

DESIGN OF TALL WIND TURBINE TOWERS UTILIZING UHPC

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Abstract

A major research focus of the wind energy industry is to reduce its capital and operation costs. One potential solution to reducing the wind energy cost has been suggested to be through the use of taller wind turbine towers. A new tower concept has been developed using Ultra-High Performance Concrete (UHPC) that would allow taller wind turbine towers to be transported to wind farm sites easily within the current transportation limitations. Three tower designs, consisting of precast UHPC or high strength concrete segments, have been completed for potential field implementation. This paper summarizes the three designs completed with a hub height of 100 m as well as laboratory tests completed on critical sections of two of the towers. Each design utilizes different connections between the precast segments. These connections rely on high strength bolts, post-tensioning, and in situ UHPC. The performance of each section is demonstrated to be excellent in this paper using the test observations and complementary analytical results.

Résumé

Un des buts principaux de l'industrie liée à l'énergie éolienne consiste à réduire ses coûts d'investissement et d'exploitation. Une solution proposée pour réduire le coût de l'énergie éolienne consiste à utiliser des éoliennes plus hautes. Un nouveau concept de mât a été développé en utilisant les bétons fibrés à ultra-hautes performances (BFUP) susceptibles de permettre que des mâts plus hauts soient transportés facilement vers les champs de production tout en tenant compte des contraintes actuelles de transport. Trois concepts de mâts, constitués de voussoirs préfabriqués en BFUP ou en béton de haute résistance, ont été mis au point pour une application possible sur site. L'article résume les 3 projets pour une hauteur de 100 m à l'axe, ainsi que les essais réalisés en laboratoire sur les sections critiques de deux des mâts. Chaque projet utilise un type de connexion différent entre les voussoirs: boulons de haute résistance, post-tension, et clavage BFUP en place. Les performances de chacune des sections s'évaluent excellentes, comme le montre l'article sur la base des observations expérimentales et des résultats analytiques complémentaires.

1. INTRODUCTION

Advancements to wind turbine components are continuously being made to reduce the cost of wind energy. Today, wind turbine towers are routinely designed with 80 m tall steel tubular towers. This material choice has been favoured because dimensions of the 80 m hub height have worked out to be just within the transportation limits. Although it is a challenging, time consuming, and costly task, as illustrated in Figure 1, the steel tower is typically transported to the site in three segments. Given that at higher elevations, faster, more sustainable wind exists that allow energy harvesting times to be increased, wind energy industry has begun exploring the design of towers with hub heights of 100 m and above. Increasing the steel towers to new heights is neither straightforward nor cost effective due to the transportation allowances of the highway system. The base dimension of the taller steel tower would require transporting of the bottom section in multiples pieces and assembling them on site. To overcome this challenge, a new tower concept has been developed that would allow a taller wind turbine tower to be designed and constructed without being constrained by the current transportation system.



Figure 1: Transportation of steel tower segments and associated challenges [1]

2. NEW TOWER CONCEPT

The new tower concept was developed utilizing the benefits of three high strength materials: Ultra-High Performance Concrete (UHPC), High Strength Concrete (HSC) and high strength prestressing strands. UHPC is a very high strength, highly durable material with compressive strengths of up to 200 MPa. It allows structural systems to be designed efficiently with overall dimensions comparable to those of equivalent steel systems. HSC on the other hand uses many of the same constituents as lower strength concretes, yet provides superior compressive strengths of up to 100 MPa. The greatest advantage of HSC is its relatively low cost. Although it does not approach the compressive strength of UHPC, the cost can be as low as 10-15 % of UHPC. An efficient design of structures with high strength cementitious materials requires high strength steel as reinforcement, which promoted the use of 1860 MPa prestressing strands as the primary reinforcement for the tower.

The new tower concept was developed with the intent that these high performance materials would provide a cost-effective solution for towers extending to 100 m and above. The new tower design is comprised of six exterior columns along with bracing elements that span the distance between each column. Although these elements may be of different forms [2], concrete panels are used as they provide the tower with a completely enclosed interior. The primary purpose of the panels is to allow the tower to act as a composite structure during

loading without the need to rigidly connect them to the foundation. Both the columns and panels extend the entire height of the turbine and are segmented into sizes that allow for easy transportation. Once on site, the pieces may be erected using two possible construction sequences. The first consists of connecting the columns and panels together in multiple segments at the base of the tower similar to the ones shown in Figure 2. These segments would then be stacked and fastened together by running vertical post-tensioning through the columns (Figure 3). At higher elevations, the number of strands required to provide the tower with sufficient load capacity is reduced. For this reason, two of the radial ducts within each column, shown in Figure 2, will be terminated at 33.5 m and 67.0 m. The centre duct will extend from the foundation to the top of the tower. The second construction sequence requires each of the columns be erected to the first post-tensioning cut-off elevation. The columns are then secured by stressing each tendon, after which, the panels are placed and connected as shown in Figure 4.

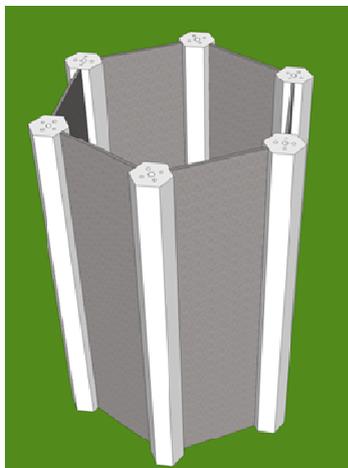
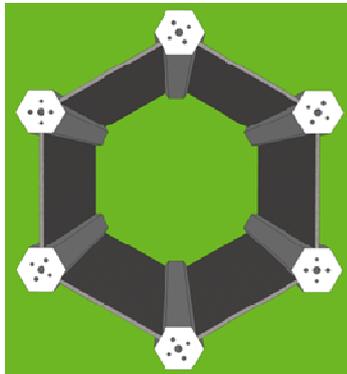


Figure 2: Tower cross-section

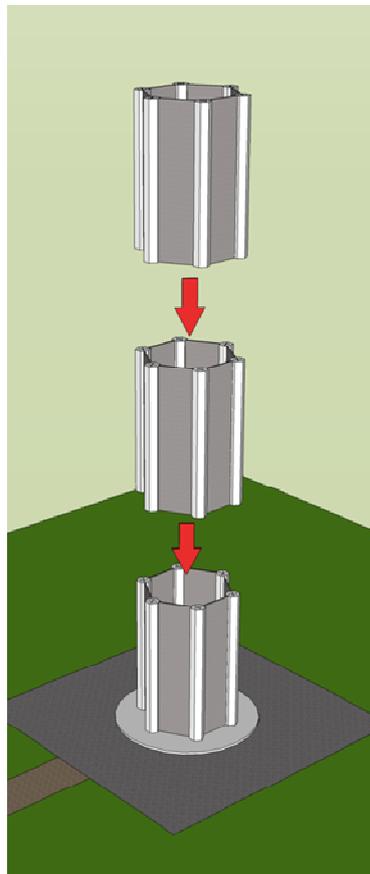


Figure 3: Assembly method 1

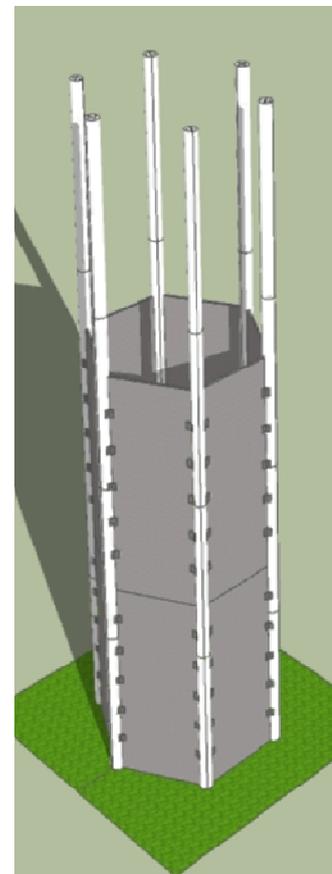


Figure 4: Assembly method 2

Using the new concept as the basis, three separate tower designs have been developed. The benefits associated with the construction and operation of the towers varies to provide the best possible solution for a number of different situations. Each design uses UHPC, HSC, or a combination of the two in all of the precast components. The first consists of HSC columns and UHPC panels. This tower design was developed to reduce to overall cost of the tower.

Although UHPC sections require significantly less material than HSC sections, the costs often make it a less economical option. This is based strictly on required material and unit cost, without incorporating the long-term benefits. The second tower concept also utilizes HSC columns but requires a thicker panel to accommodate the UHPC wet joint discussed later. With the necessary increase in the panel thickness, it was possible to use HSC rather than UHPC on both columns and panels. The third concept consists of both UHPC columns and panels. It has a significantly lower weight than those utilizing HSC because section sizes are reduced. This tower may be used in situations where poor soil conditions exist at a site. The reduced tower weight has the potential to lessen the demand on foundation soils leading to a decrease in both the foundation and transportation costs.

The use of precast concrete provides a number of advantages over cast-in-place alternatives. The pieces are cast in a controlled environment in which tighter tolerances and better material properties can be achieved. In addition, construction can often continue year-around regardless of the weather conditions that may exist on site. The use of precast structures however introduces a unique challenge that cast-in-place options do not face. This challenge is to ensure appropriate connections exist between the precast segments that will allow the structure to behave as a composite system. Precast concrete connections are commonly used throughout the construction industry, with the welded plate connection used most frequently in non-seismic regions. In the case of precast tall wind turbine towers, however, these connections are subject to significantly higher loads. Not only are these loads of high magnitudes, but more importantly fatigue demand on the connections is also increased. As part of ongoing research, three different options for connections have been proposed, which could be integrated into the precast concrete tower designs. These three connections include a horizontally post-tensioned (HPT) connection, bolted connection (BC), and UHPC wet joint connection. Each was selected to ensure satisfactory performance while providing different alternatives for the tower.

A bolted connection was developed for two primary purposes. The first was to reduce on-site construction time and ultimately reduce overall tower costs. The second was to provide a connection with a fatigue life equal to or greater than the twenty year service life of the tower. The challenges associated with the use of a bolted connection are primarily associated with the required tolerances of precast concrete members. Bolted connections are most commonly used in steel structures because of the accuracy at which members are created and holes located. By utilizing oversized holes and slip-critical connections, the connection has the potential to act as a suitable fit for the tower.

The HPT connection increases the construction tolerances when compared to the bolted connection. This connection relies on friction between the columns and panels to resist the large shear forces experienced at the interface. By running post-tensioning through the columns and panels horizontally in the perpendicular direction to the column/panel interface, the capacity of the interface to resist shear is greatly improved.

The third connection involves the use of a UHPC wet joint. This type of connection consists of two precast segments that are connected together by dowel bars protruding from each. The bars extend into a joint that is then filled with UHPC creating a single monolithic structure. The improved tensile capacity and self-consolidating features of UHPC make it a perfect candidate for use in joints. This concept has been applied to the precast tower as a way to connect the columns and panels. The performance of this type of connection has been proven in precast bridge deck applications [3].

3. COMPONENT TESTING

An experimental investigation is currently underway at Iowa State University to test each of these three connections in conjunction with the precast concrete segments to verify the overall tower design. In order to develop an appropriate testing procedure, a complete finite element model was created to identify the critical regions of the tower at two different limit states: operational and extreme. By focusing on the critical region, the primary purpose of the tests is to verify the connection design at all other locations. A series of load combinations were obtained from a U.S. based wind turbine manufacturer for their 2.5 MW turbine operating at 100 meters. A complete set of load combinations for both the operational and extreme limit states can be found in the “Guideline for the Certification of Wind Turbines” (GL 2010) [4]. Besides verifying the design of connections, the experimental investigation is also used to verify the performance of the test specimens at the two limit states. By doing so, their performance can be ensured for serviceability and extreme load conditions.

The outside diameter of the tower along its height is very comparable to a 100 m steel tower. It has a base footprint of approximately 6.0 m, which is reduced to 3.0 m at the top. The outside diameter above 33 m is controlled by the blade tip clearance. The taper in the concrete tower can be contributed to a reduction in both the individual column diameters and panel widths. By minimizing the outside diameter and cross-sectional area of the tower along its height, stresses in the panels and connections due to the torsional load demand of the head mass increase as a result. The top of the tower was found to be most critical at the governing load combination. This load combination had the largest torsional load magnitude of the set. With the location of the most critical tower section being at the top, where component sizes are at a minimum, full-scale test specimens could be constructed and tested in the lab to 100 % of the load demands corresponding to the two limit states.

The critical section of the tower top was isolated and tested using an in-plane test configuration (Figure 5). Three specimens were planned for testing using this process. The first unit, HCUP-BC, utilized HSC columns, a UHPC panel, and bolted connections. The second unit, HCUP-HPT also used HSC columns and a UHPC panel but with a HPT connection. The third unit, HCHP-UHPC, was built using HSC columns, a HSC panel, and a UHPC wet joint connection.



Figure 5: In-plane test identification and setup

4. DETAILS OF TEST UNITS

The details of each of the three in-plane test units are presented in this section. Some minor modifications were made to the details of these test specimens from the original design of the tower to improve their testability. These modifications are believed to have no effect on the performance of the complete tower cross-section. An elevation view of the in-plane test setup is shown in Figure 6. The dimensions shown in this figure are representative of all three test units. The top beam is used to replicate a rigid ring beam that accommodates the transition between the tower and nacelle on a fully erected wind turbine.

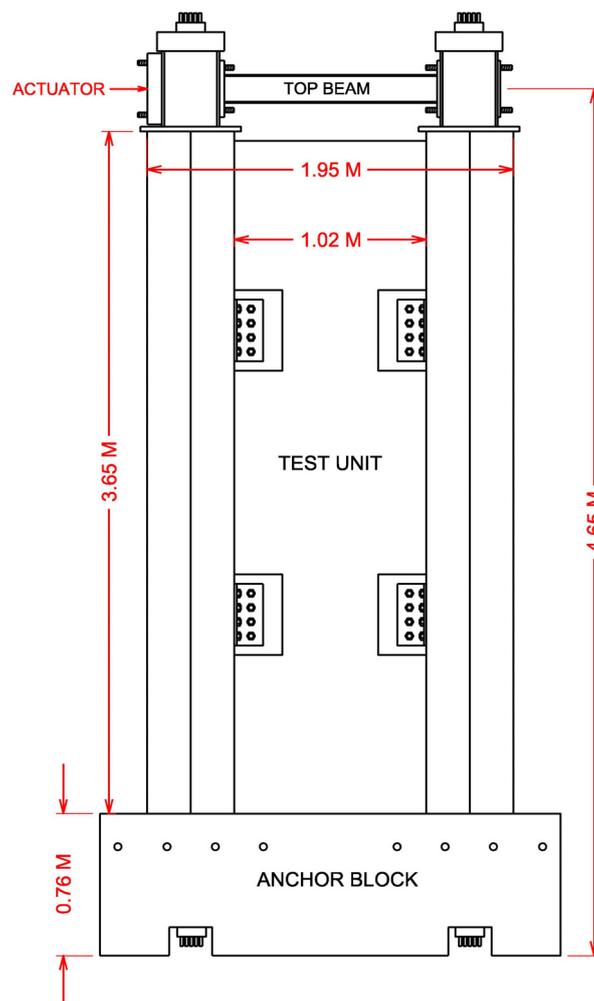


Figure 6: Elevation of in-plane test setup

Figure 7 details the cross-section of the HCUP-BC specimen, in which a steel angle connects the three components through the use of embedded plates in both the columns and panel. The connections were placed at 0.30 m intervals. The surface of the angled plate in contact with the panel is designed as a slip-critical connection. The cross-section of the HCUP-HPT specimen, which illustrates the use of flat PT ducts in the panel, is shown in Figure 8. These ducts are spaced at 1.4 m, and include four 15.2 mm diameter low-relaxation

post-tensioning strands in each. Each strand was stressed to 200 MPa resulting in a tendon force of 640 kN. The HCHP-UHPC specimen of Figure 9 relies on dowel bars protruding from both the column and panel. Once erected, UHPC is used to fill the void of the adjacent pieces to form a composite system.

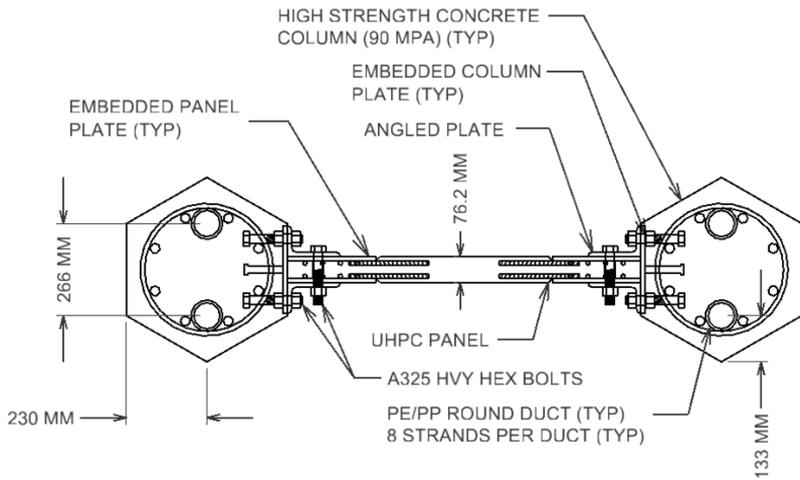


Figure 7: HCUP-BC test specimen cross-section

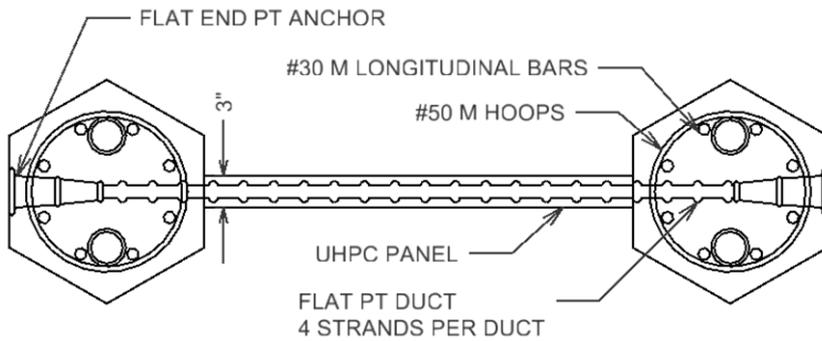


Figure 8: HCUP-HPT test specimen cross-section

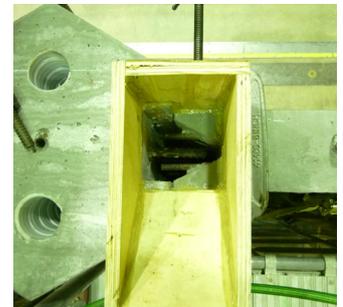
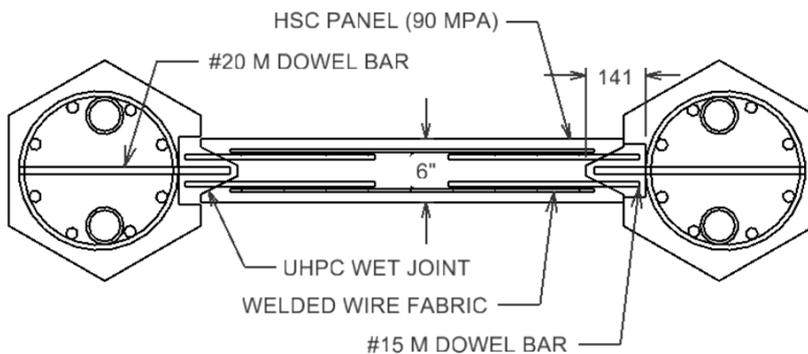


Figure 9: HCHP-UHPC test specimen cross-section

5. LOAD PROTOCOL

The load protocol used for each of the three specimens was nearly identical. The load was applied to each specimen in a quasi-static nature at 44.5 kN increments. Every two steps, the specimens were cycled three times to ensure significant load degradation was not taking place. The loads corresponding to the operation and extreme limit states were identified from the completed finite element model. Loads of approximately 445 kN and 560 kN were used for these limit states. A maximum load of 670 kN would be applied to each specimen in cases where the ultimate capacity of the specimen was not reached. This load level was controlled by the anchor block used to secure the specimens to the strong floor.

6. RESULTS

To date, two of the three in-plane tests have been completed. The HCUP-BC test was loaded to a maximum value of of 670 kN. At the operational load of 445 kN, the columns remained uncracked along the height, while the connections showed no signs of inelastic behavior. Minimal microcracking was detected in the regions of the panel surrounding the embedded connection plates. As expected, the crack angle corresponded to the direction of compression force flow expected between the connections. At the extreme load of 560 kN, many of the same observations were made. The columns and connections remained linear, while further microcracking took place. At the maximum load level, the panel was the only component showing signs of damage in the form of microcracking. At a load of approximately 50 kips, noise generated by the connections during cycling suggested some slip was taking place in the slip-critical portion of the connection. The cyclic hysteretic response of the HCUP-BC specimen is presented in Figure 10. The high-degree of energy dissipation is believed to be a direct consequence of Coulomb damping in the slip-critical portion of the connection assembly. As the specimen was subjected to cyclic loading, kinetic friction opposes the direction of motion much like many of the viscous dampers used today.

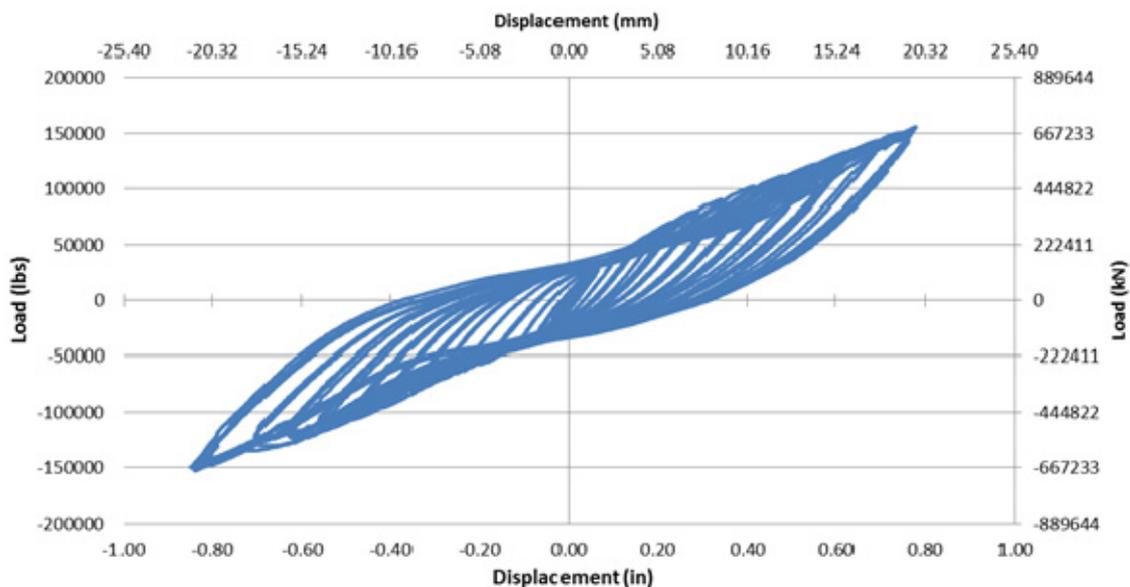


Figure 10: Cyclic hysteretic response of the HCUP-BC specimen

The HCUP-HPT specimen was also loaded to a maximum force of 670 kN. Throughout the entirety of the test, including the operation, extreme, and maximum load levels, there were no signs of cracking in any component of the specimen. The interface between the columns and panel showed no signs of slip suggesting the horizontal post-tensioning adequately increased the shear resistance. The lack of nonlinear behaviour in the specimen is well illustrated by the cyclic hysteretic response presented in Figure 11.

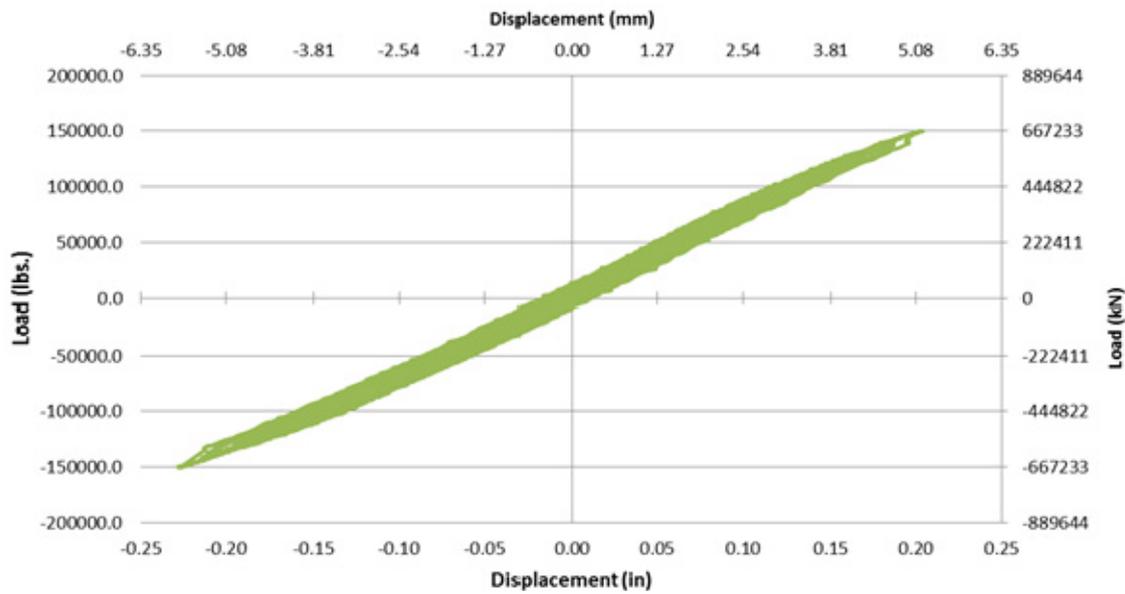


Figure 11: Cyclic hysteretic response of the HCUP-HPT specimen

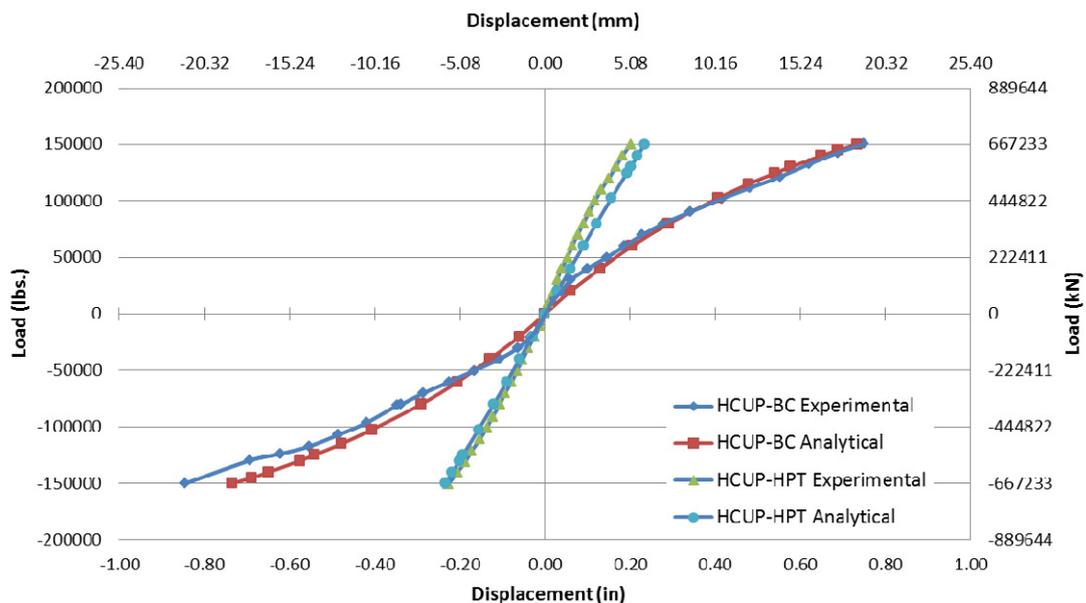


Figure 12: Force-displacement response of HCUP-BC and HCUP-HPT specimens

A comparison of the two tests has been made in Figure 12, which shows the analytical and experimental force-displacement envelopes for both the HCUP-BC and HCUP-HPT

specimens. As previously mentioned, the dimensions and materials used for each component are the same for both tests. With this in mind, it is believed that the variation in the overall stiffness is a direct result of the interface connection used. The bolted connection is localized at 1.52 m intervals, leaving the interface between these connections unattached. As a result, stress concentrations within the panel at the connection points are higher than those found in the HCUP-HPT specimen. In the HCUP-HPT specimen, the entire interface likely contributed to the shear transfer between components by means of static friction. This distributes stress within the panel more evenly than the specimen with bolted connections.

7. CONCLUSIONS

The design of a newly developed precast concrete wind turbine tower utilizing UHPC and other high strength materials has been presented. The results of two of the three test specimens used to verify the capacity of the tower and performance of the connections at different limit states have also been discussed. By observing the results of both the analytical and experimental data, it is concluded that the performance of the tower, including the designed connection, in each case would be satisfactory for the given demand over its expected life. The use of high strength precast concrete in conjunction with post-tensioning allows for the development of cost effective wind turbine tower designs. The towers presented, offer a solution to the transportation challenges currently facing the steel tower industry. The need to improve the efficiency of wind turbines is more prevalent than ever. By transitioning to taller towers, the increased energy production available at higher elevations can be used to a fuller potential.

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