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DESIGN AND CONSTRUCTION OF A 50M SINGLE SPAN ULTRA-HIGH PERFORMANCE DUCTILE CONCRETE COMPOSITE ROAD BRIDGE

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ABSTRACT

A single span 50m long prestressed road bridge was constructed under Public Works Department in the State of Negeri Sembilan, Malaysia contract recently. The bridge was constructed at a small village, Kampung Linsum, crossing a river, Sungai Linggi. To date, this bridge is the Malaysia first and may also be the world longest composite road bridge which made from ultra-high performance ductile concrete (UHPdC). This paper presents the feature of the UHPdC precast girder; brief in-sight of the manufacturing of the girder; the construction sequence of the bridge; the design method and lastly the environmental impact calculation. The midspan deflections of the bridge at different construction history were compared against the collected field data and it showed that the calculated values generally agree well with the field data.

Keywords: *Ultra high performance, Bridge, Prestressed, Ductile.*

1.0 PROJECT BACKGROUND

The Public Works Department is the first to use ultra-high performance ductile concrete (UHPdC) in a bridge girder. The road bridge was completed in January 2011 (see Figure 1). The bridge was constructed using a single U-trough girder 1.75m deep, 2.5m wide at the top, topped with a 4m wide cast in-situ reinforced concrete deck 200mm thick. The UHPdC girder ends was encased in normal strength concrete abutments at the bridge site and made integral with the abutment seating. The girder was built without any conventional shear reinforcement as the UHPdC had considerable flexure and shear capacity. The UHPdC, with the trade name "DURA®" was supplied by Dura Technology Sdn. Bhd. It has achieved up to 180 MPa compressive strength and 30 MPa of flexural strength.

Figure 1: 50m single span UHPdC road bridge crossing Sungai Linggi, Negeri Sembilan.

2.0 DESIGN METHOD

2.1 BRIDGE LAYOUT

Figure 2 presents the general layout of the bridge. The transverse width of the bridge is 4m. The bridge is simply supported over a supporting length of 49.5m.

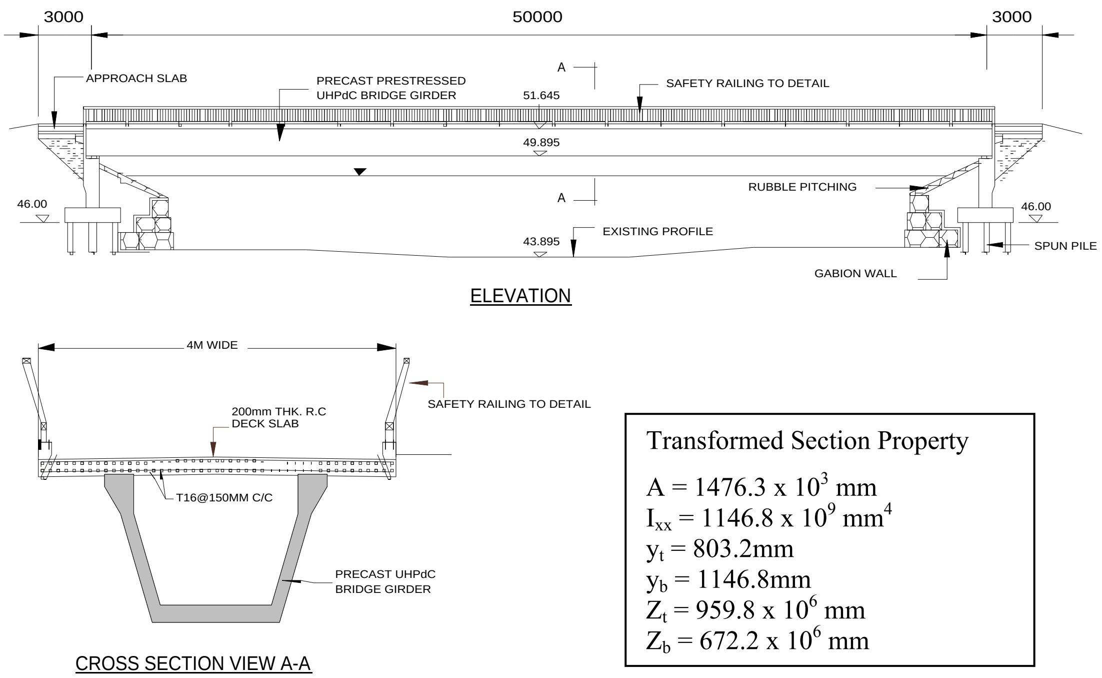


Figure 2: Layout of bridge.

2.2 SPECIFICATIONS OF BRIDGE

The specifications of the concrete bridge are as follows:

- Design life of structure: 120 years
- Number of nominal carriageway: 1
- Design traffic load: HA loading or 30 units HB loading (BS5400.2[7])
- Superstructure: Precast girder composited with 200mm thick in-situ Grade40 R.C. deck
- Bridge length: Single span of 50m
- Supported length = 49.5m
- Overall bridge width: B = 4m

2.3 LIMIT STATE DESIGN

2.3.1 GENERAL

- The bridge assumes to have relative humidity of 90% and average temperature of 30°C (this information was used for time-effect analysis)
- The strands used are 7-wire stress-relieved type and has a diameter of 15.2mm, which come with a guarantee breaking load of 260kN per strand and modulus of elasticity of 195GPa. All the tendons were stressed to 75% of its break load. The immediate losses during stressing are taken as 5%. The relaxation of tendons at different time *t* can be

calculated according to clause 3.3.4 of AS3600[8]. The tendons stress limit at SLS shall not exceed 70% of its characteristic tensile strength (i.e. 1302MPa).

- The type of reinforcement used has yield strength and breaking strength of 410MPa and 460MPa, respectively. The modulus of elasticity is taken as 200GPa. The reinforcement stress limit at SLS shall not exceed 80% of its yield strength (i.e. 328MPa).
- Grade40 concrete assumed to have characteristic compressive of f_{ck} = 40MPa and tensile strengths of $f_{tk} = 0.36\sqrt{f_{ck}}$ = 2.3MPa. The shrinkage and creep models are as per the requirement of AS3600[7]. The basic creep coefficient is taken as $\phi_{cc,b} = 2.8$.
- The allowable deflection limit due to live load (i.e HA loading in this case) is taken as L/600 according to AS5400.2[9] bridge code. Therefore, the maximum deflection shall not be greater than 82.5mm.
- HA live load is used for stress limit and deflection check at SLS.
- HB live load is used as the most adverse strength criteria at ULS.

2.3.2 UHPDC MATERIAL PROPERTIES

The characteristic compressive strength of the UHPdC is taken as f_{ck} = 150MPa. Additionally, it is possible to take into account the tensile strength of the concrete as UHPdC is superior in its fracture property. The characteristic tensile strength can be taken as f_{tk} = 10.69MPa. The modulus of elasticity is taken as E_0 = 46.5GPa.

Conventional shrinkage and creep models based on standards/codes are not available for UHPdC as it is a relatively new material. Therefore, the shrinkage and creep models used for UHPdC are based on experimental data. Knowing that the post-production shrinkage and creep are minimal, in the calculation follows, the total shrinkage of 1000με (with early autogenously shrinkage as high as 500με to 600με) is assumed to be all undertaken after the steam curing. Therefore the post-production shrinkage is considered to be negligible (i.e. $\varepsilon_{sh}(t) = 0$).

The creep model used herein is the regression fit from the experimental work conducted in University of New South Wales on Dura®-UHPdC which has similar curing method as described in Section 3.3 (see Figure 4). Equation 4 as suggested by Voo and Foster[10] is used to model the creep coefficient $\phi_{cc}(t)$ of UHPdC at any time t, where $\phi_{cc,b}$ is the basic creep coefficient of the UHPdC (which is the mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of 0.4 f_{cm}) and may be taken as and $\phi_{cc,b}$ =0.20.

2.3.3 DESIGN ACTIONS

The design loadings, bending moment and shear force values are presented in Table 1.

Table 1: Design bending moments and shear forces.

	SLS		ULS	
	Moment (kNm)	Shear Force (kN)	Moment (kNm)	Shear Force (kN)
SW of U-girder (G_1= 22.9kN/m)	7014	600	8066	690
SW of Deck (G_2 = 20.17 kN/m)	6178	504	7105	580
SW of Railing (G_3 = 0.5 kN/m)	153	12.5	176	14.4
Live Load 1 (HA)	10824	735	-	-
Live Load 2 (30 units HB)	-	-	16263	1350
Total	**24169**	**1852**	**M_{Ed} = 31610**	**V_{Ed} = 2634**

Notes:
The partial factor for UHPdC girder, RC slab and railing taken as γ_{fL} = 1.0 for SLS and γ_{fL} = 1.5 for ULS.
The partial factor for HA live load is taken as γ_{fL} = 1.2 for SLS and γ_{fL} = 1.5 for ULS.
The partial factor for 30 units HB live load is taken as γ_{fL} = 1.1 for SLS and γ_{fL} = 1.3 for ULS.

2.3.4 SECTION PROPERTIES

The effective flange width calculated as the full width for both SLS and ULS analysis. The cross section detail of the U-girder is presented in Figure 3. The U-girder consists of two slender vertical webs, each designed as a thin membrane element of 150 mm thick. The transformed sectional properties of the girder/composite bridge for the time-effect analysis are used herein corresponding to different load history of the bridge.

2.3.5 SERVICEABILITY LIMIT STATE (SLS)

The authors use the well established *Age Adjusted Effective Modulus Method* (AAEMM) by Gilbert and Mickleborough[11] to model the time-effect behaviour of the UHPdC composite bridge for a period of 30 years. The authors believe this method gives the most accurate prediction of the overall behaviour of the composite bridge at different load history during construction and during in service. Results on stresses, strains and deflections at the midspan are presented in Table 2.

In general, the stress levels for the concretes, tendons and reinforcements were confirmed to be below the specified stress limits. Calculation shows under the sustained permanent loadings for a period of 30 years, the prestressing strands will undergo maximum time-effect losses of 11.5% and 18% for the bottom tendons and top tendons, respectively. Also it has been observed that the resultant stresses of the steel reinforcements at the deck increases with time, from -20MPa to -100MPa, which indicates the inevitable creep and shrinkage behaviour of the normal strength concrete transfers significant amount of stress to the steel reinforcement.

Of particular interest, the AAEMM predicted deflection values are compared against the collected field data. Comparison show the AAEMM method generally is able to capture the overall deflection behaviour of the composite bridge at different load history during construction. AAEMM predicts the composite bridge will have a final sag deflection of 56mm after the 2nd stage PT, and the bridge shall bounce back to another 25mm after 30 years.

Table 2: Stress/stains and deflection at the midspan of the bridge (at SLS).

	Event	**1st Stg. PT**		**Add RC Deck**		**2nd Stg. PT**		**HA Loading**
	Days	28	57	57	71	71	10950	Infinity
	Analysis Type	I	T	I	T	I	T	I
	Composited?	No	No	No	Yes	Yes	Yes	Yes
Stress (MPa)	Slab Top	-	-	-	0.1	0.7	-2.2	-9.5
	Slab Bottom	-	-	-	0.2	-0.4	-2.6	-8.4
	Girder Top	-15.6	-15.8	-34.7	-34.7	-35.8	-24.1	-32.6
	Girder Bottom	-10.5	-9.6	4.2	4.2	-13.5	-13.7	2.4
	Top Reo.	-	-	-	-15	-13	-76	-121
	Bottom Reo.	-	-	-	-14	-16	-79	-119
	Top Strand	1231	1181	1110	1097	1088	1011	981
	Bottom Strand	1250	1208	1258	1262	1192	1106	1168
Strain (□□)	Slab Top	-	-	-	-78	-59	-369	-612
	Slab Bottom	-	-	-	-	-	-	-
	Girder Top	-335	-401	-807	-	-	-	-
	Girder Bottom	-226	-245	53	86	-294	-658	-311
Curvature (10^{-6} x mm^{-1})		0.0620	0.0890	0.4916	0.5484	0.3443	0.3168	0.6187
Theoretical Midspan Deflection (mm)		**-4.4**	**-0.1**	**103**	**118**	**56**	**31**	**106**
Field Measured Midspan Deflection (mm)		**-10**	**0**	**103**	**130**	**70**	**N/A**	**N/A**

- I = Instantaneous analysis T = Time-effect analysis
- Grade40 Concrete Stress Limits = -040 f_{ck} = -16 MPa (in compression) and 2.5 MPa (in tension)
- UHPdC Stress Limits = -0.60 f_{ck} = -90 MPa (in compression) and 5 MPa (in tension)
- Reinforcing Steel Stress Limit = 0.8 f_{yk} = 328 MPa
- Prestress Strand Stress Limit = 0.70 f_{pk} = 1302 MPa

The instantaneous deflection at midspan can be calculated as the superposition of the UDL part and the KEL point load of the HA loadings. Therefore the instantaneous deflection at the midspan due to HA loading is calculated to be 75mm, which is less than the allowable deflection limit of 82.5mm. Therefore the section has sufficient stiffness to pass the deflection criteria.

2.3.6 ULTIMATE LIMIT STATE (ULS)

The calculation of the design moment resistance (M_{Rd}) of UHPdC composite bridge is no difference from conventional concrete bridge where simple beam theory can be used. In this case, the critical design moment is assume to locate at a distance ± 4 m from girder midspan as that section is a joint (i.e. joint 3 of Figure 3). The joint is assumed to have no residual tensile stress during ultimate stage. The width of the R.C. deck is taken as the full width of 4m. By equating the compressive and tensile forces through the cross-section, the neutral axis depth (d_n) is found to be in the precast U-girder with $d_n = 233.7mm$. From internal forces equilibrium and taking moment about the top extreme fiber, the ultimate moment capacity can be calculated as M_u = 47279kNm. Using a member reduction factor of ϕ = 0.8, the design moment resistance can be taken as $M_{Rd} = \phi M_u = 0.8 \times 47279 = 37823 \text{ kNm} > \text{M}_{\text{Ed}} = 31610kNm$, which is greater than the design moment effect. Therefore the section has sufficient strength in flexure.

Since no stirrup is provided at any part of the UHPdC girder, the design shear resistance ($V_{Rd} = V_{yd}$) shall be set to either the design shear capacity of the web region as specified in the of Guidelines for UFC[1] or the shear transfer capacity of a dry keyed-joint specified in the experimental finding given in Voo[12], whichever is smaller.

From clause 6.3.3 of Guidelines for UFC[1], the design shear resistance can be calculated using Equation 1.

$$V_{yd} = V_{rpcd} + V_{fd} + V_{ped} \qquad \text{(Eq.1)}$$

where V_{rpcd} is the design shear capacity of a linear member that has no shear reinforcement bar, except the capacity provided by fiber reinforcement and is determined by:

$$V_{rpcd} = 0.18\sqrt{f'_{cd}}\; b_w\, d / \gamma_b = 0.18\sqrt{115.4} \times 300 \times 1728 / 1.3 = 771kN \qquad \text{(Eq. 1.1)}$$

where $b_w = the\ width\ of\ the\ web = 2x150 = 300mm$;

$d = effective\ depth = [\,(1950-100)x57 + (1950-350)x54]/(57+54) = 1728mm$;

$f'_{cd} = design\ compressive\ strength = f_{ck} / \gamma_c = 150/1.3 = 115.4MPa$

$\gamma_c = material\ reduction\ factor = 1.3$ and

$\gamma_b = member\ reduction\ factor = 1.3$.

The term V_{fd} is the design shear capacity provided by the fiber reinforcement, which is determined by the following equation:

$$\begin{aligned} V_{fd} &= \left(f_{vd} / \tan\beta_u\right) b_w\, z / \gamma_b = [8.22 / \tan(30^\circ)] \times 300 \times 1503 / 1.3 \\ &= 4938\ kN \end{aligned} \qquad \text{(Eq. 1.2)}$$

where $f_{vd} = f_{tk}/\gamma_c = 0.9 f_{spk}/\gamma_c = 0.9 \times (16 - 1.65 \times 2.5)/1.3 = 8.22MPa$ is the design average tensile strength perpendicular to diagonal cracks of UHPdC, f_{tk} is the characteristic tensile strength of UHPdC in uniaxial tension, f_{spk} is the characteristic split cylinder strength of the UHPdC (refer to Table 4). According to AS3600[8] the tensile strength can be approximated as $f_{tk} = 0.9 f_{spk}$.

The term β_u is the angle between the member axis and a diagonal crack and it shall not less than 30 degree.

$$\beta_u = \frac{1}{2}\tan^{-1}\left(\frac{2\,\tau}{\sigma'_{xu} - \sigma'_{yu}}\right) - \beta_o \geq 30^\circ \qquad \text{(Eq.1.2.1)}$$

where σ'_{xu} and σ'_{yu} are the applied average compressive stress along and perpendicular to the member. In this case σ'_{xu} is the average effective prestressing stress of the U-girder after time-effect losses. From Table 2, the average longitudinal stress can be taken as σ'_{xu} = (-24.1-13.7)/2 = -18.9 MPa and the perpendicular stress is taken as σ'_{yu} = 0.

The symbol $\beta_o = 5^\circ$ is the angle formed by a diagonal crack and a line at 45□ from the member axis, where it is not subjected to axial force.

The term τ is the average shear stress calculated from design shear force therefore it can be determined as:

$$\tau = (V_{Ed} / b_w\, d) = (2634 \times 10^3)/(300 \times 1728) = 5.08\ MPa. \qquad \text{(Eq. 1.2.2)}$$

Therefore, $\beta_u = \frac{1}{2}\tan^{-1}\left(\frac{2\,\tau}{\sigma'_{xu} - \sigma'_{yu}}\right) - \beta_o = \frac{1}{2}\tan^{-1}\left(\frac{2 \times 5.08}{18.9 - 0}\right) - 5 = 9.13^\circ$. Since the value of β_u shall not less than 30 degree. So in this calculation, $\beta_u = 30^\circ$. The term z is the distance from the location of compressive stress resultant to the centroid of tension steel, which may generally be set to $z = d/1.15 = 1728/1.15 = 1503\ mm$.

The term V_{ped} is the vertical force from the tendon component, which is taken as $V_{ped} = 0$. Finally the design shear resistance is calculated as:

$$V_{Rd} = V_{yd} = V_{rpcd} + V_{fd} + V_{ped} = 771 + 4938 + 0 = 5709 kN > V_{Ed} = 2634\ kN \quad .$$

Therefore the section has adequate shear resistance for the design shear force.

The design shear capacity corresponding to diagonal compression failure V_{wcd} may be calculated by:

$$\begin{aligned} V_{wcd} &= 0.84 f'^{\,2/3}_{cd} \sin\left(2\beta_u\right) b_w\, d/\gamma_b \\ &= 0.84\,(115.4)^{2/3} \sin\left(2 \times 30^\circ\right) \times 300 \times 1728/1.3 \qquad \text{(Eq. 2)} \\ &= 6876\ kN \gg V_{Ed} = 2634\ kN \end{aligned}$$

Therefore web shear crushing is not critical.

In the following, the shear strength of the dry keyed-joint will be calculated using the experimental and analytical finding from Voo[12]. The principle of Mohr Circle was used to predict the shear strength of the dry joint by using the minor principal strength σ_{11} as the failure criteria (noted in this case $\sigma_{11} = f_{vd}$ of Eq. 1.2). The design shear resistance at the joint ($V_{j,Rd}$ B) is taken as the superposition of the frictional force from the smooth-matched surface and the shear force contribution from the shear keys. Thus the design shear resistance of the joint can be written as:

$$V_{j,Rd} = V_{smd} + V_{kd} \qquad \text{(Eq. 3)}$$

where V_{smd} is the frictional force results from the compressive normal stress (i.e. σ_n = -18.9 MPa) due to prestressing, thus it can be expressed as:

$$V_{smd} = \mu\, A_{sm}\, \sigma_n / \gamma_b = 0.366 \times 405{,}000 \times 18.9/10^3 / 1.3 = 2155\ kN \qquad \text{(Eq. 3.1)}$$

where $A_{sm} = [(1750 \times 300) - (6 \times 100 \times 200)] = 405{,}000 mm^2$ is the area of the smooth section of the joint; γ_b is the member reduction factor which is set to $\gamma_b = 1.3$ for ULS according to Guideline to UFC[1]; μ is the friction coefficient which has calibrated against experimental specimens and it can be expressed as:

$$\mu = -0.0105\sigma_n + 0.5646 = -0.0105\,(18.9) + 0.5646 = 0.366 \qquad \text{(Eq. 3.1.1)}$$

The term V_{kd} is taken as the area of the key base times its maximum sliding shear strength (τ_{xy}), thus it can be written as:

$$V_{kd} = A_k\,\tau_{xy} / \gamma_b = 120{,}000 \times 14.93 / 10^3 / 1.3 = 1378kN \quad \text{for ULS} \qquad \text{(Eq. 3.2)}$$

where $A_k = (200 \times 100) \times 6nos. = 120{,}000mm^2$ is the total area of the shear key base and, the sliding shear strength is expressed as:

$$\tau_{xy} = \sqrt{\left(\sigma_{11} + \frac{\sigma_n}{2}\right)^2 - \left(\frac{\sigma_n}{2}\right)^2} = \sqrt{\left(8.22 + \frac{18.9}{2}\right)^2 - \left(\frac{18.9}{2}\right)^2} = 14.93MPa \qquad \text{(Eq. 3.2.1)}$$

where, $\sigma_{11} = f_{tk} / \gamma_c = 0.9 f_{spk} / \gamma_c = 0.9 \times (16 - 1.65 \times 2.5) / 1.3 = 8.22MPa$ is the design tensile strength of the UHPdC at ULS which can be taken as the split cylinder tensile strength and $\gamma_c = 1.3$ is the material reduction factor.

Finally, $V_{j,Rd} = 2155 + 1378 = 3533kN > V_{Ed} = 2634\ kN$. The shear resistance at the joint level is smaller than the shear resistance of the monolithic section (i.e. $V_{j,Rd} < V_{Rd}$), therefore the shear resistance at the keyed-joints governed.

3.0 BRIDGE CONSTRUCTION SEQUENCE

This section illustrates the construction sequence of the bridge. In brief the construction sequence can be summarized as:

Step	Activity	Date	Days
1	Fabrication of the UHPdC U-girder segments	Mid Oct 2010	-
2	Transportation of segments to job site	15th Nov 2010	20
3	Assembling of segments	16th Nov 2010	21
4	First stage post-tensioning	23rd Nov 2010	28
5	Launching of U-girder to abutments	3rd Dec 2010	38
6	Casting of in-situ RC deck	20th, 22nd Dec 2010	55, 57
7	Second stage post-tensioning	5th Jan 2011	71
8	Casting of the composite bridge to the abutment	13 Jan 2011	79

3.1 UHPDC U-GIRDER DETAIL

The precast girder consists of a total of seven segments, which consists of five standard internal segments (IS) each 8m long that weighed 18 tons, and two end standard segments (ES) each 5m long that weighed 15 tons (see Figure *3*). Unlike conventional precast concrete girders, the UHPdC girder does not have vertical shear link in its thin webs. The only conventional reinforcements used are the bursting reinforcement at the anchorage zone, lifting reinforcement at the tendon deflector positions, and horizontal shear reinforcement at the top flanges where connection with the RC deck is required.

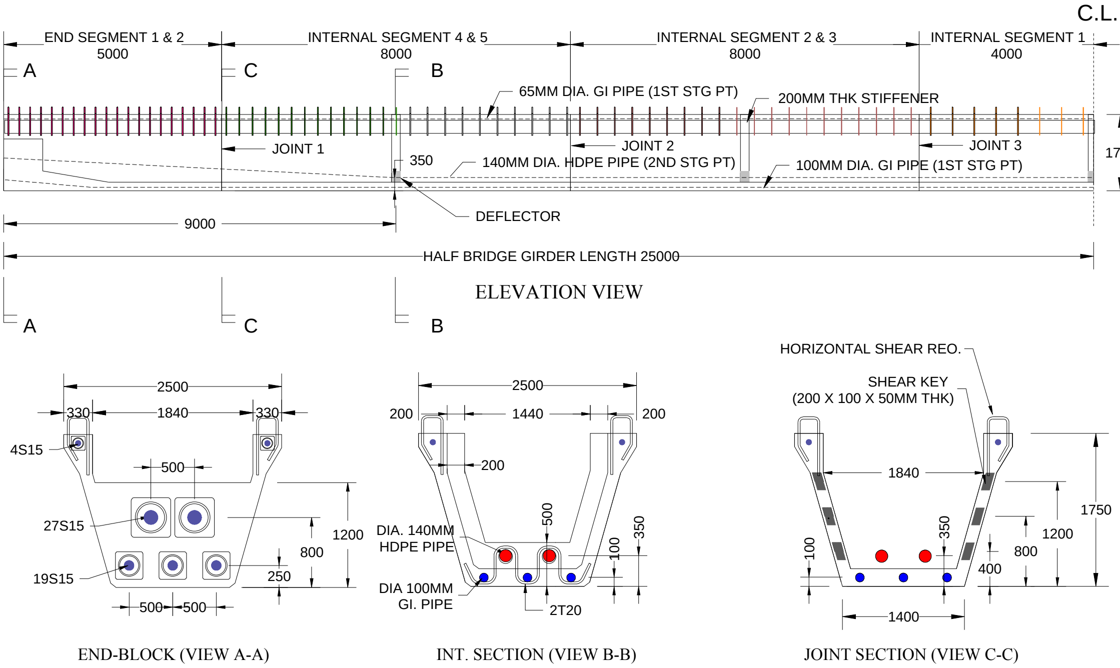


Figure 3: Detail of UHPdC girder.

3.2 MIX DESIGN OF UHPDC

The components of UHPdC are ordinary Portland cement, micro-silica, fine sand (with granular size less than 1mm), water, steel fibers and a high-range water reducing agent. In order to achieve the required performance of UHPdC, powder materials and fine aggregates are blended or proportioned to an adequate particle size distribution in order to maximize the density or compactness. Table 3 presents the general mix design used in the U-girder. Two types of steel fibers were used, that is (i) straight steel fiber with diameter and fiber length of 0.2mm and 20mm respectively; and (ii) end-hooked steel fiber with diameter and fiber length of 0.3mm and 25mm respectively. All this steel fiber has tensile strength beyond 2300MPa. The high-range water reducing agent used is a poly-carboxylate-ether based superplasticizer and no recycled wash water was used in the mixing.

Table 3: Mix design of standard DURA®-UHPdC (quantity in kg/m^3).

Ingredient	**Mass (kg/m^3)**
UHPdC Premix (Cement, micro-silica and fine sand)	2100
Superplasticizer	36
High strength steel fibers (>2300 MPa)	157 (2% by vol.)
Free water	144
3% moisture	30
Targeted W/B ratio	0.15
Total air voids	< 4%

3.3 MECHANICAL PROPERTIES OF UHPDC

UHPdC is a new generation of ultra-high performance construction material suitable for use in the production of precast elements for civil engineering, structural and architectural applications. UHPdC is a highly homogenous cementitious based composite without coarse aggregates which can achieve compressive strengths of 160 MPa and beyond. Its unique blend of very high strength micro-steel fibers and cementitious binders with extremely low water content give UHPdC the extraordinary characteristics of unique mechanical strengths, ductility comparable to steel and durability comparable to natural rock. UHPdC is highly impermeable, that is, the coefficient of water permeability and the diffusion coefficient of chloride ion are about $1/10^6$ which is 1/300 of ordinary high strength concrete. The Guidelines for UFC[1], suggested that this composite material can be used for more than 100 years without special repairs or reinforcement.

Table 4 summarized the mechanical properties of the UHPdC used in the U-girder. Each segment was cast from different batch of concrete, therefore it is important to take control samples of all the segments. Table 4 shows that the UHPdC can achieve cube compressive strength (f_{cu}) between 80MPa to 100MPa after 1 day; and 170MPa to 190MPa after 28 days. The cube compressive strength was measured according to BS 6319-2[2] using at least six cube specimens with dimension of 100mm.

Three pieces of 100mm diameter by 200mm height cylinders were tested for modulus of elasticity (E_0) and the experimental result shows the UHPdC has an average E_0 value of 46.5GPa. The E_0 values were determined according to BS 1881:121[3] under force control rate of 20MPa/min. The longitudinal strains were capture using electrical strain gauges.

Four pieces of 100mm diameter by 200mm height cylinders were tested for split cylinder indirect tensile strength test according to BS:EN 12390-6[4]. The first cracking strength (f_{ct}) is defined as the stress level in the UHPdC associated with a point in the stress-strain (or displacement) curve where the assumption of linear elasticity is no longer applicable. The split cylinder tensile strength (f_{sp}) is the point where maximum stress developed in the UHPdC after the first crack has formed.

Flexural toughness test as per ASTM-C1018[5] was used to determine the flexural properties of the UHPdC. Un-notched specimen with 100mm square cross-section and span over 300mm were used. A pre-load of 10kN was applied to the specimens and then unload to zero. This process was used for five cycles and then the specimens were loaded with midspan displacement control rate of 0.25mm/min till the end of the test. The load will decrease gradually after the peak load had achieved (i.e. f_{cf}). Experimental result shows all the UHPdC control specimens exhibit displacement-hardening behaviour after the first cracking (f_{cr}) occurred at an approximate midspan displacement of 0.03mm-0.05mm. This displacement hardening behavior is the results of the steel fibers bridging the micro-cracks and limiting the cracks from propagation. Experimental result shows the high volume of steel fibers in the concrete mix helps to increase the fracture mechanics of the composite, thus improved overall flexural toughness indexes (i.e. I_5, I_{10} and I_{20}).

The creep behavior of UHPdC was conducted at the University of New South Wales, Australia, over a period of 365 days, as per the specification of AS1012.16[6]. Four pieces of 100mm diameter by 200mm high cylinders and were pre-loaded with a compressive stress of 64MPa ($0.4f_{cm}$). The tests were conducted in an environmentally controlled room at an ambient temperature of 25°C and relative humidity of 50%. Two unloaded specimens were also measured to determine the shrinkage component of the measured strains. The experimental creep strain data are converted into creep coefficients, defined as the ratio of the creep strain to the instantaneous elastic strain, and the results are presented in Figure 4. From the results, the following equation was obtained for the creep coefficient:

$$\phi_{cc}(t)=\frac{15t^{0.4}}{60+t^{0.4}}\ \phi_{cc,28} \qquad \text{(Eq.4)}$$

where $\phi_{cc}\ (t)$ is the creep coefficient at time t, and $\phi_{cc,28}$ is the creep coefficient at 28 days. For these tests, $\phi_{cc,28} = 0.2$.

3.4 FABRICATION

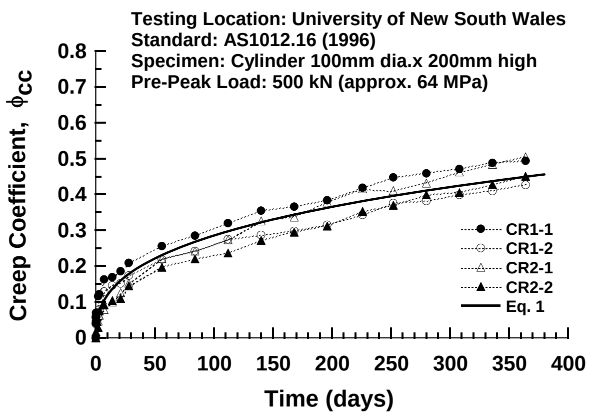
Manufacturing of the U-girder began in mid Oct 2010. Four segments are formed cast whereas three segments were matched cast against the formed cast segments (refer to Figure 5).

All the segments were steam-cured for a period of 48 hours at 90 degree Celsius as recommended by Guidelines for UFC[1]. Manufacturing of the last segment (i.e. IS1) was completed at early November. The total weight of the full girder was recorded to be 120 ton.

Table 4: Mechanical properties of UHPdC.

Ref.	Segment	End	Centre					End
		ES1	**IS4**	**IS2**	**IS1**	**IS3**	**IS5**	**ES2**
	Cast Type	FC	MC	FC	MC	FC	MC	FC
[2]	$f_{cu,\ 1\ day}$ (MPa)	85	82	80	89	99	82	100
	$f_{cu,\ 28\ day}$ (MPa)	189	180	171	183	188	183	193
[3]	$E_{o,\ 28\ day}$ (GPa)	-	-	-	-	-	-	46.5
[4]	$f_{ct,\ 28\ day}$ (MPa)	-	-	-	-	-	6.6	-
	$f_{sp,\ 28\ day}$ (MPa)	-	-	-	-	-	16.0	-
[5]	$f_{cr,\ 28\ day}$ (MPa)	14.5	15	14.0	14.5	15.5	15.0	15.5
	$f_{cf,\ 28\ day}$ (MPa)	31	30	32	30	32	29	33
	I_5	5.95	5.86	5.52	6.00	5.81	5.94	6.22
	I_{10}	13.4	13.9	12.5	13.4	13.7	13.7	14.4
	I_{20}	29.9	31.4	27.7	30.1	31.1	31.3	32.6

Notes: FC = formed cast; MC = matched cast; ES = end segment; IS = internal segment



$$\phi_{cc}(t)=\frac{15t^{0.4}}{60+t^{0.4}}\ \phi_{cc,b} \quad \text{……[Eq.4]}$$

Figure 4: Proposed creep Eq. 4 compare with experimental creep results on UHPdC.

3.5 ASSEMBLING OF U-GIRDER

A total of six 12m trucks were used to transport the seven segments to the job site. The segments were loaded onto the trucks on 15th November 2010 and on 16th November 2010 morning the segments arrived to the site and ready for assembling. Figure 6 shows a fully assembled U-girder at the job site. Due to the lightness of the segments, only one 45 ton mobile crane was used to align the seven segments.

3.6 FIRST STAGE POST-TENSIONING (PT)

First stage post-tensioning (PT) was carried out by Freyssinet PSC Malaysia on 23rd November 2010. Figure 7 shows the technician fitting the anchorage blocks for the three ducts@19S15 tendons (bottom row) and the two ducts@4S15 (top row). The central ducts (i.e. two ducts@27S15) were–for second stage PT. The bottom tendons were then stressed by a 7000kN capacity hydraulic jack. Both ends of the girder were stressed. At the end of the PT work, the midspan instantaneous hog deflection was measured to be approximately 10mm.

3.7 GIRDER LAUNCHING

Two units 160 tons mobile cranes were used to lift the UHPdC U-girder. In less than an hour, the U-girder was parked on one end of the steel framed transfer girder. Figure 8a shows the end of the U-girder was securely fastened on a trolley then gradually towed over the river. The girder was then securely positioned at the abutment of the bridge. The whole launching process took approximately 5 hours. At the end of the day, all the participants and witnesses were satisfied.

3.8 IN-SITU DECKING

Prior-to concreting, the level of the U-girder was measured and result shows the midspan deflection is close to 0mm (that is almost level). Figure 9a shows on 20th Dec 2010 (assumed day 1), the contractor concreted the first half portion of the deck. After the concrete was laid, the U-girder level was measured which showed a deflection of 25mm. After two days, the partially completed bridge had undergone further sag of another 25mm at the midspan due to the shrinkage effect from the RC deck. At this stage the net deflection is approximately 50mm. On 22nd Dec 2010 (day 3), the remaining half of the deck was concreted and the instantaneous midspan deflection recorded was 43mm, which give a total deflection of 93mm. On 24th Dec 2010 (day 5), further midspan sag deflection of 10mm was recorded which gave a total deflection of 103mm. Prior-to the second stage post-tensioning, on 4th January 2011 (day 16) the total midspan sag deflection of 130mm was recorded (refer to Figure 9b). These recorded deflection values are later compared against theoretical prediction as given in Table 2.

3.9 SECOND STAGE POST-TENSIONING

Second stage post-tensioning (PT) was carried out on 5th January 2011. The tendons were stressed by a 10000kN capacity hydraulic jack. Each duct was prestressed to a jacking force of 5265kN which gives a total prestressing force of 10530kN (refer to Figure 10a). At the end of the PT work, the internal ducts and deflectors were examined for defects. Examination shows the deflectors to be crack free (refer to Figure 10b). At this stage, the midspan instantaneous upward deflection was measured to be approximately 60mm (which result a net midspan deflection of 70mm).

3.10 LOAD TEST

Before the bridge was opened for the public use, the appointed consulting engineer from Public Works Department requested a load proof test on the bridge. Figure 11a shows an excavator weighing 22 ton was placed at the midspan of the bridge. The load test criterion is that under such static load, the bridge shall not deflect more than 16mm at the midspan and after the load is removed, the bridge shall have minimum 90% of recovery. In this test, a midspan displacement of 7mm was measured and after the excavator (i.e load) was removed the bridge has gained 100% recovery. Thus the authority has accepted the bridge deemed to comply. After a week, the bridge was open for public (refer to Figure 11b).

Figure 5: (a) Matched casting of internal segment (b) close up view.

(a) (b)

Figure 6: One unit 45 tonnes mobile crane used to assembled the U-segment and (b) full U-girder assembled on site.

Figure 7: Stressing technician preparing for first stage post-tensioning.

(a) (b)

Figure 8: (a) Launching of U-girder using steel A-frame transfer girder, (b) U-girder securely seated on abutments.

(a) (b)

Figure 9: (a) Concreting of first half of the deck, (b) RC deck completed.

(a) (b)

Figure 10: (a) Stressing technician preparing for second stage post-tensioning and (b) no sign of major cracking at the deviator.

(a) (b)

Figure 11: (a) One 22 tonnes excavator place at the bridge midspan and (b) completed bridge open for traffic.

4.0 ENVIRONMENTAL IMPACT CALCULATION (EIC)

Infinitely, the design engineers of this bridge had proposed to use two structural steel welded girders for this bridge (see Figure 12). When UHPdC girder was proposed as an alternative, the consultants were amenable as the benefits, such as negligible maintenance, eco-friendly option, aesthetically pleasing and lower cost.

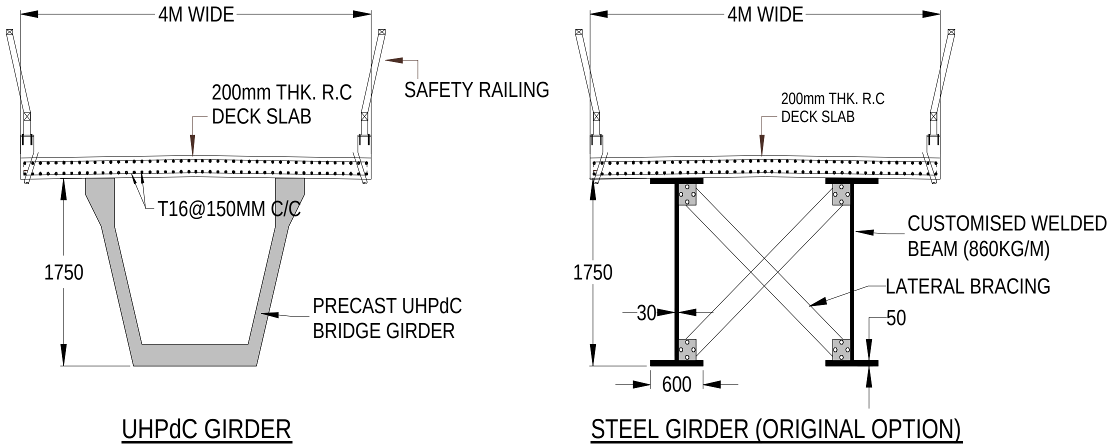


Figure 12: Comparison of UHPdC girder against steel girder composite bridge.

This section presents the environmental impact calculation (EIC) of the UHPdC composite bridge against the original steel beam composite bridge. Table 5 summaries the environmental data used in this comparative study, where detail on the derivation of the environmental impact data on the building material used can be obtained from Voo and Foster[10]. The table has been prepared to help to calculate the equivalent embodied energy (EE), CO_2 emission content and global warming potential (GWP) of particular concrete mix designs and materials.

In brief, Global Warming Potential (GWP) is a measure of how much a given mass of greenhouse gas is estimated to contribute to global warming over a given time interval. It is a relative scale which compares the gas in question to that of the same mass of CO_2. A 100-year of time horizon is most commonly used and it can be expressed as:

$$100\text{-year GWP} = CO_2 + 298\ NO_x + 25\ CH_4 \quad \text{(unit in ton of } CO_2 \text{ eq.)} \qquad \text{(Eq. 5)}$$

Table 6 summaries the material quantities and EIC of the two bridge designs. In the calculation of the material quantity, only the superstructure is considered herein. The amount of EE, CO_2 emissions and GWP are obtained from multiplying the amount of materials by the environmental data given in Table 5. Comparison of the EIC results is presented in Figure 13. In terms of material consumption, the UHPdC solution general consumed 14% more material (in term of weight) than the steel beam solution. In terms of environmental impact, the UHPdC solution is less environmental damaging with 66% less embodied energy and 57% less CO_2 emission. In terms of the 100-year GWP, the UHPdC solution gives a reduction of 52%.

Table 5: Environmental data.

	Units	Standard UHPdC (wt. 2% Steel Fiber)	Grade-40 (wt. 15% PFA)	Steel, Strand, Reo.
Density	kg/m^3	2400	2350	7840
EE	GJ/m^3	7.71	1.728	185.8
CO_2	kg/m^3	1065	297.5	17123
NO_x	kg/m^3	4.86	1.66	55.38
CH_4	kg/m^3	0.76	0.12	30.65
GWP	kg CO_2 eq. /m^3	2532	795	34392
EE	MJ/kg	3.231	0.744	23.70
CO_2	kg/kg	0.446	0.128	2.184
NO_x	g/kg	2.035	0.714	7.064
CH_4	g/kg	0.318	0.052	3.909
GWP	kg CO_2 eq. /kg	1.060	0.342	4.387

Table 6: Material quantities and environmental impact calculation (EIC).

		UHPdC (m^3)	Grade 40 Concrete (m^3)	Strands (ton)	Reo. (ton)	Steel (ton)	
No.	**UHPdC Composite Bridge**						
1	Precast U-girders	47.7	-	6.66	2.34	-	
2	RC deck	-	43.38	-	8.64	-	
	Sub-Total	47.7	43.38	6.66	10.98	-	**Total**
A	**Mass of material used (ton)**	114.48	101.9	6.66	10.98	-	**234.1**
B	**Embodied energy (GJ)**	368.0	74.97	157.8	260.23	-	**861.0**
C	**CO_2 (ton)**	50.80	12.91	14.55	24.0	-	**102.2**
D	**GWP (ton CO_2 eq.)**	120.8	34.49	29.22	48.17	-	**232.7**
No.	**Steel Welded Beam Composite Bridge**						
1	Steel welded beam	-	-	-	-	86	
2	Bracing (10% of beam)	-	-	-	-	8.6	
3	RC deck	-	43.38	-	8.64	-	
	Sub-Total	-	43.38	-	8.64	94.6	**Total**
A	**Mass of material used (ton)**	-	101.94	-	8.64	94.6	**205.2**
B	**Embodied energy (GJ)**	-	74.97	-	204.77	2242	**2521.8**
C	**CO_2 (ton)**	-	12.91	-	18.87	206.6	**238.4**
D	**GWP (ton CO_2 eq.)**	-	34.49	-	37.90	415	**487.4**

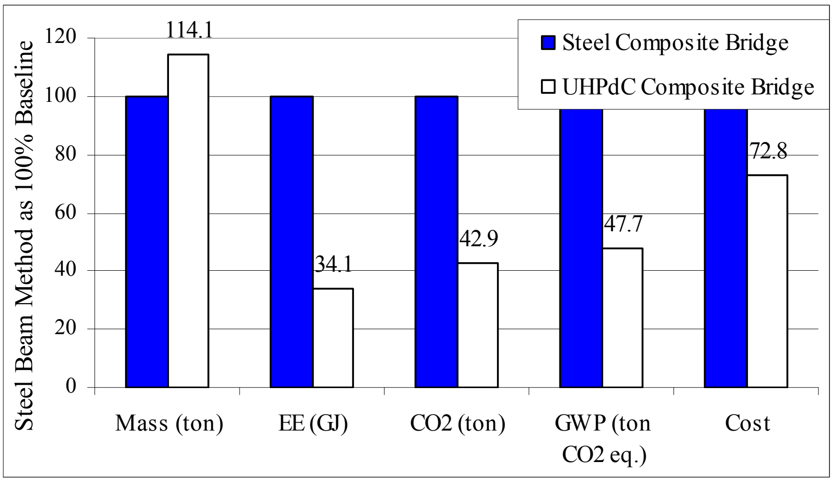


Figure 13: EIC comparison of UHPdC and steel composite bridges.

5.0 CONCLUSIONS

In January 2011, the Public Works Department had offered a tender for the construction of a single span 50m long bridge using an ultra-high performance ductile concrete composite design. It was optimized for a combination of structural, durability, sustainability and constructability aspects. To date, this bridge is Malaysia's first and perhaps the world longest road bridge with the main girders made from UHPdC. This paper presents the overview on the design and construction of the bridge. Age-adjusted elastic modulus method was used to predict the overall deflection value of the bridge corresponding to different load history during construction. Comparison shows the theoretical value generally is in well agreement with the field collected data. The bridge was then compared against conventional bridge design in term of environmental impact. In summary, the UHPdC design is confirmed to be a greener construction as the embodied energy content and CO_2 emission are approximately 66% and 57%, respectively less than of the conventional approach. In conclusion, UHPdC technology open the door for new design approach and it can make concrete structure more cost feasible, sustainable and environmental friendly.

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LIST OF SYMBOL

A_k	total area of the base of shear keys (mm^2)
A_{sm}	total area of smooth section of the joint (mm^2)
b_w	the width of the webs (mm)
d	effective depth (mm)
d_n	neutral axis (mm)
E_o	mean modulus of elasticity (GPa)
f_{cd}	design compressive strength (MPa)
f_{cf}	mean flexural strength / mean modulus of rupture (MPa)
f_{ck}	characteristic compressive strength (MPa)
f_{cm}	mean cylinder compressive strength (MPa)
f_{cr}	mean first bending cracking strength (MPa)
f_{ct}	mean first cracking strength (MPa)
f_{cu}	mean cube compressive strength (MPa)
f_{sp}	mean split cylinder tensile strength (MPa)
f_{spk}	characteristic split cylinder tensile strength (MPa)
f_{tk}	characteristic tensile strength (MPa)
f_{vd}	design average tensile strength perpendicular to diagonal crack (MPa)
I_5, I_{10}, I_{20}	mean flexural toughness indexes
L	nominal length of bridge (m)
M_{Ed}	design moment effect (kNm)
M_{Rd}	design moment resistance (kNm)
M_u	ultimate moment capacity (kNm)
V_{Ed}	design shear force effect (kN)
V_{fd}	design shear resistance provided by the fiber reinforcement (kN)
$V_{j,Rd}$	design shear resistance at joint region (kN)
V_{kd}	design shear resistance from the shear key of the joint (kN)
V_{ped}	design vertical force from the tendon component (kN)
V_{rpcd}	design shear force of linear member that has no shear reinforcement (kN)
V_{Rd}	design shear resistance (kN)
V_{smd}	design shear resistance from the frictional force of the joint results from the prestress normal stress (kN)
V_{wcd}	design shear capacity corresponding to diagonal compressive failure (kN)
V_{yd}	design shear resistance (kN)
t	time (days)
β_u	angle between the member axis and a diagonal crack (degree)
β_o	angle formed by a diagonal crack and a line 45 ° from the member axis, where it is not subjected axial force (degree)
ϕ	member reduction factor
ϕ_{cc} (t)	creep coefficient at time t
$\phi_{cc,b}$	basic creep coefficient
$\phi_{cc,28}$	basic creep coefficient at 28 days
γ_b	member reduction factor
γ_c	material reduction factor
μ	friction coeffcieint
σ_{11}	minor principal strength (MPa)
σ_n	applied normal stress results from tendon component (MPa)
σ'_{xu}	applied average compressive stress along the member (MPa)
σ'_{yu}	applied average compressive stress perpendicular the member (MPa)
τ	average shear stress (MPa)
τ_{xy}	maximum sliding shear strength (MPa)