ACI 544 Guideline - how to implement into your tunnel design and the benefits of FRC segments

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ABSTRACT
Fiber reinforcement has emerged as an alternative to steel bars in precast concrete segments due to advantages such as saving cost and reducing production time while developing a more robust product with improved handling and long-term durability. ACI recently drafted a new report as the first design guideline on FRC tunnel segments to provide specific guidance for this emerging technology. ACI document offers general information on the history of FRC precast segments from tunneling projects throughout the world; a procedure for structural analysis and design based on governing load cases, and a description of the material parameters, tests and analyses required to complete the design. This paper summarizes the design considerations in this ACI guideline prepared by the authors of this paper as the main contributors. Application of this guideline to design of a tunnel with an internal diameter of 5.5 m (18 ft) results in elimination of steel bars.

1 INTRODUCTION

Precast concrete segments are installed to support the tunnel 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 provide both the initial and final ground support as part of a one-pass lining system. These segments are typically designed to resist the permanent loads from the ground and groundwater as well as the temporary loads from production, transportation and construction. Tunnel segments are generally reinforced to resist the tensile stresses at both the Serviceability (SLS) and the Ultimate Limit States (ULS). With traditional rebar, a significant amount of labor is needed to assemble the cages and place the rebar, which results in higher production costs.

Fiber Reinforced Concrete (FRC) can be used to enhance the production, handling, and placement of precast concrete segments. This is because of the uniform dispersion of fibers throughout the segment to include the concrete cover, which is very advantageous for resisting the bursting and spalling stresses. These stresses develop as a result of the high loads induced on the segments during the TBM jacking process. The presence of fiber in the concrete matrix helps mitigate against unintentional impact loads during segment handling and tunnel construction operations.

The improved behavior of the concrete due to addition of fibers generally results in smaller crack widths (Bakhshi and Nasri, 2015) and less problems with durability over the life of the structure. An increase in the crack width contributes to the ingress of the environment agents into the concrete that can lead to the excessive water infiltration and occurrence of corrosion of steel reinforcement (ACI 544.5R, 2010). Two main deterioration mechanisms for corrosion include carbonation and ingress of chloride ions. Durability test results indicate that carbonation corrosion is limited to the surface regions of the steel fiber reinforced concrete (SFRC) and will neither lead to structural damage (cracking and/or spalling), nor protrude to deeper regions of the SFRC (ACI 544.5R, 2010). The corrosion arising from cracks and chloride diffusivity may cause a decrease of the load carrying capacity of the SFRC element; however, this is usually offset by rust formation that increases the fiber-paste friction, thus enhancing the fiber pullout response, which can increase the flexural capacity of SFRC elements (Granju and Balouch, 2005). Regarding performance of FRC under fire, similar to reinforcing bars in conventionally Reinforced Concrete (RC), steel fibres have been found to have little or no influence on the prevention of explosive spalling and monofilament polypropylene micro-fibres are commonly used as the explosive spalling prevention.

Since 1982, FRC has been used in numerous tunnel projects around the world, e.g. water/waste water, gas pipeline, power cable, subway, railway, and road tunnels, with internal diameters ranging between 7.2 ft (2.2 m) and 37.4 ft (11.4 m), as the preferred material for the construction of tunnel segmental lining (ACI 544.7R, 2016). Minimum and maximum thickness of the FRC precast segments have been 6” (0.15 m) and 16” (0.40 m), respectively. In most of the projects, small to mid-size tunnels with internal diameters ranging between 2.2 and 7 m (7.2 and 23 ft) have been reinforced with only steel...
fibers at a dosage ranging between 25 to 60 kg/m$^3$. The design has been performed using constitutive laws recommended by international codes and standards such as DBV (2001), RILEM TC 162-TDF (2003), CNR DT 204/2006 (2007), EHE (2008) and fib Model Code (2010). FRC technology has developed in recent years with the introduction of high performance concrete allowing the use of fibers as the sole reinforcement system for more challenging conditions on larger diameter tunnel projects. Tunnel rings of more than 23 ft (7 m) internal diameter have been successfully constructed with FRC segments to include Grosvenor Coal Mine, Channel Tunnel Rail Link Tunnel and Blue Plains Tunnel with internal diameters of 23 ft (7 m), 23.5 ft (7.15 m) and 23 ft (7 m), respectively. When the slenderness of a segment, defined as the ratio between the developed segment lengths and its thickness, is higher than 10, it is generally necessary to adopt a hybrid reinforcement of fibers and conventional steel bars. However, some researchers have proposed to increase the slenderness limit up to 12 – 13. Full-scale tests are needed to validate the usage of fibers with such slenderness conditions.

Regardless of the advantages of FRC segments, its use has been limited due to lack of recommendations and guidelines. Within ACI Committee 544, a working group drafted a new report (ACI 544.7R, 2016) that is the first design guideline on FRC segments. This ACI document provides a design procedure for FRC tunnel segments to withstand all the appropriate temporary and permanent load cases occurring during the construction and design life of tunnels, using specified post-crack residual tensile strength, $\sigma_p$ (ACI 544.8R, 2016). The design approach of the ACI report is applied to a case of mid-size tunnel to illustrate the applicability of the proposed design procedures.

2 DESING OF SEGMENTS FOR ULS

The design engineer should use Strength Design method introduced by ACI 318 (2014) by implementing combination of factored loads and reduced strengths to design precast concrete tunnel segments for ultimate limit state (ULS). ULS, which is a state associated with the collapse or structural failure of tunnel linings, is discussed in this section. The current practice in the tunnel industry is to design these elements for the following load cases, which occur during segment production, transportation, installation, and service conditions:

Production and Transient Stages
- Load Case 1: segment stripping
- Load Case 2: segment storage
- Load Case 3: segment transportation
- Load Case 4: segment handling

Construction Stages
- Load Case 5: TBM thrust jack forces
- Load Case 6: tail skin back grouting pressure
- Load Case 7: localized back grouting pressure

Final Service Stages
- Load Case 8: earth pressure, groundwater, and surcharge loads
- Load Case 9: longitudinal joint bursting load

Load Case 10: additional distortion
Load Case 11: other loads (for example, earthquake, fire and explosion)

Note that during the construction phase, designers may need to take into account other considerations, such as stresses due to gasket compression force, connecting dowels or bicones, vacuum erection shear cup and ring build imperfections. In the strength design procedure or ULS, the required strength (U) is expressed in terms of factored loads shown in Table 1. For load cases not covered by ACI 318 such as load cases 8 and 9, ACI 544 committee recommends using load factors and load combinations from AASHTO DCRT-1. The resulting axial forces, bending moments, and shear forces are used to design concrete strength and reinforcement. ACI 544 committee recommends a strength reduction factor of 0.70 for flexure, compression, and shear, and a strength reduction factor of 0.65 for bearing actions. The design procedure starts with selecting an appropriate geometry including selecting thickness, width and length of segments with respect to the size and loadings of the tunnel, followed by specifying compressive strength of concrete and reinforcement. Considering strength reduction factors, design strength of segments is compared with required strength, or otherwise improved.

Table 1. Required strength (U) expressed in terms of factored loads for governing load cases

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Required Strength (U)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: stripping</td>
<td>$U = 1.4w$</td>
</tr>
<tr>
<td>2: storage</td>
<td>$U = 1.4(w + F)$</td>
</tr>
<tr>
<td>3: transportation</td>
<td>$U = 1.4(w + F)xd$</td>
</tr>
<tr>
<td>4: handling</td>
<td>$U = 1.4wxd$</td>
</tr>
<tr>
<td>5: thrust jack forces</td>
<td>$U = 1.2J$</td>
</tr>
<tr>
<td>6: tail skin grouting</td>
<td>$U = 1.25(w + G)$</td>
</tr>
<tr>
<td>7: secondary grouting</td>
<td>$U = 1.25(w + G)$</td>
</tr>
<tr>
<td>8: earth pressure &amp; groundwater load</td>
<td>$U = 1.25(w + WAp) + 1.35(EH + EV) + 1.5 ES$</td>
</tr>
<tr>
<td>9: longitudinal joint bursting</td>
<td>$U = 1.25(w + WAp) + 1.35(EH + EV) + 1.5 ES$</td>
</tr>
<tr>
<td>10: additional distortion</td>
<td>$U = 1.4M_{distortion}$</td>
</tr>
</tbody>
</table>

Note: $w$ = self-weight; $F$ = self-weight of segments positioned above; $J$ = TBM jacking force; $G$ = grout pressure; $WAp$ = groundwater pressure; $EV$ = vertical ground pressure; $EH$ = horizontal ground pressure; ES = surcharge load; $d$ = dynamic shock factor

2.1 Production and Transient Stages

Load case of segment stripping represents the effect of lifting systems on stripping precast concrete segments from the forms in the segment manufacturing plant. Figure 1a shows the stripping phase which is modeled by two cantilever beams loaded under their own self-weights (w). Segment radius and cantilever length of segment subjected to bending are represented by R and a, respectively. The design is performed with regard to the specified strength when segments are stripped (i.e. 3-4 hr after casting). As shown in Figure 1b, the self-weight (w) is the only force acting on the segment, and therefore, the applied load factor in ULS is 1.4 per ACI 318 (2014). Note that ACI 544
committee does not consider a dynamic shock factor for the load case of stripping due to high quality control of the machines and equipment used in manufacturing plant. Designers can use recommendations of PCI Design Handbook to consider dynamic shock factor for this load case, in case high quality procedure is not insured.

Segment stripping is followed by segment storage phase in the stack yard where segments are stacked to gain specified strength before transportation to the construction site. As shown in Figure 2a, all segments comprising a full ring are piled up within one stack. Designers provide the distance between the stack supports considering an eccentricity of \( e = 0.1 \) m between the locations of the stacks support for the bottom segment and the supports of above segments. A simply supported beam loaded as in Figures 2b and 2c represent this load case. As shown in the figure, dead weight of segments positioned above \( (F) \) is acting on designed segment in addition to its self-weight \( (w) \). Therefore, corresponding load combination is \( 1.4w + 1.4F \) per ACI 318 (2014).

During the segment transportation phase, precast segments stored in the stack yard are transported to construction site and TBM trailing gear. Segments may encounter dynamic shock loads during this phase and usually half of the segments of each ring are transported in one car. Wood blocks provide supports for the segments. An eccentricity of \( 0.1 \) m is recommended for design. Similar to segment storage phase, simply supported beams represent the load case of transportation with dead weight of segments positioned above \( (F) \) and self-weight \( (w) \) as the acting loads on designed segment. In addition to load combination of \( 1.4w + 1.4F \) per ACI 318 (2014), a dynamic shock factor of \( 2.0 \) is applied to the forces for the transportation phase.

Segment handling from stack yard to trucks or rail cars are carried out by specially designed lifting devices or vacuum lifters. Inside the TBM, segment handling is usually carried out using vacuum lifters while other methods may be used occasionally. This load case is simulated similar to segment stripping shown in Figure 1. Self-weight \( (w) \) is the only force acting on segments and therefore, a dead load factor of \( 1.4 \) in ULS per ACI 318 (2014) and a dynamic shock factor \( (d) \) of \( 2.0 \) are recommended for design. Maximum bending moment and shear forces developed during above-mentioned stages are used for design checks.

### 2.2 Construction Stages

First load case during construction stage is TBM thrust jack forces. After assembly of a ring, the TBM moves forward, as shown in Figure 3a, by pushing its jacks on bearing pads placed on the circumferential joints of the newest assembled ring. This action results in development of high compression stresses under the pads, as well as bursting tensile stresses deep in the segment and spalling tensile forces between the pads. Maximum thrust force for each jack pair \( (J) \) is obtained by dividing maximum total thrust of TBM, if known, over the number of jack pairs.
In another approach, jack thrust forces on each pad \(J\) are estimated as the sum of forces required for boring into the rock or acting pressure on cutting face due to earth or slurry pressure, plus friction resistance between the shield surface and the ground, and hauling resistance of trailing gears, divided over the number of jack pads. Since TBM thrust jack forces \(J\) are the only forces acting on segment joints, no load combination is defined. It is recommended to apply a load factor of 1.2 on jack forces applied on each pad to take into account the higher actual segment joint geometry, considering gasket groove and the segment face, can be estimated using Equation (3). 

The strength of a partially pressurized surface. ACI 318 (2014) specifies the formula used for designing the bearing strength of concrete with a partially loaded segment face. DAUB (2013) recommends a similar formula that is specifically used for designing tunnel segment faces.

\[
f'_{c0} = 0.85f'_{c}
\]

where \(f'_{c0}\) is compressive strength of partially loaded surface and \(a_t\) is transverse length of stress distribution zone at the centerline of segment under thrust jacks.

In another approach, using Iyengar (1962) diagram shown in Figure 4a, tensile stresses are obtained considering \(\beta\) and \(b\) as the dimensions of the loaded surfaces, \(a\) as the dimension of stresses spreading surface inside the segment, and \(\alpha_{cm} (F/ab)\) as a fraction of the fully spread compressive stress.

Figure 4b, on the hand, shows typical results of a three-dimensional finite element (FE) simulation of effect of jack thrust forces on circumferential joints of a large-diameter tunnel. As shown in Figure 4b, in addition to the bursting stresses under the jacking pads, spalling stresses develop in the areas between the jacking pads due to the concentration of the jacking forces. Reinforcement is provided to control these bursting and spalling stresses.

Where \(T_{burst}\) and \(d_{burst}\) are bursting force and centroidal distance of bursting force from face of section as shown in Figure 3b, \(P_{pu}\) is the jacking force applied on each jack pad, \(h_{anc}\) is the length of contact zone between jack shoes and the segment face, \(h\) is the depth of cross section, and \(e_{anc}\) is the eccentricity of jack pads with respect to the centroid of cross section. If no specific value has been provided for \(e_{anc}\), then the eccentricity of the jacking forces is generally considered to be 30 mm (1.2 in). Note that this eccentricity (30 mm) should be compared with actual segment joint geometry, considering gasket groove and the proposed jacking shoe details and position.

High compressive stress is developed under the jacking pads due to the TBM thrust jacking forces. This compressive stress, considering \(a_{t}\) as the transverse length of the contact zone between jack shoes and the segment face, can be estimated using Equation (3).

\[
\sigma_{c,j} = \frac{P_{pu}}{a_{t}h_{anc}}
\]

Because only part of the circumferential segment face is actually in contact with the pads, the allowable compressive stresses \(f'_{c}\) can be factored to account for the strength of a partially pressurized surface. ACI 318
The load case of tail void grouting presents backfill grouting or filling the annular space with semi-liquid grouts which is required in order to control and restrict settlement at the surface and to securely lock the lining ring in position. Grout pressure has to be limited to a minimum value which is slightly higher than the water pressure, and a maximum value which is less than the overburden pressure. For the case of tail void grouting, vertical gradient of grout pressure is calculated by taking the equilibrium between the upward components of total grout pressure, lining deadweight and tangential component of grout shear strength (Groeneweg 2007). This load case is modeled by applying radial pressures increasing linearly from the crown to the invert of tunnel. Self-weight (w) and grouting pressure (G) are the acting loads on the lining at this phase, and a therefore, a load combination of 1.25 DC + 1.25 G needs to be applied in ULS following AASHTO (2010) recommendation in the absence of an ACI 318 recommendation for this load case.

In the case of localized backfilling, radial injection through holes provided in the segments is performed where annular gaps exist between the lining extrados and excavation profile after tail grouting. ITA WG2 (2000) approach is used for simulation of localized triangularly distributed backfilling pressure. As shown in Figure 5a, the lining is modeled as a 2D solid ring with a reduced flexural rigidity due to segment joints, and the interaction between the lining and surrounding ground or primary hardened grout is modeled by radial springs. Using a structural analysis package, bending moments and axial forces due to the grouting load cases are determined and checked against segment strength.

2.3 Final Service Stages

The loading in the final service stage is represented by the long-term interaction of the lining with the ground and groundwater pressure, as well as other factors specific to an individual tunnel, e.g. additional distortion, earthquake, fire, explosion, and breakouts. Longitudinal joint bursting load due to force transfer in a reduced cross section because of gasket and stress relief grooves is another critical load case in the final service stage. Due to similarity to the effect of thrust jack forces, it is not discussed further as similar analysis and design methods are applicable to this load case.

Precast concrete segments as final lining system withstand different loadings in the service stage including ground (vertical and horizontal) loads, groundwater pressure, dead weight, surcharge and ground reaction loads. As presented in Table 1, in the absence of an ACI 318 recommendation for this load case, load factors and load combinations from AASHTO (2010) are used to compute the forces. 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 (FEM) and Discrete Element Methods (DEM).

Beam-spring models are the most conventional methods. As shown in Figure 5b and 5c, a so-called two-and-a-half-dimensional, multiple-hinged, segmented, double-ring beam spring has been used to model the reduction of bending rigidity and the effects from a staggered geometry. This manipulation is achieved by modeling the segments as curved beams, flat longitudinal joints as rotational springs (Janßen joints [Groeneweg, 2007]), and circumferential joints as shear springs. Two rings are used to evaluate the coupling effects; however, with this method, only half of the segment width is considered from one ring for the longitudinal and circumferential joint zone of influence. Considering the self-weight of the lining, and distributing the ground, groundwater, and surcharge loads along the beam, member forces can be calculated using a conventional structural analysis package.

Other acceptable methods of analysis include Muir Wood (1975) continuum model with discussion from Curtis (1976), Duddeck and Erdmann (1982) and an empirical method based on tunnel distortion ratios (Sinha, 1989).
An example for design of a mid-size TBM tunnel lining with precast FRC segments is presented. It is assumed that internal diameter of the segmental ring is \( D_i = 5.5 \text{ m} \) (18 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 3.4 m (11.2 ft), respectively. A stress-strain diagram according to ACI 544.8R (2016) is adopted. Key design parameters for aforementioned load cases are the specified residual tensile or residual flexural strength \( \sigma_p \) or \( f_{D150}' \) and specified compressive strength \( f'c \). A scale factor of 0.34 is considered to convert \( f_{D150}' \) to \( \sigma_p \).

Designed demolding and 28-day \( f_{D150}' \) strengths are 2.5 MPa (360 psi) and 4 MPa (580 psi), respectively. Specified compressive strengths are 15 MPa (2,175 psi) for demolding and 45 MPa (6,525 psi) for 28-day FRC segments. As shown in Figure 6, capacity of FRC segments are calculated based on equilibrium conditions assuming a post-crack plastic behavior in the tension zone. First crack flexural strength \( f_1 \) is assumed as 4 MPa (580 psi). Design checks for the production and transitional loads are shown in Table 2. The tunnel is excavated in jointed rock. Two-dimensional DEM model shown in Figure 7a is used for calculation of tunnel lining forces in three different geological reaches defined along the alignment. Design checks for the load case of the ground and groundwater pressure is shown in Figure 7b. A TBM with maximum total thrust of 5,620 kips (25,000 kN) applied on 16 jack pairs is assumed for this project. Maximum thrust forces on each pair is therefore 351 kips (1.562 MN). The length and width of the contact area between the jack pads and segments, considering a maximum eccentricity of \( e = 1 \text{ in} \) (0.025 m), are \( a_l = 34 \text{ in} \) (0.87 m) and \( h_{anc} = 8 \text{ in} \) (0.2 m), respectively.

**Figure 5.** a) Modeling localized grouting pressure applied over 1/10th of lining perimeter, b) Double ring Beam-Spring model with radial soil springs, together with longitudinal and ring springs representing segment joints, c) Scheme of the ring joint (ACI 544.7R 2016).

**Figure 6.** Strain and stress distributions through the section as part of it undergoes tension.

**Table 2.** Segment design checks for production and transitional stages

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specified Residual Strength, MPa (psi)</th>
<th>Maximum Factored Bending Moment, kNm/m (kipf-ft/ft)</th>
<th>Resisting Bending Moment, kNm/m (kipf-ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stripping</td>
<td>2.5 (360)</td>
<td>5.04 (1.13)</td>
<td>26.25 (5.91)</td>
</tr>
<tr>
<td>Storage</td>
<td>2.5 (360)</td>
<td>18.01 (4.05)</td>
<td>26.25 (5.91)</td>
</tr>
<tr>
<td>Transportation</td>
<td>4.0 (580)</td>
<td>20.80 (4.68)</td>
<td>42.00 (9.44)</td>
</tr>
<tr>
<td>Handling</td>
<td>4.0 (580)</td>
<td>10.08 (2.26)</td>
<td>42.00 (9.44)</td>
</tr>
</tbody>
</table>
Dimensions of fully spread stresses are at = 11.1 ft /3 = 3.7 ft (1.13) m and h = 12 in (0.3) m in tangential and radial directions, respectively. Conforming to simplified equations of ACI 318 (2014), bursting force \( T_{\text{burst}} \) and its centroidal distance from the face of section \( d_{\text{burst}} \) in radial and tangential directions are:

\[
\begin{align*}
\text{Tangential direction:} & \quad d_{\text{burst}} = 0.5(h - 2e_{\text{anc}}) = \frac{0.5(12 - 2 \times 1)}{12} = 5 \text{ in (0.125 m)} \\
& \quad T_{\text{burst}} = 0.25 P_{\text{w}} \left(1 - \frac{h_{\text{anc}}}{h - 2e_{\text{anc}}}\right) = 0.25 \times 351 \times \left(1 - \frac{8}{12 - 2 \times 1}\right) = 17.55 \text{ kipf (0.078MN)}
\end{align*}
\]

Maximum bursting stresses developed in radial and transverse directions with a ULS load factor of 1.2 are

\[
\begin{align*}
\text{Tangential direction:} & \quad \sigma_p = \frac{1.2 T_{\text{burst}}}{\phi h_{\text{anc}} d_{\text{burst}}} = \frac{1.2 \times 17.32 \times 1000}{0.7 \times 8 \times 1.77 \times 12} = 174 \text{ psi (1.2MPa)} \\
\text{Radial direction:} & \quad \sigma_p = \frac{1.2 T_{\text{burst}}}{\phi a_t d_{\text{burst}}} = \frac{1.2 \times 17.55 \times 1000}{0.7 \times 34 \times 5} = 177 \text{ psi (1.22 MPa)}
\end{align*}
\]

These stresses are less than 28-day specified residual tensile strength of FRC segment as \( \sigma_p = 0.34 f'_{D150} = 0.34(580) = 197 \text{ psi (1.36 MPa)} \), and the design is valid for load case of TBM thrust jack forces.

4 CONCLUSIONS

Regardless of the advantages of FRC, its use in tunnel segments has been limited due to lack of recommendations and guidelines. This paper briefly explains the design concepts of a new ACI report (ACI 544.7R. 2016) that is the first design guideline on FRC segments. Presented design procedures include design for production and transient, construction and final service stages. Application of the design approach to a case of mid-size tunnel in jointed rock indicates that the use of fibers can lead to elimination of steel bars.

5 REFERENCES


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