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Ultra-High Performance Concrete: An Advanced Solution for Accelerated Bridge Pier Cap Rehabilitation and Bearing Replacement

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Abstract

The objective of this paper is to propose and illustrate feasibility, approach, validation as per prevailing codes and design standards, specification to rehabilitate bridge pier cap by using Ultra-High Performance Concrete (UHPC). The evaluation of existing pier caps and bearings indicates that the complete removal of existing bearing is undesirable due to 1) massive size of bearing 2) difficulty in cutting through the thick components of existing bearing 3) deeply anchored lower shoe of existing bearings 4) huge cost and time required to erect temporary support system for superstructure to facilitate the construction of new piers. To overcome these difficulties, UHPC could be cast around the lower shoe up to the existing bearing pin. This UHPC cast could be used to support jacks and temporary bearings. The new low height permanent bearing could then be installed after removing the upper shoe of the existing bearing. In the present research, first properties of UHPC are summarized followed by evaluation of case studies to check feasibility of the solution to rehabilitate pier cap by using UHPC. The complex load paths in pier cap are idealized by using validated strut and tie model as per prevailing AASHTO LRFD Bridge Design Specification.

Keywords

Bridge, Pier Cap, Rehabilitation, Composition, Bearing, Strut, Tie, AASHTO

1. Introduction

There are 614,387 bridges in the National Bridge Inventory in US. Almost 39% of these are over 50 years or older. Additional 15% of the bridges are between the ages of 40 and 49 years. On an average, the bridges in US are 43 years old. As per report, in the year of 2016 there were 9.1% bridges which were found to be structurally deficient. The estimated cost to fix these bridges was pegged at $123 billion [1]. The bridge damages were found to be to the deck area, bearings, piers and pier cap. Given that the enormous cost involved in rehabilitation, there is a need to have reliable, durable long-term solutions for bridge component rehabilitations while addressing constraints on dimension and aesthetics.

In the present case study, the bearing and pier cap need replacement. It is proposed to build the shell column around the existing columns. The existing pier is 10.5 ft × 11 Ft. The existing pier cap 11.5 ft × 12.5 ft with 6 in offset on all faces of the pier. The details of the representative pier cap are shown in Figure 1. The lower shoe of the bearing is anchored deep into the pier cap. The dimensional details of fixed bearing are shown in Figure 2. The dimensional details of the expansion bearing are shown in Figure 3. The upper and lower shoes are connected by pin. The objectives here are to study the feasibility of using UHPC for pier cap replacement, to perform preliminary analysis and design of pier cap and to review advantages, challenges in constructability.

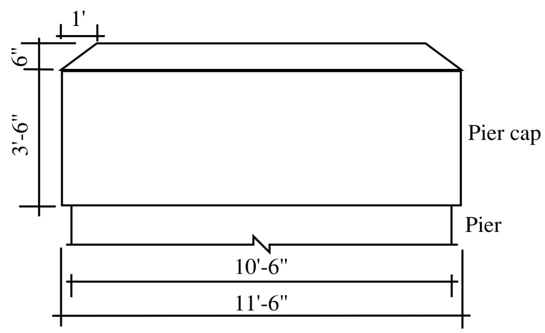

Figure 1. Representative pier cap dimension.

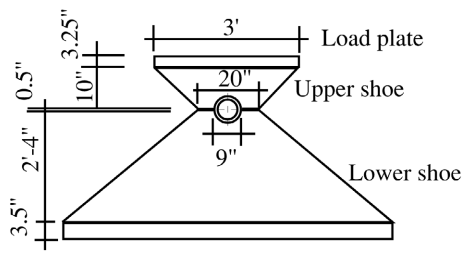

Figure 2. Fixed bearing.

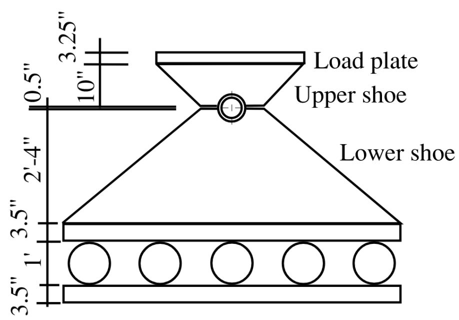

Figure 3. Expansion bearing.

The complete removal of fixed bearings is, though not impossible, difficult due to 1) massive size 2) difficulty in cutting through the thick components of existing bearing 3) anchorage of lower shoe into column. The rehabilitation of the pier cap and replacement of the bearings will require significant top portion of existing column to be demolished. All these constraints rendered to the solution where existing piers could entirely be demolished or make them redundant by creating alternate load path through shell column around them. If the column is entirely demolished, then pier cap could be constructed with new selected grade of concrete which could provide material continuity as well as load path continuity. In case, where existing piers could not be demolished, the alternative solution is to embed the lower portion of bearing in new pier cap concrete. The new low height bearing could be installed on top of this new pier cap. This condition leads to the situation where geometric as well as material non-linearity would be encountered. The partial load transfers to the existing column could not be avoided. In the long run, there exists the possibility of damage to the existing columns and pier cap. This makes it imperative to construct the new pier cap which would continue to serve its intended purpose in the event of damage and settlement of the existing column and pier cap while maintaining the geometry. This new pier cap should be able to transfer the load from superstructure to the shell column constructed around the existing pier. The new pier cap should be durable and should continue to provide load path to maintain the stability of the superstructure. The Ultra-High Performance Concrete which has been proven to be durable, stronger and far better in terms of mechanical and engineering properties compared to conventional concrete, becomes a natural choice for pier cap. The new pier cap could be used to support jacking assembly during bearing replacement stage.

2. Literature Review: UHPC Properties

The UHPC is relatively new cementitious material with mechanical and engineering properties far superior to conventional concrete [2]. The UHPC has compressive strength above 21.7 ksi. The pre-and post-cracking strength is above 0.72 ksi. Being denser and less porous, UHPC exhibits enhanced durability compared to conventional concrete. Based on research and development over the period of 2 decades, many articles and reports were published confirming enhanced engineering and material properties. The main constituents of UHPC are portland cement, fine sand, silica fumes, steel fibers, water reducing admixtures, water. The water cement ratio for UHPC is less than conventional concrete. The proportion of constituent materials varies depending on application and supplier. The typical composition of various constituent materials is shown in **Table 1** [2] [3].

Finally, In case of bridges UHPC has been used as joint fills, overlays, pier jacketing and innovative solutions for rehabilitation. The higher strength for UHPC mix proportion of Compact Reinforced Composites (CRC) in excess of 30 ksi on

2 inch cube was achieved by optimizing cementitious matrix, packing density, using high strength fibers. The composition is shown in **Table 2** [2] [4]. Similarly, the UHPC mix proportions by Teichmann and Schmidt is shown in **Table 3** [5] and by researchers at U.S. Army Corps of Engineers is shown in **Table 4** [6] [7]. From **Tables 1-4** it could be concluded that the proportions of constituents are approximately similar. The water to cement ratio for Ductal is around 0.25, for CRC 0.22, for Teichmann and Schmidt is 0.19 to 0.21 and for US Army Corps Corp-Tuf is 0.21. For normal weight conventional concrete, the water cement ratio is 0.4 to 0.7.

Table 1. Typical composition of Ductal.

Material	Amount (kg/m^3 (lb/yd^3))	Percent by Weight
Portland Cement	712 (1200)	28.5
Fine sand	1020 (1720)	40.8
Silica Fume	231 (390)	9.3
Ground Quartz	211 (355)	8.4
Superplasticizer	30.7 (51.8)	1.2
Accelerator	30.0 (50.5)	1.2
Steel Fibers	156 (263)	6.2
Water	109 (184)	4.4

Table 2. UHPC mix proportion of CRC by weight.

Material	Proportions
Portland Cement	1.0
Fine sand	0.92
Silica Fume	0.25
Glass Powder	0.25
HRWR	0.0108
Steel Fibers	0.22 to 0.31
Water	0.18 to 0.20

Table 3. UHPC mix proportions from Teichmann and Schmidt.

Material	Mix 1		Mix 2	
	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3
Cement	1235	733	978	580
Silica Powder	388	230	298	177
Fine Quartz 1	308	183	503	131
Fine Quartz 2	0	0	848	325
HRWR	55.5	32.9	56.2	33.4
Sand	1699	1008	597	354
Basalt	0	0	1198	711
Steel Fibers	327	194	324	192
Water	271	161	238	141
Water-Binder Ratio	0.19	0.19	0.21	0.21

Table 4. UHPC mix proportions of Cor-Tuf by weight.

Material	Proportions
Portland Cement	1.0
Fine sand	0.92
Silica Fume	0.25
Glass Powder	0.25
HRWR	0.0108
Steel Fibers	0.22 to 0.31
Water	0.18 to 0.20

The mechanical and engineering properties of UHPC were researched by various researchers for various curing conditions, specimen sizes, rate of loading, function of time tested after casting, biaxial compressions testing. The test conducted by Graybeal on 3 × 6 inch test cylinders cured by steam and tested at various times after casting has yielded compressive strength in the range of 18.3 ksi to 28 ksi [3]. Density of UHPC was maintained between 150 lb/ft^3 to 156 lb/ft^3. Graybeal also compared compressive strength of cylindrical specimen vs cubical specimen. Cubical specimen showed 5% higher strength than cylindrical specimen. Rate of loading found to have no effect on compressive strength modulus of elasticity and Poisson's ratio. The variation in compressive strength was also reported by Orgass and Klug [8] based on shape and size of specimens. Skazlic *et al.* [9] investigated the effect of cylinder size, and proposed the compressive strength based on their research. The available data indicates that initiation of strength gain and subsequent rate of strength gain depend on UHPC constituent materials, mix proportions and curing conditions [2].

The tensile strength study performed by researchers has indicated increased tensile strength for UHPC than conventional concrete. The idealized stress-strain response curve presented by Graybeal and Baby [10] indicates sustained tensile strength even after first cracking of UHPC. The sustained strength was attributed to the fibers content of UHPC. The flexure based test methods were proposed to determine the tensile response of fiber-reinforced concrete [2] [11] [12]. The tensile strength of UHPC was given as approximately 1.3 ksi for steam cured specimens and 0.9 ksi without any heat treatment [2] [3]. The modulus of rupture value for first cracking determined by ASTM C1018 ranged between 1.3 to 1.5 ksi based on method of steam curing, whereas an average value of 1.3 ksi was reported for untreated specimens [2] [13].

The modulus of elasticity in compression was reported to be 7250 ksi for steam cured specimen and 6200 ksi for standard laboratory conditions [3] [14]. The modulus of elasticity measured in direct tension test was reported to be 7500 ksi for steam-treated specimens whereas 6900 ksi for untreated specimens [2]. The Poisson's ratio by various researchers as reported, found to be from 0.16 to 0.2 [2]. Fatigue behavior, coefficient of thermal expansion, heat of hydration, bond strength, impact resistance, creep and shrinkage were also studied. Table 5 shows the range of various mechanical and engineering properties of UHPC [2].

Chloride induced corrosion in case of marine structures has been a major concern in reinforced concrete structures. The UHPC exhibits high durability and resistance to chlorides when exposed to sever environment as in case of marine structures [15]. In United States permeability of concrete is usually assessed as per the procedure outlined in AASHTO T 277 and ASTM C 1202 [2] [16]. The tests performed on UHPC indicate negligible chloride ion penetration [2]. Alkali Silica Reaction (ASR) in concrete results in expansion in the body of concrete. ASTM C 1260 outlines an accelerated ASR test procedure. A version of this test, modified for steam curing, performed on UHPC by Graybeal [17] found that the levels of magnitude of expansion were less than the threshold to sever environment as in case of marine structures [15]. The finding could be attributed to the fact that, the UHPC is less permeable compared to conventional concrete.

The UHPC in the present form became commercially available in United States in year 2000. The Federal Highway Administration (FHWA) started investigating the use of UHPC for highway infrastructure in 2001. The enhanced properties of UHPC have found solutions to many unique problems pertaining to highway bridge infrastructure. The UHPC has found its use in precast prestressed girders, precast waffle panels for bridge decks, joint material between precast concrete deck panels and girders etc. In Canada and Germany bridges have been built using UHPC component [2]. Though the initial cost of UHPC is high, and subjective to quantity ordered, distance to the site from its origin and many other factors, it is found to be more sustainable material with lower life-cycle cost compared to that of existing conventional concrete. The study performed by Piotrowski and Schmidt [13], shows that life cycle cost for Eder bridge replacement using UHPC would be less than that of conventional concrete. Higher strength of UHPC has been found to result in decreased component sizes and raw material for concrete [14]. The UHPC has been used in bulb-tee beams and girders [18] [19] [20] [21], waffle deck panels [22], joint between deck panels [23] [24], joint between deck bulb [25] [26], longitudinal and transverse joint between beams [27], joint fill between adjacent box beams and between precast curbs [2] [28]. One span in 10 span of Rout 624 bridge over Cat Point Creek in Richmond County, VA was built using UHPC [18]. A UHPC bridge using pi-shaped girders was constructed in Buchanan County, IA in 2008 [2] [29]. The other examples of UHPC use in bridges and components of bridge in Europe, Asia indicates the greater functional utility. In Slovenia, a bridge deck was overlaid with 1 to 1.2 inches of UHPC [30] and application in Switzerland include rehabilitation and widening of an existing bridge, protection layers to a crash and bridge piers, flooring of footbridges [31] [32]. The Sunyudo footbridge in South Korea, the longest span arch bridge, is made of pi-shaped precast post-tensioned UHPC sections [33].

Table 5. Range of UHPC material properties.

Property	Range	
Compressive strength	20 to 30 ksi	140 to 200 MPa
Tensile cracking strength	0.9 to 1.5 ksi	6 to 10 MPa
Modulus of elasticity	6000 to 10,000 ksi	40 to 70 GPa
Poisson's ratio	0.2	0.2
Coefficient of thermal expansion	5.5 to 8.5	10 to 15
	Millionths/0F	Millionths/0F
Creep coefficient1	0.04 to 0.30	0.2 to 0.8
	millionths/psi	millionths/MPa
Total shrinkage2	Up to 900	Up to 900
	Millionths	millionths

1 Depends on curing method and age of loading. 2 Combination of drying shrinkage and autogenous shrinkage and depends on curing method.

Based on literature review, it follows that the mechanical properties of UHPC are better than the conventional concrete. It also follows that UHPC has been used to solve many problems unique to bridges and transportation infrastructures. Few small bridges and components are made of UHPC. UHPC has found application in rehabilitation of various bridge components. Based on the research, UHPC could be proposed as a natural solution to the pier cap replacement problem. The major drawback of UHPC is high initial cost compared to conventional concrete which are in use for decades. Being recent addition in commercial space, there is lack of standards, codes and specification. There are few product specific guidelines provided by manufacturers. Further research, codes, realistic design guidelines could help to bring down the final cost of UHPC.

Strut and tie model analysis is an approach which is used to design the irregular concrete shapes and the D-Region in the concrete structures where forces and strains are characterized by the abrupt changes. This region is usually present at the distance of depth of the structure on either side of discontinuity in geometry or force [34]. The forces and the strains in the D-Region are of complex nature and could not be analyzed by classical beam theory which applies to the B-Region of the concrete elements. The D-region is present at beam ends, in deep beams, brackets and other complex geometric shapes could be analyzed by using this lower bound struct and tie method. The strut and tie model assumes the concrete structure as truss in equilibrium under the action of external forces. The next assumption is that the concrete element has enough deformation capacity to distribute the external forces [34]. The strut and tie model approach reduces the complex forces in concrete structure as compressive force acting along the assumed truss members called as strut and the tensile force acting along the truss members called as ties. Tyler *et al.* [35] conducted the experimental verification of concrete specimen and demonstrated that the specimen yields the safe design. Mohamed Husain *et al.* [36] demonstrated that the strut and tie method always give demand collapse load lower than the true capacity collapse load, thus implying that the solution obtained from struct and tie model is conservative while detailing the D-Region of the concrete structures and components. Panatchai *et al.* [37] derived and proposed the modified interactive strut and tie model to predict shear strength of RC corbel. The effect of compressive strength of concrete and yield stress of reinforcing steel on the behavior of self-compacted concrete deep beams were studied [38] where struts were reinforcement were based on finite element model. Jung-woong Park and Daniel Kuchma [39] compared the strut and tie model based on constitutive laws for cracked reinforced concrete with that of the strut and tie provisions in ACI 318-05 and Canadian code. The strut and tie model analysis has been used in analysis of concrete structures and components employed for wide range to functionalities. The approach essentially breaks down the complex geometry and complex flow of forces into compressive struts and tensile ties as in truss elements. The required amount of reinforcement then could be provided based on nature and magnitude of the forces in these struts and ties. The pier cap analyzed in the present study has geometric discontinuities and also the material non-linearity due to the presence of existing massive steel bearing parts. The resulting flow of forces in the proposed pier cap will be complex and discontinues. The strut and tie model as discussed in AASHTO [34] is used to analyze the UHPC pier cap.

3. Problem Statement

In the present case, the pier cap analyzed, is a representative pier cap, based on commonly found pier cap in existing old bridges. The steel superstructure truss rests on top shoe of the bearing either fixed bearing or expansion bearing. The lower shoe of the bearing is deeply anchored into the pier cap. These bearing are found to be massive. In case of tall columns, it is costly and time consuming to construct the temporary tower to support the superstructure for rehabilitation of pier caps and replacement of bearings. For these reasons *i.e.* massive size of bearing, deep anchorage of bearing, time and cost involved in the process of rehabilitation, an alternative approach is discussed in the present efforts. Instead of removing the existing bearing entirely, it is proposed to embed the lower shoe of the fixed bearing in UHPC cap. This UHPC would then provide a platform for jacking assembly during the bearing replacement stage.

3.1. Analysis and Solution Method

The representative superstructure span was loaded as per AASHTO code. The total reaction load was found to be 4000 kip. It was found that the shell column with 18 inch thickness around the existing 10.5 ft × 11.5 ft pier with normal concrete strength of 5 ksi strong enough to resist the load from superstructure. The pier cap has 6 in offset making it 11.5 ft × 12.5 ft in size at the bottom of cap. Being anchored in concrete pier cap, it is difficult to remove the lower shoe of these existing bearing without damaging the top portion of pier cap. Being very thick and massive, cutting away of lower shoe is difficult. Due to geometrical and material discontinuity alternative solution to provide any steel grillage beams around the existing bearing to transfer the load from superstructure to pier was found to be difficult and impractical. The solution to overcome these difficulties is using Ultra-High Performance Concrete (UHPC) for pier cap replacement where the top shoe of the bearing could be removed, and the lower shoe could be embedded in UHPC to support the jacks in bearing replacement process. The approach requires to study complex load paths in the proposed UHPC pier cap and provides the minimum required tension reinforcement for proposed pier cap. The pier cap analysis and the design for bottom reinforcement is discussed in this section.

3.2. Analysis Framework and Methodology

Figure 4 shows the schematic representation of pier cap and fixed type bearing. **Figure 5** shows the schematic representation of pier cap and expansion type bearing. As seen **Figure 4** and **Figure 5**, the proposed pier cap exhibits significant degree of geometrical discontinuity. The finished depth of pier cap including pedestal will be more than 3.25 ft. Refer **Figure 6**. Based on NJDOT standards the concrete component with thickness of 3 ft and more could be classified as mass concrete [40]. In order to analyze mass concrete with geometrical discontinuity, the fair idea of load paths and stresses developed in the body of mass concrete is required. AASHTO [34] outlines the concepts called as strut and tie model to analyze such structural components. The existing pier will also carry the partial load. In the event of any future deterioration of the existing pier, the new shell column will resist the entire load. Based on assumption of strut and tie model, the pier cap could be idealized as a deep concrete member. The load coming from truss vertical and bottom chord spreads into the pier cap through bottom plate of HLMR bearing. The equivalent point load is assumed to act at apex node of the 3-dimensional truss. This 3 dimensional truss could be thought of an instrument to spread the load into the pier cap.

The further simplification was achieved by dividing this 3 dimensional truss into two dimensional 4 simple triangulated trusses at each face of the pier cap. One such simple truss ABC, is shown in **Figure 6**. The idealized simple truss is devised such that the angle between strut and tie at a node should not be less than 25 degree. The angular limit ensures mitigation of wide crack opening and excessive strains in reinforcement. If the constraint for angle between strut and tie is not satisfied, then the truss configuration needs changes. In the present case study, the angle between strut and tie is 28 degrees agrees with the constraint. It is proposed that, as far as possible cold joint across the length of the tie to be avoided, else enough shear-friction capacity should be ensured. Thus, the AASHTO specified guidelines for structural modelling were followed to develop the strut and tie model. The strut and tie model as outlined in AASHTO essentially satisfies two basic requirements that the forces calculated in idealized triangulated truss members are in equilibrium and material strength is not exceeded.

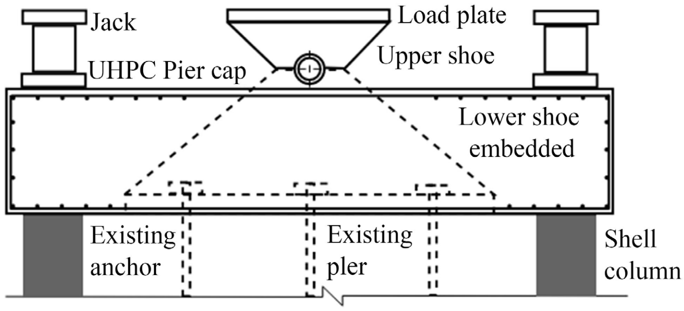

Figure 4. Schematic representation of proposed UHPC pier cap at fixed bearing.

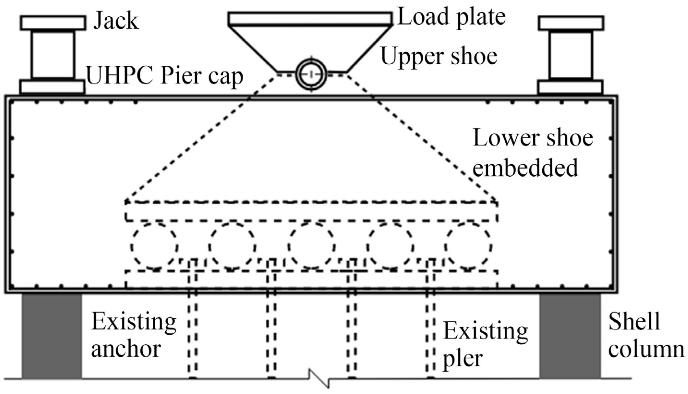

Figure 5. Schematic representation of proposed UHPC pier cap at expansion bearing.

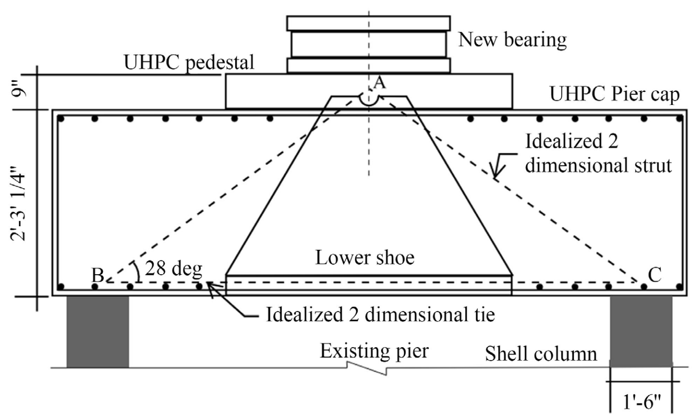

Figure 6. Schematic representation of idealized strut and tie model.

As described above the total reaction load of 4000 kip could be thought of 1000 kip load acting at mid points of each of the 4 side of masonry plate. Figure 6 shows the strut and tie model assumed at one of the face of proposed pier cap. Similar strut and tie model could be assumed at each of the remaining face. The compressive load in the struts AB and AC was found to be 875 kip. The tensile load in tie BC was found to be 941 kip. Assuming 60 ksi steel, total of 14 bars of size #11would be required to resist tensile load in the idealized tie region. Thus, the total resistance provided is 1066 kip.

The application of resistance factor of 0.9 for strut and tie model, gives the design resistance in tie as 960 kip. While calculating compressive resistance of strut and tensile resistance of tie members, the nodal regions should be checked for its capability to transfer the loads to the support. The nodal proportioning of the strut and tie modeling method ensures the capability of nodes to transfer the loads to the supports. The nodes, as per AASHTO could be characterized as CCC, CCT and CTT. The CCC nodes are the nodes where only struts intersect. At CCT tie intersect in one direction. At CTT, tie intersects in two directions. In the present case study, two nodal categories are present, CCC and CCT. Table 6 lists the parameters to be used in node proportioning as per AASHTO. Figure 7 shows the representation of CCC and CCT node region [34].

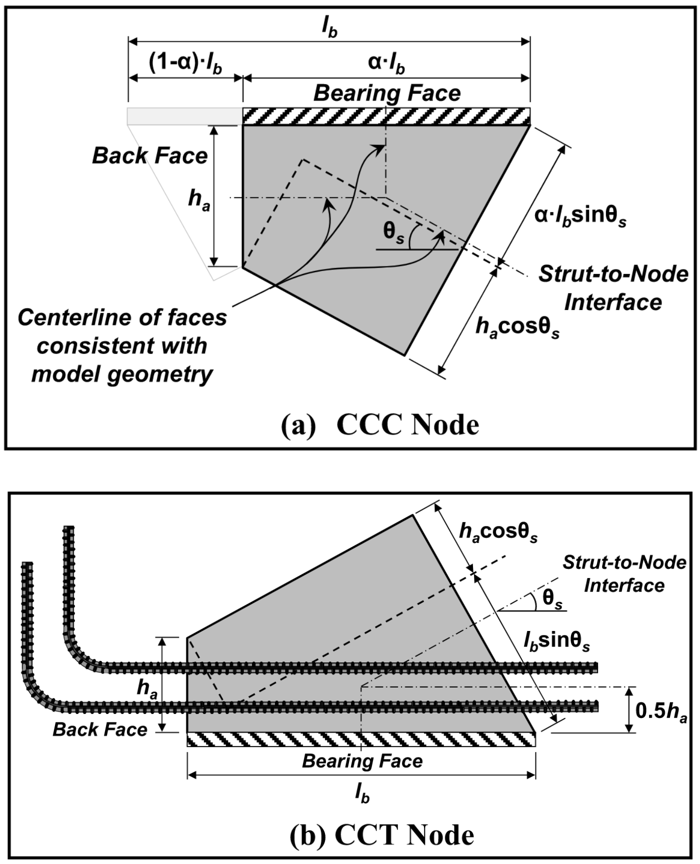

Figure 7. Representation of CCC and CCT node region (AASHTO).

Table 6. Parameters for node proportioning.

Symbol	Description	Value	Reference
A1	Area under bearing	3.5 × 4.6525 = 15.96 ft2	Provided by bearing manufacturer
A2	Notional area	14.5 × 15.5 = 224.75 ft2	Sketch drawn as per AASHTO
m	Confinement modification factor	2	AASHTO
f'c	Compressive strength of concrete	15 ksi	As per literature
v	Concrete efficiency factor	0.45	AASHTO
f'cu	Limiting compressive strength	13.5 ksi	AASHTO

Following the procedure as outlined in AASHTO and using the parameters as in Table 6, the factored resistance of the node CCC for the present case study was found to be 4431.27 kip. Similarly, the factored resistance for node CCT was found to be 7757.26 kip. The node proportioning indicates that, for the given case study the stresses are not exceeded, and the system is in equilibrium.

Based on the strut and tie model and the analysis, the assumed triangulated load path simplifies the load distribution, and gives the fair idea of the stress distribution in geometrically discontinuous proposed pier cap. Though the pier cap in general could be categorized as compression member, and which could discourage the very idea of application of strut and tie model, there exists a possibility flexural behavior due to various reasons such as accidental loads, eccentricity, change in loading conditions, excessive moments due to lateral loads and localized damage to the existing pier due to natural deterioration [41]. In such scenarios, the tie reinforcement provides the required flexural resistance and ductile behavior of pier cap. This justifies the application of strut and tie model to the proposed pier cap. The more refined strut and tie model is expected to give better results and fair distribution of forces in the pier cap.

4. Conclusion

In the present study, Ultra-High Performance Concrete (UHPC) is proposed to replace the pier cap. The engineering, chemical and mechanical properties of UHPC have been discussed. The literature review also summarizes variations in the constituent materials, properties based on application and the manufacturers. The UHPC properties were also reported based on method of testing, various curing conditions, specimen sizes, rate of loading, function of time tested after casting, biaxial compressions testing. Despite variations in properties as reported in various literatures by different research, curing methods, testing and ambient conditions, UHPC exhibits properties far better than conventional concrete. The application of UHPC has been found to address various unique problems in bridge construction and rehabilitation. In the present study, the UHPC application for reconstructing pier cap is discussed. Given the conditions at the site and difficulties in execution of alternate solutions, UHPC could prove to be a practical solution. Due to discontinuity in geometry and material, the lower bound solution of analysis called as strut and tie method as outlined in AASHTO was employed. The design load of 4000 kip was divided into 4 parts acting at each face of the pier cap. The idealized triangulated truss composed on of 2 strut members and 1 tie member was used to define the load path in the pier cap. Along the struts, resistance value found to be more than load that would be induced in idealized truss member. Similarly, the tensile load induced in idealized tie member was estimated and used to design the steel reinforcement. Due to lack of standards, codes and guidelines, reinforced concrete theory for conventional concrete has been used. Further research, development of codes, standards and guidelines could help to fine tune design process and provide more economical solution in future.

Disclaimer

The major goal here is to investigate the application of Ultra-High Performance Concrete to rehabilitate pier caps and replace bearings in existing bridges. Though, the references were made to certain UHPC manufacturers, author does not intent to promote any of these manufacturers. The references were used just to summarize the overall UHPC properties, and citations are provided at appropriate locations.

Terminology

- Structurally deficient: Bridges that require significant maintenance, rehabilitation or replacement. These bridges need yearly inspection due to decrease in critical load carrying capacity.
- UHPC: Ultra-High Performance Concrete.

Conflicts of Interest

The author declares no conflicts of interest regarding the publication of this paper.

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