Precast Segmental Box Girder with Dry Joints and External Tendons

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INTRODUCTION

In some early and a few more recent segmental bridges erected through span-by-span erection in Malaysia, especially those associated with metro rail bridges, dry joints between precast segments were primarily utilised to reduce cost and construction time as well as to eliminate potential problems with epoxy applications.

The structures were generally designed with only external post-tensioned tendons protected by High Density Polyethylene ducts. No reinforcing or prestressing steel extended across the joints.

The code of practice to be used in Malaysia for concrete bridge design is BS5400 Part 4. This code does not cover the use of external tendons or dry joints. Reference to the Highways Agency Design Manual for Roads and Bridges (DMRB) documents BD58 and BA58 can be used to supplement BS 5400 Part 4 and to provide design guidance and requirements on the use of external unbonded prestressing. However, no specific guidance on the design of dry joints is given.

It is generally accepted that dry joints give a lower ultimate moment and ultimate shear capacity than glued match-cast joints with precast segmental construction. It is therefore necessary to take this into account by introducing adjustments to the design approach and requirements. This design note compares the commonly available methods used in the design of precast segmental decks and recommends the design approach to be employed where dry joints and external tendons are used.

Design references used in the comparison include:

- BS5400: Part 4 – 1990 Code of Practice for the Design of Concrete Bridges,
- BD58/94 The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing,
- BS EN 1992-2: Eurocode 2 Design of Concrete Structures,
- AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges 2nd Edition 1999,
- Prestressed Concrete Bridges: Design & Construction by Nigel Hewson,
- Dry Joint Behaviour of Hollow Box Girder Segmental Bridges – fib Symposium,Segmental Construction in Concrete' New Delhi, 26-29.11.2004.

1. ULTIMATE MOMENT LIMIT STATE CAPACITY

Decks with dry joints behave differently in bending with ultimate loads to those using glued joints. The epoxy glue used between the segments creates a bond of greater strength than the concrete between the segments. No such bond is present with dry joints meaning that when ultimate limit state loading is applied the joints decompress and open up. This will lead to a reduction in structural stiffness and the occurrence of larger deflections with the rotations concentrated at joints. The ultimate limit state failure mechanism with dry joints and external tendons is due to concrete crushing on the compression side due to excessive strains.

As shown on Table 1, of all the design codes investigated, the only design code to recognise the different ultimate bending failure mechanism of dry jointed decks as compared to glued joints is the AASHTO Guide Specification for Design of Segmental Bridges. A lower strength reduction factor, ϕ, is applied to the ultimate bending resistance for dry joints as compared to glued joints. For glued joints $\phi = 0.90$ and dry joints $\phi = 0.85$. However, this guide specification is now superseded and AASHTO has prohibited the use of dry joints since 2003.

Table 1: Precast Segmental Decks with Dry Joints – The Ultimate Bending Moment Capacity

Design Code/Reference	Notes
BS5400: Part 4 – 1990 (Note: Not applicable to dry jointed decks)	No specific guidance given for the design of decks using external tendons with dry joints.
BD58/94: The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing	No specific guidance given for the design of decks with dry joints.
BS EN 1992-1-1:2004	No specific guidance given for the design of decks with dry joints.
AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges 2nd Edition 1999	Lower strength reduction factor, ϕ, used for dry joints as compared to glued joints. $\phi = 0.90$ *Glued Joints* $\phi = 0.85$ *Dry Joints*

It has been successfully shown that there is a good correlation in behaviour of dry jointed segmental bridge decks determined using finite element methods and test data. This is described in the paper, "Precast segmental box girder bridges with external prestressing – design and construction by Prof. Dr Ing. G. Rombach".

It is proposed that a non-linear finite element model to be created to determine the ultimate limit state response of a typical standard span. The mid-span deflection can be plotted against increasing applied live load bending moment. The bending moment being increased incrementally until the model shows the deck has failed due to concrete crushing in the extreme compression fibre. This is the approach described in BS EN 1992-2 to verify the ultimate limit state capacity. The Figure 1 from the finite element software MIDAS FEA illustrates the dry joint behaviour at the ultimate limit state, with the joints opening up over mid-span. This analytical approach can be used to determine the ULS moment capacity of the span and the increase in stress in the tendons at failure.

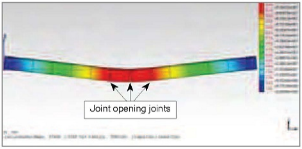

Figure 1: Dry Joint Behaviour at Ultimate Limit State

2. ULTIMATE SHEAR LIMIT STATE CAPACITY FOR SECTIONS BETWEEN JOINTS

The design rules for shear of sections between joints described in BS5400: Part 4 are based on test results for bonded tendons. Consequently their use for external unbonded tendons is inappropriate. Designing prestressed concrete bridges with external tendons in the UK requires the BD58/94 standards to be used.

The DMRB document BD58/94 does give a method for designing sections with external unbonded tendons. This is to treat the section as a reinforced concrete column section with an externally applied load. However, in general this approach is generally considered conservative and it also does not make any allowance for the opening that may occur at dry joints. The opening at the joint reduces the depth through which the web shear compression strut can pass.

The AASHTO Guide Specification for Design of Segmental Bridges also uses a strut and tie model to determine shear capacity. However, it makes no reference to the design of decks with dry joints or limitations on the size of the compression strut due to the opening of the joints. The strength reduction factors, ϕ, for both glued and dry joints is ϕ = 0.85.

It is proposed that BS EN 1992-2 to be used for the shear design of sections between joints. Specific reference is made to the design of segmental structures with precast elements and unbonded prestressing. The code makes allowance for the decompression of joints under ultimate limit state loading and the subsequent reduction in depth through which the compression strut can pass.

3. SHEAR CAPACITY OF DRY JOINT BETWEEN SEGMENTS USING SHEAR KEYS

Typically, the shear design of dry joints relies on the friction capacity of the concrete interfaces between shear keys and the shear resistance of the shear key. A comparison between the various codes of practice and methods available for the design of joints in segmental bridges has been completed. Specific reference has been made to the design of dry joints. Details of the comparison are provided in Table 2 and an example of typical shear capacities are given in the attached calculations shown in Table 3.

Table 2: Precast Segmental Decks with Dry Joints – The Dry Joint Ultimate Shear Capacity

Design Code/Reference	Equation	Notes
BS5400: Part 4 – 1990 (Note: Not applicable to dry jointed decks)	ULS Shear Capacity, $V = 0.7 \times (\tan \alpha_2) \times \gamma_{fl} \times P_h$	α_2 = Joint friction angle γ_{fl} = Prestressing force partial safety factor = 0.87 P_h = Horizontal component of force after losses (kN)
BD58/94 The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing	No guidance given for the design of decks with dry joints	
BS EN 1992-1-1:2004	$v_{Edi} \leq v_{Rdi}$ Design shear stress, $v_{Edi} = \beta \times V_{Ed} / (z \times b_i)$ Design shear resistance, $v_{Rdi} = (c \times f_{ctd}) + (\mu \times \sigma_n) \leq 0.5 \times v \times f_{cd}$	V_{Ed} = Shear force (kN) z = Lever arm of composite section (mm) β = longitudinal force ratio b_i = width of interface (mm) c = Friction coefficient = 0.35 σ_n = Compressive stress in concrete after all losses (N/mm^2) f_{ctd} = Design tensile strength (N/mm^2) μ = Friction Coefficient = 0.60
AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges, 2nd Edition, 1999	Shear Strength, $V_{uj} = \phi_j \times V_{nj}$ Nominal Shear Capacity, $V_{nj} = A_k \times \sqrt{f'_c} \times (12 + 0.017 f_{pc})$ $+ (0.6 \times A_{sm} \times f_{pc})$	ϕ_j = Strength reduction factor for the design of dry joints = 0.75 A_k = Area of the base of all shear keys in failure plane (in^2) A_{sm} = Area of contact between smooth surfaces on failure plane (in^2) f_{pc} = Compressive stress in concrete after all losses (psi) $\sqrt{f'_c}$ = Characteristic concrete compressive stress (psi)
Prestressed Concrete Bridges: Design & Construction by Nigel Hewson	ULS Shear Capacity, $V = \dfrac{[(1.4 f_c \times A_{sk}) + (0.6 f_c \times (A_w - A_{sk}))]}{F.O.S}$	f_c = coexistent compressive stress on the web (N/mm^2) A_{sk} = Area of Shear Key (mm^2) A_w = Area of Web (mm^2)
Dry Joint Behavior of Hollow Box GirderSegmental Bridges – fib Symposium, Segmental Construction in Concrete, New Delhi, 26-29.11.2004	Shear Strength, $V_{uj} = \phi_j \times V_{nj}$ Nominal Shear Capacity, $V_{nj} = (\mu \times \sigma_n \times A_{joint}) + (0.14 \times A_k \times f_{ck})$	ϕ_j = Strength reduction factor for the design of dry joints = 0.5 A_{joint} = Area under compression (mm^2) A_k = Area of the base of all shear keys in failure plane (mm^2) σ_n = Compressive stress in concrete after all losses (N/mm^2) f_{ck} = Characteristic concrete compressive stress (N/mm^2) μ = Friction Coefficient = 0.65

Table 3: Sample Calculations of Dry Joint Shear Capacities using Various Codes

	Section: Deck Segment **Calcs for: Joint Shear Capacity Comparison – Typical Example**	**Date: 8/5/2012** **By: twk/ttw**
	Prestressed Concrete Bridges: Design & Construction ULS Shear Capacity $V = ((1.4f_c \times A_{sk}) + (0.6f_c \times (A_w - A_{sk}))) / \text{F.O.S}$ where f_c = 6 N/mm^2 A_{sk} = 480000 mm^2 A_w = 539750 mm^2 F.O.S = 2.0	V = 2124 kN
AASHTO 12.2.21	**AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges 2nd Edition 1999 (Clause 12.2.21)** Shear Strength $V_{uj} = \phi_j \times V_{nj}$ $V_{nj} = A_k \times \sqrt{f'_c} \times (12 + 0.017f_{pc}) + (0.6 \times A_{sm} \times f_{pc})$ where ϕ_j = 0.75 A_k = 744 *in*^{2} f'_c = 5800 *psi* f_{pc} = 870 *psi* A_{sm} = 93 *in*^{2} Note: The use of Type B (Dry) Joints was prohibited by AASHTO 2003	V_{uj} = 5226 kN
BS5400:4 6.3.4.6	**BS5400 Part 4 1990 – Code of Practice for Design of Concrete Bridges (Clause 6.3.4.6)** ULS Shear Capacity $V = 0.7 \times (\tan \alpha_2) \times \gamma_{fl} \times P_h$ where α_2 = 0.942 Rads γ_{fl} = 0.87 P_h = 15000 kN	**NOT APPLICABLE TO DRY JOINTS** V = 12561 kN
	Dry Joint Behaviour of Hollow Box Girder Segmental Bridges – fib Symposium Segmental Construction in Concrete, New Delhi 26-29/11/2004 Shear Strength $V_{uj} = \phi_j \times V_{nj}$ $V_{nj} = (\mu \times \sigma_n \times A_{joint}) + (0.14 \times A_k \times f_{ck})$ where ϕ_j = 0.5 A_{joint} = 539750 mm^2 f_{ck} = 40 N/mm^2 σ_n = 6 N/mm^2 A_k = 480000 mm^2 μ = 0.65	V_{uj} = 2397 kN
1992-2 6.2.5	**BS EN 1992-2 – Eurocode 2 Design of Concrete Structures** Design Shear Stress $\nu_{Edi} = \beta \times V_{Ed} / (z \times b_i)$ Design Shear Resistance $\nu_{Rdi} = (c \times f_{ctd}) + (\mu \times \sigma_n) \leq 0.5 \times \nu \times f_{cd}$ where V_{Ed} = 1200 kN z = 1250 mm β = 1.0 b_i = 425 mm c = 0.35 σ_n = 6 N/mm^2 f_{ctd} = 1.67 N/mm^2 μ = 0.6	ν_{Edi} = 2.26 N/mm^2 ν_{Rdi} = 4.18 N/mm^2 V = 2223 kN

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