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Innovative ultra-high performance concrete structures

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ABSTRACT: The properties of Ductal® ultra-high performance fibre reinforced concrete are summarized together with the associated design recommendations as a basis for the use of the material in two structures in the city of Calgary – Canada. The first application was in a 20-mm thick shell canopy with no conventional reinforcement and the second was in a post-tensioned, 33.6-metre T-girder that is the central, suspended span of a pedestrian bridge. In the second application, only a small amount of glassfibre and stainless steel reinforcement was used. Both structures are believed to be milestones in the use of this type of concrete, being the first shell structure in the first instance and in the second, the largest component cast in a single pour at the time of its production. Full-scale tests on both structures were performed prior to final construction. A shell canopy was subjected to independent, full factored wind and snow load cases. The actual T-girder was subjected to 90% of the factored service load, first uniformly applied over the top of the slab and then eccentrically, over a lateral half of the slab. A summary of the design assumptions and choices, tests and results, and building processes is presented.

1 INTRODUCTION

Among the new concrete types developed during the last two decades, ultra high performance fibre reinforced concretes (UHPFRC) have very high compressive strength and durability with good flexural resistance and ductility. One such concrete available commercially is known as "Ductal®". This new material was selected for a pedestrian bridge and the roof system of a light rail transit (LRT) station in Calgary, Canada, because of its predicted long durability and superior aesthetics.

The 33.6-m prestressed drop-in girder is thought to be the largest single volume piece of Ductal® cast in a single pour (up to the time of its production). The canopies for the LRT station (Adeeb et al. 2005, Perry et al. 2005) are believed to be the first case of using this material in the structural form of an unreinforced shell.

Although this material had been used previously around the world in various projects, such as a pedestrian bridge in Seoul, Korea (Ricciotti 2001) and a highway bridge in Iowa, USA, (Perry et al. 2006), the City of Calgary required testing of these structures prior to accepting its use in these novel applications.

Experimental programs were therefore developed where representative sections of the bridge were loaded to failure or subjected to fatigue tests prior to casting of the actual girder. The girder was also tested before it was installed. For the LRT station, a full-scale prototype canopy was tested for both static and dynamic responses.

2 BACKGROUND RESEARCH

2.1 *Materials*

The U.S. Department of Transportation 2006 indicates that UHPC "…tends to have compressive strength over 150 MPa, internal fibre reinforcement to ensure

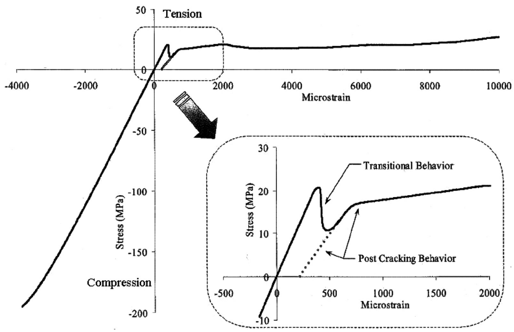

Figure 1. Experimental stress-strain curve for Ductal® (Chanvillard & Rigaud, 2003).

nonbrittle behavior and a high binder content with special aggregates. Furthermore, UHPC tends to have a very low water content and can achieve sufficient rheological properties through a combination of optimized granular packing and the addition of high-range water reducing admixtures".

The same reference indicates that the only UHPC currently available in North-America is Ductal®. Aspects about the characteristics of this material have been described elsewhere (Chanvillard & Rigaud 2003, Centre Scientifique et Technique du Bâtiment 2005, Rosa 2005, Brandao 2005). Some properties are presented in Table 1.

The fibres used in the concrete for the canopies were 12-mm organic PVA (poly-vinyl alcohol), while 13-mm steel fibres were used in the bridge concrete.

2.2 *Design Recommendations*

Design stress-strain curves in compression and tension are detailed by the U.S. Department of Transportation 2006b and reproduced in Figure 1. Chanvillard & Rigeau 2003 presented more details on the tensile behaviour of this material. According to these authors, the theoretical tensile strength of a steel-fibre Ductal® section is equal to the fibre strength, 2,500 MPa, multiplied by its volume percentage, 2%, times a coefficient that takes into account the fibre orientations, assumed equal to 0.6, times a fibre/matrix efficiency coefficient, equal to 0.5.

The calculation gives a theoretical tensile strength of 15 MPa, which is of the same order of magnitude as the average results of the direct tensile tests reported. Flexural tests on varying sizes specimens were also reported. From these results, after correcting the flexural tensile strength by a scale factor, the authors estimated a direct tensile strength of 11.5 MPa, which is the value that was used in structural design calculations for the bridge section tested here.

Furthermore, design recommendations are based on another study where diagonal and vertical

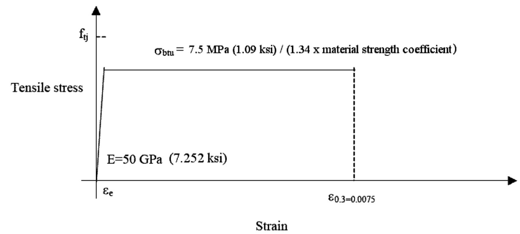

Figure 2. Recommended tensile stress-strain curve for design with Ductal® (Behloul, 2006).

Table 1. Ductal® concrete properties after Heat Treatment.

Characteristic	Result
Compression Strength, N/mm^2 (1)	152.3 (±6.3)
(standard deviation in brackets)	251.4 (±5.5)
Tensile Strength at First Crack, N/mm^2 (2)	8
Density kN/m^3 (2)	25
Flexural Strength N/mm^2 (2)	30 to 50
Tensile modulus of elasticity, N/mm^2 (2)	50 to 60 × 10^3

(1) Test result: 1st value for the LRT Station (Ductal® with PVA fibres), 2nd for the bridge (Ductal® with steel fibres)
(2) Lafarge North America

70 × 70 × 280 mm samples were cut from a 3-m long I-beam, tested in flexure, and the results compared to those from samples moulded to the same size and tested under the same conditions (Behloul 2006). The objective of this last study was to measure the influence of changing fibre orientation during placement of the material on the resulting strength (horizontal samples should also have been tested but were lost during the cutting process).

The authors introduced another fibre orientation factor, the "K-factor", which was the maximum flexural strength obtained from moulded prisms divided by the flexural strength obtained from prisms cut from the I-beam. A lower-bound behaviour for design was suggested as presented in Figure 2. The 1.34-value shown in this graph is in respect of this last fibre orientation K factor. The recommended design curve is more conservative than the curve presented in Figure 2, but takes into account other aspects that may be present in a field application.

3 CONSTRUCTION AND TESTING

3.1 *Canopies for the Light-Rail Transit Station*

Adeeb et al. (2005) describe the analyses, design and construction of the structure. The station roof canopy was designed as a series of 20-mm thick shell panels,

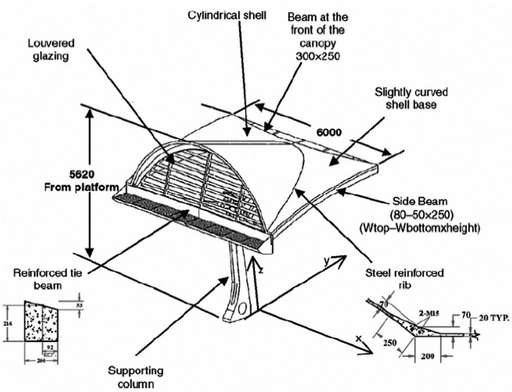

Figure 3. Schematic of a single canopy unit (Adeeb et al., 2005).

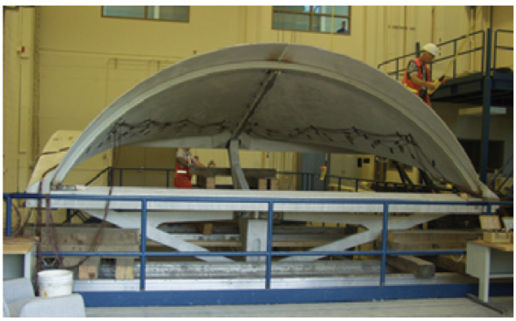

Figure 4. Full scale canopy prototype (Adeeb et al., 2005).

each supported by a single column. The design objective of the canopy shell was to use a thickness which balanced the minimum to facilitate the injection of the material into a thin section while minimizing the thickness for weight and consumption of material. A schematic of a single unit is presented in Figure 3.

3.1.1 *Testing*

The full-scale prototype of a single panel (Figure 4) was tested to factored wind and snow loadings determined by finite element analyses performed by Montreal-based Strudes Inc. The boundary conditions in the test were those deduced from finite element analyses on sets of three joined panels as intended in the actual structure. The canopy withstood the maximum snow and uplift loads without damage.

Dynamic tests were performed to determine the frequency of natural vibration. The natural frequency of the first mode of vibration obtained in the experimental program, 2.0 Hz, was smaller than that obtained from the FEA (2.62 Hz), indicating that the model was stiffer than the actual structure. The modal shapes in the two cases were similar.

Figure 5. Canopy construction with temporary scaffolding (Adeeb et al., 2005).

Figure 6. The LRT Station. (Lafarge Canada Inc.).

3.1.2 *Construction*

Two half shells were cast by injection moulding into closed steel forms at a specific pressure and flow rate determined from pilot tests. To avoid shrinkage cracks, the moulds were rotated at specified times during initial curing and the half shells demoulded 12 hours after casting. Other parts of the precast structure include the columns, tie beams, struts, and troughs. The columns were installed first and temporary scaffolding assembled. The canopies were placed onto the scaffolding, and the struts attached to the shells and the columns with welded connections (Figure 5).

The LRT station was completed on schedule and opened to the public in June, 2004. (Figure 6)

3.2 *Pedestrian Bridge*

The 33.6 m girder is a precast drop-in "T-section" for of a single span 53 m pedestrian bridge between two high performance concrete abutments. The bridge stretches across 8 lanes of traffic (Figure 7). The girder is 1.1 m in depth at mid-span with a 3.6 m wide deck and weighs approximately 100 tons. GFRP (glass fibre reinforced plastic) and stainless steel bars were also utilized as a redundant, passive reinforcing system.

Figure 7. The bridge. The drop in girder is the part with the thin web and wider flanges than the abutments which stretch out from the column's construction.

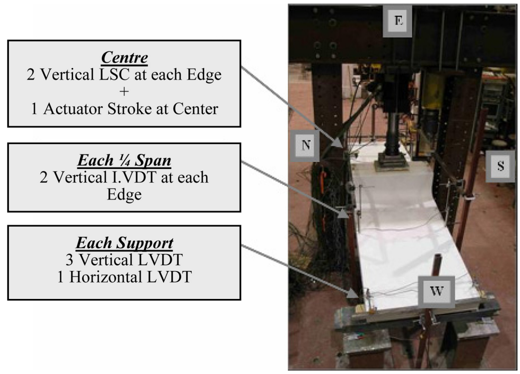

Figure 8. Bridge section test instrumentation.

3.2.1 *Testing*

Load tests were performed on three, one-metre long, full-width and full-depth slab sections and on the actual girder.

3.2.1.1 Section tests

The tests were designed to load the specimens as simply-supported beams across their width with a central point load in terms of both width and length. The centre-to-centre span width was 3.3 m. One support was designed to act as a pin support and the other as a sliding support. Displacements were measured using displacement transducers and recorded electronically. Vertical displacements were measured at the mid and quarter-points on each side, across the width of the cross-section.

At each edge of the cross-section, three vertical displacements were measured over the supports (one at each side and one at the centre). The horizontal displacements were also measured at each support. Fourteen displacement readings were thus obtained throughout the testing in addition to the stroke of the actuator. An overall view of the test arrangement and displacement transducer location is shown in Figure 8.

The UHPFRC was reinforced with 13-mm long steel fibres (2% by volume). In the first and second tests (Tests #1 and #2), the sections were reinforced with GFRP bars and tested to failure under monotonic loading. The specimens cracked and failed at similar loads.

The third specimen (Test #3) had no reinforcement other than the steel fibres. Initially, the specimen was loaded until its first crack. Subsequently the specimen was subjected to one million cycles between 20 and 80% of the design service load, followed by a second million load cycles over a load range of 20 to 80% of the observed first-crack load.

As the specimen did not fail under this loading regimen, nor was there any observed degradation of stiffness, a third million load cycles were applied between 20 and 80% of the failure load of the sections with GFRP reinforcement.

Static tests were performed to evaluate the specimen stiffness several times during the fatigue test. The service load range was not observed to cause damage to the specimen. Some stiffness degradation was noted during the beginning of the third million cycles of loading, but stabilized at about two thirds of the original stiffness. Following this fatigue testing, the specimen was loaded to failure, with collapse occurring at a load higher than predicted.

From the section tests the following could be observed:

- A smaller value than expected (39 GPa) was estimated for the elastic modulus from the static tests with GFRP and a higher value of 51 GPa was estimated from the dynamic tests and without GFRP. The elastic modulus for this material is typically in the range of 55 GPa. This low value of 39 GPa is an anomaly that requires further investigation. The inclusion of the GFRP may have impacted this theoretical calculated value;
- The first crack (that could be observed) occurred with little difference among the three specimens, leading to an estimate of 1st crack tensile strength of over 8.0 MPa;
- In all cases, the failure load was consistently higher than the designer's prediction;
- The stiffness and failure load of specimen #3, delivered without GFRP reinforcement, were about 30% greater than the values obtained from Tests #1 and #2. The presence of the reinforcing bars may be causing reorientation of fibres local to the bar, and weakening the composite. Further investigation is necessary;
- After loading to cracking, specimen #3 showed a 10% stiffness loss, with no further loss of stiffness detectable after the first million (with a load range between 20 and 80% of designed service load) and 2nd million cycles (with a load range between 20

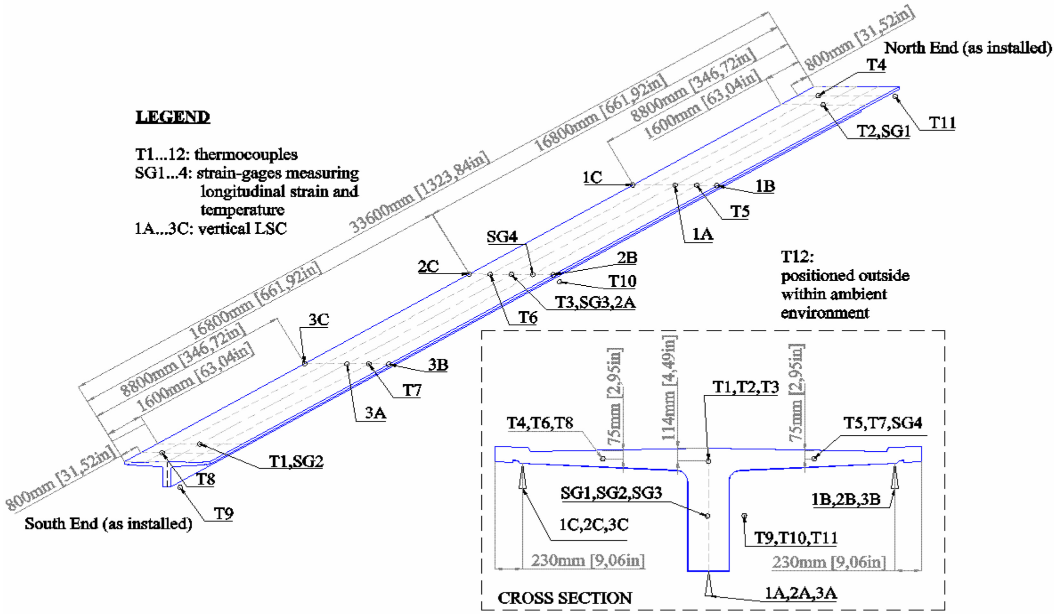

Figure 9. The bridge full-scale test instrumentation.

and 80% of actual cracking load) After the third million cycles (between 20 and 80% of expected failure load), the stiffness was 65% of its initial value;

- Little increase in crack width was observed after the first and second million cycles. The width of the crack grew approximately 140% within the third million cycles, mainly at the beginning of those cycles, suggesting some strengthening mechanism is in effect.

3.2.1.2 Full-Scale tests

Embedded thermocouples, strain gauges, load cells and displacement transducers were used as instrumentation during two simply-supported load tests of the girder. 0.9 × 1.7 × Service Load was first applied concentrically and uniformly over the top of the slab, and then eccentrically over half of the top of the slab (a half created along the longitudinal centre-line) (Figures 9 and 10).

For both load cases, the displacements recovered fully upon load removal, indicating elastic behaviour (no short term viscous behaviour). The average E value from both load cases was estimated as 57 GPa. Results showed the measured self-weight to be close to predicted and displacements to be smaller than predicted.

3.2.2 *Construction*

To ensure proper and efficient mixing, this material should be batched in a high shear mixer. Since the precast facility was only able to mix 1.25 m^3 per batch in a high shear mixer, the entire UHPFRP amount was prepared over a 16 hour batch cycle and poured into 4 ready mix trucks until the beginning of the girder casting. The ready mix trucks were filled in a specific filling order which ensured the same average pot life across the entire prepared material. The UHPFRP was kept agitated in the ready mix trucks at a low revolution until ready for casting. Workability and plastic behavior remained consistent throughout the entire agitation and casting period. Favorable cool temperatures during the batching and casting periods assisted in maintaining the rheological behaviour of the material.

Figure 10. The bridge full-scale test.

Figure 11. Casting of the girder nearing completion.

This unprecedented, monolithic girder pour was achieved by filling up the stem of the T-section from one end of the girder. Random fibre orientation in the 3.6 m deck was ensured by using a concrete bucket of the same width as the top flange of the girder (Figure 11). After filling the mould, the top surface of the girder was completely enclosed due to the longitudinal curvature and transverse drainage slopes and to ensure no de-hydration of the cast material.

4 CONCLUSION

Both structures are milestones in the use of the UHPFRC and both were thoroughly tested prior to construction. In each case, the test results indicated good performance, normally better than predicted. Full scale load tests further validated that the designs provided a good prediction of the material performance and an acceptable design.

Some questions raised from the experimental programs still need to be answered through further research, such as the observed greater stiffness and strength in the section non-reinforced with GFRP and submitted to cyclic loading – compared to the same specimen shape reinforced with GFRP and tested statically.

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