Numerical Modelling & Design of Unreinforced Construction Joint for Reinforced Concrete Substructures with Concrete Shrinkage & Creep

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***Abstract*—Nowadays rapid urban growth always requires intensive development of underground infrastructure networks. New reinforced concrete substructures may need to be built adjacent to existing ones sharing a common diaphragm wall with unreinforced construction joints at the roof and the base slabs. These unreinforced construction joints need to provide adequate support for the adjacent existing substructure during construction stage and for long term when the time dependent creep shrinkage movement of the substructure becomes significant. This paper presents design details and a soil structure interaction finite element computer model for the unreinforced construction joints. These are applied to the impact assessment for the Perth City Busport substructure construction adjacent to the existing Perth-Fremantle Rail Tunnel. The effects on the existing rail tunnel structure due to the short term and long term shrinkage and creep movements of the Busport are found to increase the lateral displacement, bending moment and shear by up to 30%. However, the shrinkage and creep effect steadies at about 500 days after casting.**

***Index Terms*—unreinforced construction joint, shrinkage, creep, substructure, numerical modelling**

I. INTRODUCTION

Reinforced concrete structures require joints where two successive placements of concrete meet. They may be designed to permit movement and/or to transfer load. There are three types of joints, namely expansion joints, contraction joints and construction joints [1]. Unreinforced construction joints would need to be provided between any two adjoining structures of different ownerships to accommodate the construction sequence together with the short term and long term shrinkage and creep effects without adversely affecting the structures.

A. Unreinforced Construction Joints for Substructures

Unreinforced construction joints can only carry compression but no tension. This presents problems for substructure base slabs which are often under tension after completion of the substructure and recharge of ground water to the original level. Tension would further increase upon continual concrete shrinkage and creep [2]. Adequate steel reinforcements are required to prevent excessive cracks in the base slab [3]. Waterproofing strategy for the substructure will need to maintain water ingress within limits prescribed by the design criteria. To block water ingress, waterproofing membrane bonded to the adjacent substructure, large movement capacity water-bar, continuous hydrophilic seal, polyurethane injection (with re-injectable grout tubes for retroactive sealing of gaps between the adjoining structures) should be provided in the joint [4].

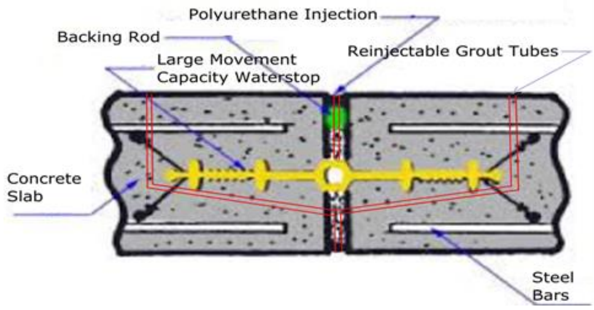

Figure 1. Typical details for unreinforced construction joint.

B. Concrete Shrinkage and Creep Strains

The total measured strain $\epsilon(t)$ at any time t is the sum of elastic strain ϵ_e, shrinkage strain ϵ_{cs} and creep strain ϵ_{cc} as follows [5]:

$$\epsilon(t) = \epsilon_e + \epsilon_{cs} + \epsilon_{cc}. \tag{1}$$

1) Shrinkage srain

Based on [6] the design shrinkage strain ϵ_{cs} at time t [in days] after set of concrete is the sum of the autogenous shrinkage ϵ_{cse} and the drying shrinkage ϵ_{csd} as follows:

$$\epsilon_{cs} = \epsilon_{cse} + \epsilon_{csd} \tag{2}$$

where

$$\epsilon_{cse} = \epsilon^*_{cse}(1 - e^{-0.1t}) \tag{3}$$

$$\epsilon^*_{cse} = (0.06f'_c - 1) \times 50 \times 10^{-6} \tag{4}$$

$$\epsilon_{csd} = k_1 k_4 \epsilon_{csd.b}, \tag{5}$$

f'_c = Concrete 28 day characteristic strength [MPa]

$$k_1 = \frac{\alpha_1 t^{0.8}}{t^{0.8} + 0.15 t_{h,}} \tag{6}$$

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t_h = thickness of concrete [mm],
$k_4 = 0.7, 0.65$ *or* 0.5 respectively for arid, interior or tropical, near coastal environments,

$$\alpha_1 = 0.8 + 1.2e^{-0.005t_h} \tag{7}$$

$$\epsilon_{csd.b} = (1 - 0.008f'_c) \times \epsilon^*_{csd.b} \tag{8}$$

$$\epsilon^*_{csd.b} = 1000 \times 10^{-6} \tag{9}$$

2) Creep srain

Reference [6] relates the design creep strain ϵ_{cc} at any time t [days] after concrete set at a sustained stress σ_o with the creep coefficient $\emptyset_{cc}$ and the mean 28-day concrete modulus E_c as follows:

$$\varepsilon_{cc} = \emptyset_{cc}\sigma_o/E_c \tag{10}$$

$$\emptyset_{cc} = k_2k_3k_4k_5\emptyset_{cc,b} \tag{11}$$

where $\emptyset_{cc.b}$ =2.8 for Grade 40; 2.4 for Grade 50 concrete,

$$k_2 = \frac{\alpha_2 t^{0.8}}{t^{0.8}+0.15t_h} \tag{12}$$

$$k_3 = 2.7/[1+\log(\tau)] \tag{13}$$

$$\tau = \text{time after loading} \geq 1 \text{ day} \tag{14}$$

k_4 is as defined for (5),
k_5 =1.0 for f'_c ≤50MPa or

$$k_5 = 2 - \alpha_3 - 0.02(1-\alpha_3) \text{ if } 50 < f'_c \leq 100\text{MPa} \tag{15}$$

$$\alpha_3 = 0.7/(k_4\alpha_2) \tag{16}$$

$$\alpha_2 = 1 + 1.12e^{-0.008t_h} \tag{17}$$

C. Concrete Modulus of Elasticity

Reference [6] relates the concrete modulus of elasticity E_c with its density ρ [kg/m^3] and f_{cmi}[MPa] as follows:

$$\rho^{1.5} \times 0.043\sqrt{f_{cmi}} \text{ [MPa] if } f_{cmi} \leq 40\text{MPa} \tag{18a}$$

$$\rho^{1.5}(0.024\sqrt{f_{cmi}} + 0.12) \text{ [MPa] if } f_{cmi} > 40\text{MPa} \tag{18b}$$

where f_{cmi} is given in Table 3.1.2, AS3600, Ref. [6]).

TABLE I. 28 DAY CONCRETE PROPERTIES (TABLE 3.1.2 REF. [6])

f_c' (MPa)	20	25	32	40	50	65	80	100
f_{cmi}'(MPa)	22	28	35	43	53	68	82	99
E_c (GPa)	24	26.7	30.1	32.8	34.8	37.4	39.6	42.2

From (10) the concrete modulus E_{cc} at a creep strain ϵ_{cc} is given by [7]

$$E_{cc} = \frac{E_c}{1+\emptyset_{cc}} \tag{19}$$

II. NUMERICAL MODELLING OF UNREINFORCED CONSTRUCTION JOINT, SHRINKAGE AND CREEP STRAINS

Numerical modelling has adopted the 2D finite element software Plaxis 2D, [8] as explained below.

A. Unreinforced Construction Joint

Unreinforced construction joints are modelled as Plaxis node-to-node anchor elements which can only carry compressive axial loads without any bending resistance. The numerical model keeps the joint at zero tension by prestressing the node-to-node anchor at 0kN whenever tension develops during the construction staging calculations. In actual case during the service life of the substructure any gaps or cracks developed in the unreinforced construction joints are to be mitigated by non-shrink grout to ensure water tightness.

B. Shrinkage Strain

In Plaxis numerical analysis model, reinforced concrete structures are often represented by Plaxis plate elements. Concrete shrinkage strains are applied to each of these plate elements by adopting a prestressing force N_{cs} to a fictitious node-to-node anchor added to the middle of the plate element.

$$N_{cs} = \frac{\epsilon_{cs}L_o}{L_f}(EA)_f \tag{20}$$

In (20) L_o is the original length of the plate element, L_f, $(EA)_f$ are the original length and the original stiffness of the fictitious node-to-node anchor respectively. The fictitious node-to-node anchor element will have negligible length, axial and bending stiffnesses compared to the plate element.

C. Creep Strain

Concrete creep strains are applied to each of the structural members of the substructure (represented by Plaxis plate elements) by adopting a prestressing force N_{cc} to a fictitious node-to-node anchor added to the middle of the plate element.

$$N_{cc} = \frac{\epsilon_{cc}L_o}{L_f}(EA)_f \tag{21}$$

The creep strain is obtained by (10) using the elastic strain of the substructure with the concrete modulus E_{cc} at creep calculated from (19). The combined effect due to concrete shrinkage and creep can now be applied to the structure by prestressing the respective fictitious node-to-node anchor element to a total prestress of $N_{sc} = N_{cs} + N_{cc}$.

III. NUMERICAL MODELLING OF PERTH CITY BUSPORT SUBSTRUCTURE CONSTRUCTION

TABLE II. SOIL SHEAR STRENGTH PARAMETERS

Soil	Unit weight [kN/m^3]	Undrained shear strength, S_u [kPa]	Cohesion c' [kPa]	Friction angle '
Fill	18	NA	0	33°
LSA	17	20	0	25°
SS	19	NA	0	35°
GFU	20.5	NA	0	34°
UGU	20	0.60 $_v$'> 40	0	31°
LGU	17.5	0.62 $_v$'> 80	6	29°
KPF	19	125	250	30°

The new Perth Busport is located adjacent to the south of the existing Perth-Fremantle Rail Tunnel T6, Perth, Australia. It is an underground structure 210m long from east to west and about 45m wide constructed in 0.60m thick diaphragm walls. The excavation is about 6.5m deep adopting top down construction. The site geology comprises about 2.5m thick granular fill over-lying successive layers of alluvium soil of LSA, SS, UGU, GFU and LGU underlain by KPF rock with the design groundwater table at 1.5m depth as shown in Fig. 2. The design geotechnical parameters are shown in Table II and Table III [9].

TABLE III. SOIL STIFFNESS & PERMEABILITY PARAMETERS

Soil	Drained Elastic Modulus, E [MPa]	Poisson's ratio,	Permeability	
			Horizontal k_h [m/s]	vertical k_v [m/s]
Fill	20	0.25	1.7 x10^{-4}	0.1k_h
LSA	4	0.45	1.2 x10^{-6}	0.1k_h
SS	50 (z≤5m); 100 (z>5m)	0.25	1.7 x10^{-4}	0.1k_h
GFU	100	0.30	2.8 x10^{-5}	0.1k_h
UGU	70	0.35	1.2 x10^{-6}	0.1k_h
LGU	80	0.30	1.2 x10^{-7}	0.1k_h
KPF	500 -1000	0.25	impermeable	

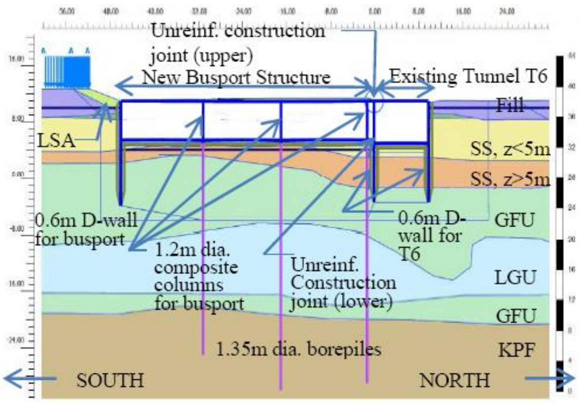

Figure 2. Geotechnical cross section of new Perth Busport.

Two unreinforced construction joints are formed between the existing tunnel T6 and the new bus port (Fig. 2). The excavation and construction will need to be carried out carefully to avoid detrimental impact on the existing tunnel T6, as the existing tunnel T6 has not been designed to carry out-of-balance horizontal loads.

TABLE IV. EXISTING PERTH RAIL TUNNEL T6 PARAMETERS

Structural elements	EA (kN/m)	EI (kNm^2/m)	w (kN/m)	Toe Level (mRL)
D-wall (70% EI) (0.6m thick)	14.8E6	437E3	4.2	-3.5
D-wall (50% EI) (0.6m thick)	10.4E6	312E3	4.2	-3.5
Roof slab (70% EI) (0.7m thick)	16.2E6	663E3	17.5	10.97
Roof slab (50% EI) (0.7m thick)	11.6E6	474E3	17.5	10.97
Base slab (70% EI) (0.75m thick)	17.4E6	816E3	5.3	5.0
Base slab (50% EI) (0.75m thick)	12.4E6	583E3	5.3	5.0

NB: Grade 50 concrete for D-walls, Grade 40 for slabs

The existing Perth Rail Tunnel T6 was recently completed in 2012 and is assumed to have achieved the long term concrete shrinkage and creep strains when the bus port construction commences. The short term and long term stiffnesses of the Rail Tunnel T6 slabs and D-walls respectively adopt 70% and 50% of the uncracked concrete modulus before shrinkage and creep as recommended by CIRIA 580 [10]. The structural parameters for T6 are shown in Table IV.

The new bus port structure parameters are presented in Table V. The lower unreinforced construction joint at the base slab is modelled by a Plaxis node-to-node anchor adopting non-shrink grout. As the bus port roof slab is always in compression, the upper unreinforced construction joint at the roof slab is modelled by a hinge joint allowing free rotation of the bus port roof slab with respect to the existing Tunnel T6 roof slab. Concrete shrinkage and creep strains are assumed to be less significant for the D-walls, composite columns and borepiles than the bus port roof slabs and base slabs which are up to 45m wide.

TABLE V. NEW PERTH BUSPORT PARAMETERS

Structural elements	EA (kN/m)	EI (kNm^2/m)	w (kN/m)	Toe Level (mRL)
Bus port Roof Slab (0.8m thick) (at 28 days)	18.6E6	989E3	20	10.97
Bus port Roof Slab (0.8m thick) (at 1.5 yrs)	16.7E6	889E3	20	10.97
Bus port Roof Slab (0.8m thick) (at 100 yrs)	15.9E6	850E3	20	10.97
Bus port Base Slab (0.75m thick) (at 28 days)	17.4E6	815E3	5.3	5.475
Bus port Base Slab (0.75m thick)) (at 1.5 yrs)	15.6E6	732E3	5.3	5.475
Bus port Base Slab (0.75m thick)) (at 100 yrs)	14.9E6	700E3	5.3	5.475
Bus port D-wall (70% EI) (0.6m thick)	14.8E6	437E3	4.2	-4
Bus port D-wall (50% EI) (0.6m thick)	10.4E6	312E3	4.2	-4
Bus port composite columns (1.2m dia) (70%EI)	4.78E6	395E3	4.22	NA
Bus port composite columns (1.2m dia) (50%EI)	3.66E6	293E3	4.22	NA
Bus port borepiles (1.35m dia) (70%EI)	5.1E6	579E3	5.3	-25~-30
Bus port borepiles (1.35m dia) (50%EI)	3.6E6	414E3	5.3	-25~-30
Unreinforced concrete joint (50mm gap non-shrink grout)	34.7E6	NA	NA	5.475

NB: Grade 50 concrete for D-walls, Grade 40 for slabs

The adopted concrete modulus is presented in Fig. 3. Staged construction modelling follows Table VI.

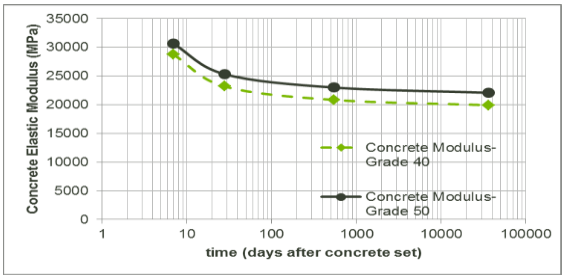

Figure 3. Concrete elastic modulus (with creep).

TABLE VI. CONSTRUCTION STAGING FOR NEW BUSPORT

Stage	Description
1	Install Perth Rail Tunnel T6 D-walls. Adopt 70% stiffness parameters.
2	Install Perth Rail Tunnel T6 roof slab. Adopt 70% stiffness parameters.
3	Excavate to Perth Rail Tunnel T6 base slab
4	Cast Perth Rail Tunnel T6 base slab. Adopt 70% stiffness parameters.
5	Recharge ground water table. Adopt 50% stiffness parameters.
6	Install Busport D-walls, plunge columns and borepiles. Adopt 70% stiffness parameters for Busport, 50% for T6.
7	Excavate to Busport roof slab soffit
8	Cast Busport roof slab. Adopt 28day stiffness parameters for Busport, 50% stiffness parameters for T6.
9	Busport roof slab shrinks at 28 day. Adopt 28day stiffness parameters with shrinkage & creep strains for Busport, 50% stiffness parameters for T6.
10	Dewater & excavate to Busport base slab soffit
11	Cast Busport base slab. Adopt 28day stiffness parameters for Busport base slab and roof slab, 50% stiffness parameters for T6.
12	Busport base slab shrinks at 28 days. Adopt 28day stiffness parameters with shrinkage & creep strains for Busport base slab and roof slab, 50% stiffness parameters for T6.
13	Construct composite columns. Adopt 70% stiffness parameters for columns.
14	Busport substructure completes at 1.5 yrs and ground water table recharges to normal. Adopt 1.5yr stiffness parameters for Busport base slab and roof slab, 50% stiffness parameters for others.
15	Busport roof slab and base slab shrink and creep at 1.5 yrs. Adopt 1.5yr stiffness parameters for Busport base slab and roof slab with shrinkage & creep strains, 50% stiffness parameters for others.
16	Busport roof slab and base slab undergo long term shrinkage and creep for 100 years adopting 100yr stiffness parameters.
17	Busport roof slab and base slab undergo long term shrinkage and creep for 100 years. Adopt 100yr stiffness parameters with shrinkage and creep strains

The fictitious node-to-node anchors for the application of concrete shrinkage and creep strains to the bus port substructure are 0.2m long with an axial stiffness $(EA)_f$ of 1000kN/m – about ten thousand times less stiff than the Busport structure. (Note that Plaxis node-to-node anchors do not have bending stiffness or weight.) The estimated concrete shrinkage and creep strains and the applied prestress forces N_{sc} are presented in Fig. 4, Fig. 5, Fig. 6 and Fig. 7 below.

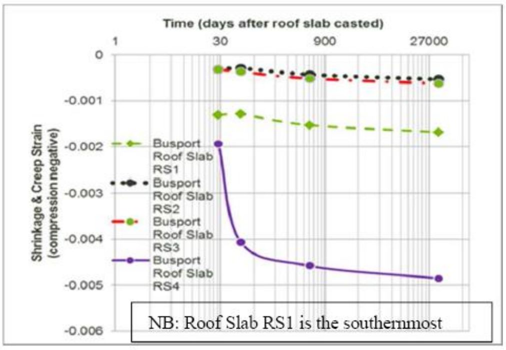

Figure 4. Concrete shrinkage & creep strains for Busport roof slab.

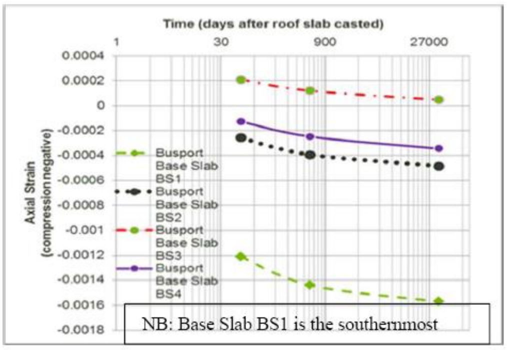

Figure 5. Concrete shrinkage & creep strains for Busport base slab.

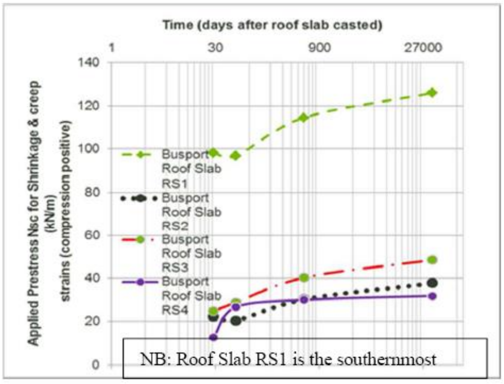

Figure 6. Applied prestress force for Busport roof slab concrete shrinkage & creep analysis

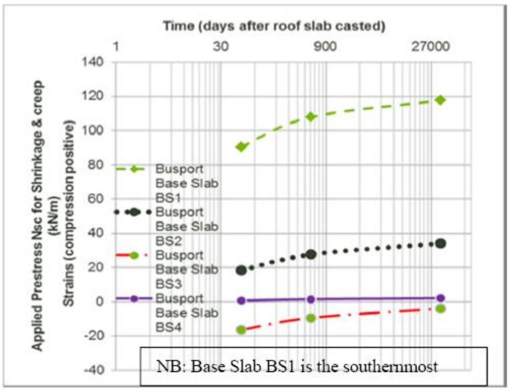

Figure 7. Applied prestress force for Busport base slab concrete shrinkage & creep analysis.

The unreinforced construction joints adjoining the existing southern D-wall of Tunnel T6 have been provided with waterproofing membrane bonded to the adjacent substructure, large movement capacity angle water-bar, continuous hydrophilic seal and polyurethane injection (with re-injectable grout tubes for retroactive sealing of gaps) as shown in Fig. 8 and Fig. 9.

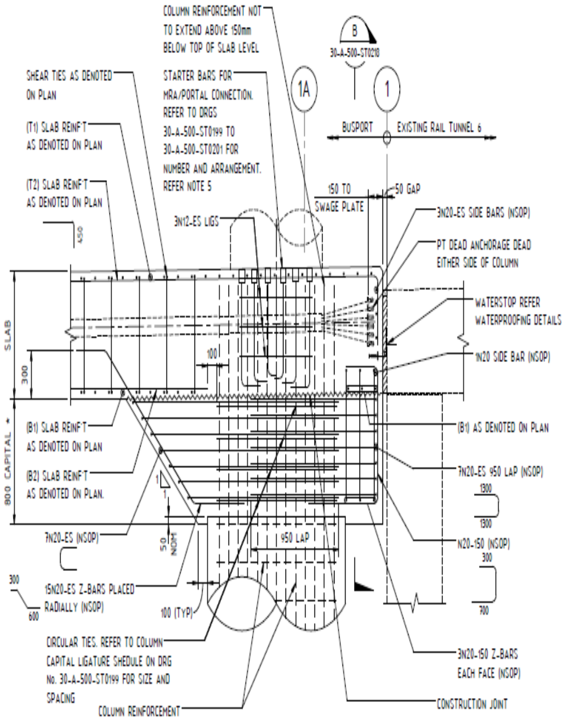

Figure 8. Unreinforced construction joint details at Busport roof slab.

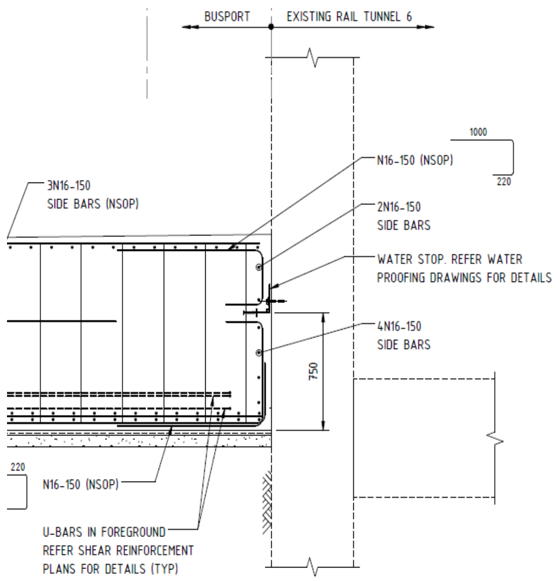

Figure 9. Unreinforced construction joint details at Busport base slab.

IV. Summary & Discussion of Analysis Results

A. *Numerical Analysis Graphical Outputs*

The numerical analysis results are presented in Fig. 10 to Fig. 14 for bending moment, shear force and lateral displacements for the Perth Rail Tunnel southern diaphragm wall and the Busport roof slab and base slab.

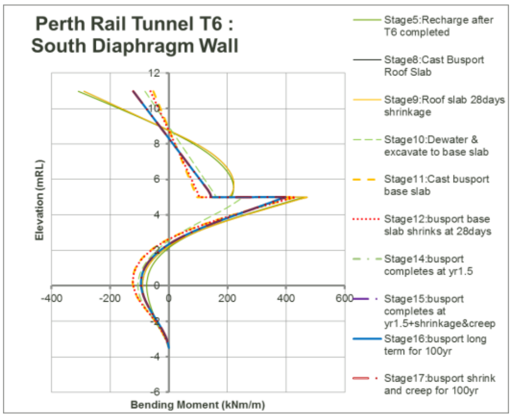

Figure 10. Bending moment diagram for T6 south D-wall.

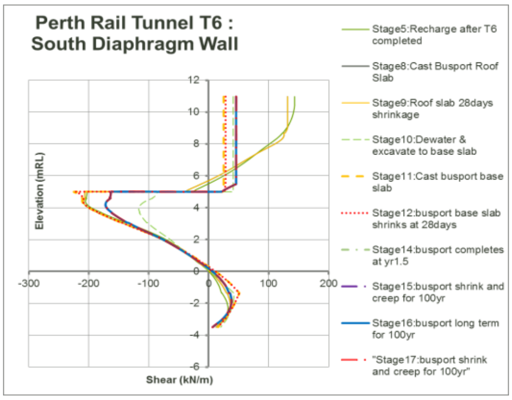

Figure 11. Shear force diagram for T6 south D-wall.

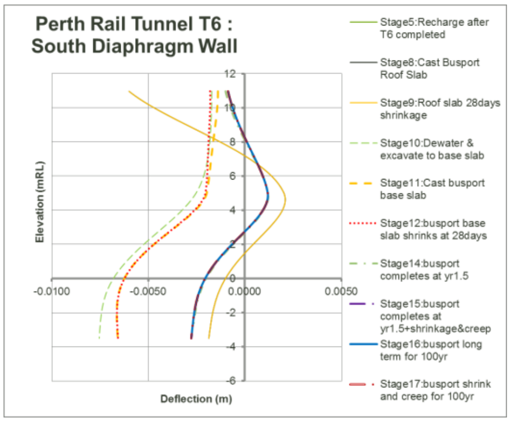

Figure 12. Lateral displacement for T6 south D-wall.

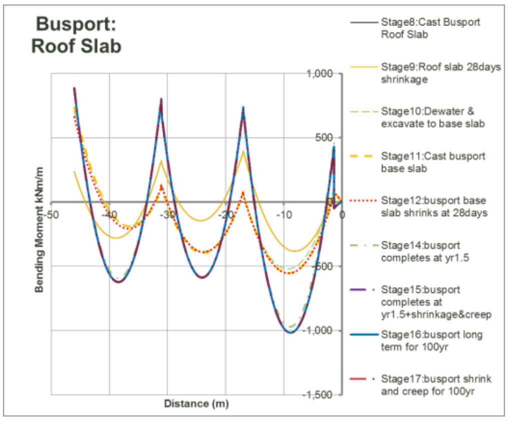

Figure 13. Bending moment diagram for Busport roof slab.

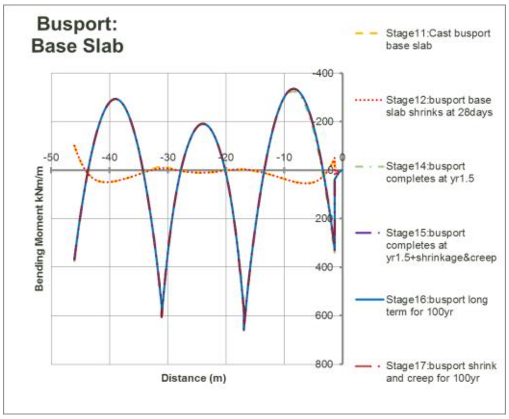

Figure 14. Bending moment diagram for Busport base slab.

B. Discussion of Analysis Results

Once the excavation of the Busport structure starts, lateral resistance on the southern side of the Tunnel T6 structure will be removed as a result of unloading and reduction in groundwater pressure. Tunnel T6 will horizontally sway towards the Busport side and the rail tunnel structures will deflect. Effect of concrete shrinkage and creep is most significant from 28 days to 1.5 years after the Busport roof slab is casted. There is an increase of the Tunnel T6 and the Busport roof slab maximum lateral deflections and bending moments up to about 30 % and 10% respectively when the Busport base slab shrinks at Stage 12. However, the shrinkage and creep effects fade significantly afterwards. The unreinforced construction joint at the Busport roof slab is always under compression while the one at the Busport base slab has been kept in zero tension in the numerical model by prestressing the Plaxis node-to-node anchor element.

V. Conclusions and Recommendations

Numerical modelling adopting Plaxis node-to-node anchor elements for unreinforced construction joints and concrete shrinkage and creep strains is presented. The modelling technique has been applied to the impact assessment for Perth City Busport. The effect of shrinkage and creep is found to increase the lateral displacement, bending moment and shear by up to 30%. However, the shrinkage and creep effect steadies after the concrete has been casted for about 500 days. Water tightness control for unreinforced construction joints is recommended by using waterproofing membrane bonded to the adjacent substructure, large movement capacity angle water-bar, continuous hydrophilic seal, polyurethane injection (with re-injectable grout tubes for retroactive sealing of gaps between the adjoining structures).

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References

[1] S. Graham. (Jan. 1, 2011). Concrete Joints and Their Applications. [Online]. Available: http://www.networx.com/ article/concrete - joint-types
[2] A. P. Lampropoulos, O. T. Tsioulou, and S. E. Dritsos, "Restrained concrete shrinkage in case of strengthened bridge piers by concrete jacketing," presented at the 4th ECCOMAS Thematic Conf. on Computational Methods in Structural Dynamics and Earthquake Engineering, Kos Island, Greece, 12-14 June 2013.
[3] P. Brinckerhoff. (Dec. 2008). Guidelines for Waterproofing of Underground Structures. [Online]. Available: http://ondemandweb.pbworld.net/pbucontent/aicc/waterproofing
[4] Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules and Rules for Buildings EN 1992-1-1.
[5] B. E. Usibe, I. P. Etim, and J. O. Ushie, "Prediction of creep deformation in concrete using some design code models," *IOSR Journal of Mechanical and Civil Engineering*, vol. 4, no. 3, pp. 49-53, Nov.-Dec. 2012.
[6] Concrete Structures, Australian Standard AS3600 - 2009 (with Amendment Nos. 1 and 2).
[7] R. Park and T. Parley, *Reinforced Concrete Structure*, 1st ed. U.S.A.: John Wiley & Sons, Inc, 1975, ch. 10.2.7, pp. 457-461.
[8] *Plaxis 2D 2012 Reference Manual*, Delft, the Netherlands, 2012, pp. 40.
[9] City Busport Alliance, "Geotechnical design report – underground structure," Perth, W.A., 2014, pp. 24.
[10] A. R. Gaba, B. Simpson, W. Powrie, and D. R. Beadman, *CIRIA C580, Embedded Retaining Walls – Guidance for Economic Design*, 1st ed. London, U.K.: CIRIA, 2003, ch. 4.2.3, pp. 124.

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