Highlights:

- Effect of Steel FRC on flexural behavior of segmental tunnel joints is studied
- Micro steel fibers show more positive influence than macro fibers on joint behavior
- Simple relations are proposed for seismic design of Steel FRC segmental joint
Graphical Abstract

Segmental Joint

Steel Fiber Reinforced Concrete

Macro fiber
Micro fiber

$M-\theta$ behavior curve

EARTHQUAKE

Segmental Joint Seismic Design Parameters

Moment Demand Ratio $\alpha_{R_0}$

Ductility Demand Ratio $\mu_{\theta_0}$

IM (PGA/g)
Numerical-aided design of fiber reinforced concrete tunnel segment joints subjected to seismic loads

Abstract

In this paper, the effects of different steel fiber reinforced concrete (SFRC) composites on the flexural response of segmental joints under seismic actions is investigated numerically based on experimental results. The results in terms of moment – rotation (M – θ) curves derived from an experimental test set-up, are used to calibrate and verify a finite element numerical model of the joint. From seismic analyses, the SFRC mixes show to enhance the seismic performance of the joint compared to plain concrete or traditional reinforced concrete. Finally, equations are proposed to estimate the joint’s moment demand/capacity ratio and rotational ductility for seismic design.

KEYWORDS: Segmental Tunnel; Segmental joint; Steel Fiber Reinforced Concrete; Hybrid Fiber; Seismic Performance

1. Introduction

In infrastructural development, tunneling projects generally consume a considerable amount of national budgets, justifying the need of more research focused on cost reduction and productivity enhancement methods. The use of fibers in cementitious composites has been introduced as a potential solution to increase productivity by cutting costs and saving time in tunneling projects [1-3]. Today, Fiber Reinforced Concrete (FRC) is commonly considered a suitable alternative to traditional concrete reinforcing solutions in designing structures for both SLS and ULS conditions [4]. In the past few years, the use of structural fibers as partial, or even full replacement of traditional reinforcement in segmental linings of tunnel, has gained great interest in the tunneling industry [5-7]; particularly, after the realize of the Model Code 2010 [8], which gather bases to design FRC elements [[7, 9-11];[1]].

Structural fibers are produced from different materials (e.g., steel, polypropylene, glass, carbon). Likewise, various geometries (length, thickness, shape, anchorage) have been designed.
to meet specific technical requirements. Application of steel fibers has shown to improve the
tensile post-cracking behavior of concrete composites by generating a crack-bridge mechanism
to control crack width [12], resulting in more economic designs of structural elements, e.g.
tunnel lining segments, generally designed in accordance with main standards [6, 13, 14].
Presence of fibers properly distributed [15] throughout the tunnel lining segment can
significantly reduce detrimental consequences due to extreme loads from TBM jacks [16-18], fire
exposure [19, 20] or explosion [19, 21].

In order to take advantage of the properties of each specific fiber geometry and composition,
hybrid fiber reinforced concretes (FRCs) have been introduced [22, 23]. In this sense, short
fibers enhance the micro-cracking and cracking control within the range of SLS while longer
fibers are capable to bear stresses even in ULS conditions [24]. In any case, the type and
specifications of a fiber concrete mix is chosen to meet the target design requirements which
requires experience and testing efforts [25, 26].

Currently, segmental tunnels are widely used in seismic areas around the world, such as
Mexico, Chile, Japan, Iran, the United States, and elsewhere. In such seismically active regions,
the performance and vulnerability of infrastructure that can be subjected to earthquake loads, is
of great concern. Despite being less vulnerable than above ground structures, minor to extreme
incidents of damage to underground tunnels have been reported in past earthquakes [27-29]. It
must be highlighted that tunnel deformations and water leakage, even reduced, can provoke
severe damages to the structures above due to differential time-differed settlements. Therefore,
for the reliable implementation of FRC in tunnel linings in such regions, extensive research on
the seismic performance and vulnerability of tunnels linings is of paramount importance to
guarantee the safety and integrity of the surrounding structures.
In segmented lining tunnels, the longitudinal joints, which connect the adjacent segments of each ring, also known as “segmental joints”, have a significant effect on the global response [30-32]. Under seismic deformations in the transversal direction, segmental joints provide the necessary capacity for the tunnel section to accommodate the seismically generated ovaling deformations and other transversal distortions, playing an important role in preventing significant damage to the segment lining [32]. Despite the proved influence of the joint’ properties on the tunnel response, for simplicity or uncertainties when assigning rotational stiffness, segmental joints are usually simplified as hinges in the design process by considering no flexural capacity [33]. Yet, research conducted on the behavior of segmental joints (plain type joint) indicate a semi-rigid flexural response [34, 35], falling between the two extremes of perfect hinge (no bending bearing capacity) and fully rigid (continuous lining) configurations. In a segmental lined tunnel, transfer of load and bending moment between adjacent segments occurs due to interaction (contact) between the concrete segments, analogous to TBM jack load on segments, which the material resistant properties, the configurations and geometry of the jacks and thrust eccentricity strongly affects the joint’ response [34].

El Naggar et al. [36] proposed a simplified analytical procedure to evaluate the in-plane response of segmented lining tunnels, considering linear elastic properties for the lining and simulating the segmental joints using a rotational stiffness. In a comprehensive study by He and Koizumi [37], the seismic response of shield tunnels in the transverse direction was studied with consideration of segmental joint effects. A series of shaking table model tests were carried out, followed by numerical simulations including 2D dynamic Finite Element Method (FEM), and static analyses based on the seismic deformation method by considering simple beam-spring model. The segmental joints were modeled using short beam elements with reduced axial and
flexural rigidity in the static FEM analysis. In the beam-spring model, the segmental joints were modeled using a rotational spring with a constant value of rotational stiffness. The tunnel lining material was considered as linear elastic, defined by a modulus of elasticity. In a study by Chow et al. [38], the seismic in-plane response of a segmental tunnel lining was evaluated by a 2D FEM model; simulating the segmental joints as hinges. The authors concluded that a pseudo-static analysis could effectively reproduce seismically induced force demands in the tunnel lining. Do et al. [35] studied the influence of the segmental and longitudinal joints’ properties on the overall 2D seismic response of segmental lining tunnels using a finite difference element model. This study indicated the dependency of segmental joints’ behavior on surrounding soil conditions, segment configuration, and its material properties; emphasizing on the better performance of segmental tunnels over continuous lining tunnels under seismic loads.

Properties of the segment material have proven significant influence on the behavior of segmental joints, especially under seismic deformations [32, 34]. Past research on segmental joint behavior has been mainly focused on plain or conventionally reinforced concrete as the segment lining material, generally modeled as a linear elastic material [30]. The safe and reliable application of fibers in segmented lined tunnels requires understanding the performance of its segmental joints, especially for tunnels constructed in seismic regions. In this regard, limited research has been conducted [39-42] to study the effect of FRC composites on the segmental joint behavior. Yet, an extensive lack of research exists considering different FRC composites with focus on seismic performance of segmental joints.

In this paper, an extensive numerical research on the flexural response of segmental joints in SFRC precast segmental linings subjected to earthquake loading is carried out considering the involved mechanical and geometric nonlinearities. The main goal is to study the effects of
different SFRC composites by comparing to that of unreinforced and conventionally reinforced concrete alternatives. These alternatives have been considered to be used in a particular segment geometry of the metro Line 7 of Tehran (Iran). To this end, six Hybrid Steel FRC (HSFRC) mixes comprising the use of different fiber combinations (0.3% and 0.5% volume content of micro and macro size) were investigated. Posteriorly, the post-cracking tensile behavior of these HSFRC composites was obtained by performing flexural 3-Point Bending Tests (3-PBTs) on notched specimens. As a result, stress-strain relationships were obtained for the HSFRC mixes by using the RILEM TC 162-TDF [43] specification. The suitability of these constitutive equations was confirmed using a numerical model capable to reproduce the 3-PBTs.

The experimental results[43] obtained from a test set-up designed for characterizing the mechanical response of segmental joints are used to confirm the accuracy of a non-linear FEM implemented to simulate the moment-rotation behavior of segmental joints. The model has been posteriorly used to derive curves for different HSFRC segmental joints. Finally, these relationships are assigned to the joints of the Tehran’s Metro Line7 precast segments and 2D seismic analyses is carried out in order to obtain the mechanical behavior of the different parts of the ring. In this sense, the results derived are analyzed aiming at assessing and compare the vulnerability of the different HSFRC segmental joints. As a result, straightforward relationships are proposed for obtaining the flexural response of HSFRC segmental joints under seismic loads as function of the residual flexural strength capacity of the composite.

Finally, conclusions based on the results are established along with the limitations and range of application of these. Further research is also proposed in order to expand the conclusions and increase the range of validity and applicability.
2. Experimental program

2.1. Introduction

The mechanical properties of FRC composites are determined by a series of standardized experimental tests. In particular, the values obtained from experimental tests are used to derive the stress ($\sigma$) – strain ($\varepsilon$) constitutive relationships of the material. In this research, the mechanical properties of the SFRC are used in the numerical phase to study the behavior of the segmental joints subjected to seismic loads.

2.2. Materials and mechanical characterization

Six SFRC mixes (1-6) and one Plain Concrete (PC) mix (7) have been produced (dosages presented in Table 1) in this experimental program. The micro and macro steel fibers geometric and mechanical properties are presented in Table 2 and Figure 1. Posteriorly, a characterization phase (Figure 2) was carried out at 28 days involving (2/dosage) compressive[44] and (2/dosage) indirect tensile[45] tests on 150x300 mm cylinders, and (3/dosage) 3 point bending tests (3PBTs) on 150x150x600 mm prismatic notched beams according to [46]. LVDT’s were installed onto the tip of the notch and a servo-controlled hydraulic press was used.

<table>
<thead>
<tr>
<th>Mix no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
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<tr>
<td>Macro steel fiber ratio (%)</td>
<td>0.5</td>
<td>-</td>
<td>0.3</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
<td>-</td>
</tr>
<tr>
<td>Micro steel fiber ratio (%)</td>
<td>-</td>
<td>0.5</td>
<td>0.3</td>
<td>0.5</td>
<td>0.3</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>Portland Cement type</td>
<td>II</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
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<td>Natural sand (kg/m$^3$)</td>
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<td>1172</td>
<td>1172</td>
<td>1172</td>
<td>1172</td>
<td>1172</td>
</tr>
<tr>
<td>Gravel (kg/m$^3$)</td>
<td>632</td>
<td>632</td>
<td>632</td>
<td>632</td>
<td>632</td>
<td>632</td>
<td>632</td>
</tr>
<tr>
<td>Water-Cement ratio</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
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</table>

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Geometry</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>Aspect Ratio</th>
<th>Tensile strength (MPa)</th>
<th>Elasticity module (GPa)</th>
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<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Macro steel fiber</td>
<td>Hooked</td>
<td>50</td>
<td>0.80</td>
<td>62.5</td>
<td>1169</td>
<td>210</td>
</tr>
<tr>
<td>Micro steel fiber</td>
<td>Smooth</td>
<td>13</td>
<td>0.17</td>
<td>76.5</td>
<td>2100</td>
<td>210</td>
</tr>
</tbody>
</table>

Figure 1: Macro (left) and micro (right) size steel fiber

Figure 2: Material characterization tests: (a) compressive; (b) indirect tensile and (c) tensile-flexural concrete strengths.

2.3. Results and analysis

The slump values, mean compressive ($f_{cm}$) and indirect tensile ($f_{ctm,i}$) strength results along with corresponding coefficient of variations are listed in Table 3. As it can be noticed, the slump of the PC mix (7) ranged between 10 and 15 cm. The addition of fibers led to a 50% of workability reduction for the HSFRC mixes. Nevertheless, this reduction does not imply a problem in terms of production since high energetic vibration is induced into the molds during the casting (very dry concrete with slumps < 3 cm are frequently used).

It must also be highlighted that the inclusion of fibers leads to an increase of both the compressive and tensile strengths of FRC mixes comparing to plain concrete (41.1 N/mm$^2$ and
3.4 N/mm², respectively). This effect has also been reported by other authors \([26]\) and can be particularly attributed to the capacity of short fibers to bridge the internal micro-cracks that govern the resistant mechanism during the tests.

Table 3: Mean values of compression and indirect tensile strength (CV in %)

<table>
<thead>
<tr>
<th>Mix no.</th>
<th>Macro steel fiber ratio (%)</th>
<th>Micro steel fiber ratio (%)</th>
<th>Slump (cm)</th>
<th>Compression strength (N/mm²)</th>
<th>CV (%)</th>
<th>Tensile Strength (N/mm²)</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>-</td>
<td>7-9</td>
<td>46.4</td>
<td>4.82</td>
<td>4.2</td>
<td>4.24</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>0.5</td>
<td>5-7</td>
<td>68.8</td>
<td>5.91</td>
<td>5.2</td>
<td>6.86</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>0.3</td>
<td>5-7</td>
<td>58.3</td>
<td>0.70</td>
<td>4.7</td>
<td>0.64</td>
</tr>
<tr>
<td>4</td>
<td>0.5</td>
<td>0.5</td>
<td>4-6</td>
<td>60.8</td>
<td>3.29</td>
<td>6.7</td>
<td>4.61</td>
</tr>
<tr>
<td>5</td>
<td>0.5</td>
<td>0.3</td>
<td>5-7</td>
<td>54.0</td>
<td>2.49</td>
<td>5.5</td>
<td>3.29</td>
</tr>
<tr>
<td>6</td>
<td>0.3</td>
<td>0.5</td>
<td>5-7</td>
<td>65.6</td>
<td>2.25</td>
<td>5.0</td>
<td>2.21</td>
</tr>
<tr>
<td>7</td>
<td>-</td>
<td>-</td>
<td>10-15</td>
<td>41.1</td>
<td>2.59</td>
<td>3.4</td>
<td>1.79</td>
</tr>
</tbody>
</table>

Figure 3 gathers the averaged load – midspan deflection (F-δ) relationships for each mix obtained in the 3-PBTs. In 3-PBTs, there exist a proven relationship (Equation 1) \([49]\) that relates CMOD and δ. This relationship has been used to derive the CMOD values from those δ measured experimentally.

\[
CMOD = 1.18 \delta_m + \beta \quad \beta = -0.0416 \text{ mm}
\]  

(1)

The flexural tensile strength \(f_{LOP}\) and the residual tensile flexural strengths \(f_{RI}\) for different crack mouth opening displacements (CMOD\(_i\)), CMOD\(_i\) = 0.5, 1.5, 2.5, and 3.5 mm for \(i = 1 – 4\), respectively are presented in Table 4. The \(f_{LOP}\) and \(f_{RI}\) values were calculated from the F-δ relationships (Figure 3) using the analytical procedure proposed by RILEM TC 162-TDF \([43]\). Classification of each SFRC composite according to the Model Code 2010 (MC-2010)\([8]\) is also presented in Table 4.
3. Simulation of the mechanical behavior of the HSFRC composites

3.1. Introduction

The RILEM TC 162-TDF [43] procedure for deriving stress-strain (σ-ε) relationships for FRC from 3-PBTs results is taken into account herein. Besides, the resulting σ-ε relationships for the HSFRC mixes are validated using numerical modeling.
3.2. Constitutive relationships

According to RILEM TC 162-TDF [43], tension and compression in FRC can be simulated based on the $\sigma$-$\varepsilon$ diagram shown in Figure 4.

Figure 4: Stress-Strain relationship for fiber reinforced concrete

The average $\sigma$-$\varepsilon$ values and young modulus ($E_c$) for the HSFRC concrete mixes are reported in Table 5. In FE modeling, the uniaxial tensile strength ($f_{ct}$) should be used as $\sigma_1$. According to equation (5.1-7) of the MC-2010[8], $f_{ct}$ can be assumed as the indirect (splitting) tensile strength of concrete ($f_{cm}$).

Table 5: Stress-strain properties of SFRC composites (based on RILEM TC 162-TDF [43])

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Mix Abbrev.</th>
<th>$\sigma_1$ (MPa)</th>
<th>$\sigma_2$ (MPa)</th>
<th>$\sigma_3$ (MPa)</th>
<th>$\varepsilon_1$ (‰)</th>
<th>$\varepsilon_2$ (‰)</th>
<th>$\varepsilon_3$ (‰)</th>
<th>$E_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5Mac</td>
<td>6.31</td>
<td>5.23</td>
<td>2.58</td>
<td>0.192</td>
<td>0.292</td>
<td>25</td>
<td>32945</td>
</tr>
<tr>
<td>2</td>
<td>0.5Mic</td>
<td>8.54</td>
<td>7.87</td>
<td>2.96</td>
<td>0.223</td>
<td>0.323</td>
<td>25</td>
<td>38306</td>
</tr>
<tr>
<td>3</td>
<td>0.3Mac0.3Mic</td>
<td>7.31</td>
<td>6.74</td>
<td>1.76</td>
<td>0.206</td>
<td>0.306</td>
<td>25</td>
<td>35446</td>
</tr>
<tr>
<td>4</td>
<td>0.5Mac0.5Mic</td>
<td>7.73</td>
<td>5.75</td>
<td>3.68</td>
<td>0.212</td>
<td>0.312</td>
<td>25</td>
<td>36453</td>
</tr>
<tr>
<td>5</td>
<td>0.5Mac0.3Mic</td>
<td>7.82</td>
<td>5.82</td>
<td>2.44</td>
<td>0.213</td>
<td>0.313</td>
<td>25</td>
<td>36664</td>
</tr>
<tr>
<td>6</td>
<td>0.3Mac0.5Mic</td>
<td>7.03</td>
<td>6.03</td>
<td>2.86</td>
<td>0.202</td>
<td>0.302</td>
<td>25</td>
<td>34761</td>
</tr>
</tbody>
</table>

3.3. Numerical validation of the $\sigma$-$\varepsilon$ constitutive relationships

For validating the suitability of these constitutive relationships for simulating the mechanical performance of the HFRC composites (Table 5), a numerical 3D-FE model has been developed.
by using the ABAQUS [47] package. This model is capable of representing the boundary
(supports and notch) and loading conditions existing in the 3-PBT test. The loading rate effects
were taken into account by including a dynamic solution [48].

The mesh of the beam model (see Figure 5) consists of 3D stress hex elements [48]. Each
beam model is adequately meshed in the three special dimensions. The damaged plasticity model
[48] was used to simulate the nonlinear behavior of the material and the damage development
due to cracking. The $F - CMOD$ relationships resulting from the FE model are compared with
those obtained experimentally. Figure 5 gathers those obtained for mix 3 (0.3Mac0.3Mic); the
other hybrid mixes following similar patterns. The results indicate that trends of the F-CMOD
curves derived numerically fit reasonably well with those obtained experimentally.

![Figure 5: Load-CMOD curves for 0.3mic 0.3mac (mix 3): Experimental and Finite element results](image)
Maximum loads \( (F_{\text{max}}) \) and load values corresponding CMOD\(_i\), \( i=1-4 \) for the numerical and experimental curves, along with relative error values, are listed in Table 6. The numerical values overestimate those experimentally obtained by a maximum of 12.1\% for \( F_{\text{max}} \) and 28.0\% (\( F_{R1} \)), 17.7 (\( F_{R2} \)), 18.9\% (\( F_{R3} \)) and 37.8\% (\( F_{R4} \)) for the post-cracking forces. Although these differences might be considered as high, these can be accepted since: (1) the intrinsic scatter of the material lead to variation coefficients of \( F_{Ri} \) easily higher than 25\% in the 3-PBTs and (2) the aim of the posterior numerical simulation of segmental joints’ is to compare rather qualitatively the mechanical behavior of the different mixes.

Table 6: Experimental and numerical values for \( F_{\text{max}}, F_{R1}, F_{R2}, F_{R3} \) (kN) and relative errors, \( \delta \) (%)

<table>
<thead>
<tr>
<th>Hybrid Mix No.</th>
<th>Mix Abbrev.</th>
<th>( F_{\text{max}} ) (kN)</th>
<th>( F_{R1} ) (kN)</th>
<th>( F_{R2} ) (kN)</th>
<th>( F_{R3} ) (kN)</th>
<th>( F_{R4} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.3Mac0.3Mic</td>
<td>1.95/1.84</td>
<td>15.7</td>
<td>1.10/1.11</td>
<td>0.9</td>
<td>0.82/0.51</td>
</tr>
<tr>
<td>4</td>
<td>0.5Mac0.5Mic</td>
<td>2.47/2.17</td>
<td>24.2</td>
<td>1.79/1.59</td>
<td>11.2</td>
<td>1.23/1.07</td>
</tr>
<tr>
<td>5</td>
<td>0.5Mac0.3Mic</td>
<td>1.90/1.86</td>
<td>28.0</td>
<td>1.30/1.07</td>
<td>17.7</td>
<td>0.93/0.72</td>
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<tr>
<td>6</td>
<td>0.3Mac0.5Mic</td>
<td>2.06/1.89</td>
<td>19.4</td>
<td>1.25/1.24</td>
<td>0.8</td>
<td>1.02/0.82</td>
</tr>
</tbody>
</table>

4. Calibration of nonlinear FE model for segmental joints

4.1. Introduction

As previously mentioned, segmental joints strongly govern the overall behavior of a segmental lining tunnel. Different parameters influence the behavior of segmental joints: (1) magnitude and position of normal forces acting at the joint; (2) joint geometry and constituent material and (3) soil properties [35, 50].

In this regard, joints might be subjected to rotation, this leading to concrete-to-concrete contact between the adjoining segments, and thus bending moment and load transfer. This could result in potential damage in those cases in which high stress concentration occur. This is particularly relevant when the tunnel is subjected to asymmetric deformations in the transversal
direction, generated by seismic shear waves (S-waves). This problem is analogous to the damage inflicted by TBM thrust forces on segments in the assembly phase of segmental tunnels [51, 52]. The damage can have detrimental consequences on the performance of a segmental tunnel at both SLS and ULS levels [53], including spalling of the segment edges, cracking of the lining, loss of water tightness and other negative effects on both durability and safety.

4.2. Experimental results used for calibration (Hordijk and Gijsbers research)

Different researchers have proposed theoretical models to describe the moment-rotation behavior of segmental joints [54-56]. However, it must be highlighted that none of these models account for the post-cracking behavior of the material, neither the type nor amount of reinforcement. Contrarily, experimental research on the moment-rotation behavior of segmental joints carried out for validating existing and newly proposed theoretical models using numerical approaches is significant. Hordijk and Gijsbers [57] conducted an extensive experimental program to study the flexural behavior of segmental joints with no packing materials (Figure 6). In the test procedure, the segments were initially loaded with a normal force to replicate the initial state of stress in the tunnel lining. Thereafter, a bending moment was then increasingly applied by the means of an eccentric jack load. The values of bending moment and joint rotation were continuously recorded. The resulting values of moment and rotation form the moment-rotation diagram of the segmental joint.
4.3. Nonlinear FE model development

A 3D nonlinear FE model meant to simulate the behavior of segmental joints has been implemented in ABAQUS [47]. The Hordijk and Gijsbers [57] experimental test set-up has been simulated with the model and the experimental results have been compared with those obtained numerically in order to assess the goodness of the model.

For this purpose, three-dimensional hex elements [48] with a mesh size equal to 0.04 m were used to adequately mesh the adjoining segments (Figure 7). The bolt was modeled using three-dimensional hex elements [48] with a mesh size equal to 0.01 m and considered as an embedded region [48] within the segments. In the normal (axial) direction of the segmental connection, the connecting steel bolt was modeled with linear behavior, and full surface-to-surface contact [48] was considered to simulate the segment-to-segment interaction. To calculate the joint rotation...
values, the measurements at the 50 mm distance over the joint were considered to sufficiently represent the actual rotations of the joint.

To validate the FE model, the experimental tests corresponding to the specimens with a bolt in the positive bending direction [57] are simulated. The geometrical and average mechanical properties of these test specimens are gathered in Table 7.

In Figure 8, the numerically obtained moment-rotation curves for different levels of normal force are compared to those obtained by [57] for the case of a segment joint with a bolt in the positive bending direction. Both the experimental and numerically obtained moment-rotation diagrams constitute of a linear and a nonlinear region. According to [57], the initial rotational stiffness and maximum bending moment capacity was barely affected by the presence of bolts. Unlike theoretical models[54, 55], the experimental results showed a direct relation between the initial rotational stiffness and the normal force. Moreover, the maximum moment capacity and corresponding rotation of the segment joint increases with the normal force, implying more ductile behavior. This favorable behavior is of paramount significance for segmented lining tunnels in ULS conditions under seismic actions. By comparing the experimental and numerical results, it can be seen that a good match has been obtained for all cases of normal force, confirming the suitability of the model for dealing with the simulation of this particular boundary and loading conditions.
Figure 7: Test set-up for segmental joint behavior of Tehran Metro Line 7 tunnel: Schematic representation and geometry (left) and FE model (right).

Table 7: Geometrical and mechanical properties of test specimens (bolt in the positive bending direction) [57]

<table>
<thead>
<tr>
<th>Specimen geometry (mm)</th>
<th>Concrete properties (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness ($T$)</td>
<td>350</td>
</tr>
<tr>
<td>Width ($B$)</td>
<td>500</td>
</tr>
<tr>
<td>Contact height of joint ($t_j$)</td>
<td>158</td>
</tr>
</tbody>
</table>
Figure 8: Comparison of numerical and experimental results of test specimens (bolt in positive bending direction)

5. Study Case: Metro Line 7 of Tehran (Iran)

5.1. Geometry and material properties definition

In order to evaluate the suitability of using a HSFRC mix for the design and construction of a real tunnel, the line 7 urban metro tunnel of Tehran (Iran) is selected [58]. Figure 9 presents the tunnel section and surrounding soil profile (31 m of thickness with the constitutive properties presented in Table 8 [59]) along with the segmental lining. This tunnel lining has an internal diameter of 9.16 m with a 6+1 key segmental configuration (1.50 of width and 0.35 of thickness). Steel bolts (SS400 steel type with a characteristic yielding strength $f_{y/k} = 400$ Mpa) are used for connecting the segmental joints. The concrete reinforcement configuration of the segment is gathered in Figure 10. The volume fraction of the rebar in the segment is about
1.75%. The mechanical behavior of the steel rebar was modeled using a bilinear diagram (elastic perfect-plastic behavior) with characteristic tensile strength ($f_y$), and elastic modulus ($E$) values equal to 400 MPa and 210 GPa, respectively. The concrete was modeled based on the properties for the plain concrete mix, mix 7, with $f'_{c} = 40$ MPa and $E = 33320$ MPa.

### Table 8: Geotechnical properties of the soil profile

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Cohesion (Kpa)</th>
<th>Internal Friction Angle (degrees)</th>
<th>Elastic Modulus (Mpa)</th>
<th>Poisson Ratio</th>
<th>Dry Specific Weight (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ET-1</td>
<td>31</td>
<td>28</td>
<td>35</td>
<td>0.35</td>
<td>1.70</td>
</tr>
<tr>
<td>ET-2</td>
<td>15</td>
<td>33</td>
<td>75</td>
<td>0.30</td>
<td>1.84</td>
</tr>
<tr>
<td>ET-3</td>
<td>30</td>
<td>33</td>
<td>50</td>
<td>0.32</td>
<td>1.90</td>
</tr>
</tbody>
</table>

Figure 9: The segmental tunnel lining and surrounding soil profile
Figure 10: Steel reinforcement configuration for typical segment of the tunnel lining (in mm)

<table>
<thead>
<tr>
<th>Label</th>
<th>No</th>
<th>Diameter (φ mm)</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>16</td>
<td>4024</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>16</td>
<td>4250</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
<td>10</td>
<td>1390</td>
</tr>
</tbody>
</table>

5.2. Moment-Rotation Curves obtained numerically

To obtain the behavior of HSFRC joints, the connecting segments of Metro Line 7 of Tehran (Iran) were modeled using the developed FE model and considering the geometrical properties presented in Figure 7. Properties of the different HSFRC lining materials (refer to section 3) were implemented in the CDP material model [48]. As a basis for comparison, the plain concrete and conventionally (rebar) Reinforced Concrete (RC) cases were investigated too. The results obtained from the model when only the gravity force is active, lead to a normal stress applied at the segmental joints of 10 MPa. The test simulations were performed for only the positive direction of rotation, corresponding to a positive (bolt in tension) bending moment (see Figure 7). The resulting moment-rotation curve of the segmental connection was derived by recording bending moment and rotation values of the connection ends.
The moment-rotation curves (Figure 11) of the segmental joints (Figure 7) are comprised of linear and nonlinear behavior branches. In the linear range, the different initial (elastic) stiffness values for the different materials is due to the difference in their elastic moduli. In the nonlinear range, the SFRC mixes exhibit higher ultimate moment capacity than the plain (20-35%) and RC (9-23%) cases. The RC joint behavior approximately follows that of the 0.5Mac case. In other words, the joint behavior in a segment with 0.5% macro steel fiber volume content, produces the same flexural capacity of a conventional RC segment with about 1.75% rebar volume ratio. This observation implies the considerable economic efficiency of SFRC over RC for segmented lined tunnels in terms of the steel amount used, with respect to segmental joint behavior. Among the SFRC composites, the non-hybrid 0.5Mic and 0.5Mac composites display the best and worst flexural performance, respectively. The hybrid composites exhibit a relatively similar flexural behavior, both in the linear and nonlinear phases. In terms of economic feasibility, a 0.5% macro steel fiber volume content in a segmented lining is more efficient and also has more favorable segmental joint behavior than plain, RC or other SFRC mixes, despite their higher steel fiber ratios in total (e.g. the hybrid SFRC mixes with a total steel fiber volume content of 0.8%).

From the moment-rotation curves, the elastic rotational stiffness \(K_{\theta e}\) are obtained (initial tangential stiffness of the moment-rotation curve) and the yield moment of the segmental joints \(M_y\) are estimated by applying the methodology of the idealized force-displacement curve procedure [60]. The yield rotation \(\theta_y\), which determines the point of transition from linear to nonlinear behavior in the segmental joint, is then defined as the ratio of the yield moment to the elastic rotational stiffness, i.e. \(M_y / K_{\theta e}\). The nonlinear branch is approximated with a one-degree polynomial (linear approximation) resulting in a representative value of stiffness for the
nonlinear softening branch ($K_{\theta 2}$). This procedure leads to a bi-linear approximation of the moment-rotation curves (Figure 12).

The attained absolute values of $M_y$, $K_{\theta e}$ and $K_{\theta 2}$, along with normalized values to Plain concrete results (in parentheses), for the different reinforcing concrete alternatives studied herein are gathered in Table 9. It must be emphasized that the SFRC configurations show higher yielding moments than the plain concrete case, from around 16 to 40 %. All the obtained parameters for the RC case has relatively close values to the 0.5Mac case, falling within a maximum 6% margin from each other. The non-hybrid 0.5Mic and 0.5Mac composites display the highest and lowest values of $M_y$, respectively. The same trend is followed in the stiffness parameters, $K_{\theta e}$ and $K_{\theta 2}$. The SFRC composites display higher values of $\theta_y$, from 7 to 24 %, than plain concrete. The hybrid 0.3Mac0.3Mic composite shows the highest yield rotation. Values of $\theta_y$ for the hybrid composites are generally higher than the non-hybrid cases.

![Figure 11: Moment-Rotation curves of segmental joints for different lining material](image)

Figure 11: Moment-Rotation curves of segmental joints for different lining material
Figure 12: Bi-linear approximation of the moment-rotation curve of segmental joint (e.g. 5Mac segment material)

Table 9: Absolute and normalized (to Plain concrete) values of \( M_y \), \( K_{\theta e} \) and \( K_{\theta 2} \) for segmental joints

<table>
<thead>
<tr>
<th>Segment material</th>
<th>( M_y ) (kN.m)</th>
<th>( K_{\theta e} ) (kN.m/rad)</th>
<th>( \theta_y ) (rad)</th>
<th>( K_{\theta 2} ) (kN.m/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5Mac</td>
<td>233(1.16)</td>
<td>64871(1.03)</td>
<td>359(1.13)</td>
<td>14385(1.50)</td>
</tr>
<tr>
<td>0.5Mic</td>
<td>279(1.39)</td>
<td>68902(1.09)</td>
<td>405(1.27)</td>
<td>28097(2.92)</td>
</tr>
<tr>
<td>0.3Mac0.3Mic</td>
<td>262(1.30)</td>
<td>65877(1.04)</td>
<td>397(1.25)</td>
<td>15443(1.61)</td>
</tr>
<tr>
<td>0.5Mac0.5Mic</td>
<td>273(1.36)</td>
<td>66752(1.06)</td>
<td>410(1.29)</td>
<td>21337(2.22)</td>
</tr>
<tr>
<td>0.5Mac0.3Mic</td>
<td>254(1.26)</td>
<td>66457(1.05)</td>
<td>382(1.20)</td>
<td>21437(2.23)</td>
</tr>
<tr>
<td>0.3Mac0.5Mic</td>
<td>262(1.30)</td>
<td>65426(1.03)</td>
<td>401(1.26)</td>
<td>15469(1.61)</td>
</tr>
<tr>
<td>RC</td>
<td>224(1.11)</td>
<td>64555(1.02)</td>
<td>347(1.09)</td>
<td>13527(1.41)</td>
</tr>
<tr>
<td>Plain</td>
<td>201</td>
<td>63255</td>
<td>318</td>
<td>9615</td>
</tr>
</tbody>
</table>

5. Seismic Performance of HSFRC Segmental Joints

5.1. Overview

In the previous section, the numerical \( M – \theta \) relationships for plain, RC and fiber reinforced concrete segmental joints were obtained for the line 7 urban metro tunnel of Tehran (Iran). In this section, the seismic load is included into the analysis aiming at determining the structural response of the segmental joints.
A 2D numerical model of the line 7 Tehran metro soil-structure system (Figure 9) is built in ABAQUS, considering lining materials and corresponding segmental joint behaviors of the SFRC composites. A representative set of earthquake records is chosen and one-dimensional equivalent linear Site Response Analysis (SRA) is conducted to obtain the imposed ground displacements. Quasi-static transversal analysis is used to evaluate the seismic response of the tunnel system.

5.2. Seismic induced ground displacements

Seven ground motions (Table 10) were selected as input motions in outcrop conditions for SRA simulating different real earthquake scenarios. The earthquake records were chosen to be compatible, i.e. with the geo-seismic specifications of the tunnel site. To calculate the tunnels response to varying levels of seismic intensity, each record is scaled to four levels of Peak Ground Acceleration (PGA): 0.2g, 0.5g, 0.7g, and 1.0g.

Table 10: Specifications of the ground motion set

<table>
<thead>
<tr>
<th>No</th>
<th>Date</th>
<th>Earthquake name</th>
<th>Record name</th>
<th>Magnitude (Ms)</th>
<th>Station number</th>
<th>PGA (g)</th>
<th>dt (sec)</th>
<th>T_{max} (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>01/17/94</td>
<td>Northridge</td>
<td>NRORR360</td>
<td>6.8</td>
<td>24,278</td>
<td>0.51</td>
<td>0.010</td>
<td>39.970</td>
</tr>
<tr>
<td>2</td>
<td>06/28/92</td>
<td>Landers</td>
<td>LADSP000</td>
<td>7.5</td>
<td>12,149</td>
<td>0.17</td>
<td>0.010</td>
<td>49.970</td>
</tr>
<tr>
<td>3</td>
<td>04/24/84</td>
<td>Morgan Hill</td>
<td>MHG06090</td>
<td>6.1</td>
<td>57,383</td>
<td>0.29</td>
<td>0.005</td>
<td>29.969</td>
</tr>
<tr>
<td>4</td>
<td>10/17/89</td>
<td>Loma Prieta</td>
<td>LPAND270</td>
<td>7.1</td>
<td>1,652</td>
<td>0.24</td>
<td>0.005</td>
<td>39.595</td>
</tr>
<tr>
<td>5</td>
<td>10/17/89</td>
<td>Loma Prieta</td>
<td>LPGIL067</td>
<td>7.1</td>
<td>47,006</td>
<td>0.36</td>
<td>0.005</td>
<td>39.945</td>
</tr>
<tr>
<td>6</td>
<td>10/17/89</td>
<td>Loma Prieta</td>
<td>LPLOB000</td>
<td>7.1</td>
<td>58,135</td>
<td>0.44</td>
<td>0.005</td>
<td>39.940</td>
</tr>
<tr>
<td>7</td>
<td>10/17/89</td>
<td>Loma Prieta</td>
<td>LPSTG000</td>
<td>7.1</td>
<td>58,065</td>
<td>0.50</td>
<td>0.005</td>
<td>39.945</td>
</tr>
</tbody>
</table>

The EERA [61] package was used for the linear 1D site response analysis of the soil profile in free field condition, using the above ground motion set. To perform SRA, shear modulus (G) and damping ratio (D) variations with shear strain (γ) of each soil layer was introduced. Moreover, variation of shear wave velocity ($V_{s30}$) with depth of the soil profile is also taken into account (see Figure 13). For the seismic bedrock, the curves provided by Schnabel et al. [62]...
and a shear wave velocity equal to 760 m/s was used. For each input earthquake record, variation of the maximum shear strain with depth is calculated from SRA. These plots were then used to obtain the induced cumulative peak ground displacements in each soil layer and depth of the soil profile.

(a)  

![Graph showing dynamic properties of the soil profile: (a) G/Gmax vs. γ & Damping vs. γ curves; (b) shear wave velocity vs. depth](image)

Figure 13: Dynamic properties of the soil profile: (a) G/Gmax vs. γ & Damping vs. γ curves; (b) shear wave velocity vs. depth

5.3. Numerical Modeling & Seismic Analysis

A 2D numerical model of the soil-structure section is built in ABAQUS in a plain strain condition (Figure 14). Both the tunnel lining and surrounding soil are modeled using solid continuum elements of type linear quadrilateral (CPE4R)[48]. The lining segment and soil layers were meshed to 0.2m and 1.0m size elements, respectively. To simulate the segmental joints, the obtained moment-rotation curves(Figure 11) were assigned to connector elements[48]. The soil-structure interaction is effectively considered in the model by defining the tunnel lining as an embedded region[48] within the soil medium. The Mohr-Coulomb yield criterion was assigned
to the soil elements. For the tunnel lining, properties of the considered SFRC concrete mixes were incorporated using the damaged plasticity material model [48]. As a basis for comparison, plain and RC cases was also used as the lining material. The CDP model[48] was also used to model the concrete in the plain and RC cases \(f'_c = 40 \text{ MPa}, E = 33320 \text{ MPa}\). In the RC case, the mechanical behavior of steel rebar was modeled using a bilinear diagram (elastic perfect-plastic behavior) with characteristic tensile strength, and elastic modulus values equal to 400 MPa and 210 GPa, respectively.

To avoid boundary effects, the lateral extent of each side boundary (horizontal distance from the tunnel center to each side boundary) is properly selected from a sensitivity analysis, approximately \(2.5H\) \((H = \text{average overburden depth, measured from ground surface to tunnel center})\), equal to 40 m. The lateral boundaries of the soil-structure model are free to move in the horizontal direction and restrained from vertical movement. The bottom boundary of the model is fixed in both directions of translational displacement.

![Figure 14: Finite Element Model of the soil-structure system](image)
Prior to seismic deformation loading, the steady state of the tunnel lining i.e. after tunnel excavation and lining installation should be initially established. For this purpose, the confinement convergence method was followed which involves assigning a relaxation factor to reach an equilibrium state. To ensure realistic results and validate the soil-structure model, the value of this factor was determined so that the numerically obtained values for ground surface settlement approximate those from actual instrumental readings at the location of the tunnel section of interest. After excavation, segment installation, and contact grouting of the tunnel extrados, the maximum ground surface settlement at the point of the tunnel section of interest was recorded equal to 41.74 mm [58]. Following this approach, a value equal to 0.37 was used as the relaxation factor, resulting in a ground surface settlement value of 39.78 mm from the numerical model.

The peak displacement profiles, calculated from SRA results, are applied on the lateral boundaries of the soil-structure model in a quasi-static manner to obtain the tunnels’ ovaling response. The analysis was repeated for variations of the tunnel lining material, and thus segment joint behavior, earthquake record, and corresponding scale factor, resulting in a total number of 224 analysis cases.

5.4. Results & Discussion

The structural performance of the SFRC joints for the analyzed segmental tunnel of the Tehran line 7 metro, subjected to seismic loads, is analyzed in terms of both the flexural and the rotation capacity. In this regard, both the bending moment ($R_{\theta,j}$) and rotation ($\mu_{\theta,j}$) demand ratios of the segmental joint (Equations 2 and 3, respectively) are defined:

$$R_{\theta,j} = \frac{M_{D,j}}{M_y}$$  \hspace{1cm} \text{(2)}
Where $M_{D,j}$ and $\Theta_{D,j}$ are the maximum bending moment and joint rotation demands (among all segmental joints in the analyzed tunnel ring of Figure 14) in each seismic analysis case (obtained from each quasi-static analysis case, as detailed in section 5.3), respectively; and $M_y$ and $\Theta_y$ are the yield bending moment and yield rotation of the segmental joint, respectively (section 4.4). Both $R_{\Theta,j}$ and $\mu_{\Theta,j}$ indicate the level of demand provoked by the seismic load on the segmental joint compared to its yield capacity. Therefore, higher values of these parameters imply less favorable structural performance, or in other words, greater seismic vulnerability of the segmental joint.

$R_{\Theta,j}$ – IM and $\mu_{\Theta,j}$ - IM relationships, IM being the seismic Intensity Measure (PGA/g), are presented in Figure 15 and 16, respectively, for the different constitutive joint cement-based materials analyzed.
Comparing the flexural response curves ($R_{\theta,j}$ and $\mu_{\theta,j}$ vs. $IM$ curves), the reinforced material (HSFRC and RC) segmental joints show less vulnerability than the plain concrete case at all levels of seismic intensity. This better performance increases with the seismic intensity, reaching up to around 25% (disregarding mix 1 here) at approximately PGA=1.0g. Furthermore, HSFRC joints generally show a better seismic performance than those made of RC. Flexural response curves for the non-hybrid SFRC mix 1, nearly match that of conventional reinforcement at all PGA values, indicating similar seismic performance. This SFRC mix showed to be the most vulnerable among the SFRC joints. This phenomenon is anticipated due to the similar M-\(\theta\) behavior curves of this SFRC mix and RC (Figure 11).
By observing the flexural response curves of the HSFRC mixes for seismic intensities up to around 0.3g, it can be noted that the seismic vulnerability of all the HSFRC mixes, apart from mix1, are quite close to each other. For PGA>0.3g, the difference between the HSFRC joint performances grows. In this region, the HSFRC mix 3 segmental joint displays the best flexural ductility performance among other materials. On the other hand, the segmental joint with the highest flexural strength (by comparing the $R_{\Theta,j}$ vs. $IM$ curves) is HSFRC mix 6.

These observations show the better performance of the hybrid SFRC mixes over the non-hybrid ones (mixes 1 and 2). The advantageous behavior of hybrid SFRC composites can be due to the favorable dual action resulting from the presence of both micro and macro steel fibers, as reported by previous researchers[63, 64]. The bridging mechanism in micro fibers, initiates at an earlier stage of seismic intensity and by propagating the damage into multiple micro-cracks, prevents the formation of one major crack. As the seismic load intensifies, the minor cracks transform into one major crack. At this point, the macro fibers are fully activated and counteract the expansion of the formed crack. This resistance comes from their high ductility and toughness characteristics due to their hooked end shape and greater length. Moreover, since both fibers have high tensile strengths (>1000 MPa), both exhibit high ductile behavior, thus leading to a gradual fiber pullout mechanism rather than a sudden brittle failure[65].

Based on the general shape of the obtained curves, 2nd degree polynomials(Equation 4) provide suitable equations to approximate (fit) the flexural response curves of Figure 15 and 16:

$$F_2(PGA) = a_2 \times (PGA)^2 + a_1 \times (PGA) + a_0$$  \hspace{1cm} (4)

Where $F_2(PGA)$ is the flexural response parameter, $R_{\Theta,j}$ or $\mu_{\Theta,j}$ (dependent variable), PGA is the independent variable and $a_i$, $i = 0, 1$ and 2 are the polynomial coefficients. Using regression
analysis, the coefficients are calculated for the $R_{\theta.j}$ or $\mu_{\theta.j}$ curves of the HSFRC joints and tabulated in Table 11 and Table 12, respectively.

### Table 11: Coefficients of 2nd degree polynomial fit for $R_{\theta.j}$ vs. IM curves

<table>
<thead>
<tr>
<th>Tunnel Lining Material</th>
<th>$a_2$</th>
<th>$a_1$</th>
<th>$a_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix 1</td>
<td>-0.7457</td>
<td>2.6960</td>
<td>0.0838</td>
</tr>
<tr>
<td>Mix 2</td>
<td>-1.3619</td>
<td>2.9051</td>
<td>0.0347</td>
</tr>
<tr>
<td>Mix 3</td>
<td>-1.4321</td>
<td>3.0807</td>
<td>0.0348</td>
</tr>
<tr>
<td>Mix 4</td>
<td>-1.3396</td>
<td>2.9325</td>
<td>0.0317</td>
</tr>
<tr>
<td>Mix 5</td>
<td>-1.3903</td>
<td>3.0651</td>
<td>0.0331</td>
</tr>
<tr>
<td>Mix 6</td>
<td>-1.5828</td>
<td>3.0278</td>
<td>0.0331</td>
</tr>
</tbody>
</table>

### Table 12: Coefficients of 2nd degree polynomial fit for $\mu_{\theta.j}$ vs. IM curves

<table>
<thead>
<tr>
<th>Tunnel Lining Material</th>
<th>$a_2$</th>
<th>$a_1$</th>
<th>$a_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix 1</td>
<td>-0.0079</td>
<td>0.0286</td>
<td>0.0009</td>
</tr>
<tr>
<td>Mix 2</td>
<td>-0.0153</td>
<td>0.0327</td>
<td>0.0004</td>
</tr>
<tr>
<td>Mix 3</td>
<td>-0.0148</td>
<td>0.0301</td>
<td>0.0006</td>
</tr>
<tr>
<td>Mix 4</td>
<td>-0.0146</td>
<td>0.0319</td>
<td>0.0003</td>
</tr>
<tr>
<td>Mix 5</td>
<td>-0.0150</td>
<td>0.0330</td>
<td>0.0004</td>
</tr>
<tr>
<td>Mix 6</td>
<td>-0.0172</td>
<td>0.0330</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

In order to extend the conclusions of the obtained flexural response curves, general relationships are determined to assess the flexural response of any HSFRC segmental joint. In this regard, general relations are established between the flexural response parameters and definitive parameters of a HSFRC composite. In this regard, the residual flexural strength parameters, $f_{r4}$ and $f_{r2}$, of HSFRC composites are related to the flexural response parameters, $R_{\theta.j}$ and $\mu_{\theta.j}$, resulting in $R_{\theta.j}(PGA, f_{r4}, f_{r2})$ and $\mu_{\theta.j}(PGA, f_{r4}, f_{r2})$ relationships. To this end, the relationship between $f_{r4}$ and $f_{r2}$ with the polynomial coefficients of the polynomial fitting curves, i.e. $a_i$, $i=0, 1$ and 2 (Equation 4) is found in the following general form (Equation 5):
\[ a_i = g_i(f_{r4}, f_{r2}) = b_0 + b_1 \times f_{r4} + b_2 \times f_{r2} + b_3 \times (f_{r4})^2 + b_4 \times (f_{r4} \times f_{r2}) + b_5 \times (f_{r2})^2; \quad i = 0, 1, 2 \]  

The polynomial coefficients, \( b_i \), are calculated from regression analysis for each of the flexural response parameters, \( R_{\theta, j} \) and \( \mu_{\theta, j} \) and are presented in Table 13 and Table 14, respectively. The \( b_i \) values are substituted in Equation 5 to obtain \( a_i(f_{r4}, f_{r2}) \), which are themselves substituted in Equation 4 to obtain general relations for the flexural response parameters of HSFRC segmental joints (Equation 6):

\[ R_{\theta, j}(P{GA}, f_{r4}, f_{r2}) = \mu_{\theta, j}(P{GA}, f_{r4}, f_{r2}) = g_2(f_{r4}, f_{r2}) \times (P{GA})^2 + g_1(f_{r4}, f_{r2}) \times (P{GA}) + g_0(f_{r4}, f_{r2}) \]  

<table>
<thead>
<tr>
<th>( g_i(f_{r4}, f_{r2}) )</th>
<th>( R_{0, j} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b0 )</td>
<td>( b1 )</td>
</tr>
<tr>
<td>g2</td>
<td>-0.8318</td>
</tr>
<tr>
<td>g1</td>
<td>3.7490</td>
</tr>
<tr>
<td>g0</td>
<td>-0.1898</td>
</tr>
</tbody>
</table>

Table 13: Coefficients of \( g_i(f_{r4}, f_{r2}), i=0,1 \) and 2 for \( R_{0, j} \)

<table>
<thead>
<tr>
<th>( g_i(f_{r4}, f_{r2}) )</th>
<th>( \mu_{0, j} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b0 )</td>
<td>( b1 )</td>
</tr>
<tr>
<td>g2</td>
<td>-0.0118</td>
</tr>
<tr>
<td>g1</td>
<td>0.0474</td>
</tr>
<tr>
<td>g0</td>
<td>-0.0026</td>
</tr>
</tbody>
</table>

Table 14: Coefficients of \( g_i(f_{r4}, f_{r2}), i=0,1 \) and 2 for \( \mu_{0, j} \)

6. Conclusions

The flexural behavior of segmental joints in precast segmental linings, constructed of hybrid and non-hybrid Steel Fiber Reinforced Concrete (SFRC) under earthquake loading is
investigated. The focus of this paper is to study the effects of different composites of SFRC, as the tunnel’s lining material, on the seismic performance of segmental joints. A standard experimental test set-up for studying the behavior of segmental joints is numerically simulated to obtain the moment-rotation behavior of SFRC segmental joints. The obtained behavior curves are assigned to the joints in a case study segmental tunnel and seismic analyses are carried out. Based on the conditions and assumptions specified in this research (joint type, normal force, etc.), the main results can be summarized as follows:

- The SFRC segmental joints generally show higher yielding moments \((M_y)\) than the plain concrete case, from around 16 to 40 %. Among the SFRC composites, the non-hybrid 0.5% micro fibers and 0.5% macro fibers composites led to the highest and lowest values of \(M_y\), respectively.

- In the hybrid composites, the addition of micro fibers shows a greater effect than macro fibers in increasing the yielding bending moment and the linear and nonlinear stiffness of segmental joints.

- The SFRC composites generally produced higher values of yield rotation \((\theta_y)\), from 7 to 24 %, than plain concrete.

- By comparing the flexural response curves \((R_{\theta, j} \text{ and } \mu_{\theta, j} \text{ vs. } IM \text{ curves})\), the SFRC and RC segmental joints show less vulnerability than those made of PC at all levels of seismic intensity. This better performance increases with seismic intensity.

- Flexural performance of the non-hybrid SFRC mix 1 (0.5% macro fibers), nearly match that of RC at all PGA values, indicating similar seismic performance. This macro-SFRC mix showed to be the most vulnerable among the SFRC joints.
• For PGA<0.3g, seismic vulnerability of all the SFRC mixes, apart from mix1 (0.5% macro fibers), are quite close to each other. For PGA>0.3g, the Hybrid SFRC mix 3 (0.3% macro fibers and 0.3% micro fibers) segmental joint, displays the best flexural ductility performance among other materials. On the other hand, the segmental joint with the highest flexural strength is displayed by the Hybrid SFRC mix 6, containing 0.3% and 0.5% volume content of macro and micro steel fibers respectively.

In summary, the application of SFRC composites as the lining material in segmental tunnels, results in generally better flexural performance of the segmental joints over plain concrete. Micro steel fibers show a more positive influence than macro fibers on the moment-rotation behavior, in terms of yielding bending moment, linear and nonlinear stiffness of segmental joints. From the seismic analyses of SFRC segmental tunnels, steel fibers show to improve the seismic performance of segmental joints over RC and plain concrete in terms of both flexural strength and ductility. This study helps better understand and compare the effect of different SFRC composites in the seismic response of segmental joints in SFRC segmental tunnels, providing better insight for the design of such tunnels, especially in seismic zones. The positive outcomes derived from this study have been reported to the Iranian authorities and tunnel designer. In this regard, it must be emphasized that the tunnels designers are already considering the FRC as a structural material in the tunnels to be constructed in Iran.

As pointed out by limited past research [57], the presence of the connecting bolt has insignificant effect on the flexural behavior in segmental joints. Therefore, the proposed relations in this research are potentially applicable for the design of HSFRC segmental joints, regardless of the specifications of the connecting bolts. Nevertheless, further research on various aspects of
the connecting bolt (material, distribution, geometry, etc.) is required to better understand its influence on the performance of joints in precast segmented lining tunnels.

References


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