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Development of a ductile steel bridge substructure system

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ABSTRACT

Described in this paper is the evaluation of a series of design concepts which attempt to improve the inelastic cyclic response of steel bridge substructures. The bridge system under consideration consists of hollow circular steel piles welded to steel cap beams. Described first is the motivation for the use of this type of structure, followed by a discussion of the research methods which include large scale reversed cyclic testing supplemented by finite element analysis. Next, the performance of the current as-built system, the fillet welded connection, is evaluated. This connection is shown to perform poorly with little inelastic deformation capacity prior to failure. A variety of alternative connections are then proposed and evaluated. These alternative connections include modified weld detailing and plastic hinge relocation approaches. Alternative weld detailing focuses on the complete joint penetration weld with reinforcing fillet welds. The plastic hinge relocation alternatives include a gusseted connection, a reduced column section, and the recently proposed grouted shear stud (GSS) connection. Alternative weld details produce only slight improvement in performance. Of the plastic hinge relocation concepts, the grouted shear stud (GSS) connection offers the most promising approach to improve inelastic cyclic response.

1. Introduction

Because of their rapid construction, bridge systems of the type shown in Fig. 1 have been used in the State of Alaska dating back to the early 1970s, thus forming an early example of accelerated bridge construction. The system consists of driven, circular steel pile-columns which are then field fillet welded to a steel cap beam which has traditionally consisted of a double HP-section. Current bridge design methodology intends for flexural hinging to develop in the column (or pile) elements of the substructure system in order to act as fuse links when subjected to a design level seismic event [1]. As a result, the connections of the hollow steel piles to the soffit of the cap beam must not only be able to develop the flexural strength of the pile, but must also be capable of withstanding considerable cyclic inelastic rotations to ensure a ductile system response. While such a bridge system is rapid to construct, there are concerns over the ability of the connection to be capacity protected against brittle failure, i.e., cracking in or near the welded region [2].

The sparse published research [3,4], points to limited ductility capacity of connections fabricated by directly welding hollow steel pipes to connecting members. Steunenberg et al. [3] tested a single steel pile welded to a steel plate anchored in a concrete block. The connection of the pile to the plate was accomplished with a complete joint penetration weld placed in the overhead position to simulate field conditions. Although this specimen was able to develop hinging in the form of pipe wall local buckling, which mitigated connection cracking, the force–displacement response indicates that this connection is of limited ductility capacity. Because only one specimen was evaluated, there was no attempt at improving performance. Nishikawa et al. [4] sought to prolong the life of circular steel columns by controlling the growth of outward local buckling. This was to be achieved by placing an outer reinforcing ring around the column with a specified gap. The gap was intended to ensure that the outer ring provided no strength or stiffness to the pipe until buckling occurred. The experimental results showed that this retrofit was moderately successful in that a slight increase in post-buckling ductility capacity over the non-retrofitted specimen was observed. The important result is that connection cracking did not occur prior to pile wall local buckling. While the two studies just described are of limited applicability to the scope of the research discussed in this paper, both did indicate that basic welded connections may be capable of precluding connection fracture and developing relatively stable pile wall local buckling.

The research described in this paper represents a six year period of study that contained several phases and goals. This paper describes these phases of study, and the development accomplished at each step, which led to several options for the design of circular pipe to cap beam connections in bridge systems. The first portion of this paper discusses the research methods including test setup, instrumentation, and loading protocol, as well as an overview of the finite element analysis. The second portion examines the behavior of the as-built, fillet weld

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Fig. 1. Driven steel pile bridge pier.
Courtesy of AKDOT.

connection and a discussion of alternatives that were considered which focused on weld detail improvement. The third portion focuses on the use of plastic hinge relocation methods to design these bridge systems. The most desirable of these approaches, the grouted shear stud (GSS) connection, has been discussed by the authors elsewhere [5–8], and more recently in this journal [9]. Therefore, only a brief description of the GSS connection is provided in this paper. The final section of the paper presents research conclusions and design recommendations. The research has been accomplished through a combination of large scale experimental testing and finite element analysis.

2. Research methods

2.1. Test specimen and laboratory setup

The primary goal was to model as accurately as possible a typical steel bridge pier for the evaluation of connection behavior. While the test specimens were modeled after those typically used in Alaska, the results of the evaluation are generally applicable to similar bridge piers irrespective of geographic location, notwithstanding the corresponding seismic hazard. The use of full scale, two pile pier specimens ensured that the axial forces due to overturning and proper boundary conditions were captured. Although laboratory limitations were present, an attempt was made to minimize the influence of these limitations.

An important aspect of the specimen design was the coordination with Alaska Department of Transportation and Public Facilities (AKDOT) engineers to ensure that the design was representative of their existing steel bridge inventory. Table 1 contains a sample of this steel bridge inventory. As shown in Table 1, pile heights range from 10 to 20 ft. (3.05 to 6.10 m) above grade, and the pile diameters vary between 12 and 30 in. (305 and 762 mm). Taking into account laboratory limitations, and the use of pinned based supports to model the point of inflection that would exist in the actual system when subjected to double bending, the target test pile height was nominally 12 ft. (3.66 m), corresponding to a field pile height of approximately 24 ft. (7.32 m). ASTM A500 Grade B & C (dual certification) piles with a diameter of 16 in. (406.4 mm) and a wall thickness of ½ in. (12.7 mm), designated as HSS16x0.500 (HSS406.4x12.7), were used. The resulting D/t ratio was 32, within the range of AKDOT practice.

The design of the test specimen cap beam was controlled by capacity design principles. In order to ensure that flexural hinging occurred at the tops of the piles, other failure modes such as beam hinging, flange bending, etc. had to be capacity protected. From the design calculations detailed by Cookson [10] and Fulmer et al. [11], a double wide HP14x89 (HP360x132) cap beam fabricated from ASTM A572 Grade 50 steel was found to remain elastic when subjected to the anticipated overstrength demands of the expected pile hinging mode of failure. Tests in later phases of this research utilized a double HP14x117 (HP360x174) cap beam because of the larger cap beam forces generated by flexural hinge relocation. To mitigate flange bending, full depth transverse stiffeners were placed at the location of the extreme fibers of the HSS piles. The design resulted in the specimen and laboratory setup depicted in Figs. 2 and 3. In Fig. 3, note the red frames installed on both sides of the test specimen, each fitted with adjustable channel outriggers. When positioned against the webs of the cap beam, the outriggers with attached end rollers provided lateral support to the test pier as cyclic loading was applied. It was observed during initial tests that the lateral frames were not required, and were eliminated in subsequent tests.

2.2. Lateral loading of test specimens

Quasi-static, cyclic lateral loading was applied by either a 220 kip (979 kN) or 440 kip (1960 kN) MTS servo-controlled actuator, each with a total stroke capacity of 40 in. (102 cm). When the actuator was installed at its neutral position, this accommodated 20 in. (50.8 cm) of stroke in the push and pull directions. The applied load history is termed the three cycle set history, described in detail elsewhere [9] and briefly here as follows. This load history begins in a force-controlled mode in which the load is applied in single reverse cyclic increments, up to the first yield force, in increments of ¼ of the first yield force. Following first yield, the loading is applied in displacement controlled three cycle sets of displacement ductility increments. Displacement ductility μ_Δ, is defined as the imposed displacement divided by the idealized yield displacement. The idealized yield displacement is defined as the experimental first yield displacement multiplied by the ratio of the nominal strength M_p divided by the strength at first yield M_y. The ratio M_p/M_y is essentially a constant value of 1.3 for pipe piles with reasonable diameter to thickness ratios, i.e., $D/t < 50$. Increasing levels of displacement ductility were imposed in the order of 1, 1.5, 2, 3, 4, 6, etc. until failure. Failure was defined as a sudden rupture accompanied by a corresponding sudden drop in strength, i.e., brittle failure, or as a more gradual but significant strength loss as cyclic loading progressed.

Table 1
Sample of AKDOT steel bridge pier inventory.
Courtesy AKDOT.

Bridge	Weld type	Weld size, in. (mm)	Pile dia., in. (mm)	Wall thick., in. (mm)	Pile ht. above ground, ft. (m)	Number of piles per bent	Cap beam	No. of spans	Span length, ft. (m)
208	Field fillet	0.25 (6.4)	12 (305)	N/A	10 (3.0)	4	HP14x73 (HP360x108)	3	75 (23)
1196	Field fillet	0.25 (6.4)	12 (305)	0.833 (21.2)	14 (4.3)	4	HP14x73 (HP360x108)	3	33 (10)
1754	Field fillet	0.75 (19)	30 (762)	N/A	16.5 (5.0)	4	2W36x282 (2W920x420)	3	50 (15)
1820	Field fillet	0.375 (9.5)	16 (406)	N/A	20 (6.1)	4	2HP10x57 (2HP250x85)	3	35 (11)
1136	Field fillet	0.375 (9.5)	16 (406)	0.5 (12.7)	10 (3.0)	2	2HP14x89 (2HP360x132)	1	80 (24)
1945	Field fillet	0.3125 (7.9)	20 (508)	0.625 (15.9)	20 (6.1)	3	2W18x35 (2W460x52)	23	30 (9.1)
1714	Field fillet	0.375 (9.5)	12 (305)	0.375 (9.5)	N/A	2	W24x84 (W610x125)	1	74 (23)