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Shakedown Behavior of a Continuous Steel Bridge Girder Strengthened With Post-Installed Shear Connectors

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ABSTRACT

An economical method for strengthening non-composite steel girder bridges is to create composite action by "post-installing" shear connectors to engage the steel girder and concrete deck to act as a unit. Concepts of inelastic moment redistribution can be used to further increase the load-carrying capacity of continuous bridges. This paper presents results from experimental testing of a large-scale girder specimen with post-installed shear connectors under large repeated loads leading to moment redistribution at the shakedown limit state.

1. Introduction

Many continuous steel bridges constructed prior to 1970 have a non-composite floor system comprised of a concrete deck over steel girders with no shear connectors. In these bridges, the deck serves primarily as a driving surface and helps to transfer the traffic loads to the steel girders, which are the primary load-resisting components.

As infrastructure components continue to age and truck load demands increase, many older bridges will need to be strengthened to avoid load-posting and to maintain the safety of the structure. One economical method of strengthening such bridges is to create composite action by using "post-installed" shear connectors, which provide a mechanical connection between the steel girders and the existing concrete deck. A steel-concrete composite section is significantly stronger and stiffer than a non-composite section, especially in regions dominated by positive bending.

The proposed method for strengthening existing continuous non-composite steel girder bridges is comprised of (1) post-installing shear connectors in positive moment regions to create composite action, and (2) allowing yielding of the steel girders at interior piers, so that inelastic redistribution of moments occurs at strength limit states. The inelastic redistribution of moments is most efficient for well-braced, compact steel girders, which can develop the full plastic flexural capacity without local or lateral-torsional buckling, but the concept can be applied to other girder shapes using lower values of strength.

An experimental program was conducted at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin to investigate the use of post-installed shear connectors and moment redistribution for strengthening existing non-composite continuous steel I-girder bridges. Throughout the many phases of testing, the large-scale girder specimen was subjected to a variety of loading conditions, including fatigue loading, large repeated loads to simulate heavy truck traffic, and monotonic loading to failure. This paper presents and discusses the results from the testing conducted under large repeated loads, which were applied at magnitudes well into the inelastic range resulting in significant moment redistribution.

2. Background

2.1. Composite beam behavior

In a composite beam, the steel girder is mechanically attached to the concrete deck by shear connectors so that the two elements work together to resist the bending, creating a stronger and stiffer section [1]. The most common type of shear connector in practice is the headed stud, which is welded to the top flange of the steel girder and embedded into the concrete deck during casting. Developing composite action is very efficient in regions of beams subjected to positive bending, where the concrete deck is in compression. In negative bending regions,

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however, the low tensile strength of concrete generally prevents any significant strength and stiffness gains from composite action.

A fully composite beam contains an adequate number of shear connectors to develop the full plastic flexural capacity of the composite section. The deformation of the shear connectors in a fully composite beam is small, so that the relative sliding of the concrete slab across the top flange of the steel girder (or "slip") is negligible.

Partially composite beams do not have enough shear connectors to develop the full plastic capacity of the composite section, and thus the strength of the beam is controlled by the strength of the shear connection. The composite ratio is defined as the ratio of the number of connectors provided in a partially composite beam to the number of connectors that would be required for the beam to be fully composite. Partial-composite action is efficient in the sense that low composite ratios provide significant strength gains over non-composite behavior. Thus, partially composite beams are often used in building design. However, bridge girders tend to be fully composite because fatigue, rather than strength, typically controls the shear connector design [2]. A key difference between partially and fully composite design is that the slip, or the relative longitudinal motion between the top of the steel girder and the bottom of the concrete deck, is significant in a partially composite girder and may need to be considered in design.

2.2. Post-installed shear connectors

While welded shear studs are very common in new construction, creating composite action in existing non-composite structures requires a different type of connector that can easily be "post-installed." A few researchers investigated different types of post-installed connectors comprised of bolts, threaded rods, and coiled spring pins in the 1980s and 1990s [3–5]. In more recent years, extensive research was conducted at the University of Texas at Austin to develop various types of post-installed shear connectors that exhibit good structural behavior [2]. The testing reported in this paper uses an adhesive anchor post-installed shear connector, shown in Fig. 1, developed by these researchers.

The adhesive anchor, comprised of a 22 mm diameter ASTM A193 B7 threaded rod, is installed entirely from the underside of the bridge deck. First, a 25 mm diameter hole is drilled through the top flange of the steel girder using a magnetic drill. Next, a slightly smaller 24 mm diameter hole is drilled into the concrete deck using a rotary hammer drill. After cleaning the hole with compressed air and a brush, a structural adhesive is injected into the hole and the threaded rod is inserted. Once the adhesive has suitably cured (approximately 1 h), a torque wrench is used to tighten the nut shown in Fig. 1 below the girder flange. It is recommended that provisions for minimum spacing, edge distance, and top cover provided for welded shear studs in current bridge design codes be followed for these post-installed connectors.

This adhesive anchor connector generally has significantly better fatigue performance as compared to conventional welded shear studs [2,6]. This means that partially composite design can be used to achieve the necessary static strength using fewer shear connectors, while maintaining adequate fatigue strength for the expected remaining life of the bridge.

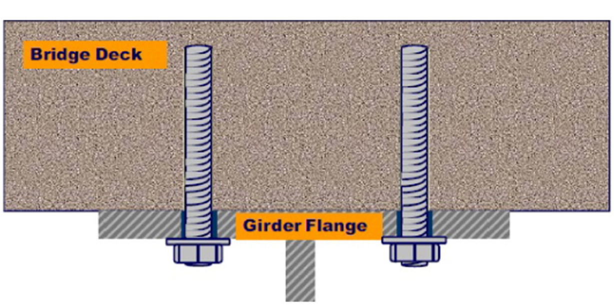

Fig. 1. Adhesive anchor shear connector [2].

2.3. Inelastic moment redistribution and shakedown

Moment redistribution has been allowed in bridge design codes in the United States for more than 40 years. This allows for interior pier sections of continuous steel bridges to be loaded well into the inelastic range, undergo some yielding and plastic rotation, and automatically redistribute moments to adjacent span regions. This behavior is allowed to occur at large overloads and at strength limit states, not at service levels of loads.

Shakedown refers to a limit state in statically indeterminate structures that involves the accumulation of inelastic deformation with repeated cycles of load. When yielding and moment redistribution occur due to a very heavy truck crossing a bridge, residual deformations, stresses, and moments remain in the girders once the load is removed. If these residual moments offset the applied moments from the next passage of an equally heavy truck so that additional yielding does not occur in the girders, the girders are said to have "shaken down," and all future cycles of equal or lesser load will be resisted elastically. The shakedown limit load, which is greater than the load at first yield but less than the static plastic collapse load, can be computed using simple concepts of plastic theory [7]. Any level of load below this limit can be repeatedly applied to a structure in this manner without causing continuous accumulation of inelastic deformation. Shakedown can be observed experimentally by the stabilization of deflections from one cycle of load to the next.

Research involving shakedown began in Germany in the 1920s [8], and more details about the general behavior is discussed elsewhere [9]. Direct application of shakedown and moment redistribution concepts to bridges commenced in the 1970s under research sponsored by the American Iron and Steel Institute [10–12], which led to the publication of a guide specification for Alternate Load Factor Design (ALFD), also referred to as "autostress design," in the mid-1980s [13]. However, all of the experimental work to develop these design procedures was performed under static loading conditions and was conducted for the most part on small-scale, steel-only specimens. The design procedure was later verified by a field test using heavy trucks on a newly constructed bridge [14] and by extensive testing of a 0.4-scale model of a composite bridge system under simulated moving loads [15] in the late 1980s.

In the mid-1990s however, researchers at the University of Minnesota [16,17] and at the University of Adelaide [18] conducted moving load tests on composite and partially composite bridge systems and girders and observed that the deflections and slips did not seem to truly stabilize with increasing cycles of load. This indicates that permanent deformations continued to accumulate with each cycle. These researchers concluded that composite structures may not be able to achieve shakedown due to the lack of ductility and repeatability of behavior in the shear connectors or in the concrete deck under large loads. It is worth noting that none of these tests were performed on specimens larger than 1/2-scale, and deck thicknesses did not exceed 100 mm. This might have had a detrimental effect on the performance of the shear connection and the concrete under repeated loading.

Meanwhile, efforts were undertaken in the mid-1990s at the University of Missouri to simplify the moment redistribution design procedures [19,20] to the current code provisions in the United States [21]. The procedure begins with conducting an elastic analysis to determine the moment envelope for design or evaluation. If the elastic moment at any of the interior piers exceeds the capacity, the excess moment can be redistributed to the adjacent span regions, provided that all of the design requirements in those regions are satisfied after redistribution. A maximum amount of redistribution from a single interior pier is limited to 20% of the elastic moment at the pier section to limit the amount of permanent deformation from the allowed inelastic

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