1. Executive Summary

The test 4 specimen consisted of a composite connection configuration utilizing shear stud connectors placed on the outside of the pile section which was inserted into a larger stub pile section that contained a matching pattern of shear studs as shown in Figure 4. The annular space formed between the two components was skewed to one side in an effort to evaluate the performance of the system with the maximum possible construction tolerance offset in the connection corresponding to a magnitude of ±1”. The annular space was filled with flowable grout allowing for composite action to develop the moment connection by transferring forces from the pile to the cap beam via the stub pile, while avoiding inelasticity in critical welded regions. The results of experimental testing showed the connection capable of successfully developing a flexural hinging mode of failure in the form of pile wall local buckling at the intended region regardless of the construction tolerance offset. All regions which were considered to be in critical need of remaining elastic were shown through data analysis to remain within the intended elastic range as the full flexural hinging capacity of the piles developed. Ultimately, the limit state of pile wall local buckling was shown to control at a reliable displacement ductility level of 3 associated with 8.27” of displacement or 8.1% drift. The behavior of the system as well as the performance of the connection has also been verified with FEM simulations.
2. **Introduction**

The fourth test of this research project was aimed at evaluating the capability of a grouted shear stud composite connection to effectively form flexural hinging in the pile section. The primary difference between this test and the prior similar test was the presence of construction tolerance as the piles were offset by nominally ±1” which corresponded to the maximum possible magnitude for this scale of the connection. The configuration which was developed and shown to perform well in test 2, consisted of a 24”x0.500” stub pipe pile section that was connected to the cap beam by a complete joint penetration weld (CJP) with a 3/8” reinforcing fillet weld. The inner diameter of the stub pile contained 12 lines of welded 3/4” diameter 2-1/2” long shear stud connectors located at 30˚ on centers around the pipe with 4 shear studs in a given line. Similarly the top of the HSS16x0.500 pile section had 12 lines of 4 matching shear studs welded at 30˚ on center around the cross section offset by 15˚ radially and 2-1/2” vertically from the studs inside the larger stub pile. In the case of test 4, a maximum possible construction tolerance offset of the connection was provided during construction to evaluate the effect of a minimal shear stud overlap at an extreme fiber the connection. The offset shear stud pocket, shown in Figure 1, was grouted utilizing non-shrinking flowable grout to complete the moment resisting connections of the bent as shown in Figure 2. Further design and construction details are provided in subsequent sections of this report.

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**Figure 1. Offset Annular Shear Stud Pocket**
3. Bent and Connection Design

Similar to past configurations, ASTM A500 Gr. B HSS16x0.500 piles were chosen as the column elements of the bent to produce reasonable aspect and D/t ratios. The construction of the cap beam consisted of double ASTM A572 HP14x117 sections to provide both a capacity protected cap beam as well as adequate bearing seat width for single span girders, should a designer choose not to utilize continuous spans. A 24”x0.500” pipe section manufactured to the material standards of ASTM A500 Gr. B was utilized as the stub pile element to provide an adequate gap for the placement of shear studs as well as to accommodate construction tolerances.

As was the case in test 2, the design of the composite connection was based on the assumption that the total nominal strength of the shear stud connectors, on the pile side, should be capable of developing yielding of the HSS16x0.500 gross cross section. From known, or anticipated, material properties a required number of 3/4” diameter shear studs could be determined and distributed around the cross section in an even pattern. A matching number of studs were then placed on the stub pile side to facilitate load transfer in a strut and tie mechanism between the studs on either side of the connection. The nominal capacity of a single shear stud was determined utilizing the provisions of “AASHTO LRFD Bridge Design Specifications” (AASHTO, 2007) Section 6.10.10.4.3 as well as the ANSI/AISC 360-05 (AISC, 2005) Section I3 both of which
provide the same model shown in Eq. 1. As indicated in Eq. 1, the model is a function of concrete compressive strength as well as the cross sectional area of the shear stud with an upper bound of stud shear failure. Although the model is intended for use (in both codes) for composite construction between beams and a slab or bridge deck, it has been assumed conservative for use in this design given the highly confined nature of the annular grout pocket.

\[
Q_n = 0.5A_{sc}(f'cE_c)^{0.5} \leq A_{sc}F_u \quad \text{Eq. 1a (ksi)}
\]

\[
E_c = 1746(f'c)^{0.5} \text{ AISC}, 1820(f'c)^{0.5} \text{ AASHTO} \quad \text{Eq. 1b (NWC)}
\]

Utilizing this model along with expected material properties of the grout and specified material properties of ASTM A108 shear stud connectors, it was found that a minimum of 43 shear stud connectors were necessary based on an anticipated yield stress of 54 ksi for the HSS16x0.500 ASTM A500 Gr. B piles. To generate a symmetrical condition, 12 lines of 4 shear studs at 30° on centers were utilized providing 48 shear studs on the pile side or 96 total per connection as shown in Figure 3. The details of the connection are provided in Figure 4 where the 1” (nominal) construction tolerance offset is also shown. It should also be noted that the 24”x0.500” stub pile section, acting non-compositely at the cap beam connection, was designed to remain elastic at the full flexural over strength capacity of the piles.

![Figure 3. Pile Side Shear Studs (48 per pile)](image)

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4. **Construction Process**

Welding of the cap beam elements, stub piles to the cap beam, and shear stud connectors was conducted by the fabricator to eliminate the necessity of any field welding. Hence, only grout placement was necessary to complete the connection at the laboratory. To ensure that the maximum possible construction tolerance offset was provided, the cap beam was placed on the piles such that the ends of the shear stud connectors located on the north extreme fibers of each pile were in contact with the inner wall of the stub pile section as shown in Figure 5.

The selection of a grout product was focused on adequate workability as well strength capacity and as a result an extended working time non-shrinking grout product, BASF Masterflow® 928, was chosen. Emphasis was also placed in the selection process to ensure that a widely available material was selected. To assist in minimizing the possibility of air voids within the annular grout...
pocket, the decision was made to pump grout vertically (as in test 2) from the bottom of the connection to the top where 1-1/6” diameter holes had been left in the cap beam flange to allow air to escape the connection. A hand operated pumping system was utilized along with shut off valves that were cast into place and later removed to facilitated pumping of the grout as shown in Figure 6 and Figure 7. In addition to the pocketed connections, 4”x8” cylinders were also cast for testing to ensure a minimum compressive strength of 5-6 ksi was obtained by the grout prior to testing of the specimen.

Figure 5. North Side Connection Offset
Figure 6. Grout Pumping System

Figure 7. Cast In Place Shut-Off Valve
5. **Load History and Instrumentation**

The test specimen was laterally loaded using a 440 kip MTS hydraulic actuator at North Carolina State University’s Constructed Facilities Laboratory as shown in Figure 8. The applied load history, termed a three cycle set history, consisted of an initial elastic portion based on the anticipated yield force of the system and a second section based on the experimentally determined yield displacement of the system. More specifically, single reverse cyclic load controlled cycles of ¼ first yield force increments were applied up to the first yield force which is determined in accordance with Eq. 2. In Eq. 2, S represents the section modulus of the HSS 16x0.500 pile, $f_y$ represents the anticipated yield stress of the pile material, and X represents shear span from the pinned supports to the critical section. In the case of test 4, the average material yield strength was found to be 56.5 ksi for the A500 Gr. B pile material based on in-house material testing. This value, combined with the shear span of 8.65’ from the pinned supports to the base of the composite connection, resulted in a first yield force of 93.3 kips.

The second section of the load history was defined by displacement controlled incremental ductility levels where ductility 1, or effective yield, is defined by Eq. 3 and subsequent displacement ductility levels are defined by Eq. 4. In Eq. 3, $\Delta'_y$ represents the experimentally determined first yield displacement while $M_p$ and $M_y$ represent the full plastic moment capacity and the first yield moment capacity of the pile section respectively. In the case of test 4, the experimentally determined first yield displacement of 2.11” resulted in a ductility 1 displacement of 2.76” nominally identical to that of test 2 which was 2.82”. The load and displacement histories resulting from this testing method are provided in Figure 9 and Figure 10 respectively.

![Figure 8. Laboratory Set-Up](image-url)
\[ F_y = 2(S)(f_y)/(X) \quad \text{Eq. 2} \]
\[ \Delta_y = (M_p/M_y)\Delta' = \mu_1 \quad \text{Eq. 3} \]
\[ \mu_i = i(\mu_i) \quad \text{Eq. 4} \]

Figure 9. Test 2 Load History
The instrumentation utilized in test 4 consisted of traditional laboratory instrumentation as well as the Optotrak motion sensing system which tracks the location of LED markers. The traditional equipment consisted of inclinometers located 8” above the center line of the pinned bases to monitor drift magnitudes, linear string potentiometers attached to the bases to monitor any unanticipated base sliding, and strain gauges located inside each pile on the extreme fibers as shown in Figure 11. Locating the strain gauges inside the pile allowed for monitoring of the strain gradient along the length of the connection throughout the test. As was mentioned, in addition to the traditional instrumentation, a 2” spaced grid of Optotrak LED markers was placed on the east face of each pile in both the composite connection and critical pile regions as shown in Figure 12. Post processing of the local deformation data recorded by the Optotrak system allows for additional strain calculations over a larger area and at higher levels of strain than is allowed by the traditional electric resistance strain gauges.

Figure 10. Test 2 Displacement History
6. Experimental Summary

The results of experimental testing showed the specimen to perform in an adequate manner with no appreciable signs of damage that could be related to strength loss or any upper bound limit state throughout the second ductility level as is shown in Figure 13 and Figure 14. However, minor signs of inconsequential damage were experienced as early as the negative ¼ yield cycle where small cracks were observed at the base of the grout pocket of the south column as shown in Figure 15. Throughout the remaining elastic cycles as well as the ductility 1 and 2 cycles, more small cracks developed in addition to the formation of small gaps between the grout block and the pile and/or stub pile walls on the tensile faces of each column.
Figure 13. Test 4 Force-Displacement Hysteresis

Figure 14. Test 4 Force-Displacement Envelopes
The first signs of the development of a critical limit state occurred in the second negative cycle of the ductility 2 loading stage where a small magnitude of pile wall local buckling began to develop on the south side (compression face) of the north pile along with a small amount of slip between the grout block and the stub pile wall as shown in Figure 16. Progressing into the third positive cycle of the ductility 2 loading stage led to the development of a similar magnitude of local buckling on the north face of the south pile. This observed response is similar in nature to that of test 2 where minor amounts of local buckling developed in the third positive and negative cycles of ductility 2. As in test 2, the observed local buckling at this stage of the load history was not significant enough to produce any noticeable strength loss in either the full force-displacement response or the force-displacement response envelopes. In addition to the local buckling, grout spalling below the first level of shear studs was experienced in the ductility 2 level but appeared to be inconsequential to the response of the system.

Figure 15. South Pile, -Fy Cycle, -23.3 kips, -0.57 in, Small Cracks
Figures 16. North Pile South Face, Ductility 2 Cycle -2, -128 kips, -5.52 in

Progressing into the first positive and negative cycle of ductility 3, 8.27” of displacement as shown in Figure 17, led to an increase in the magnitude of local buckling on both piles but again no recognizable strength loss in the system. However, the second and third cycles of the ductility 3 level produced propagation of the pile wall local buckling which consequently led to strength reductions of approximately 6% and 13% respectively. Additionally during these cycles, grout located below the first row of shear studs began to spall as shown in Figure 18.

As the test progressed into the ductility 4 cycles (11.03” of displacement as shown in Figure 19), intense local buckling developed inducing strength reduction magnitudes of approximately 25% to 40% throughout the ductility level. Although the magnitude of the local buckling increased in this ductility level as shown in Figure 20, the formation of small cracks which were noted to develop in the pile wall in test 2 at this stage of loading were not noted in this test. In addition to the propagation of buckling, continued spalling of grout below the first level of shear studs was experienced as can also be seen in Figure 20. At the completion of the ductility 4 cycles, the damage as well as the associated strength loss, was significant but the bent remained in-tact and did not exhibited any pile wall cracking. The test was therefore continued to a ductility level of 6.
Figure 17. Ductility 3 Cycle 1, 130 kips, 8.27 in

Figure 18. Ductility 3 Cycle -3, -119 kips, -8.27 in
Figure 19. Ductility 4 Cycle 1, 105 kips, 11.03 in

Figure 20. Ductility 4 Cycle 3, South Pile North Face, 87 kips, 11.03 in
As the test specimen was loaded to the first positive cycle of the ductility 6 level, associated with 16.55” of displacement as shown in Figure 21, small cracks began to develop on the north face of the south column at the location of local buckling. Loading reversal towards the first negative ductility 6 displacement led to cracking through the pile wall on the north face of the south column as well as the south face of the north column. Interestingly, as shown in Figure 22, the severity and orientation of the buckled pile wall induced fracture along the seam weld of the pile. Although this was inconsequential to the performance of pier due to the severe strength degradation which had already occurred, this cracking mechanism had not been seen in any past test. At this point, approximately 60% strength loss had been experienced along with considerable cracking so the specimen was returned to a neutral position and the test was assumed to be concluded.

Figure 21. Ductility 6 Cycle 1, 67 kips, 16.55 in
As has been noted throughout this report, the test specimen performed in a manner which was nominally identical to that of test 2 which had no construction tolerance offset. For additional comparison of the global behavior of both tests, Figure 23 provides the force displacement hysteretic response from each test to allow for a direct assessment regarding the effects of the tolerance offset. As is shown, it appears from this test that the global response of the pier was unaffected by the minimum overlapping of shear connectors on the side of the connection with the largest annular space. In addition to this conclusion, the similarity of two tests can also be viewed as a validation of the behavior of the connection since no adverse effects due to the offset were noticeable.
Figure 23. Offset (Test 4) vs. Ideal (Test 2)

Following testing, Optottrak data analysis was conducted to evaluate the effectiveness of the connection configuration to reduce strain demands within the connection itself and appropriately relocate damage. As shown in Figure 24 through Figure 27, the connection did remain in the elastic range while large inelastic demands were forced below the bottom of the connection which terminated at 23” below the cap beam. As was intended in the design of the connection, a critical region developed immediately below the capacity protected connection on both extreme fibers of the south pile in both directions of loading. Strains in the approximate region of 30000-40000 (µε) were developed in this critical region prior to the formation of local buckling at the ductility 3 stage of loading. It should be noted that in some cases, such as Figure 24, the vertical strain profile appears to enter the positive range on a face that should be experiencing negative strains or vice versa. This condition is generated in a region where significant local buckling has occurred and the orientation of the Optottrak marker is no longer correctly indicative of engineering strains. Although not shown in this report for brevity, nearly identical response was exhibited by the north pile. Also, the strain elevations presented here are nearly identical to those of the presented in the test 2 summary report again indicating little or no effects were experienced due to the tolerance offset.
Figure 24. Test 4 – South Column North Face – Positive Cycle 1 Vertical Strain Profile

Figure 25. Test 4 – South Column South Face – Positive Cycle 1 Vertical Strain Profile
Figure 26. Test 4 – South Column North Face – Negative Cycle 1 Vertical Strain Profile

Figure 27. Test 4 – South Column South Face – Negative Cycle 1 Vertical Strain Profile
In an effort to quantify the behavior of the connection throughout its length, internal strain gauges were placed inside the pile at the extremities of the section prior to construction as shown in Figure 28 and as has been discussed in prior sections of this report. Post test data analysis produced the strain elevation presented in Figure 29 through Figure 32. The strain elevations presented indicate a relatively linear strain gradient prior to the large strain accumulation which occurs at the base of the connection as local buckling develops. Also it is shown, as expected, that the top strain gauge in all cases provides a reading close to zero since by design full force transfer should have taken place at this point in the connection. Given the relatively linear nature of the strain gradient, it is not immediately apparent that a reduction in the number of shear studs (or overall size of the connection) is warranted as may be the case if the majority of the length was experiencing virtually zero strain. However, it is possible that should the number of shear stud connectors be reduced, higher demands on the studs further into the connection would generate larger strains higher into the connection while the overall behavior of the system remains un-altered. An analytical or experimental study with a reduced number of shear studs would be required to verify this possibility.

Figure 28. Internal Strain Gauges
Figure 29. Test 4 – North Column North Face Connection Strain Elevation

Figure 30. Test 4 – North Column North Face Connection Strain Elevation
Figure 31. Test 4 – North Column South Face Connection Strain Elevation

Figure 32. Test 4 – North Column South Face Connection Strain Elevation
7. Finite Element Analysis Verification

In an effort to verify the global behavior of the pier and the formation of local buckling within the pile section as well as to better understand the behavior of the connection, finite element modeling of the pier and connection was conducted. No construction tolerance offset was considered in the model such that the results are applicable to both tests 2 and 4. The model utilized 4-node shell elements as well as 20-node solid brick elements along with a non-linear combined isotropic/kinematic hardening material model which considered expected stress strain behavior of the ASTM A500 Gr. B pile material. For simplicity the grout material was modeled with elastic material behavior and the resulting analytical errors will be discussed where applicable. Due to the harsh geometric conditions required to directly model the shear studs, connection between the grout and pipe elements was achieved by utilizing fastener definitions to join appropriate areas of each mesh. The model, shown in Figure 33, was subjected to a quasi-static displacement history matching that of the test 2 specimen (nearly identical to that of the test 4 specimen shown in Figure 10).

![Figure 33. Abaqus FEM Pier Model](image)

Similar to both experimental specimens, the FEM simulation began to develop local buckling of pile wall below the base of the connection at the ductility 2 displacement level as can be seen in Figure 34. In addition to this similarity, the simulation also indicates tensile strains in the pile wall prior to buckling in the range of approximately 25000-35000 (µε), shown in Figure 35,
comparable to that of the experimental evaluations which indicated approximately 30000-40000 (µε). As was the case with the experimental evaluation, the buckling which developed in the simulation at the ductility two level increased in magnitude throughout the ductility 3 and 4 cycles shown in Figure 36 and Figure 37 respectively. As expected, the propagation of buckling initiated a loss in the strength capacity of the system resulting in a hysteretic response similar in nature to that of the experimental specimen as shown in Figure 38.

Figure 34. Minor Local Buckling of Pile at Ductility 2
Figure 35. Longitudinal Strain Prior to the Development of Buckling

Figure 36. Propagation of Local Buckling
Figure 37. Propagation of Local Buckling

Figure 38. FEM vs. Experimental (Test 2)
In an effort to quantify the behavior the grouted connection, Von Mises stresses within the grout section are shown in Figure 39 and Figure 40 at a ductility level of 2 as this corresponds to the maximum moment demand placed on the connection. It should however be noted that it would be inappropriate to draw conclusions regarding the exact values of the Von Mises stresses since the grout block is modeled with elastic material properties which develop stresses much higher than that of the capacity of the actual grout material. Hence, it is more appropriate view the results as a reasonable estimate of the actual stress condition by considering only the stress variations (i.e. stress gradient) within grout block while disregarding the actual values.

As shown in Figure 39, the grout block is most highly stressed at the top row of shear connectors on the pile stub side of the connection which is also a logical conclusion based on the design action. Similarly, Figure 40 indicates that the grout at the bottom row of shear studs is most highly stressed on the pile side of the connection which is logical in accordance with the design action as well as the experimental data produced from the internal strain gauges. Regardless of the fact that stress intensities are highest at the extreme ends of the connection (top outside and bottom inside), the simulation also indicates that considerable levels of stress are also being developed beyond the extremities at the other rows of shear studs. In addition, reasonable levels of strain are shown to develop by the simulation in the pile section within the connection on both the compressive and tensile faces of the pile as shown in Figure 41 through Figure 43. The magnitudes of strain provided by the simulation are generally below the yield strain of the pile material again consistent with the data produced from the internal strain gauges. Although these results are not fully conclusive and are based in part on some simplifying modeling assumptions, they seem to indicate that a reduction in the number of shear connectors could produce undesirable results.

Figure 39. Von Mises Stresses in Grout Block at Ductility 2
Figure 40. Von Mises Stresses in Grout Block at Ductility 2

Figure 41. Longitudinal Pile Strains at Ductility 2
Figure 42. Longitudinal Pile Strains with the Connection above the First Row of Shear Stud Connectors

Figure 43. Longitudinal Pile Strains with the Connection above the First Row of Shear Stud Connectors
8. General Conclusions

The physical results of experimental testing, as well as the results of data analysis, indicate that the composite connection design considered in this test was effective in producing a desirable failure mode in the form of flexural hinging of the HSS16x0.500 pile wall regardless of the construction tolerance offset. In addition, the results of the FEM simulation also support the conclusion that flexural hinging of the pile is the controlling failure mode of this system neglecting any construction tolerance offset. Consequently, undesirable failure modes such as weld cracking or tearing of the pile wall prior to local buckling appear to be avoidable for this connection configuration. As was the case in test 2, some damage did develop within the grout block at the base of the connection in the form of cracking and spalling, though no adverse consequences were induced on the structural response due to these actions. Although these actions may be considered as low level response limit states such as a serviceability limit state (hence some repair after seismic loading would be necessary) they would not constitute an ultimate limit state. As has been noted, the ultimate limit state in this case would be considered pile wall local buckling likely corresponding to a reliable displacement ductility level of 3 depending on the exact allowable level of strength loss. In regards to all aspects of the response which have been considered in this report, the construction tolerance offset produced no adverse effects as the specimen performed in a nominally identical manner to that of the test 2 specimen with no construction tolerance offset.

Taking into account the results produced by the strain gauges placed within the connection region as well as the results of the FEM simulation, it is not immediately apparent that a reduction in the number of shear connector or the overall size of the connection should be reduced. Hence, the design methodology of developing the axial yield capacity of the pile to determine the required number shear stud connectors appears to be adequate and no optimization options are immediately evident. However, should a more direct investigation relating to optimization of the design methodology may produce results that suggest otherwise.

9. Project Direction

From the results of test 2 and 4, it is evident that controlling the propagation of local buckling could enhance the performance of the system by helping to maintain post buckling strength. One proposed method to achieve this goal is shown in Figure 44 which provides the details of a buckling restrained grouted shear stud connection which is scheduled as test 5 of this research project. As shown, the connection is identical to that of the basic grouted shear stud connection with the exception of being 10” longer than required by the design methodology. During construction a small annular block-out will be provided around the HSS16x0.500 pile wall through the additional 10” of length such that the pre-buckling response of the system will be unaltered. Following the initiation of buckling, the intent of the system is to allow the pile wall to contact the surrounding grout block which in turn will restrain the propagation of buckling and corresponding strength loss.
FEM simulation of the system has shown the design concept to be of potential value provided the appropriate gap dimensions are utilized. Currently, a parametric FEM study is being conducted in an effort to determine suitable gap dimensions. Should the gap be too large the method will be ineffective as a large magnitude of buckling and ovilization will be required for the pile wall to contact the grout block. Conversely, should the gap be too small the pile wall would contact the base of the block-out due to typical flexural displacements. Currently it has been found that a gap size of ½” produces reasonably effective results. As shown in Figure 45 through Figure 47, a ½” gap size does not alter the pre-buckling response of the system but begins to restrain the propagation of buckling and control strength degradation at the third ductility level. Further analysis will be conducted as necessary to determine an appropriate detailing scheme.
Figure 45. Standard vs. Buckling Restrained (1/2” Gap) FEM Response

Figure 46. Buckling Restrained Grouted Shear Stud Simulation (1/2” Gap)
In addition to the buckling restrained grouted shear stud connection, the current project directions also includes plans for evaluation of a truss style system as well as shake table testing of scaled grouted shear stud connections. The following sequence of testing is proposed:

Test 5: buckling restrained grouted shear stud connection

Test 6: buckling restrained grouted shear stud connection with additional axial load

Test 6: grouted shear stud connection with additional axial load

Test 7-9: → truss system evaluation

→ shake table testing of grouted shear stud connection

→ shake table testing of buckling restrained composite shear stud connection
10. References
