CRACKING SERVICEABILITY LIMIT STATE FOR PRECAST CONCRETE TUNNEL SEGMENTS

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Abstract: Ultimate limit state (ULS) and serviceability limit state (SLS) are specified in national and international design codes for design of structural members such as segmental tunnel linings. SLS is associated with the availability of a structure for users, such as the constraints on deflection or deformation and cracking of a member under loading. Cracking SLS can be very detrimental for durability for segmental tunnel linings, especially if the tunnel is exposed to an aggressive environment or excessive water inflow. This paper focuses on required cracking serviceability verification of concrete tunnel segments with particular attention to different types of reinforcement including reinforcement bars and fibers. Calculation of crack width under SLS loads, and maximum allowable crack width for tunnel linings are discussed with respect to various guidelines and standards. In order to illustrate the applicability of the proposed approaches, SLS crack verification for a case of mid-size tunnel is presented. Results show that fiber reinforcement results in the reduction of crack width in tunnel segments at SLS comparing to conventional reinforcement.

1 INTRODUCTION

Precast concrete segments are installed to support the excavation behind the Tunnel Boring Machine (TBM) in soft ground and weak rock applications. The TBM advances by thrusting off the completed rings of precast concrete segments that typically provide both the initial and final ground support as part of a one-pass lining system. These segments are designed to resist the permanent loads from the ground and groundwater as well as the temporary loads from production, transportation and construction (ACI 544.AR, 2015). In conformance with ACI 318-Building Code Requirements for Structural Concrete and Commentary (ACI 318, 2014), the design engineer may use the LRFD method to design precast concrete tunnel segments. LRFD is a design philosophy that takes into account the variability in the prediction of loads and the variability in the properties of structural elements. LRFD employs specified limit states beyond which a structure no longer satisfies the design performance to achieve its objectives of constructability, safety, and serviceability. The limit states for shield tunnel design are principally divided into the ultimate limit state and the serviceability limit state (JSCE, 2007). Ultimate limit state which is a state associated with the collapse or other forms of structural failure of tunnel linings have been discussed elsewhere (ACI 544.AR, 2015; Bakker and Blom, 2009; de la Fuente et al. 2011). Serviceability limit state (SLS) is a state beyond which specified service requirements for a tunnel are no longer met. The SLS in segmental tunnel lining systems is associated with excessive stresses, deflections and cracking of concrete segments as well as excessive stresses and deformations of segment joints. These serviceability limit states may cause reduction of tunnel inner space due to excessive deformations, and durability and watertightness issues due to steel bar corrosion and water leakage from segment cracks or enlarged gap between segment joints (JSCE, 2007; Mendez Lorenzo, 1998; Çimentepe, 2010). Required design checks for satisfying the serviceability limit states of tunnel segments include stress, deformation and cracking SLS verifications in segments. Stress verification is checked in concrete and reinforcement at the main section of segments and the segment joints, as well as stress verification in connecting bolts in the joints. Deformation verification includes ring deformation, joint opening, and joint offset. Cracking SLS verification is checked for shear cracking and flexural crack width. Because tunnels are usually excavated at our below the groundwater level or exposed to an aggressive environment, cracking SLS are often very detrimental for durability of segmental tunnel linings. The SLS of flexural cracking including methods of calculation and allowable values of crack width are discussed in this paper with particular attention to the type of reinforcement.

2 SLS DESIGN LOADS AND METHODS OF ANALYSIS

The flexural cracking SLS design for the tunnel segments is performed considering all different types and combination of loads acting on tunnel linings during production, transport, construction, and final service stages. The types, combination, of loads, recommended load factors, and calculation methods for flexural crack width and limiting values are discussed in this section.

Critical load cases for segment design are divided into 3 categories: production and transient loads, construction loads, and service loads. Production and transient load cases include segment demolding, storage, transportation and handling, while construction loads include TBM thrust jack forces, tail skin and localized back grouting pressure. Final service loads include earth pressure, groundwater and surcharge loads, longitudinal joint bursting load, and special loads such as earthquake, fire, explosion and loads induced due to additional distortion (ACI 544.AR, 2015). During production and transport stages of demolding and handling, segments are modeled as cantilever beams with the self-weight (DC) as the only force acting on segments and no load combination is defined. For the load cases of storage and transportation, the dead weight of segments positioned above (F) is also acting on the designed segment in addition to its self-weight (DC). Therefore, a load combination of DC + F is taken into account. In the load case of TBM thrust jack force, jack force (J) is the only force acting on segment joints, and no load combination is considered. The self-weight of the lining (DC) and the grouting pressure (G) are acting on the tunnel lining at construction load cases of tail skin and localized back grouting pressure, and a load combination of DC + G needs to be applied. In the final service stages, vertical (EV) and horizontal earth pressure (EH), groundwater pressure (WAP), lining self-weight (DC) and surcharge load (ES) act on the lining and considered in the SLS design of lining. Load combinations from AASHTO (2010) are used to verify the segment design at the final service stage as DC + WA_P + EH + EV + ES. Note that a load factor of 1.0 is commonly used for all above-mentioned load cases and combinations.

The methods to analyze the segments for above-mentioned load cases are presented in ACI 544.AR (2015) and Bakhshi and Nasri (2013a and 2013b). Production and transient load case are modeled by cantilever or simply supported beams using simple equations in the references. Method of Groeneweg (2007) and International Tunnelling Association guidelines (2000) are used for load cases of tail skin and localized back grouting, respectively. The effect of ground, groundwater and surcharge loads as the major final service stage load case is analyzed using elastic equations, beam-spring models, Finite Element Methods and Discrete Element Methods (Bakhshi and Nasri, 2014b). Other acceptable methods of analysis include Muir Wood's continuum model (1975) with discussion from Curtis (1976), method of Duddeck and Erdmann (1982) and an empirical method based on tunnel distortion ratios (Sinha, 1989; Deere *et al.*, 1969) that was originally developed by Peck (1969). Results of analyses following above-mentioned analyses, including bending moments, axial and shear forces are used for verifications of SLS states of flexural cracking in segments.

3 CRACKING VERIFICATION

Cracking in segments is a major cause for reduction in serviceability due to reduction of watertightness and reinforcement corrosion. In particular, cracking has a significant effect on the durability of the tunnel in an environment with frequent freeze-thaw cycles. Examination using appropriate methods should be carried out to ensure that cracking in segments does not impair the serviceability, durability or intended purposes of the tunnel lining. Among possible cracks induced in segments under service loads are mainly the cracks due to bending moment and axial force. Cracking should be examined by ensuring that the flexural crack width is not greater than the allowable crack width. The flexural crack width calculation for reinforced concrete (RC) and fiber reinforced concrete (FRC) segments is presented in the following sections, and compared in the following section for a case of tunnel with medium size diameter. Note that maximum shear force developed at segment joints as a result of a beam-spring type of modeling should be limited to shear crack capacity to satisfy SLS of shear cracking.

3.1 Calculation of Flexural Crack Width for RC Segments

Crack width in tunnel segments due to bending moments and axial forces is calculated using ACI 224.1R (2007), JSCE (2007) and EN 1992-1-1 (2004) formulas as shown in Eqs. 1, 2 and 3, respectively.

[1] $w = 0.011\beta\, f_s \sqrt[3]{d_c A} \times 10^{-3}$ and $w = 2\dfrac{f_s}{E_s}\beta\sqrt{d_c^{\,2} + \left(\dfrac{s}{2}\right)^2}$

[2] $w = s\left(\dfrac{f_s}{E_s} + \varepsilon'_{csd}\right);\ s > 0.55\left(\dfrac{15}{f'_c + 20} + 0.7\right)\cdot\dfrac{5(n+2)}{7n+8}\cdot\left(4\cdot d_c + 0.7\cdot(s-\phi)\right)$

[3] $w = s_{r,\max}\left(\dfrac{f_s - k_t\dfrac{f_{ct,eff}}{\left(\dfrac{A_s}{A}\right)}\left(1 + \dfrac{E_s}{E_{cm}}\cdot\dfrac{A_s}{A}\right)}{E_s}\right) \geq s_{r,\max}\left(0.6\dfrac{f_s}{E_s}\right)$

where w is maximum crack width (mm), β is ratio of distance between neutral axis and tension face to distance between neutral axis and centroid of rebars, f_s is stress in rebars (MPa), d_c is concrete cover over rebars (mm), A is effective tension area of concrete around rebars divided by number of rebars (mm^2), E_s is modulus of elasticity of rebars (MPa), s is maximum rebar spacing (mm), ϕ is rebar diameter (mm), ε'_{csd} is compressive strain due to shrinkage and creep equal to 150x10^{-6}, f'_c is specified compressive strength of concrete (MPa), n is number of layers of tensile rebars, $s_{r,max}$ is maximum crack spacing (mm), k_t is a factor depending on the duration of loading (0.6 for short and 0.4 for long term loading), $f_{ct,eff}$ is concrete tensile strength (MPa), A_s is the area of rebars (mm^2), and E_{cm} is modulus of elasticity of concrete (MPa).

3.2 Calculation of Flexural Crack Width for FRC Segments

Fib Model Code (2010), Italian standard CNR-DT 204 (2006), RILEM TC 162-TDF (2003) recommendation, and German DAfStb guideline (2012) are among available references to calculate crack width in concrete sections reinforced by fibers without conventional reinforcement. Eq. 4 represents the approach of Model Code (2010) and CNR-DT 204 (2006), while Eqs. 5 and 6 show the approach of RILEM TC 162-TDF (2003) and DAfStb (2012), respectively.

[4] $w = \varepsilon_{fc,t}\, h$

[5] $w = \varepsilon_{fc,t}\,(h - x)$

[6] $w = 0.14\,\varepsilon_{fc,t}$

where w is maximum crack width (mm), $\varepsilon_{fc,t}$ is the strain at extreme tensile fiber, h is the section thickness (mm) and x is the distance from extreme compression fiber to neutral axis (mm). Note that according to these references, sections can be designed without conventional reinforcement only if the minimum rebar area required for SLS ($A_{s,min}$) obtained by Eq. 7 is zero or negative.

[7] $$A_{s,\min} = (k_c k\ f_{ctm} - f_{Ftsm})\frac{A_{ct}}{\sigma_s}$$

where f_{ctm} is the average concrete tensile strength, f_{Ftsm} is the average residual strength of FRC (MPa), A_{ct} is the area of concrete within tensile zone (mm^2), σ_s is the yielding stress of the rebars (MPa), k is the coefficient taking into account non-uniformity of self-equilibrating stresses recommended as 0.8, and k_c is defined by Eq. 8.

[8] $$k_c = \frac{1+\dfrac{e}{0.4h}}{1+\dfrac{6e}{h}}; \quad if\ e = M/N < 0.4\,; \qquad k_c = \frac{1+\dfrac{0.4h}{e}}{2.5\left(1+\dfrac{h}{6e}\right)}; if\ e = M/N \geq 0.4$$

3.3 Maximum Allowable Crack Width for Tunnel Linings

Cracking in tunnel segments is controlled by limiting the crack width to specific levels to prevent durability issues due to increased permeability, excessive water leaks, and reinforcement corrosion. The allowable crack widths are recommended by standards and guidelines considering the function, importance, service, life span, purpose, surrounding environment, and surrounding soil conditions of the tunnel (JSCE, 2007). ACI 224 (2007) limits allowable cracks in structures exposed to the soil to 0.30 mm (0.012 in). Similar to ACI, European standard (EN 1992-1-1, 2004) recommends an allowable crack width of 0.30 mm (0.012 in) for RC members. In a more restricted manner, fib Model Code (2010) limits the allowable crack width to 0.2 mm (0.008 in) if leakage in the structure to be limited to a small amount and only some surface staining is acceptable. Among references specific to tunnel segments are Singapore Land Transport Authority design criteria (2010), German recommendations (DAUB, 2013) and JSCE standard (2007) that specify the allowable crack width to 0.30 mm (0.012 in), 0.2 mm (0.008 in), and 0.004d_c, respectively, where d_c is the concrete cover over the rebars. As the most comprehensive guideline, Austrian Society for Concrete and Construction Technology (ÖVBB, 2011) specifies the allowable crack width in segments based on the tunnel function, and corresponding watertightness requirements, which is shown in Table 1.

4 DESIGN EXAMPLE FOR SLS OF FLEXURAL CRACKING

An example is presented for flexural cracking SLS design of a mid-size TBM tunnel lining with both precast RC and FRC segments. It is assumed that internal diameter of the segmental ring is D_i = 5.74 m (18.83 ft), and the ring composed of 5 large segments and one key segment (one-third of the size of large segments). Width, thickness and curved length at centerline of the large segments are 1.5 m (5 ft), 0.3 m (12 in) and 3.56 m (11.7 ft), respectively. ULS design checks for this example has been presented elsewhere (Bakhshi and Nasri, 2014a). Key design parameters for RC segments are specified compressive strengths (f'_c) at early-age and 28 days of 15 and 45 MPa, respectively, in addition to 20#4 Grade 60 rebars (each with nominal area of 129 mm^2 or 0.2 in^2) designed in two rows as shown in Figure 1. Note that maximum rebar spacing (s) is 230 mm (9 in).

Table 1: Allowable crack width for SLS design of tunnel segments (ÖVBB, 2011)

Requirement Class	Designation	Application	Requirement	Allowable Crack Width
AT1	Largely dry	- One-pass lining with very tight waterproofing requirements - Portal areas	Impermeable	0.20 mm (0.008 in)
AT2	Slightly moist	- One-pass lining for road and railway tunnels with normal waterproofing requirements (excluding portals)	Moist, no running water in tunnel	0.25 mm (0.010 in)
AT3	Moist	- One-pass lining without waterproofing requirements - two-pass lining systems	Water dripping from individual spots	0.30 mm (0.012 in)
AT4	Wet	- One-pass lining without waterproofing requirements - two-pass lining as drained system	Water running in some places	0.30 mm (0.012 in)

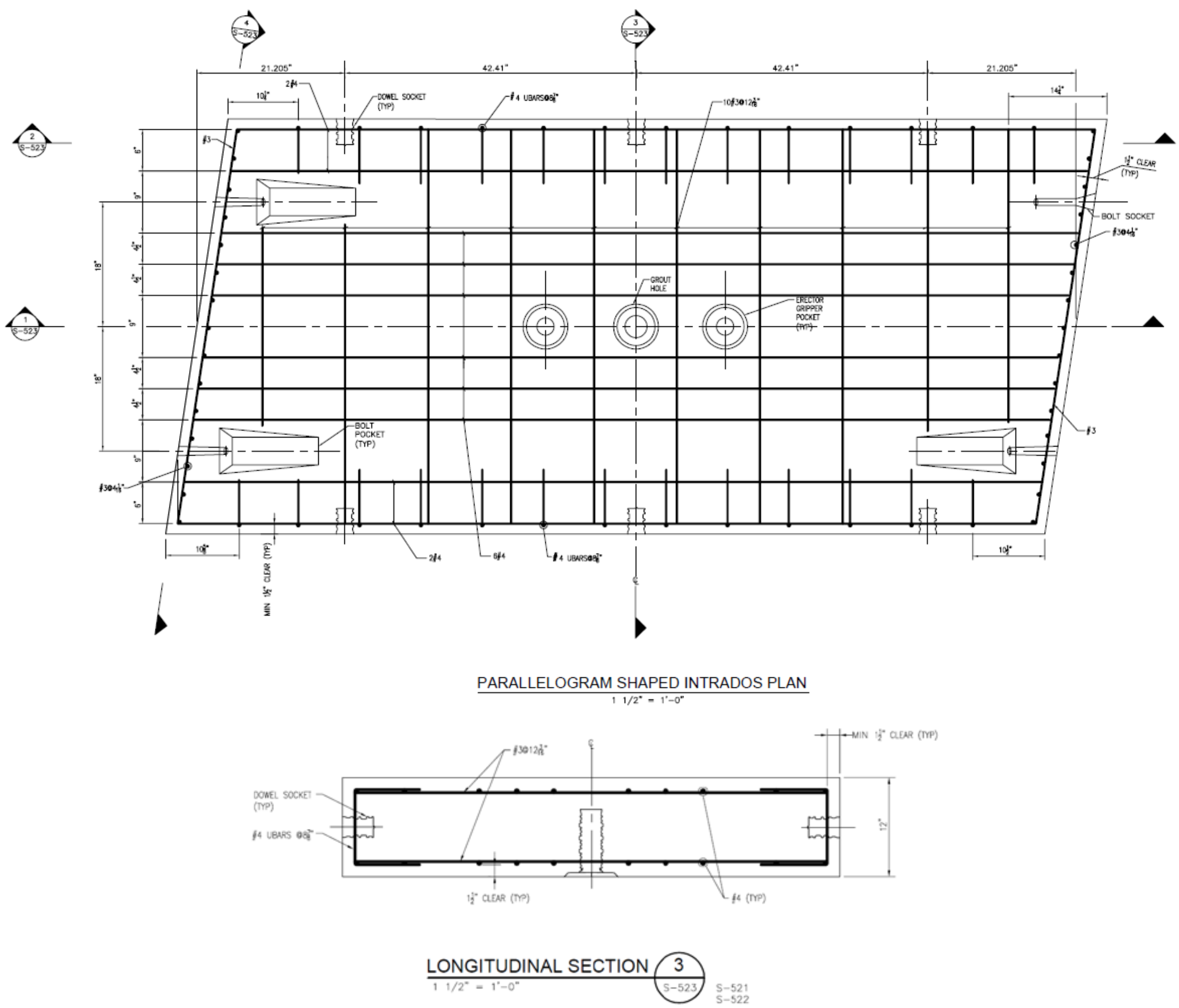

Figure 1: Designed reinforcement for a typical segment of a 5+1 ring with 5.74 m internal diameter

Table 2: SLS design checks for production and transient stages

Phase	Specified Residual Strength (MPa)	Maximum Developed Bending Moment (kNm/m)	Cracking bending moment capacity (kNm/m)
Demolding	2.5 (early-age)	3.6	34.5
Storage	2.5 (early-age)	18.0	34.5
Transportation	4.0 (28d)	14.9	57.0
Handling	4.0 (28d)	7.2	57.0

SLS design checks for the service load conditions are more critical in this example since the tunnel is excavated in soft ground, and ground and groundwater pressures induce significant bending moments in the lining. As results of two-dimensional FE analyses, maximum bending moment and corresponding axial forces in the final service stage due to SLS load combination are $M_{service}$ = 239 kN.m (177 kips-ft) and $N_{service}$ = 2,068 kN (465 kips), respectively. Cracking bending moment capacity in presence of such significant axial force is calculated by Eq. 9 as 194 kN.m (143 kips-ft) and, therefore, the segment section is expected to be cracked under this service load combination.

[9] $$M_{cr} = (f_r + \frac{N}{A_g}).s = (3.75 \times 1000 + \frac{2068}{1.5 \times 0.3}) \cdot \frac{1.5 \times 0.3^2}{6} = 194\,kN.m\,(143\,kips - ft)$$

In Eq. 9, A_g (m^2) and s are the section gross area and the uncracked section modulus of segments. Flexural analysis was performed following the assumptions that plane sections remain plane (compatibility), $\Sigma F_x = N_{service}$, $\Sigma M_z = M_{service}$ (Equilibrium), and $\sigma = E\varepsilon$ (constitutive relations for linear material). Depth of neutral axis (x) and stress in the extreme compression fiber (f_{top}) are obtained as results of the flexural analyses on RC and FRC cross sections. As shown in Figure 2, x, f_{top}, F_{st} and F_{sb} are calculated as 148 mm (5.8 in), 18.45 MPa (2.676 ksi), 1,956 kN (440 kips) and 2,122 kN (477 kips) in RC segments, respectively, where F_{st} and F_{sb} are forces in top and bottom steel bars. The stress in tensile rebars required for crack width analysis is therefore obtained as 89 MPa (12.93 ksi). Crack width calculations are made for RC segments using the methods presented in section 3.1. Results shown in Table 3 indicate that the maximum calculated crack width in RC segments ranges between 0.0711 mm (0.0028 in) using EN 1992-1-1 (2004) method to 0.142 mm (0.0056 in) using ACI 224.1R Frosch method (2007), with an average value of 0.1120 mm (0.0044 in). In FRC segments, x and f_{top} are calculated as 179 mm (7.04 in) and 17.09 MPa (2.479 ksi), as shown in Figure 3. The strain at tension face required for crack width analysis is therefore obtained as 0.00033. Results of crack width calculation shown in Table 3 indicate that maximum crack width in FRC segments ranges between 0.0421 mm (0.0017 in) using RILEM (2003) method to 0.1020 mm (0.0040 in) using fib Model Code (2010) method, with an average value of 0.06365 mm (0.0025 in). Note that minimum rebar area required for SLS ($A_{s,min}$) using Eq. 7 is negative, confirming a valid design without conventional reinforcement. Comparing predicted crack width in RC segments with FRC segments indicates that the use of fiber reinforcement results in the reduction of the maximum crack width by 43%. Maximum crack width values calculated in Table 3 are 0.142 mm (0.0056 in) for RC and 0.1020 mm (0.0056 in) for FRC segments, both below the most restrict criteria for allowable crack width of tunnel segments as 0.20 mm (0.008 in), which satisfies the SLS of flexural cracking in this design example.

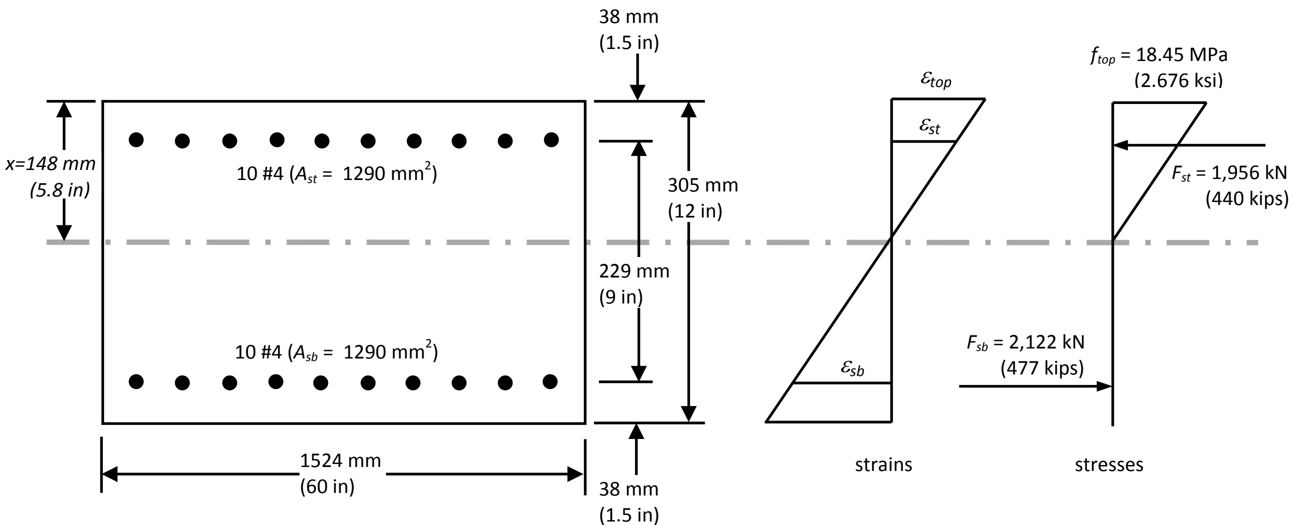

Figure 2: Flexural cross sectional analysis on RC segments reinforced with 20#4 bars ($A_{\#4}$ = 129 mm^2)

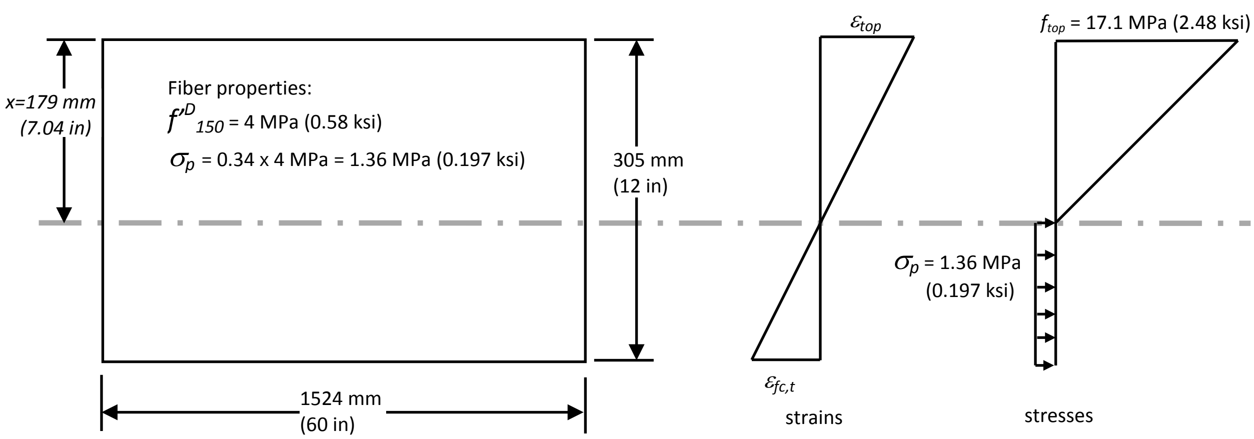

Figure 3: Flexural cross sectional analysis on FRC segments with ASTM residual parameter (f'_{D150}) of 4 MPa (580 psi)

Table 6: Results of crack width analyses for RC and FRC segments under critical service load conditions

Maximum Crack Width in RC Segments		Maximum Crack Width in FRC Segments	
ACI 224.1R (2007) - Gergely & Lutz	0.099 mm (0.0039 in)	fib Model Code (2010) CNR-DT 204 (2006)	0.102 mm (0.0040 in)
ACI 224.1R (2007) - Frosch	0.142 mm (0.0056 in)	RILEMTC 162-TDF (2003)	0.042 mm (0.0017 in)
JSCE (2007)	0.136 mm (0.0053 in)	DAfStb (2012)	0.047 mm (0.0018 in)
EN 1992-1-1 (2004)	0.071 mm (0.0028 in)		

5 CONCLUSIONS

Serviceability limit states (SLS) cause reduction of tunnel inner space due to excessive deformation, and durability and watertightness issues due to rebars corrosion and water leakage from segment cracks or enlarged gap between segment joints. Verification methods for SLS of cracking was explained including presentation of types, combination and factors of SLS loads, methods of calculations for crack width, and limiting values for design checks. The SLS of flexural cracking in both RC and FRC segments, as the major cause of durability issues, was verified for a case of mid-size tunnel. Results show that both rebars and fibers can effectively control the crack width in mid-size tunnel segments below the most restrict reference criteria. Comparing crack width in RC segments with FRC segments indicate a better performance in favor of fibers by as much as an average value of 43%.

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