ENVISIONING CONNECTION DETAIL FOR CONNECTING CONCRETE FILLED TUBE (CFT) COLUMNS TO CAP BEAM FOR HIGH SPEED RAIL APPLICATION

Quarterly Progress Report

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BACKGROUND

The demand for mass transit in urban transportation has increased substantially with an increase in the population. Conventional modes of transportation such as road and air travel have caused congestion and traffic problems in many cities around the world. A direct consequence of this is an increase in carbon emissions from automobiles which have contributed to the greenhouse effect. Besides environmental degradation, the cost of road and air travel has been rising with the cost of fuel.

To achieve high mobility for mass transit, there have been innovations in rail transportation. One such example is the introduction of High-speed railway (HSR) which offers a robust alternative to traditional trains, vehicular and air traffic systems for mid-range distances. As technological innovations improve over time, the concept of high speed has also evolved. Since its introduction in 1964, the speed of HSR has increased from 200 Km/hr to about 500 Km/hr. HSR services now operate in more than 16 countries and carry millions of passenger trips per year. It is estimated that a total of 37,000 Km of HSR lines are in operation [Kang, 2018].

HSR offers an efficient, fast and safe transportation system. HSR is one of the best examples of low carbon logistics and offers an additional benefit of lesser greenhouse emissions. Countries with a wide network of high-speed trains have shown a reduction in volumes of road and air traffics. On the other hand, the HSR system is expensive and generally cannot be used for freight transport. The high cost is generally attributed to many factors including dedicated infrastructure, energy requirements, high maintenance etc.

HSR SYSTEMS IN ASIA AND EUROPE

The introduction of HSR in Europe and Asia in mid-sixties has been a cornerstone in the economic development of many countries. HSR now operate up to 16 countries, and many new countries like Turkey, Saudia Arabia etc are developing HSR systems. Figure 1 shows the percentage of HSR in the world by mileage.

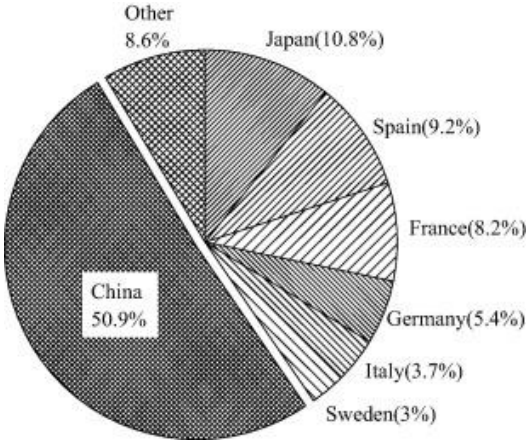

Figure 1: HSR mileage by countries [Hu et. al 204]

Japan was the first country to developed HSR, commonly known as Shinkansen, which operated between Tokyo and Osaka in the sixties. Later on, the Shinkansen network was expanded to different cities covering a total distance of 2800 km. The average

speed of these HSR ranges from 260 Km/hr to 320 Km/r. The success of the system relies on dedicated tracks with no level crossings which has resulted in zero crashes or fatalities since 1964 when these HSR were introduced. Japan is now looking into commercial deployment of Maglev train systems which can achieve up to 550 Km/hr.

China started as late as 2007-2008, and, since then it has developed the largest network of HSR in the world. As of 2016, China has 22,000 Km [Qin, 2017]. The Chinese high-speed train operates at a speed of 350-380 Km/hr. China has also developed Maglev technology which operates on a short route with a maximum speed of 430 Km/hr [Gourvish, 2010]. Figure 2 summarizes different generations of technologies used in different countries.

Europe has an estimated HSR line of almost 6637 Km which are mainly located in France, Germany, Spain, and Italy. Additional high-speed lines are either being constructed or planned for future [Dobruszkes, 2011]. In France, the TGV (Train à Grande Vitesse) was introduced in 1981 between Paris and Lyon. The total network of HSR in France is about 1,800 Km. With technological advancements, the HSR has improved, and, by 2008 the trains achieved the highest operational speed in the world at 320 Km/hr. Spain launched the high-speed train also known as AVE (Alta Velocidad Espanola) in 1992 between the Madrid-Sevilla corridor and now has the largest HSR system in Europe only next to China. Germany developed ICE (Inter City Express) in 1991 which is the long-distance rail transport system. Until 2018, Germany has an HSR network of almost 1658 Km. Other European countries like Turkey, Poland, Switzerland and, Belgium have also developed HSR networks.

Generation	Japan	France	Germany	China
1G	**S-0** 210 km/h 1964 **S-100** 270 km/h 1975 **E-2** 275 km/h 1997	**TGV-PSE** 280 km/h 1981 **TGV-A** 300 km/h 1989	**ICE-1** 250 km/h 1991 **ICE-2** 280 km/h 1996	**CRH1** 200 km/h **CRH2** 200 km/h **CHR5** 200 km/h 2007 **Imported 1G**
2G	**S-500** 300 km/h 1998	**TGV-Mediterranee** 320 km/h **TGV-EST** 2001 320 km/h 2007	**ICE-3** 300 km/h 2002 (Valaro)	**CRH2** 300 km/h **CRH3** 300 km/h 350 km/h **EST** 2008 **Developed 2G**
3G	**Fastech360** 2011 Reduced to 320 km/h, as E5 was put in service	**AGV360** 2008 Tested 360 km/h **EST** in Italy, by 2013 (not over) 300 km/h **EST**	**ICE350E** 2006 Tested 350 km/h **EST** in Spanish (not in service)	**CRH380-A** 2010 **CRH380-B** 350-380 km/h In service **Created 3G**

Figure 2: Footmarks of technology development for high-speed trains around the world

HSR SYSTEM IN UNITED STATES

The U.S. started its efforts in HSR in the 1960s which resulted in the introduction of Metroliner between New York and Washington D.C. The Federal Railway Administration (FRA) has identified different corridors for connecting populated centers with HSR which are in the range of 100-600 miles. Figure 3 shows different planned corridors. Recent developments include HSR systems planned for California and Texas.

The California High-Speed Rail Authority (CHSRA) envisions a speed of 220 mph with an initial stretch between Los Angeles and San Francisco. The Texas Central Railway and private companies are conducting a study on Dallas to Houston corridor which is almost 250-miles long.

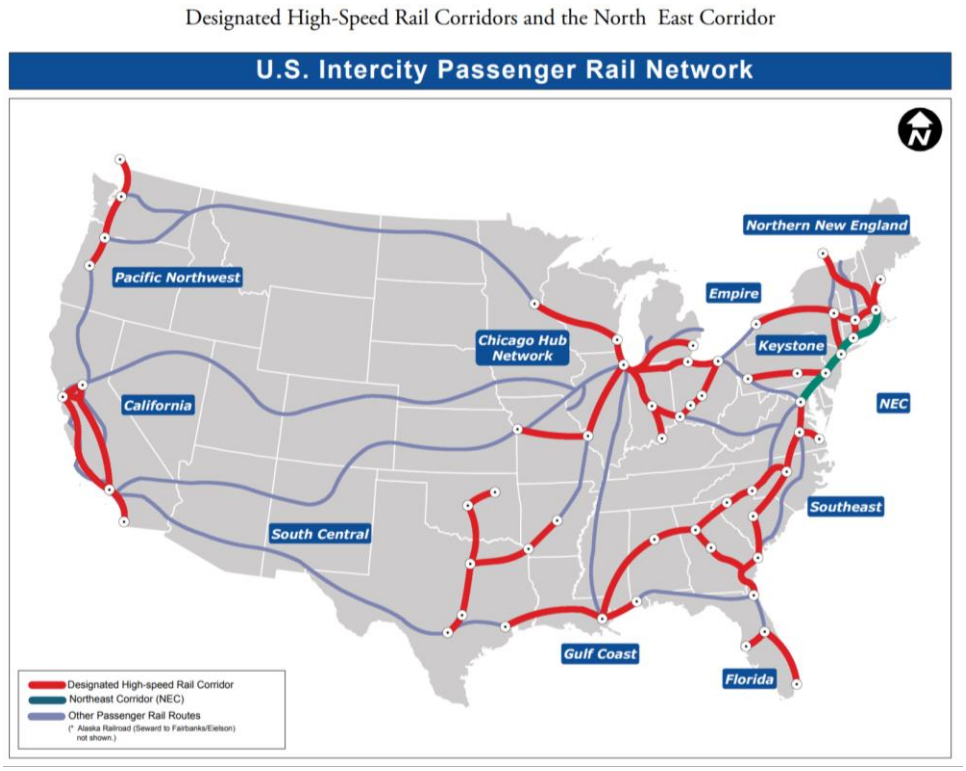

Figure 3: Planned High Speed Rail Corridors in The U.S. [USDOT]

Due to the geography and terrain of California, most of the rail network will use structures which are elevated. The substructure height for these structures may go up to a few hundred feet. The construction of substructure for these bridges using traditional construction methodologies may not be efficient as these require the use of climbing formwork which is time-consuming and costly. Alternate solutions consist of the use of prefabricated columns but the delivery and erection of these long columns is a challenging task.

HSR BRIDGE STRUCTURES

In general, HSR bridges have stringent requirements for design and construction. Based on the speed, location and traveler's comfort, the structures have limitations on geometric aspect like slopes and curvatures of railway track. The track smoothness requires deflections checks which are different from conventional train bridges [Hu et al., 2014]. Besides, the bridge structures are subjected to dynamic effects of high-speed trains, fatigue and rail-deck interaction which demand bridges with higher stiffness and dynamic stability. Other structural considerations for HSR bridges include seismic performance, creep and thermal effects.

In China, the HSR lines mainly consist of standardized simply supported beams for shorter spans and continuous beams for longer spans [Hu et al.,2014]. For medium spans, Concrete Filled Tubes (CFT) arches, tied steel arch, and rigid frame prestressed structures have been used. A major portion of the Beijing-Shanghai HSR consists of simply supported prestressed box girders with a span of 32 m. Due to diverse train, long span bridges on railway network have been constructed. Most of the bridges on the German HSR network consist of simply supported bridges and continuous box girder bridges. For substructure, the most common kind of pier columns consists of reinforced concrete single columns, wall piers and, multi-column piers.

Several different design codes have been developed for HSR. China developed the Code for Design of High-Speed Railway and International Union of Railways (UIC) code provides several design leaflets for HSR. American railway engineering and maintenance-of-way association (AREMA) do not have specific requirements for high-speed rails.

The initial cost of HSR requires a large capital and due to their stringent requirements for alignment control, the construction time is an issue. For substructure construction, the use of robust construction technologies has not been used extensively for the HSR network which causes delays in delivery of the projects. To address these issues, accelerated bridge construction (ABC) technologies can assist in reducing construction time and providing solutions that are resilient and safe for construction workers. Many ABC technologies have been developed for pier construction including prefabricated piers, precast segmental and CFT columns. The use of prefabricated segmental and CFT columns have not been extensively used in HSR bridge construction.

Concrete Filled Tubes (CFT)

Concrete filled tubes (CFTs) offer an alternative to prefabricated construction which is suitable for HSRs and offer additional benefits of ABC. The use of CFTs in ABC technologies can benefit the construction industry by providing alternate structural elements which have both structural strength and ductility especially for high seismic regions such as California.

Concrete Filled Tubes (CFTs) consist of an encasing tube of steel which is filled with concrete. The tube is located at the periphery and provides protection to the in-fill concrete. The tube acts as a formwork element which reduces the cost of the structure. Most of CFTs do not require the use of any mild reinforcement inside the tube which reduces the labor cost of fabrication and assembling at the site. The structural performance of CFT is also superior to an equivalent concrete column. The steel tube offers confinement to the concrete which improves the structural capacity and ductility of the section. Under a combination of cyclic and axial load, these columns perform elastically up to a certain level of displacement after which the tubes buckle locally.

Significant research efforts have been carried out on CFT column behavior under different loading condition. Different design codes provide design expression for the CFTs including AASHTO, AISC, and ACI but the provisions of these design codes vary from each other. In building structures, CFT to steel beam connection has been extensively evaluated and used in practice (Azizinamini, 1989). However, in bridge

design, there is a lack of connection design for CFT to foundation and pier cap which hinders the use of CFT especially in the U.S. Recently, there has been a lot of research effort invested in developing design recommendations on connections of CFT in bridge structures.

CFT CONNECTION TO FOUNDATION/CAP BEAM

The connection of steel tube with foundation or pier cap is to be designed to prevent any premature failure. A number of designs for CFT connection have been used in practice. The most common type of connection detail consists of exposed connection and embedded connection as shown in Figure 4. The exposed connection relies entirely on anchor bolt and thick end plate, both of which are vulnerable to damage during earthquakes. The embedded connection perform better as additional resistance is contributed by the steel tube and the concrete. However, high forces during lateral movement can cause crushing in the vicinity of the steel plate which reduces the structural performance.

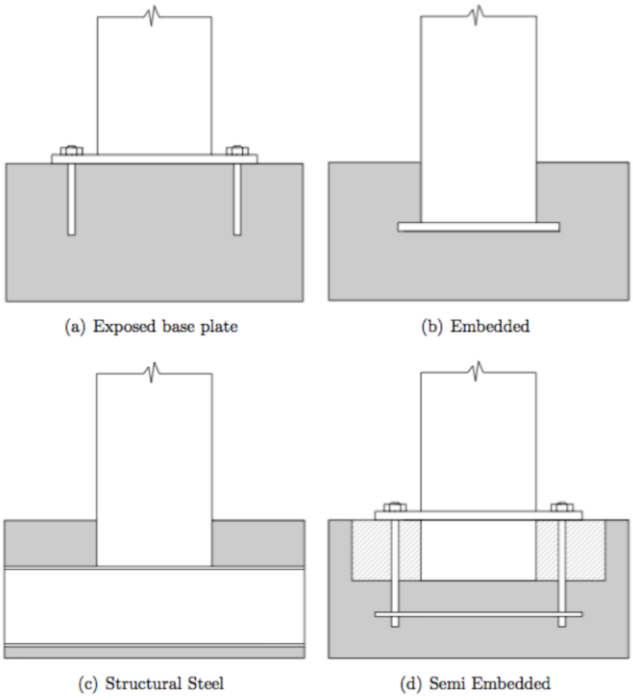

Figure 4: CFT connection a) exposed connection and b) embedded connection, c) structural steel connection, and d) semi-embedded connection (Kingsley 2005)

Hsu et al. (2006) proposed a new type of connection for square CFT by strengthening the embedded connection with stiffeners which would minimize the tendency of concrete to crush. The experimental results, under combined axial and lateral load, showed that the embedment improved the structural behavior of the connection and addition of stiffeners improved the energy dissipation capacity.

Kim et al. (2015) experimentally evaluated different types of CFT to foundation connections and compared the results on the effectiveness. Five connection details

were considered with different combinations of high-tension bolts, deformed bars, and anchor frames, etc. Results showed that all columns exhibited local buckling along with anchor failures. The high-tension bolts exhibited superior structural performance when compared with anchor bolts or anchor frames.

Stephens et al. (2015) carried out an extensive experimental study on the connection of CFT column with precast concrete elements. The authors proposed different connection details (Figure 5) for foundation and cap beam which were numerically and experimentally tested and design expressions were proposed.

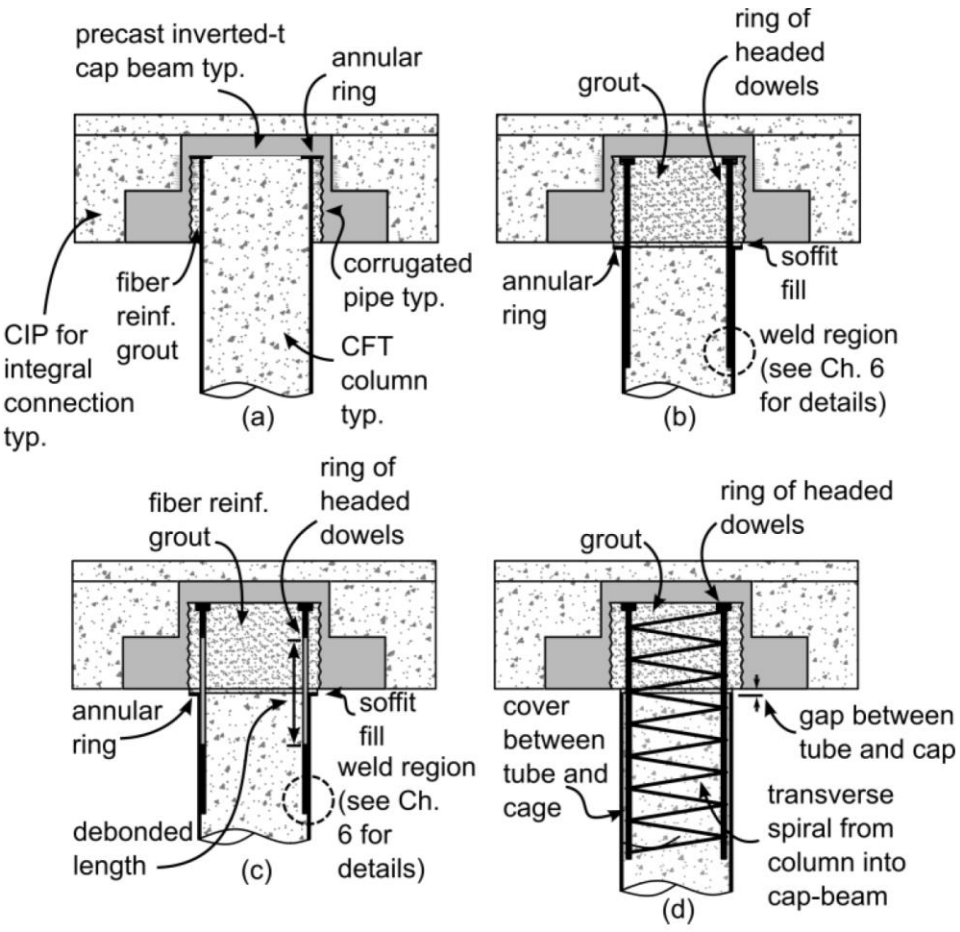

Figure 5: Proposed Connection types (Stephens, 2015)

Lehman et al. (2012) carried out an experimental study to evaluate ductile connections of CFT without the use of any dowels or internal reinforcement. The research used grouts to fill the voids once the CFT is placed in a precast member. In this case, a high-strength fiber-reinforced grout was filled to achieve composite action.

Recent research activities have looked into the use of ultra-high performance concrete (UHPC) for connecting different bridge elements. UHPC is a class of material with excellent material and durability properties. The material can exhibit compressive stress between 22-26 ksi and average tensile stress of almost 1.2 ksi. The use of UHPC

for connection of precast concrete columns has been investigated by Shafieifar et al. (2018). The UHPC band at the interface of concrete column to cap beam shifted the plastic hinge away from the capacity protected element. Tazrav et al. (2015) experimentally evaluated the connection of reinforced concrete column to footing using duct connection incorporating UHPC and concluded that the connection was similar to the corresponding conventional connection. Other researchers incorporated UHPC in different configurations to protect structural elements during seismic activity. UHPC was used in plastic hinge zones of the column for pocket connections which were evaluated by Mohebbi et al. 2018.

The objective of the research is envisioning of details for CFT connections for prefabricated substructure elements incorporating a UHPC based connection. The connection will be evaluated with a combination of analytical and numerical study. Following inception, an experimental study will be carried out on the CFT connection details. The initial FE element model will be calibrated based on the results of the experimental study and a parametric study will be carried out to understand the behavior of the connection. A formulation will be developed to provide designers with guidelines to calculate the capacity of the section.

CONCEPT DEVELOPMENT

The main concept of the connection is based on the use of a UHPC layer located near the potential plastic hinge zone. A cavity is created in the foundation or cap beam which accommodates the CFT column end dowel bars. The force and moment transfer is accomplished through the use of a corrugated interface between the UHPC and normal concrete. The UHPC provides anchorage and develops the reinforcing dowels over a shorter length. Slippage between the UHPC and the CFT is accomplished through the use of the stud bolts.

The sequence of construction is suitable for ABC applications with prefabricated foundation or cap beams. As shown in Figure 6, the corrugated interface is introduced at the time of casting of concrete foundation along with reinforcing dowels. The CFT column is then placed inside the foundation and UHPC band is cast for a limited length. At the final stage, normal concrete is cast inside the remaining CFT tube. The casting of UHPC in the potential plastic hinge region results in improved structural behavior at the critical section. The strategic use of UHPC reduces project cost.

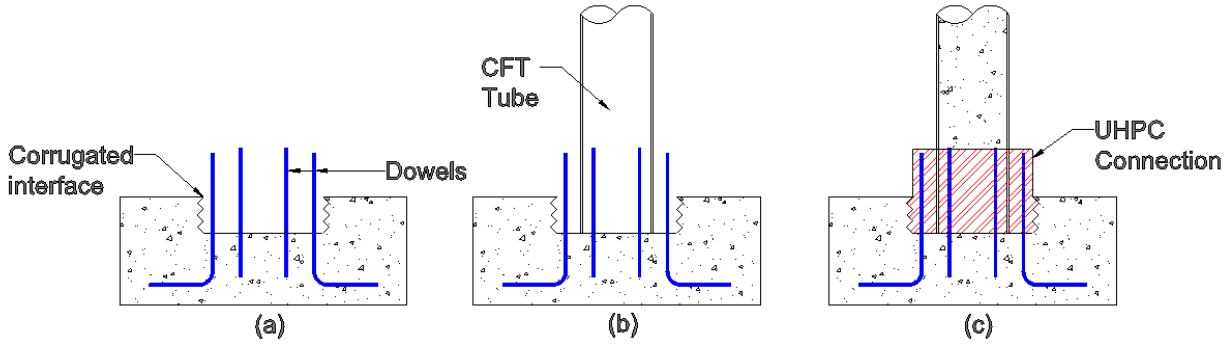

Figure 6: Proposed Connetion Detail

EXPERIMENTAL STUDY

In this Task, a scaled model of a typical foundation/cap beam connection is designed as shown in Figure 7. Moment-curvature analyses were performed at critical sections A-A, B-B, and C-C.

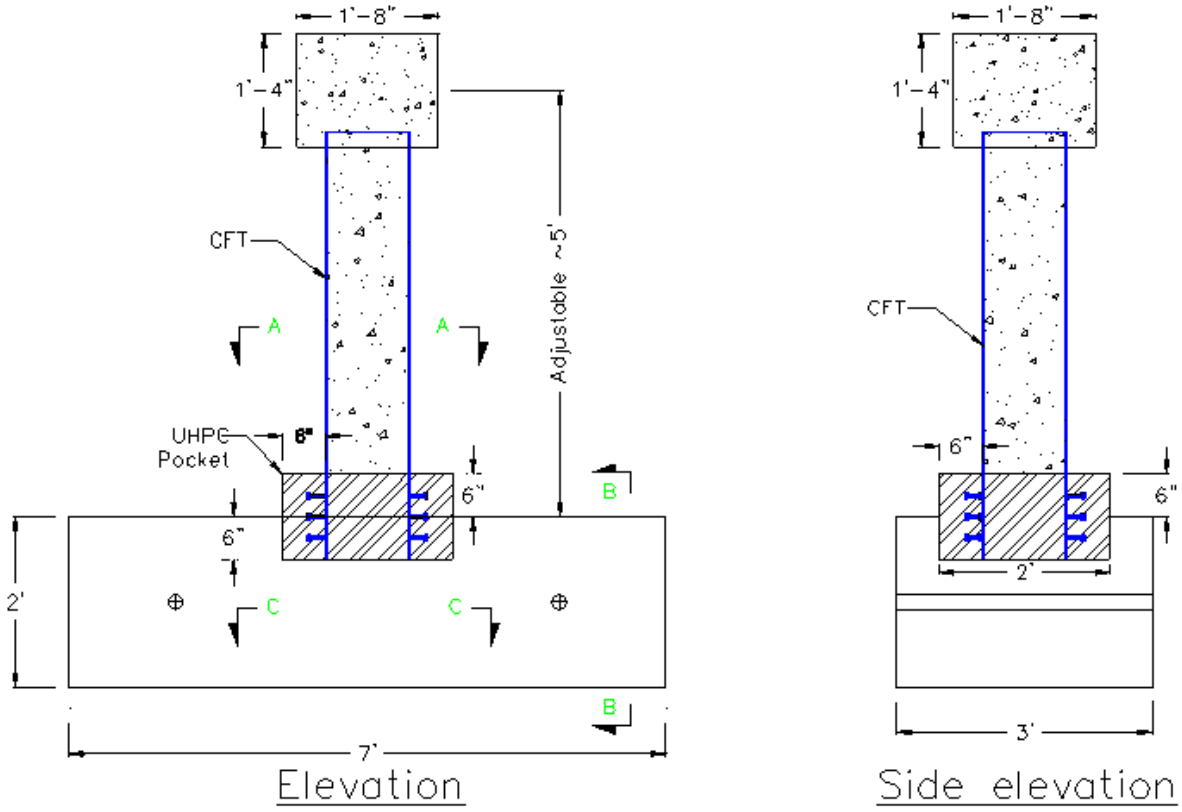

Figure 7: Scaled down experimental setup (reinforcement not shown for clarity)

The outer diameter of the CFT column is 12.75" with a wall thickness of 0.25-in. Different design codes limit the depth-to-thickness ratio (D/t) of the column for local stability. In this case, D/t of the column is 51. Assuming yield strength of steel tube as 60 ksi and concrete compressive stress as 6.5 ksi, the axial capacity of the column P_o is about 1320 Kips. At critical sections, the moment curvatures at section A-A is shown in Figure 8. To keep the foundation/cap beam element capacity protected the moment capacity at section B-B and section C-C are determined to be at least 1.3 times that of section A-A which is higher than AASHTO-LRFD requirements

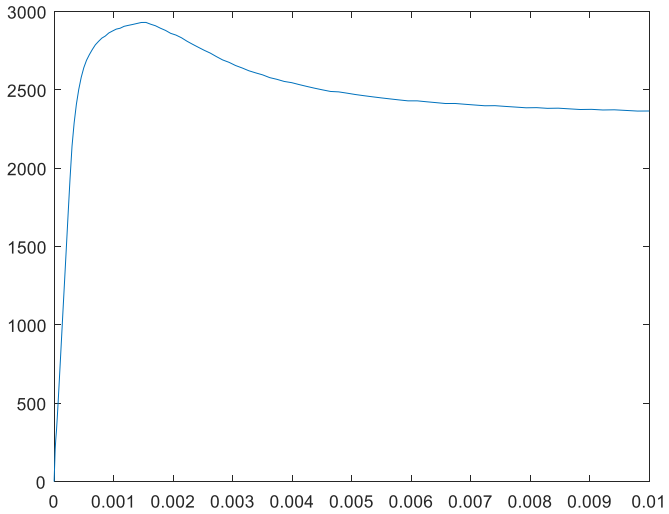

Figure 8: Moment Curvature at Section A-A (Kips-in)

Based on the moment-curvature at critical sections, the reinforcing steel is designed as shown in Figure 9.

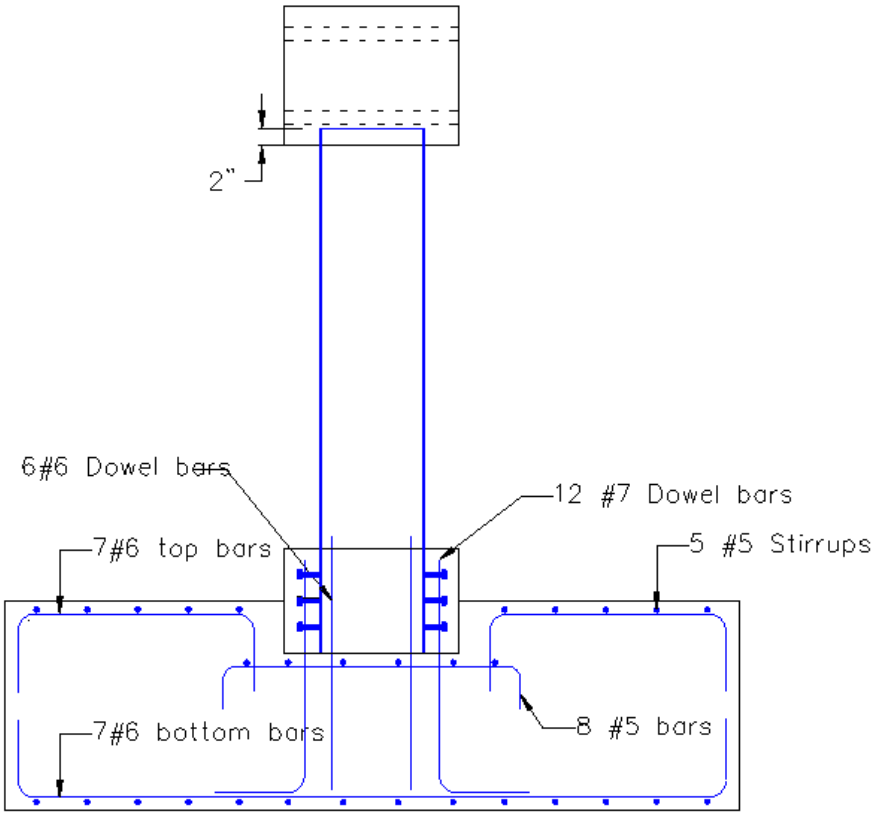

Figure 9: Reinforcement detail

EXPERIMENTAL PROGRAM

The experimental program for the CFT to footing connection is being conducted at Florida International University (FIU). In the first series of testing, partially embedded connection with UHPC encasement is being constructed.

According to the proposed methodology, a recess in the precast footing was constructed using a Styrofoam mold. The Styrofoam was corrugated at the periphery with corrugated spikes. Before concreting, the mold was embedded in the footing and normal strength concrete was cast. After curing, the Styrofoam was removed and the interface at the plane of the recess was chipped off to allow for improved bond between the footing concrete and the UHPC. A slot was cut at the bottom of the CFT to allow UHPC to flow inside the tube. A total of 18 shear connectors were welded (using MIG welding method) equidistant to the CFT tube in a radial pattern. The shear connectors used in the specimen had a shaft diameter of 0.5 inch, head diameter of 1-inch and a total length of 2.5-inch. An encasement formwork was assembled with a 24-inch Sonotube and the CFT tube was placed inside the recess. The UHPC was cast into the 6-inch recess and an additional 6-inch of encasement. Figure 10 shows various photographs of the construction sequence described above.

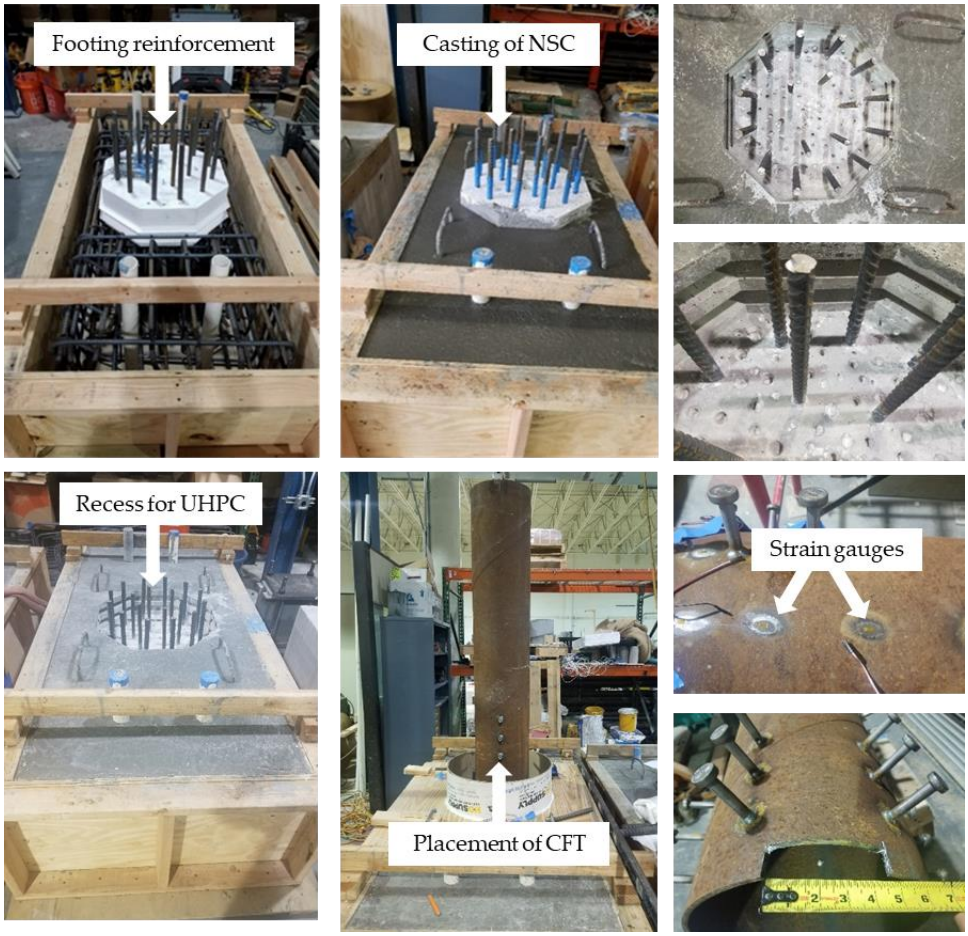

Figure 10. Construction of Specimen

TEST SETUP AND TESTING PROTOCOL

The dowels were typically instrumented with strain gauges at critical locations. Two strain gages were located inside the concrete footing and one was located in the UHPC step. Similarly, strain gauges were also installed on the outer surface of the CFT tube at different depths. Coupling nuts were also welded on the steel tube to allow for installation of linear potentiometers. Figure 11 shows the instrumentation plans.

The column will be tested under a constant axial load and a drift controlled lateral load. The axial load on the CFT column will be between 5% to 10 % of the axial capacity (P_o). The cyclic load will be applied using a hydraulic ram with a capacity of 140 Kip. In initial cycles, low displacement cycles will be applied to the column to estimate idealized yield displacement of the column (Δ_y). The test setup is shown in Figure 12.

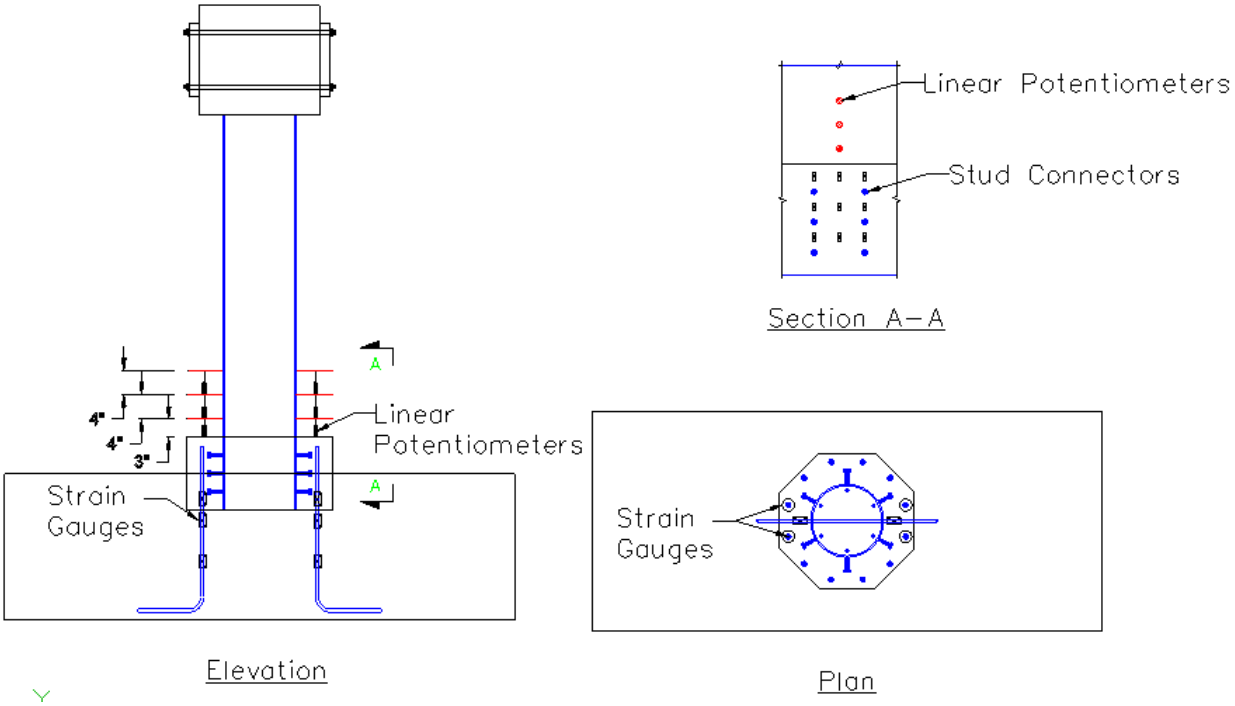

Figure 11. Instrumentation Plan

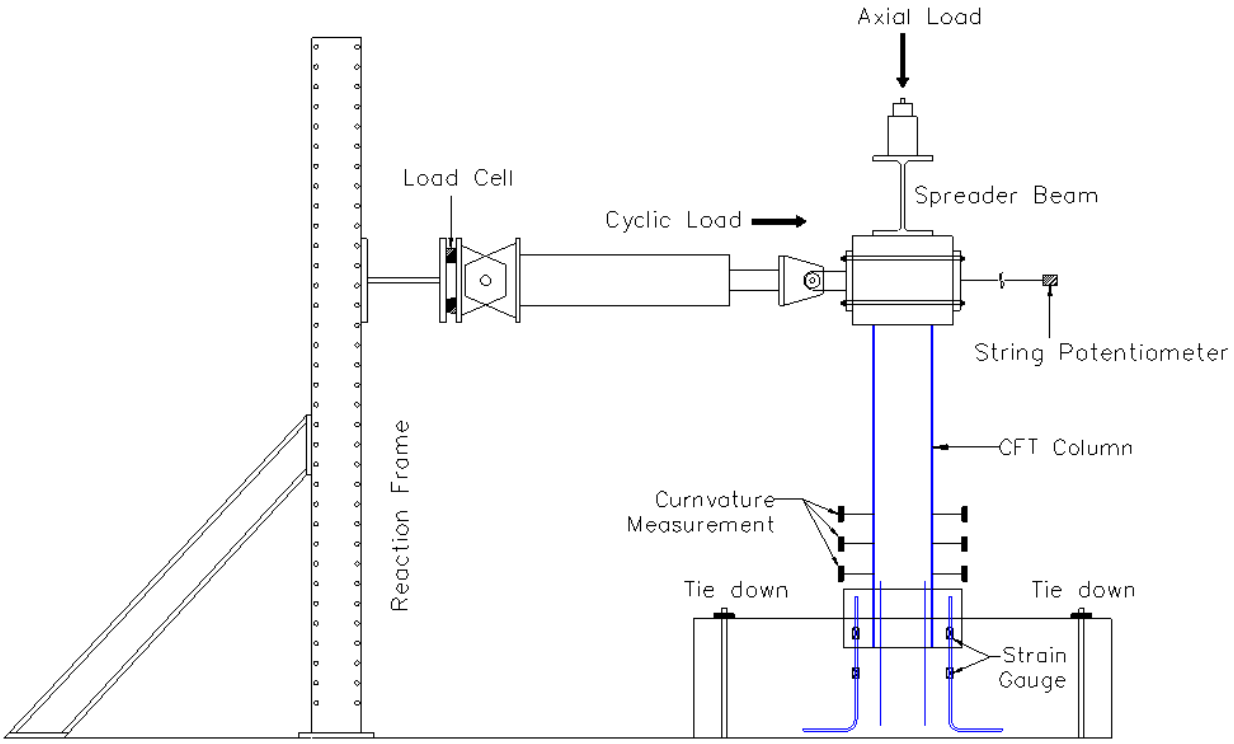

Figure 12. Load test setup

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