher deck strength did, however, significantly increase the capacity of shear connectors and thereby had an effect on the need for additional shear connectors as part of a strengthening program.

As a result of the unexpected end restraint for Bridge 1, Bridge 2 was more extensively instrumented with strain gages and deflection dials. The additional instrumentation confirmed the existence of end restraint in Bridge 2.

For post-tensioning alone, field measured strains and deflections for Bridge 2 were only about one-half those computed for a simple span, right angle bridge. Essentially there was no difference between exterior and interior beams; for both types of beams the measured quantities were very close to those computed on the basis of fixed end

conditions. Comparison with the plexiglas model indicated that more of the end restraint was due to skew than to construction details at the abutments of Bridge 2.

For the truck loading, measured strains and deflections were one-half to two-thirds of those expected. The measured quantities generally lay midway between simple span and fixed end conditions or closer to the fixed end condition. For Bridge 2, post-tensioning did not affect truck load distribution.

Again, as was the case for Bridge 1, the post-tensioning did not cause as much compression strain as desired, but truck loading also did not cause as much tension strain as expected. The two effects essentially compensated.

As a result of the field work for both Bridge 1 and Bridge 2, it appeared that significant end restraints existed as a result of construction details for single span, right angle composite bridges and as a result of both skew and construction details for single span, skewed composite bridges. The restraint reduced the effect which truck loading had on the bridge beams and also reduced the effect which post-tension strengthening had on bridge beams.

5.2 Conclusions

The following conclusions were developed as a result of this study:

(1) The capacity of existing shear connectors must be checked as part of a bridge strengthening program. Since strength of

deck concrete has a significant effect on the need for additional shear connectors, determination of the concrete deck strength in advance of bridge strengthening is recommended.

(2) The ultimate capacity of angle-plus-bar shear connectors can be computed on the basis of a modified AASHTO channel connector formula and an angle-to-beam weld capacity check.

(3) Existing shear connector capacity can be augmented by means of double-nutted high strength bolt connectors. Ultimate capacity of a high strength bolt connector can be computed directly from the AASHTO formula for a welded stud.

(4) Post-tensioning did not significantly affect truck load distribution, either for right angle or for 45° skewed bridges.

(5) Approximate post-tensioning and truck load distribution for actual bridges can be predicted by orthotropic plate theory for vertical load; however, the agreement between actual distribution and theoretical distribution is not as close as that measured for the laboratory model in Phase I.

(6) The right angle bridge (Bridge 1) exhibited considerable end restraint at what would be assumed to be simple support. The construction details at bridge abutments seem to be the reason for the restraint.

(7) The 45° skewed bridge (Bridge 2) exhibited more end restraint than Bridge 1. Both skew effects and construction details at the abutments accounted for the restraint.

(8) End restraint in Bridges 1 and 2 reduced tension strains in the steel bridge beams due to truck loading, but also reduced the compression strains caused by post-tensioning. In effect, the truck tension strain losses compensated for the post-tensioning compression strain losses.