Malaysia's 1st "Ultra-High Performance Ductile Concrete" Composite Bridge in a Marine Environment




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INTRODUCTION

Recently Westports Malaysia Sdn. Bhd. called for tenders to expand its container cargo terminal at Pulau Indah, Port Klang. The project included the construction of four new access bridges (namely Bridge24, Bridge25, Bridge26 and Bridge27) connecting the new wharf to the container stacking yard. Of the four bridges, Bridge25 was to be designed as a special access bridge for overweight and oversized cargo with trailer payloads of up to 3,072 metric tonnes.

Project owner Westports Malaysia Sdn. Bhd. had appointed HSS Integrated Sdn. Bhd. (HSSI) as the Engineer for the project and Putra Perdana Construction Sdn. Bhd. as the Contractor.

HSSI specified the use of Grade150 "Ultra-High Performance ductile Concrete" (UHPdC) precast prestressed beams for Bridge25 in order to carry the exceptionally heavy live loads while maintaining a shallow beam depth of 1m.

The material has also been reported to be highly durable and has the ability to provide a service life in excess of 100 years (JSCE, 2006). Being located in a marine environment, Bridge25 would benefit from UHPdC's extra resistance against chloride attack, which would be a major advantage.

The other three bridges adopted conventional Grade50 concrete composite bridge decks as they were designed for normal highway bridge loadings.

FEATURES OF BRIDGE25

The superstructure of Bridge25 consisted of six 13.0m spans with five of the spans at 22.5m width and the sixth span at 40.5m width. The substructure of the bridge was founded on 800mm diameter Grade 80 spun concrete piles driven to set at an average pile depth of approximately 36m. The piles were framed into Reinforced Concrete (RC) crossheads measuring 1.5m wide by 0.6m deep.

The structural analysis indicated that a total of 77 (seventy-seven) 1,400mm deep by 1,600mm wide conventional Grade50 precast concrete T-beams spaced at 2.0m centres would be required for the whole bridge deck. However, as the T-beams came with a limitation of insufficient freeboard (600mm) below the bridge soffit, the beam depths had to be reduced.

The UHPdC option was considered in order to achieve the same high load carrying capacity required with a shallower beam depth, so that a minimum freeboard of 1.0m from the high tide water level could be achieved.

Figure 1: Construction of Bridge25 using UHPdC beams

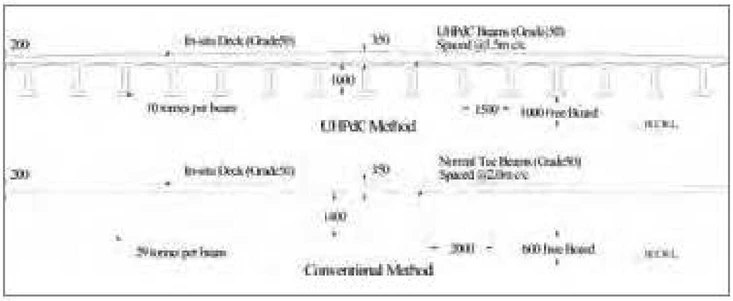


Figure 2: Sectional views of Bridge25 deck options

Figure 2 gives a comparison of the cross-sections of the composite bridge decks for both the UHPdC and the conventional Grade50 concrete T-beam options. For the UHPdC beam option, a total of 102 precast UHPdC beams were required for Bridge25, with each UHPdC beam spaced at 1.5m centre to centre.

The UHPdC beam option also gave a significant dead weight saving of approximately 66% per beam compared to the conventional Grade50 precast concrete beam design.

The composite bridge deck would be completed with a Grade50 in-situ RC deck, with an average thickness of 275mm.

Accordingly, a proposal for using Grade150 UHPdC beams was presented to the Client and subsequently approved.

OVERSIZED CARGO LOAD

In Malaysia, most bridges are designed to the highway bridge traffic loadings specified in the Design Manual for Roads and Bridges (BD 37/01). Bridge25 however, has to be designed for use by special trailers known as "Gold holder 24 lines".

These trailers have a total of 96 axles, arranged with 24 axle lines spaced at 1.5m centre to centre longitudinally and 3 axles side by side at 3.0m centre to centre transversely.

Figure 3 shows an example of the type of overweight and oversized live load that will be using Bridge25.

According to the specialist transporter's specifications, these trailers are able to transport cargo payloads up to a maximum of 3,072 metric tonnes at a time.

Factored live axle line loads of 387kN/axle and 458kN/axle for the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) respectively were used in the structural analysis of the multi-span composite bridge.

Figure 3: Example of similar oversized cargo using the Gold holder 24 lines trailer

GRADE150-UHPdC

The raw materials for the Grade150 steel fibre reinforced Ultra-High Performance ductile Concrete (UHPdC) used in the precast pretensioned beams include Type I Ordinary Portland Cement, densified silica fume containing more than 92% silicon dioxide with particle sizes ranging from 0.1µm to 1µm and surface fineness of 23,700m²/kg, and washed-sieved fine sand with particle sizes ranging between 100µm and 1,000µm. A polycarboxylic ether (PCE) based super plasticiser was used to ensure good workability of the mix. The micro steel fibres specified for the mix were required to have an ultimate tensile strength of 2,500MPa. The formulation of the mix however has been patented under the trade name DURA®.

For this project, a benchmark value for performance was set for the DURA® UHPdC material to achieve. It was specified that the average 28-day cube compressive strength and modulus of rupture should not be less than 150MPa and 20MPa, respectively.

UHPdC PRESTRESS BEAM

Figure 4 shows the cross-sectional dimensions of the UHPdC beam used on Bridge25. The total length of the beam was 12.1m. The top flange was 1,490mm wide and reinforced with 6 pieces of 15.2mm diameter strands, while the bottom flange was 500mm wide and reinforced with 18 pieces of 15.2mm diameter strands. The web was designed as a thin membrane element of 175mm thickness. Unlike conventional RC beams where steel reinforcement or stirrups are used as primary resistance against all major tensile/shear forces that may occur in the stress/load path inside the beam, the UHPdC beams do not have any conventional

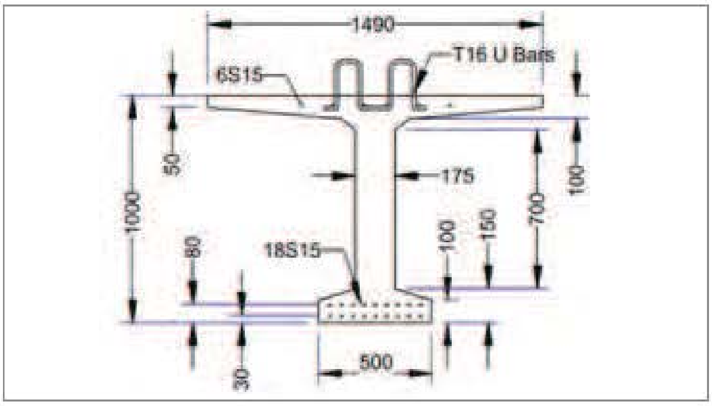


Figure 4: UHPdC beam section details (in mm)

steel reinforcement or stirrups in its section other than the starter bars in the top flange. These starter bars are required only for making the connection to the in-situ concrete deck. Instead, steel fibres are used to enhance the tensile/shear strength of the UHPdC and to improve beam ductility.

LIMIT STATE DESIGN

Bridge25 was designed as a six-span continuous composite bridge with rigid joints at the supports.

Table 1 summarises the critical design force effects both in terms of SLS and ULS from the structural analysis of Bridge25.

Prior to construction of Bridge25 and to verify the strength of the precast UHPdC beam's strength, the Client and the Engineer had requested for full scale performance load tests on the UHPdC beams both in flexure and in shear until failure.

For the purpose of the verification load tests, only the precast beams (i.e. without the RC deck) were tested. First principles of solid mechanics were used to calculate the design load actions on the beams in the absence of the deck. The calculations show that the precast UHPdC beam only (without the deck) will resist 74% of the design bending moment effect and 79% of the design shear force effect of the composite section. These values are tabulated in Table 1

The UHPdC beam manufacturer had guaranteed that the precast UHPdC beam only (without the deck) would be able to resist a minimum design moment of $M_{Rd\,beam}$ = 3,750kNm and a minimum design shear force of $V_{Rd\,beam}$ = 1,420kN.

Table 1: Design force effects

	Bridge		Beam Only	
Design Forces	SLS	ULS	SLS	ULS
Positive Moment, kNm	2,720	3,228	2,013	2,389
Negative Moment, kNm	-2,853	-3,386	-	-
Shear Force, kN	1,299	1,541	1,026	1,218

Two prototype UHPdC beams were then manufactured and subjected to the strength verification tests as described below.

DESTRUCTIVE PERFORMANCE LOAD TEST

Figure 5a shows the set-up for the flexural strength verification test. The flexural beam was set in a three-point test configuration with a simply supported span of 11.9m. The applied force from the hydraulic jack was placed at the centre of the span with a stiff steel plate/beam to distribute the load across the top flange of the beam. One end of the beam was supported on a pinned support, while the other end was sitting on a pin and roller support. The pins and rollers were greased to minimise friction in order to give the required freedom of rotation and horizontal translation.

Three sets of Linear Variable Differential Transformers (LVDTs) were used to capture the vertical displacements of the beam during testing. LVDT1 was the major interest of the test as it was located at the mid span of the beam (i.e. where the applied load was situated). LVDT2 and LVDT 3 were placed at both supports to monitor support stiffness.

Figure 5c shows the set-up for the shear strength verification test. The beam was simply supported over a span of 5.67m between centre lines of the supports. The applied concentrated load was similarly placed at the top flange of the beam in a three-point test configuration. The ratio of shear span to effective depth used in the shear test was 2.

The results of both the flexural and shear tests are presented in Figure 5b and Figure 5d respectively, where $P_{cr,exp}$ denotes the applied load measured at first structural cracking, determined by visual tracing of cracks on the specimens or as detected on the load versus displacement curves (whichever is lower), and the symbol $P_{u,exp}$ denotes the maximum applied load recorded at the end of each test.

As the cracks were extremely fine and difficult to be seen by the naked eye, water was sprayed onto the surface of the beam at each load step, to help obtain a clearer trace of the cracks.

In the flexural strength test, the first flexural cracks were observed at the applied load of $P_{cr,exp}$ = 870kN. Using a microscopic crack detector, the crack widths observed were in the order of 0.01mm under this load. The cracking moment capacity (M_{cr}) of the beam can therefore be calculated as follows:

M_{cr} = Applied load x Span/4 + moment due to self weight of girder = 870 x 11.9/4 + 8 x 11.9²/8 = 2,730 kNm.

The resulting M_{cr} proved that the UHPdC beam did not crack at the Design SLS load condition (see Table 1).

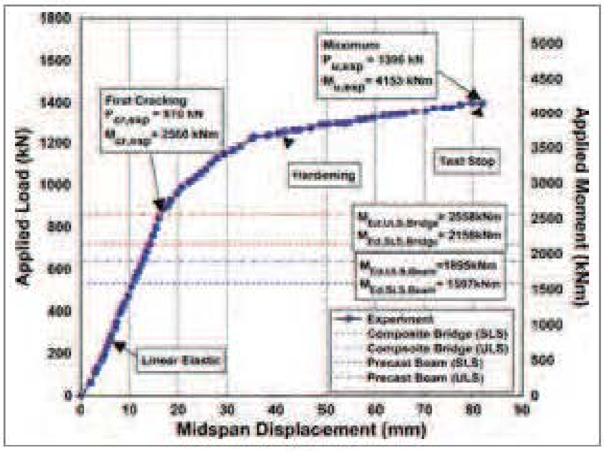
As the applied load increased further, more cracks appeared but these were fine and uniformly distributed across the span. Observations showed that these multiple flexural micro-cracks, which appeared "smeared" across the bottom flange/web area, had crack widths of approximately 0.2mm to 0.3mm at the applied load of P = 1,230kN, corresponding to the guaranteed load carrying capacity in flexure ($M_{Rd,beam}$). As a result, the Design ULS load condition was met.

The maximum applied load captured in the flexural test was P = 1,396kN, which corresponded to a maximum applied moment of 4,153kNm at the mid span, confirming that the UHPdC beam had ample positive moment resistance over the design positive moments shown in Table 1.

The resulting plot of the applied load versus mid-span displacement curve of the test beam in Figure 5b also showed that the test beam exhibited linear elastic behaviour prior to cracking. The mid-span deflection at first cracking was captured to be 16mm. The beam was able to undergo a further 64mm of mid-span deflection before the maximum applied load of P_u= 1,396kN was reached.

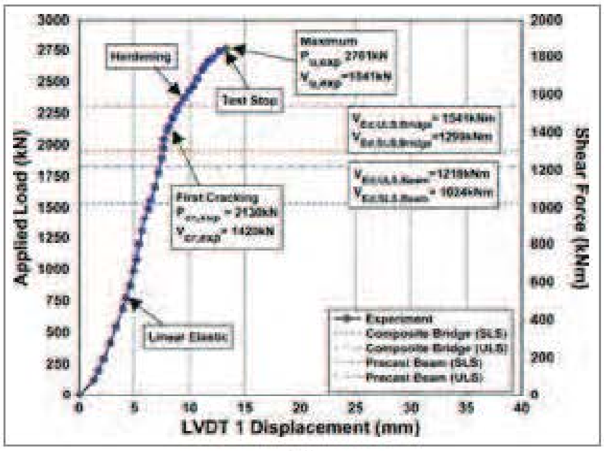
In the shear strength test, the measured first cracking load was P_{cr} = 2,130kN (i.e. V_{cr} = 1,420kN) which co-incidentally equalled the guaranteed shear force capacity of the UHPdC beams ($V_{Rd,beam}$).

(a)



(b)

(c)



(d)

Figure 5: (a) Flexural strength verification test set-up, (b) Flexural test experimental result compared against design moments criteria, (c) Shear strength verification test set-up, (d) Shear test experimental result compared against design shear forces criteria

Assuming that the shear force was taken by only the rectangular section of the web, the cracking shear strength of the beam can be approximated as follows:

$$\tau_{cr} = 2130 \times 2/3/(175 \times 1000)$$
$$= 8.1\text{MPa}$$

Figure 5d shows the plot of the applied load versus displacement curve of the beam tested in shear. Beam deflection generally showed linear elastic behaviour before cracking. The monolithic section underneath the applied load (i.e. near LVDT1) was captured with a deflection of 8mm at the first web shear cracking load effect of V_{cr} = 1,420kN.

After first shear cracking, the beam exhibited displacement hardening behaviour until the maximum applied load of P_u = 2,761kN (V_u = 1,841kN) was recorded.

The shear test clearly demonstrated that the UHPdC beam section had sufficient reserves in shear resistance beyond the design shear force (see Table 1).

QUALITY CONTROL AND INSPECTION PROCEDURES

The UHPdC supplier had assured that each batch of UHPdC would have a minimum average cube compressive strength and flexural strength of 150MPa and 20MPa, respectively.

Very strict quality control and inspection procedures were implemented during the production of the 104 prestressed precast UHPdC beams for this project. (102 nos. for the bridge construction and 2 nos. for the destructive load test). Manufacturing of the first UHPdC beam started in early April 2012 and all 104 beams were completed only at the end of July the same year.

Each single piece of precast beam was produced from a new batch mixing of the UHPdC material, and control samples were collected from every batch of the UHPdC mixes.

For this project, a total of 104 sets of UHPdC samples were collected (each set consisting of a minimum of six 100mm cubes and a minimum of one prism). Figure 6 presents the statistical data on the various strength test results of the control specimens.

The cube compressive strength f_{cu} for each batch was determined using a minimum of three cube specimens. The early age 1-day strengths and the 28-day strengths were measured. These are presented here in Figure 6a and Figure 6b, respectively. In general, the UHPdC material was able to achieve 1-day and 28-day characteristic strengths of 67MPa and 151MPa, respectively.

The flexural toughness test in accordance with ASTM-C1018 (1997) was carried out to determine the flexural properties of the UHPdC. Figure 6c and Figure 6d show respectively the frequency distribution of first cracking flexural strengths (f_{cr}) and the moduli of rupture (f_{ct}) of the UHPdC. The test results show that the characteristic first cracking strength and modulus of rupture after 28 days are 10.8MPa and 22.2MPa, respectively.

Though every batch of UHPdC was tested and found to satisfy the required standards, the Contractor and Engineer were both concerned that the material testing results might not fully cover and demonstrate the structural performance of the UPHdC beams. As such, the Engineer further requested for five additional UHPdC beams to be load tested (non-destructively) up to their Design SLS loadings as given in Table 1.

The Client then proceeded to randomly select five beams (i.e. Beams 22, 49, 55, 78 and 95) to be subjected to the SLS load proof test. The passing criterion for the SLS load proof test was for the beams to be able to carry the specified SLS loading without cracking.

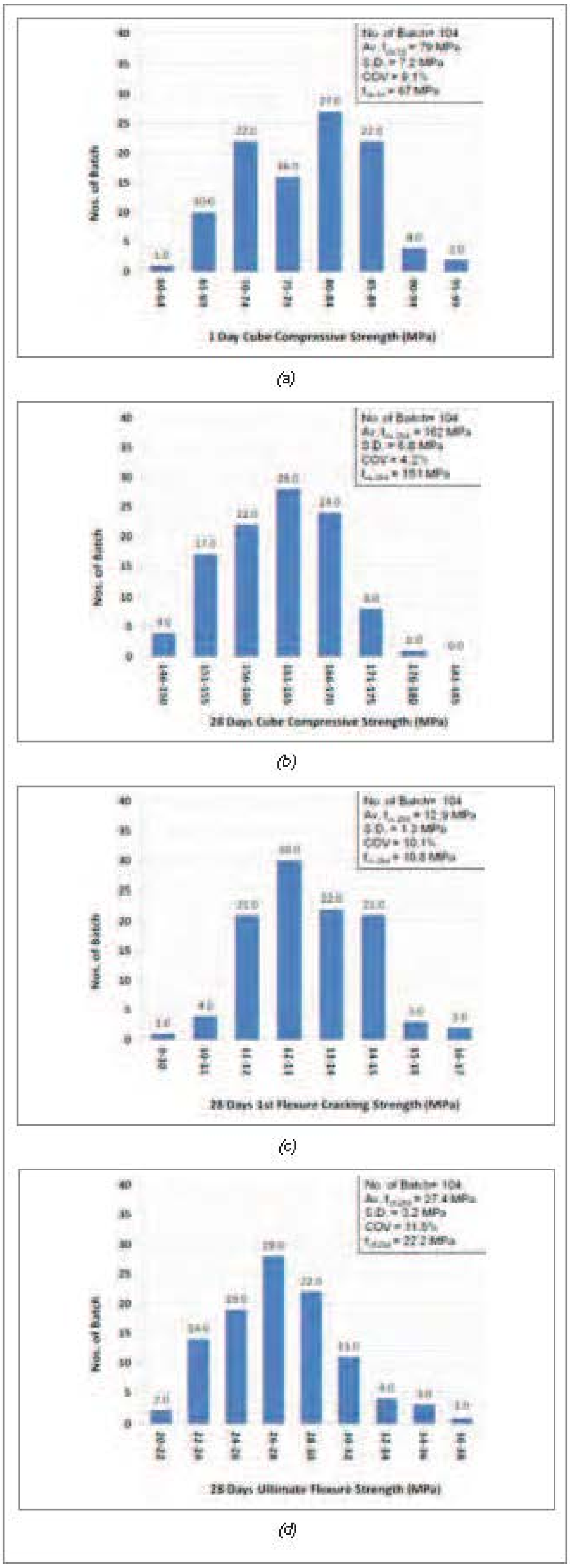


Figure 6: Frequency distribution of (a) 1-day compressive strengths, (b) 28-day compressive strengths, (c) 28-day first flexural cracking strengths and (d) 28-day moduli of rupture for the UHPdC material used in the precast beams

(Continued on page 32)

All five selected beams passed the SLS load proof tests without cracking.

ENVIRONMENTAL IMPACT CALCULATIONS (EIC)

This section illustrates an example of Environmental Impact Calculations (EIC) for Bridge25, based on the two different deck options shown in Figure 2. The purpose of this exercise was to illustrate how the advancement of UHPdC technology could help to reduce the carbon foot-print or to reduce the consumption of primary energy to construct the same bridge.

Undertaking a rigorous EIC is a complex exercise and the data required for the calculation varies from country to country due to different local practices and the technologies available. Table 2 summarises the inventory data of the materials used for this comparative study on the two bridges. Details on the derivation of this inventory data can be obtained from Voo and Foster (2010). The table has been prepared for determining the equivalent Embodied Energy (EE), CO_2 content and 100-year Global Warming Potential (GWP) of each particular concrete mix design and the materials used. The information may be updated more frequently as the industry continues to improve its processes.

Table 2: Inventory data for construction material (Voo and Foster, 2010)

	Units	UHPdC	G50	Strand & Reo.
Density	kg/m³	2388	2344	7840
Cement	kg/m³	720	480	-
EE	GJ/m³	7.77	2.70	185.8
CO_2	kg/m³	1065	480	17123
NO_x	kg/m³	4.86	1.66	55.4
CH_4	kg/m³	0.76	0.12	30.7
100-yr GWP	kg CO_2 eq./m³	2532	978	34392

Elrod (1999) defines GWP as a measure of how a given mass of green house gas is estimated to contribute to global warming over a given time interval. It is a relative scale that compares the gas in question to that of the same mass of CO_2 and a 100-year of time horizon is most commonly adopted, as per the Kyoto Protocol. The GWP formulation can be ambiguous and the adequacy of the GWP concept has been widely debated since its introduction. To date, very little work has been done on this area and the formulations of the 100-year GWP have yet to be unified.

However, Voo and Foster (2010) for the first time suggested that the 100-year GWP can be expressed as:

$$\text{100-yr GWP} = CO_2 + 298\ NO_x + 25\ CH_4 \tag{1}$$

In this comparative study, calculation of material quantities will only cover the super structure, whereas the substructure is assumed to be the same for both cases. A comparison of the EIC results is presented in Figure 7.

In terms of material consumption, the UHPdC option consumed 27% less raw material than the conventional option. In terms of environmental impact, the UHPdC technology has 20.6% less embodied energy and 19% less CO_2 emissions. In terms of the 100-year GWP, the UHPdC solution provides for a reduction of 14.5% over that of the conventional solution.

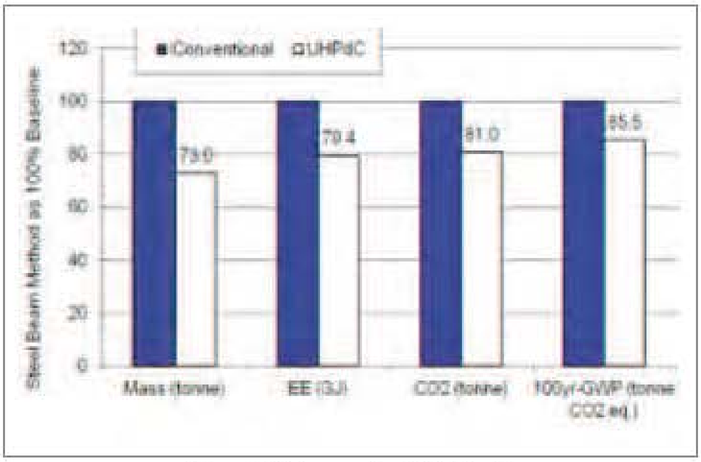


Figure 7: EIC comparison for Bridge25

It also needs recognition that in this example, only the savings at the level of the superstructure have been considered. Further savings will result from the lighter weight of the UHPdC solution requiring a smaller substructure, foundations and lighter machinery and lower transport costs.

DURABILITY DESIGN

To date, there is no single agreed or unified method in the world for obtaining a measure of the 'durability' of a concrete structure in aggressive marine conditions. However, the most commonly accepted model of service life prediction concerning the corrosion of the reinforcing bars was developed by Tuutti (1982). Figure 8 shows the schematic evolution of damage of RC structures due to steel corrosion. In this model, the service life is composed of two periods. The first is the initiation period (t_i) related to the penetration of the chlorides or carbon dioxide, i.e. the aggressive agents, until depassivation of the steel reinforcing bars and the beginning of corrosion in the bars. Second is the propagation period (t_p) where corrosion proliferates. Such a model proposes that service life is to be determined as a function of an acceptable limit of corrosion.

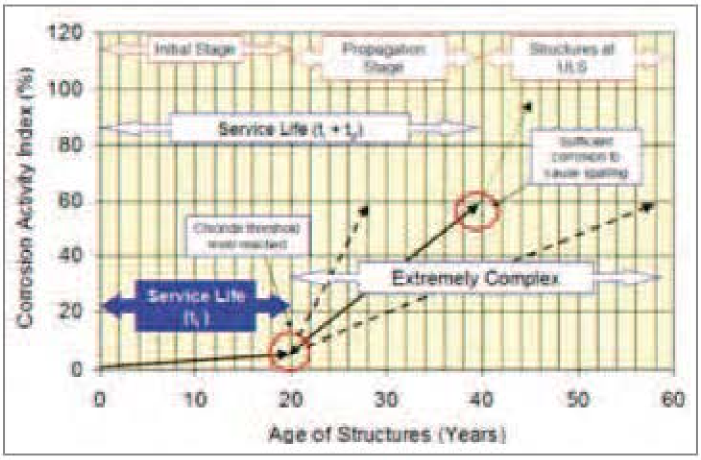


Figure 8: Corrosion model of RC structures

When modelling the initial phase, corrosion is triggered either by carbonation or when the critical corrosion-inducing chloride content is exceeded. The initial phase ends after steel depassivation and corrosion are initiated. Today, many well-tried models are available for the initial phase.

Once steel depassivation has occurred, reinforcement corrosion is dependent on the material quality and the environmental conditions, which must be taken into account in design and into consideration for structural safety.

The consequences of reinforcement corrosion in concrete include the loss of reinforcement cross-section, the development of tensile stress in concrete due to expansion caused by corrosion by-products and a change in the mechanical properties at the boundary between reinforcement and concrete.

The effects of corrosion can be divided into those concerning the reinforcement, the surrounding concrete and the bond between the concrete and the steel.

Equation 2 is the equation that expresses the process of chloride ingress from outside with the minimum number of required input parameters, where C_s is the chloride ion concentration at the exposed surface of the structure, C_x is the chloride concentration at depth X after time t, D_c is the chloride ion diffusion constant of the concrete material and *erf* is the error function (standard mathematical function).Abundant data for C_s and D_c based on this model have been obtained from many kinds of tests and surveys to estimate service life of existing structures. It would otherwise be difficult to verify the validity of the model due to the time dependent nature of the data for the various parameters such as temperature, humidity, carbonation, absorption into hydrated compounds and so on. Thus, Equation 2 is appropriate for the purposes of the comparative study in this paper.

$$C_x = C_s\left[1 - erf\left(\frac{X}{2\sqrt{D_c t_i}}\right)\right] \qquad (2)$$

The results in Table 3 show that with a concrete cover of 50mm, and without intervention or any active corrosion prevention system, corrosion of the reinforcing steel in the Grade50 concrete beams will initiate after just 10.5 years. In contrast, depassivation in the UHPdC beam will not start for 154 years. So, without regular maintenance, or passive or active corrosion protection systems, many conventional concrete structures in marine environments fail at an early age.

Table 3: Durability calculation in marine environment (for air-borne salt)

Exposure	Air-borne salt	
Concrete Type	G50	UHPdC
Cement (kg/m³)	480	720
f_{cs} (MPa)	50	150
X (mm)	50	22
C_s (kg/m³)	6.403	6.403
C_x (kg/m³)	1.68	2.52
D_c (mm²/s)	3.0×10^{-6}	6.87×10^{-8}
Time (years), t_i	10.5	153.5

In comparison UHPdC structures have the potential for significant savings in maintenance costs and a longer service life, leading to sustainable solutions.

This is particularly true if the structural element is pre-compressed to avoid cracking under service conditions.

CONCLUSION

The Bridge25 project provided a unique set of challenges which afforded the Engineer an opportunity to explore the use of UHPdC technology in the design of the multiple span composite bridge for the marine environment.

In the process of design, the Engineer gained valuable exposure to the properties of the material, which imparted unique mechanical behaviour to the beams made from it. These included the high strength obtained from a 1m deep section, the generous reserve capacity after cracking and the ductility that was seen in the test results.

The various and multiple tests gave confidence to the Engineer with regard to the ability of the UHPdC beams to fulfil its role. Such confidence can only be the result of the meticulous selection of materials and careful control of the manufacturing processes.

Bridge25 has provided a live platform to compare and contrast the performance of UHPdC against conventional concrete in terms of strength, durability, material consumption, embodied energy, CO_2 content, embodied energy and global warming potential.

The experience with UHPdC has certainly left the Engineer in a much better position to tackle the questions of durability and sustainability. ■

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