Seismic fragility curves for vulnerability assessment of steel fiber reinforced concrete segmental tunnel linings

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Abstract

In recent years, application of fibers as a replacement for conventional reinforcement in segmented lined tunnels has gained great interest due to resulting cost and time savings. In seismically active regions, the seismic vulnerability of underground tunnels is of great concern. The aim of this paper is to investigate the effects of different composites of Steel FRC (SFRC), as the tunnel's lining material, on its seismic vulnerability, compared to each other and to that of unreinforced and conventionally reinforced concrete cases, employing analytical fragility curves. Results show that steel fibers, especially micro size fibers and for higher states of damage, display better seismic performance over conventional steel rebar in reinforced concrete linings. For best performance, a hybrid SFRC mix containing both micro and macro fibers, with a higher content of micro fibers over macro ones, is a technically preferable option for the design of segmental lining tunnels in seismic zones.

KEYWORDS: SFRC; Segmental Tunnel; Seismic Vulnerability; Fragility Curve

1. Introduction

In developing countries, underground railway networks, commonly known as ‘subways’, are rapidly growing in urban regions. Tunnels are the key elements in subway networks and considered as important national infrastructures and therefore, their requisition for public transport is of great interest. Tunneling projects consume a considerable amount of national budgets, signifying the necessity for more research on cost reduction and productivity enhancement methods in this regard. Using fibers in the concrete mix has been introduced as a potential solution to reduce costs and save times in tunneling projects [1-3].

Fibers were employed to enhance the structural performance of hardened concrete under tensile and flexural actions, commonly known as Fiber Reinforced Concrete (FRC). Today, fiber reinforced concrete is widely used as a reliable design option in structures such as industrial

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pavement slabs, shotcrete of tunnels and in the precast industry [4]. In the past few years, application of fibers as a replacement of traditional reinforcement for different structures under bending and shear forces, namely segmental linings of tunnel, has gained great interest [5-7]. Fibers of different materials (Steel, Polypropylene, Glass, carbon, etc.) and geometries (length, thickness) are used in fiber reinforced concrete. In recent years, research on simultaneously taking advantage of the beneficial effects of different types of fibers, by incorporating the different fibers into a single concrete mix, has led to the development of new cementitious materials, called hybrid FRCs [8, 9]. The enhancement in engineering properties of hybrid composites, composed of two different fiber types has been shown in previous research [10-13]. In one type of hybrid FRC, different sizes of a single type (material) of fibers are combined. In any case, the type and specifications of a fiber concrete mix is chosen to meet the required conditions [14]. In seismically active regions, the performance and vulnerability of infrastructure such as underground tunnels, under earthquake loads, is of great concern. Despite being less vulnerable than above ground structures, minor to extreme incidents of damage to underground tunnels has been reported in past earthquakes [15-17]. Therefore, for the reliable implementation of FRC in tunnel linings in such regions, extensive research on the seismic performance and vulnerability of such tunnels is vital.

In assessing the global seismic performance and behavior of structural elements, fragility curves play an important role. These curves plot the probability of exceeding a defined damage state at a certain value of seismic intensity. Fragility curves are a robust tool for evaluating the seismic vulnerability of structures. Various methods have been used to develop seismic fragility curves, including expert-based/engineering judgement, empirical, and analytical methods [18, 19]. So far, the fragility curves developed for underground structures, including tunnels, are generally based on engineering judgment, or obtained using tunnel damage data from past seismic events [20-22], making its applicability limited. Fragility curves generated from analytical approaches can be readily applied to different structural configurations and seismic zones. These approaches have been mainly employed so far to develop fragility curves for building and bridge structures, barely implemented for segmented lining tunnels.

The aim of this paper is to evaluate the seismic performance of TBM constructed tunnels with Steel FRC (SFRC) precast segmental linings, using analytical fragility curves. The focus of this paper is to investigate the effects of different composites of SFRC, as the tunnel's lining material, on its seismic vulnerability, compared to each other and to that of unreinforced (plain), and conventionally Reinforced Concrete (RC) cases.

In this study, a comprehensive experimental program is initially conducted to acquire the mechanical properties of different SFRC mixes, primarily the post-crack tensile behavior. The mechanical properties of the SFRC composites are necessary in the numerical phase of the study to reliably model the behavior of the SFRC tunnel lining and calculate its response under seismic loading. For this purpose, the basic engineering properties of the SFRC composites are determined using compressive and splitting tensile strengths, and also flexural properties from 3-point bending tests (3PBTs).

Six concrete mixes, comprised of different combinations, using 0.3% and 0.5% steel fiber volume content of micro and macro size, is investigated. Hybrid Steel FRC mixes, i.e. combinations of micro and macro size steel fibers in a single concrete mix, are also investigated at a comprehensive level. As a result, stress-strain relationships are developed and validated for the SFRC mixes, using the RILEM TC 162-TDF [23] guidelines. After completing the experimental phase, a procedure similar to previous analytical fragility studies [24, 25] is followed.

Underground tunnels are usually considered as non-redundant key elements in a lifeline network (transportation network, water supply network, etc.) which consume a considerable amount of time and capital for its construction. Therefore, any disruption in a tunnel's operation due to a seismic event, can potentially lead to catastrophic socio-economic losses and human fatalities. In this regard, application of new methods and materials for tunnel construction in seismically active regions should consider seismic safety and vulnerability aspects of tunnels. This study investigates the seismic vulnerability of SFRC segmental tunnels using fragility curves, providing a tool for city planners and disaster management authorities to evaluate the potential seismic vulnerability and damage of such tunnels against the economic savings of SFRC usage. Moreover, this paper helps better understand and compare the effect of different SFRC composites in segmental tunnel linings in the seismic performance of SFRC segmental tunnels, providing insight for the design of such tunnels in seismic zones.

2. Experimental program

Application of steel fibers has shown to saliently improve the tensile behavior of concrete composites by creating a crack-bridge mechanism to control crack width [26]. This advantageous behavior of SFRC over conventional rebar has resulted in more economic designs of structural elements, generally designed in accordance with main standards [27]. In large structural elements, such as tunnel lining segments, the usage of fibers in full/partial replacement of reinforcement bars can provide significant cost reduction and increase in production speed. Moreover, the proper distribution of fibers throughout the tunnel lining segment [28] can significantly reduce spalling effects and detrimental consequences due to extreme loads from TBM jacks [29, 30], fire exposure [31, 32] or explosion [31, 33].

For the reliable implementation of steel fibers in a concrete mix, the main standards require experimental tests to determine the mechanical properties of SFRC. The obtained engineering properties are used as a basis to obtain the constitutive relationships of SFRC for describing the response under normal stresses. In this research, the mechanical properties of the SFRC composites are used in the numerical phase to reliably model the behavior of the SFRC tunnel lining and obtain its seismic response. In this regard, an experimental campaign is conducted using various mixes of micro and macro steel fibers, comprised of compressive, splitting tensile and 3-point bending tests (3PBTs).

Properties of the two types of steel fibers investigated are as follows:

Macro fibers: Hooked ends, 50 mm length with 0.8 mm diameter.
 Micro fibers: Flattened ends, 13 mm length with 0.17mm diameter.

The fibers used in this research are chosen due to their availability and current usage in fiber reinforced concrete applications in Iranian construction projects. In Table 1 and Table 2, properties of the fibers and corresponding volume content in each concrete mix are shown, respectively. The used fibers are displayed in Figure 1. A plain concrete mix (mix No. 7) is also produced for comparison purposes. In Table 3, the materials and corresponding proportions used for the concrete mixes is given.

Table 1: Properties of fibers

Fiber type	Geometry	Length (mm)	Diameter (mm)	Aspect Ratio	Tensile strength (MPa)	Elasticity module (GPa)
Macro steel fiber	Hooked	50	0.80	62.5	1169	210
Micro steel fiber	Smooth	13	0.17	76.5	2100	210

Table 2: Fiber volume fraction in concrete mixes

Mix no.	1	2	3	4	5	6	7
Macro steel fiber ratio (%)	0.5	-	0.3	0.5	0.5	0.3	-
Micro steel fiber ratio (%)	-	0.5	0.3	0.5	0.3	0.5	-

Figure 1: Micro and macro size steel fiber

Table 3: The materials used for all the concrete mixes

Portland Cement type II (kg/m ³)	Natural sand (kg/m ³)	Gravel (kg/m ³)	Water-Cement ratio
400	1172	632	0.375

The mechanical concrete characterization was carried out by means of: (1) two 150 mm cubes for compressive testing at 28-days; (2) two 150×300 mm cylinders for the splitting tensile strength tests and (3) three 150×150×550 mm beams for the 3-PBTs over a span of 500 mm (Figure 2). Cube and cylinder specimens were tested according to the ASTM standard [34] while beam specimens according to RILEM TC 162-TDF [23].

Figure 2: Specimens of the experimental tests

Mean compressive (f_{cm}) and splitting tensile ($f_{ctm,i}$) strength values of FRC mixtures at 28 days along with slump values are listed in Table 4. The $f_{cm, 28}$ of plain concrete (mix 7) was about 40 MPa. The workability of fresh plain concrete, measured by the slump test, before addition of fibers was almost the same in all mixtures (in the range of 10-15cm) but reduced significantly after addition of fibers, i.e. from 40 to 60% in different mixtures. Nevertheless, this reduction does not imply a problem in terms of production since high energetic vibration is induced in to the molds during the casting.

From the results gathered in Table 4, it can also be noticed, that both micro and macro size steel fibers increase the compressive and tensile strength of concrete mixes compared to plain concrete, up to around 38 and 75%, respectively. In this regard, the presence and content value of micro size fibers has been more effective in increasing the compressive and tensile strengths of the SFRC composites. These beneficial effects are more obvious for hybrid composites.

Table 4: Compression and Tensile Strength values

Mix	Macro steel fiber ratio (%)	Micro steel fiber ratio (%)	Slump (cm)	Compression strength (MPa)	Tensile strength (MPa)
1	0.5	-	7-9	46.44	4.23
2	-	0.5	5-7	68.83	5.21
3	0.3	0.3	5-7	58.25	4.66
4	0.5	0.5	4-6	60.83	6.69
5	0.5	0.3	5-7	54.02	5.49

6	0.3	0.5	5-7	65.61	5.04
7	-	-	10-15	41.1	3.4

In the 3-PBT method, the flexural tensile behavior of SFRC is determined by testing a simply supported notched beam under three-point loading configuration. In this testing method, the tensile performance of SFRC is obtained in terms of the area under the load-deflection curve or by the load bearing capacity at a certain deflection or crack mouth opening displacement (CMOD) [23]. In order to calculate flexural tensile residual strength (f_R), it is necessary to measure the deflection. However, if only residual flexural tensile strengths are to be calculated, one can choose between the measurement of deflection and/or CMOD.

According to RILEM guidelines, the beams are notched using wet sawing. Each beam is turned 90 degree from the casting surface and the notch is then sawn through the width of the beam at midspan (see Figure 3).

In this study, only deflection at midspan was recorded during the 3-PBTs. Load–deflection curves were determined by loading the 28-day prism specimens. Both load and deflection were measured with a servo controlled hydraulic machine and a Linear Variable Displacement Transducer (LVDT), respectively (Figure 3). Displacement control is the recommended loading strategy for determining the complete load deflection relationship, i.e. including the post peak region. The RILEM TC 162-TDF recommends a testing machine be operated in a way that the measured deflection of the specimen at midspan increases at a constant rate of 0.2 mm/min until the specified final deflection is reached. During testing, load and deflection values at midspan are recorded continuously [23].

Figure 3: Setup for 3PBTs

Mean load–deflection relationships or curves, based on the LVDT’s recorded values, for all the prism specimens are illustrated in Figure 4. Average values of maximum and residual flexural strength values [35] for each SFRC mix is presented in Table 5. These load–deflection curves are used to determine stress-strain relationships for the concrete mixes. In the following figures throughout the paper, the “Mic.” and “Mac.” abbreviations stand for Micro and Macro fiber, respectively.

Figure 4: Load deflection curve for all concrete mixes

Table 5: Average values for f_{max} , f_{r1} , f_{r2} , f_{r3} , f_{r4} (Mpa) and CV (%)

Mix No.	Mix. Abbrev.	f_{max}/CV	f_{r1}/CV	f_{r2}/CV	f_{r3}/CV	f_{r4}/CV
1	0.5Mac	4.2/3.2	3.3/3.5	3.2/8.4	2.5/7.7	2.4/8.1
2	0.5Mic	6.0/4.1	4.0/4.6	5.2/7.2	4.0/11.1	3.0/11.3
3	0.3Mac0.3Mic	5.2/3.7	4.9/4.1	4.2/8.3	2.9/9.1	2.0/7.6
4	0.5Mac0.5Mic	6.1/5.1	5.7/5.9	4.7/12.1	4.2/18.8	3.4/15.2
5	0.5Mac0.3Mic	5.3/2.8	3.9/3.3	3.0/6.5	2.5/8.1	2.0/9.2
6	0.3Mac0.5Mic	5.4/4.7	5.0/4.4	4.3/8.9	3.3/7.7	2.9/8.7

By comparing mixes 1 and 2, for equal amounts of micro and macro sized fibers, the former behaves better in terms of Serviceability Limit States, SLSs (deflection < 5mm), while macro fibers are more effective for Ultimate Level States, ULSs (deflection > 5mm). It can be noted that 0.3Mac0.5Mic leads to a growing increase in values of f_{ri} (i=1 to 4) compared to

0.3Mac0.3Mic, i.e. from around 2% for f_{r1} , up to about 45% for f_{r4} . This observation also points out the higher influence of micro fibers in improving flexural strength of in the range of SLS deflections.

3. σ - ϵ relationships for SFRC mixes

The RILEM procedure for developing stress-strain relationships for SFRC using 3-point bending test results is employed for the SFRC mixes. Flexural strength and load-deflection curves are obtained in the process. Finally, the resulting stress-strain relationships for the hybrid steel fiber reinforced concrete mixes are validated using numerical modeling.

According to RILEM [23], tension and compression stresses in steel fiber reinforced concrete are characterized based on the stress-strain diagram shown in Figure 5. Refer to RILEM [23] for further details.

Figure 5: Stress-Strain relationship for fiber reinforced concrete

From the load-deflection curves, the stress-strain behavior for each concrete mix can be derived. According to RILEM, tension and compression stresses in steel fiber reinforced concrete are characterized based on the stress-strain diagram, previously shown in Figure 5. The average stress-strain values for the SFRC concrete mixes are reported in Table 6.

Table 6: stress-strain properties of SFRC composites (based on RILEM TC 162-TDF)

Mix No.	Mix Abbrev.	σ_1 (Mpa)	σ_2 (Mpa)	σ_3 (Mpa)	ϵ_1 (‰)	ϵ_2 (‰)	ϵ_3 (‰)
1	0.5Mac	6.31	5.23	2.58	0.192	0.292	25

2	0.5Mic	8.54	7.87	2.96	0.223	0.323	25
3	0.3Mac0.3Mic	7.31	6.74	1.76	0.206	0.306	25
4	0.5Mac0.5Mic	7.73	5.75	3.68	0.212	0.312	25
5	0.5Mac0.3Mic	7.82	5.82	2.44	0.213	0.313	25
6	0.3Mac0.5Mic	7.03	6.03	2.86	0.202	0.302	25

Regarding the significance of hybrid composites in this research, the calculated stress-strain models are validated for the hybrid mixes (mixes 3 to 6). In this regard, numerical models of hybrid SFRC beam specimens are developed and loaded according to the 3PBT protocol. Relevant calculated stress-strain relationships for hybrid SFRC are assigned to the beam model. Resultant load-deflection relationships of the numerical model are then compared to that of the 3-point bending load experiments to verify the calculated stress-strain models.

The beam specimens subject to 3-point loading were modeled in the ABAQUS [36] finite element software, using 3D stress hex elements (Figure 6). Each beam model is adequately meshed in the longitudinal, transverse and thickness directions, respectively. All experimental conditions including beam properties and simple support conditions were simulated in the finite element model. The notch was modeled by cutting through the initial beam element at midspan according to the notch dimensions (15 mm wide, 25 mm deep), then meshing the final model. Between the three crack models available for modeling fiber reinforced concrete, the Concrete Damaged Plasticity (CDP) model [37] was used. This model has the ability to characterize the complete inelastic behavior of concrete, both in tension and compression, including damage characteristics. The obtained constitutive relationships for hybrid SFRC (Table 6) are implemented in the CDP material model and assigned to the beam specimens. The plasticity parameters of the CDP model was set according to Table 7: Plasticity parameters for CDP model (refer to [37] for details on these parameters). To consider loading rate effects, a dynamic solution is employed for analysis.

Figure 6: The finite element model of the beam under 3PBT

Table 7: Plasticity parameters for CDP model

Dilation Angle	Eccentricity	f_{b0} / f_{c0}	K	Viscosity Parameter
15	0.1	1.16	0.667	0.001

In Figure 7 to 10, load vs. deflection relationships, measured at midspan of the beam, resulting from Finite Element (FE) modeling, are compared to that of mean experimental results. Maximum loads (F_{max}) and load values corresponding to f_{ri} , $i=1, 2, 3$ and 4, for the numerical and experimental curves, along with relative error values, are tabulated in Table 8. The results indicate that maximum load values from the FE output, using the calculated material model for hybrid SFRC, is well matched to the experimental results, with relative errors less than 14% in all mixes. The numerical results show lower initial stiffness values than experiment. The initial stiffness in the load-deflection curve is directly related to the elastic modulus of the SFRC composites. Since the elastic modulus used for the numerical simulations is based on the proposed relation by RILEM (refer to Figure 5), which due to design safety considerations, is determined to be lower than anticipated experimental results, lower initial stiffness values are generated in the numerical results. From the results obtained, the proposed material models for the hybrid mixes can be reliably employed in the modeling process of hybrid SFRC segmental linings.

Table 8: Experimental and numerical values for F_{max} , F_{r1} , F_{r2} , F_{r3} (kN) and relative errors, δ (%)

Hybrid Mix No.	Mix Abbrev.	F_{max} (kN)		F_{r1} (kN)		F_{r2} (kN)		F_{r3} (kN)		F_{r4} (kN)	
		ABQ/EXP	δ (%)	ABQ/EXP	δ (%)	ABQ/EXP	δ (%)	ABQ/EXP	δ (%)	ABQ/EXP	δ (%)
3	0.3Mac0.3Mic	1.95/1.84	6.0	1.91/1.61	18.6	1.10/1.11	0.9	0.95/0.77	23.4	0.82/0.51	60.8
4	0.5Mac0.5Mic	2.47/2.17	13.8	2.44/1.85	31.9	1.79/1.59	12.6	1.40/1.31	6.9	1.23/1.07	15.0
5	0.5Mac0.3Mic	1.90/1.86	2.2	1.89/1.36	39.0	1.30/1.07	21.5	1.06/0.88	20.5	0.93/0.72	29.2
6	0.3Mac0.5Mic	2.06/1.89	9.0	2.06/1.66	24.1	1.25/1.24	0.8	1.09/1.08	0.9	1.02/0.82	24.4

Figure 7: Load-deflection curves for 0.3mic 0.3mac (mix 3): Experimental and FE results

Figure 8: Load-deflection curves for 0.3mic 0.5mac (mix 5): Experimental and FE results

Figure 9: Load-deflection curves for 0.5mic 0.5mac (mix 4): Experimental and FE results

Figure 10: Load-deflection curves for 0.5mic 0.3mac (mix 6): Experimental and FE results

4. Derivation of Analytical Fragility Curves

4.1. Overview

The general approach employed for deriving the analytical fragility curves is based on the methodology followed in the SYNER-G [25] framework, described by Argyroudis et al [24]. In this procedure, a typical soil profile and segmental lined circular tunnel of an urban subway (metro) tunnel is selected. A representative set of earthquake records is chosen to reflect the effects of ground motion characteristics on the soil-structure response. 1D equivalent linear Site Response Analysis (SRA) is then conducted to obtain seismically imposed shear displacements of the soil profile in free field condition.

A 2D numerical model of the soil-structure system is built using nonlinear continuum elements in a plain strain condition, considering the segmental joint behavior and capturing the Soil Structure Interaction (SSI) effects. The ABAQUS finite element software has been used for the numerical modeling and seismic analysis phases. The nonlinear static behavior of the surrounding soil is accounted for using the Mohr-Coulomb criterion. The nonlinear dynamic properties of the soil profile are inherent in the variations of shear modulus and damping ratio over the shear strain, employed in the site response analysis. The behavior of the lining material is considered using the obtained nonlinear stress-strain response curves for the SFRC mixes, aforementioned in section 3. The damaged plasticity model [37] is used to properly model the mechanical contribution of the fibers in SFRC composites, due to its capability to characterize the complete inelastic behavior and damage characteristics of concrete, both in tension and compression. Segmental joint behavior is evaluated using a separate numerical model, based on a standard test setup for such joints, then implemented in the soil-structure model. In this regard, a pre-calculated normal force is initially applied on the joint ends, followed by an increasing bending moment. To obtain the moment-rotation behavior of the joint, average rotation and bending moment values are continuously recorded. The model is verified using surface settlement values from actual instrumentation readings on the ground surface of the considered tunnel section [38].

Quasi-static transversal analysis is used to evaluate the nonlinear ovaling response of the tunnel system. In this type of analysis, the calculated seismic ground displacements are applied to the soil-structure system in a quasi-static manner. In general practice, this approach is conventionally used for the transversal seismic analysis of underground structures, due to being cost effective and sufficiently accurate for most cases [39]. In the present study, an extensive fragility analysis, comprised of more than 200 analysis cases, is carried out using various ground motion records and lining materials. Therefore, the implementation of the straightforward quasi-static analysis is preferred over the much more elaborate dynamic time history analysis.

Levels of seismic damage to the tunnel are categorized using damage states. A damage index is defined to quantify the damage states, expressing the effects of ground motion intensity and the non-continuous geometry of a segmented tunnel on the lining response (damage). The fragility curves are derived for each damage state and lining material, implicitly considering the tunnel geometry, soil properties and ground motion characteristics. In contrast, the current empirical

and expert-judgment based fragility curves for tunnels are incapable to consider the various properties of the tunnel, soil and earthquake motions of interest.

4.2. Damage States and Indexes

Damage states express a qualitative description of the damage level for a structure, generally based on experienced damage from past seismic events. Unlike building and bridge structures, a significant absence of references exists for damage indexes for tunnels. The damage index proposed by Argyroudis et al [24] for intact circular tunnels, defined as the ratio of actual (demand) and capacity bending moments on the tunnel cross section, resulting from both static and seismic loads, is one of the latest few efforts in this regard. In the present case of a segmented lining tunnel, a more specific definition is required to reflect the geometric discontinuities, i.e. segmental joints, of such tunnels. Moreover, a damage index definition based on displacement parameters rather than forces/moments is preferred, regarding the close correlation between damage and the predominant ovaling response of circular underground tunnels under seismic actions. Therefore, the damage index in this study is defined by means of equation 1:

$$DI = \left(\frac{\varepsilon_{D,demand}}{\varepsilon_{D.capacity}} \right)_{lining} + \left(\frac{M_{demand}}{M_y} \right)_{joint} \quad (1)$$

The proposed damage index is composed of two terms, the “lining term,” and the “joint term.” In this definition, $\varepsilon_{D,demand}$ is the diametric strain demand on the lining, defined as the ratio of maximum diametric deformation to the lining diameter due to a certain seismic action. The parameter $\varepsilon_{D.capacity}$ is the diametric strain capacity of the lining, calculated using the equation 2 [40]:

$$\Delta D_{max}/D = \pm \frac{1}{3} K_1 F \gamma_{max} \quad (2)$$

$$K_1 = \frac{12(1 - \nu_m)}{2F + 5 - 6\nu_m} \quad (3)$$

$$\text{Flexibility Ratio: } F = \frac{E_m(1 - \nu_l^2)R_l^3}{6E_l I_{l,1}(1 + \nu_m)} \quad (4)$$

Where:

- γ_{max} = maximum free-field shear strain of the ground (calculated from site response analysis)
- E_m = elastic modulus of surrounding soil
- ν_m = Poisson's ratio of surrounding soil
- R_l = Nominal radius of tunnel lining

ν_l = Poisson's ratio of tunnel lining
 $I_{l,1}$ = Moment of Inertia of tunnel lining, per unit width along the tunnel axis

The soil profile of interest in this study is layered and thus, the weighted average of the soil layers over layer heights is used to calculate mean values for the soil parameters, E_m and ν_m .

In equation 1, M_{demand} is the maximum bending moment demand on the segmental joints during a certain earthquake event; M_y is the yield bending moment of the segmental joints. The assessment of M_y is detailed in section 5. For a continuous non-segmented lining, the second term, i.e. the “joint term” will be eliminated. The remaining term, the “lining term,” is equivalent to the damage index proposed by Argyroudis et al [24] and compatible with the use of moments, based on the equal displacement approximation. It is notable that the defined damage index here only considers flexural damage of the joints and damage induced due to ovaling (diametric strain) in the lining. Thus, to characterize other possible forms of damage, e.g. shear failure of the SFRC lining[41], other suitable damage indexes should be derived.

Based on experiences from past earthquakes and limited conducted studies [24, 25, 42], by applying expert judgment, suitable damage states and corresponding threshold values of the defined damage index is considered (Table 9). Due to the overt lack of reference on fragility analysis of tunnels, especially for segmented lining ones, and of FRC lined tunnel cases, the proposed threshold values of damage index are not yet fully documented, but provide a realistic description for the expected damage.

Table 9: Damage States and threshold Damage Index values for the tunnel lining

Damage State (DS)	Damage Index (DI) Range	DI threshold value
DS0: None	$DI \leq 1.20$	-
DS1: Minor	$1.20 < DI \leq 1.90$	1.55
DS2: Moderate	$1.90 < DI \leq 2.70$	2.30
DS3: Extensive	$2.70 < DI \leq 3.00$	2.85
DS4: Collapse	$DI > 3.00$	-

4.3. Fragility Curve Parameters

The fragility curves derived from analytical approaches are generally described as a lognormal probability distribution function (see equation 5).

$$P_f(d_s \geq d_{s,i}|S) = \Phi \left[\frac{1}{\beta_{Tot}} \cdot \ln \left(\frac{S}{S_{m,i}} \right) \right] \quad (5)$$

Equation 5 states that the probability of exceeding a specified damage state (d_{si}) due to a certain value of seismic intensity (S) is described as a standard lognormal cumulative probability function (Φ). In this research, the damage states are given in Table 9. The Peak Ground Acceleration (PGA) is considered as S . S_{mi} is the median threshold value of S , required to cause the i^{th} damage state (d_{si}). β_{tot} is the total lognormal standard deviation, estimating the total variations in the fragility functions.

To obtain S_{mi} , the damage values are plotted against the seismic intensity parameter, $\ln(DI)$ vs. PGA plots, and by using the damage index limits, median threshold values for each damage state is acquired. This process is further explained in section 6.2. The total lognormal standard deviation, β_{tot} , is assumed to result from three main sources of uncertainty [21]:

$$\beta_{Tot} = \sqrt{\beta_{DS}^2 + \beta_C^2 + \beta_D^2} \quad (6)$$

Where β_{DS} , β_C and β_D , models the uncertainties due to the definition of damage states, capacity (resistance) of the tunnel and input seismic motion (demand), respectively. Due to lack of significant references and in line with Argyroudis et al [24], β_{DS} is taken equal to 0.4 and β_C is assigned a value equal to 0.3. The average standard deviation of the damage indices, calculated for the different ground motions, at each value of PGA is used as β_D [24].

5. Numerical Modeling of Segmental Tunnels

5.1. The Soil-Structure System

A typical section from the line 7 urban metro tunnel of Tehran (Iran) is selected for this study [43]. Figure 11 schematically displays the tunnel section and surrounding soil profile, along with the segmental lining. This segmental circular (bored) tunnel has an internal diameter equal to 9.16 m. The precast segments are 0.35 m thick, and have a width equal to 1.50 m. Each ring consists of six segments, plus a key segment at the crown. The typical reinforcement configuration in the segments is displayed in Figure 13. Steel bolts are used to connect the segments to each other in each ring. The segment lining and joints for SFRC material are modeled (Figure 12) to consider the properties of the SFRC composites by employing their calculated σ - ε constitutive models (Table 6) to model the lining and derive moment-rotation behavior curves for the joints. These aspects are explained in detail in the following sections.

The surrounding medium at the tunnel section of interest consists of a three-layered soil profile (Figure 11) with a total thickness of 31 m. The top layer consists of silt and sandy clay, with a thickness of 5 m. The middle layer is mainly comprised of gravelly sand, with traces of silt and clay, and has a thickness equal to 19 m. The bottom layer is a 7 m deposit of clayey sand. The significant geotechnical properties of each layer of the soil profile are given in Table 10. The mechanical behavior of the soil medium is modeled using the Mohr-Coulomb criterion.

Table 10 : Geotechnical properties of the soil profile

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

Figure 11 : The segmental tunnel lining and surrounding soil profile

Figure 12: Modeling aspects of SFRC segmental tunnel lining and joint

Figure 13: Steel reinforcement configuration for typical segment of the tunnel lining (in mm)

5.2. Segmental Joint Behavior

In segmented lining tunnels, the longitudinal joints, which connect the adjacent segments of each ring, also known as “segmental joints”, strongly govern the overall behavior of a segmental lining tunnel. Under seismic deformations in the transversal direction, segmental joints provide the necessary capacity for the tunnel section to accommodate the seismically generated ovaling deformations, playing an important role in preventing significant damage to the segment lining [44].

Properties of the segment material have proven significant influence on the behavior of segmental joints, especially under seismic deformations [44, 45]. In this regard, joints might be subjected to rotation, 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 [46, 47].

To realistically model the rotational and radial behavior of the segmental joints, the segmental connection of the tunnel was numerically modeled separately, according to a standard test setup [48]. The test setup is schematically illustrated in Figure 14. A normal stress equal to 10 MPa,

based on a preliminary gravity analysis of the tunnel section, was initially applied to the connection ends (segments). Increasing values of bending moment was applied using a non-concentrated load applied at the connection end.

The connecting segments are modeled using properties of the different lining materials, i.e. plain, conventionally reinforced (RC) and SFRC (refer to Table 6), implemented in the CDP material model [37]. In the RC case, the rebar configuration (Figure 13) is modeled for each connecting segment. Modeling of the plain, RC and SFRC concrete materials are elaborated in section 5.4. The connecting steel bolts (SS400 steel type, characteristic yield strength, $f_{yk} = 400$ MPa, elastic modulus, $E = 200$ GPa) was modeled as an elastic perfect-plastic material and simulated as an embedded region [37] within the segments.

The resulting behavior curve of the segmental connection was derived by recording bending moment and average rotation values of the connection ends. The obtained moment-rotation curves are presented in Figure 14. From these curves, the yield moment of the segmental joints (M_y) were estimated by applying the methodology of the idealized force-displacement curve procedure [49]. The attained values of M_y are used to calculate the “joint term” of the defined damage index in equation 1. In the normal (axial) direction of the segmental connection, the linear properties of the lining material and connecting steel bolt was used to model the compressive and tensile actions, respectively.

From the moment-rotation behavior curves of Figure 14, it can be noted that the SFRC composites present higher initial (elastic) stiffness values than the plain and RC cases. Likewise, in the nonlinear range, the SFRC mixes exhibit a better behavior than the plain concrete and RC cases, especially for higher rotation demands (rotation > 0.004 rad). Among the SFRC composites, the 5Mic composite displays the best flexural performance.

(a)

(b)

Figure 14 : Segmental joint behavior: (a) Test setup (b) Moment-Rotation curves for different lining materials

5.3. Earthquake Induced Ground Displacements

Seven ground motions were selected as input motions in outcrop conditions for the site response analysis (Table 11). The earthquake records were chosen to be compatible, i.e. in soil conditions, fault rupture mechanism, seismic wave propagation and so on, with the geo-seismic specifications of the tunnel site. These earthquake records were selected from a set of 20 ground motions, used in the ATC-55 project (FEMA-440 report) [50]. To calculate the tunnels response to varying levels of seismic intensity, each record is scaled to four levels of PGA: 0.2g, 0.5g, 0.7g, and 1.0g.

Table 11 : Specifications of the ground motion set

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

The EERA [51] package was used for the linear 1D site response analysis of the soil profile, using the above ground motion set. To perform the ground response analysis, 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 inputted. These diagrams are presented in Figure 15. For the seismic bedrock, the curves provided by Schnabel et al. [52] and a shear wave velocity equal to 760 m/s was used.

For the site response analysis, each layer of the soil profile is discretized into sufficient number of sub-layers varying from 1.5 to 4.0 m thick. In the iterative procedure, the ratio of effective and maximum shear strain is assumed equal to 0.5. For each input earthquake record, variation of the maximum shear strain with depth is calculated from the site response analysis. These plots were then used to obtain the induced cumulative peak ground displacements in each soil layer and depth of the soil profile. These displacement profiles were applied in a quasi-static manner on the lateral boundaries of the soil-structure model to obtain the response of the tunnel lining.

Figure 15 : Dynamic properties of the soil profile: (a) G/G_{max} vs. γ & Damping vs. γ curves; (b) shear wave velocity vs. depth

5.4. Numerical Model Development

A 2D numerical model of the soil-structure section is built using nonlinear continuum elements in a plain strain condition, considering the segmental joint behavior and capturing the SSI effects. The ABAQUS finite element software has been used for the numerical modeling and analysis phases. Both the tunnel lining and surrounding soil are modeled using solid continuum elements. The Mohr-Coulomb yield criterion was assigned to the soil elements. The SSI effect is effectively considered in the model by defining the tunnel lining as an embedded region[37] within the soil medium. The obtained $M-\theta$ relationships (Figure 14) were assigned to connector elements [37] to simulate the segmental joints' behavior.

For the tunnel lining, properties of the considered SFRC concrete mixes were incorporated. The obtained constitutive relationships for hybrid SFRC (Table 6) are implemented into the CDP material model and assigned to the tunnel lining. As a basis for comparison, plain (unreinforced) and conventionally reinforced concrete was also used as the lining material. In the RC lining material, 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 in the plain and RC linings was modeled based on the mechanical properties for the plain concrete mix, mix 7, with $f'_c = 40$ MPa and $E = 33320$ MPa, again implemented in the CDP material model. The compression branch of Figure 5, with negligible tensile strength, was considered as the behavior curve for

plain concrete. For all different lining materials, the plasticity parameters presented in Table 7 was set for the CDP model.

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 = 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.

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 [43]. 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.

6. Seismic Fragility Analysis

6.1. Quasi-Static Seismic Analysis

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. A schematic representation of the ovaling response and an example output of the tunnel lining are illustrated in Figure 16 (In figure (b), the surrounding soil elements have been hidden for better visualization). 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 (for each damage state: minor, moderate and extensive).

Figure 16 : Typical 2D quasi-static analysis results: (a) ovaling response of soil-structure system; (b) maximum diametric strain of the lining

6.2. Fragility Curves for SFRC Segmental Lining Tunnels

Relevant output results of the quasi-static analyses are used to compute the damage indexes for each analysis case. As previously mentioned, the procedure for developing analytical fragility curves follows that of similar studies for different structures [24, 25]. Natural logarithms of damage index ($LnDI$) versus PGA (seismic intensity parameter) data points are plotted and linearly approximated to derive the fragility curves for each damage state. In this linear regression, PGA and $LnDI$ are taken as the independent and dependent variables, respectively. From these diagrams, median threshold PGA values for each damage state are determined using the corresponding threshold damage index values defined in Table 9. In Figure 17, median threshold values of PGA for different damage states are attained using this approach for a typical example, i.e. SFRC mix No. 1 (0.5% macro fiber). The obtained median threshold value of PGA and corresponding total standard deviation (β_{Tot}) is used as the input parameters for the lognormal cumulative distribution function, described as the fragility curve.

In Figure 18, the fragility curves for each lining material is illustrated, with corresponding lognormal distribution parameters, i.e. median and standard deviation tabulated in Table 12. For better insight, the fragility curves of all different lining materials are plotted together on a single chart for each damage state and represented in Figure 19.

Table 12 : Median (μ) and standard deviation (β_{Tot}) values of the fragility curves

Tunnel Lining	Damage State
---------------	--------------

Material	Minor		Moderate		Extensive	
	μ (g)	β_{Tot}	μ (g)	β_{Tot}	μ (g)	β_{Tot}
Mix 1	0.704	0.758	1.019	0.758	1.190	0.758
Mix 2	0.790	0.726	1.132	0.726	1.317	0.726
Mix 3	0.823	0.717	1.158	0.717	1.340	0.717
Mix 4	0.843	0.685	1.186	0.685	1.373	0.685
Mix 5	0.844	0.717	1.226	0.717	1.433	0.717
Mix 6	0.759	0.739	1.092	0.739	1.273	0.739
RC	0.640	0.692	1.003	0.692	1.201	0.692
Plain	0.453	0.478	0.653	0.648	0.881	0.753

Figure 17 : Estimation of median threshold values of PGA for different damage states (Concrete Mix 1)

Figure 18 : Fragility Curves for different tunnel lining materials

Figure 19 : Fragility Curves for each Damage State

Comparing the fragility curves for SFRC, conventional reinforcement and plain concrete lining materials, the reinforced lining materials intuitively show less vulnerability than the plain concrete case at all levels of seismic intensity. Furthermore, SFRC segmental lined tunnels generally show a better seismic performance than the conventional reinforced case. Fragility curves for moderate and extensive damage states of 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 lining materials. Judging by the moment-rotation curves of Figure 14, this vulnerability can be due to lower flexural capacity of its segmental joints than the other SFRC mixes. By observing the fragility functions for the SFRC mixes, it can be noted that the seismic vulnerability of mixes 2, 3, 4 and 6 are quite close to each other. Yet, the difference in seismic vulnerability between all SFRC mixes lies in a 10% margin of each other.

The SFRC mix No. 5, containing 0.3% and 0.5% volume content of macro and micro steel fibers respectively, displays the best seismic performance among other lining materials. This can be because 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 [53].

6.3. Verification of Fragility Curves

In current literature, no documented research exists on the seismic fragility analysis of segmented lined tunnels and/or tunnels with SFRC linings. Therefore, a fully proven comparison of the obtained fragility curves from the present study with other works is not possible. Yet, the obtained fragility curves for the conventionally reinforced lining in this study are compared to those based on empirical data and expert judgment, neglecting the discontinuity effects due to segmental joints.

In this regard, the fragility curves corresponding to bored tunnels with good quality construction from HAZUS [21], which is based on expert judgment, are used for this purpose. These fragility curves are classified based on the tunnel construction quality and tunnel construction method, i.e. bored and cut-and-cover, with qualitative descriptions of the different damage states. The empirical fragility curves presented in ALA [22] are developed from damage data from past seismic events for various elements in a water system network, including tunnels. These curves are categorized based on the tunnel construction method, type of surrounding medium, i.e. alluvial soil or rock, and the quality of tunnel construction and maintenance. The soil-structure system of the present study roughly falls in the category of good quality construction and alluvial

geologic medium. In both references, the fragility curves are only available for the minor and moderate damage states, the extensive (heavy) damage state curves are not provided.

In Figure 20, the numerical fragility curves for the conventionally reinforced lining case is compared to relevant curves from HAZUS and ALA. It can be noted that in both damage states, the numerical fragility curves show less probability of damage than the empirical and expert-judgment curves. The HAZUS fragility curves are closer to the numerical ones, especially for the minor damage state, with the maximum difference approximately equal to 25% occurring at $PGA=1.2g$ in the moderate damage state. The variations between the numerical and empirical/expert-judgment curves can be mainly due to dissimilarities in the soil conditions, lining discontinuity and lining material, which are not fully considered in the latter (non-numerical) curves.

Figure 20: Comparison of numerical with empirical and expert-judgment fragility curves (Conventionally Reinforced lining):
(a) Minor damage state, (b) Moderate damage state

6.4. General Fragility Functions for SFRC Segmental Lining Tunnels

Based on the general shape of the obtained fragility curves, 4th degree polynomial equations provide suitable equations to approximate (fit) the curves (equation 7):

$$P_4(PGA) = a_4 \times (PGA)^4 + a_3 \times (PGA)^3 + a_2 \times (PGA)^2 + a_1 \times (PGA) + a_0 \quad (7)$$

Where $P_4(PGA)$ is the probability of damage (dependent variable), PGA is the independent variable and a_i , $i = 0, 1, 2, 3$ and 4 are the polynomial coefficients. Using regression analysis, the coefficients are calculated for fragility curves of the SFRC mixes and tabulated in Table 13 to 15, for minor, moderate, and extensive damage states, respectively.

Table 13: Coefficients of 4th degree polynomial fit for fragility curves (minor damage state)

Tunnel Lining Material	Minor Damage State				
	a ₄	a ₃	a ₂	a ₁	a ₀
Mix 1	0.2586	-1.1017	1.2650	0.2855	-0.0242
Mix 2	0.2647	-1.1813	1.5102	0.0508	-0.0146
Mix 3	0.2609	-1.1825	1.5592	-0.0171	0.0113
Mix 4	0.2703	-1.2467	1.6991	-0.1162	-0.0072
Mix 5	0.2542	-1.1617	1.5539	-0.0415	-0.0099
Mix 6	0.2637	-1.1576	1.4284	-0.1326	0.0181

Table 14: Coefficients of 4th degree polynomial fit for fragility curves (moderate damage state)

Tunnel Lining Material	Moderate Damage State				
	a ₄	a ₃	a ₂	a ₁	a ₀
Mix 1	0.1877	-0.8999	1.3033	-0.9670	-0.0040
Mix 2	0.1576	-0.8001	1.2609	-0.1890	0.0017
Mix 3	0.1493	-0.7714	1.2431	-0.2066	0.0030
Mix 4	0.1368	-0.7357	1.2423	-0.2501	0.0058
Mix 5	0.1292	-0.6878	1.1519	-0.2107	0.0041
Mix 6	0.1692	-0.8396	1.2816	-0.1590	0.0003

Table 15: Coefficients of 4th degree polynomial fit for fragility curves (extensive damage state)

Tunnel Lining Material	Extensive Damage State				
	a ₄	a ₃	a ₂	a ₁	a ₀
Mix 1	0.1418	-0.7211	1.1414	-0.1542	0.0007
Mix 2	0.1063	-0.5864	1.0262	-0.2002	0.0046
Mix 3	0.0988	-0.5579	1.0014	-0.2078	0.0053
Mix 4	0.0816	-0.4962	0.9558	-0.2289	0.0072
Mix 5	0.0776	-0.4642	0.8853	-0.1983	0.0057
Mix 6	0.1195	-0.6372	1.0716	-0.1865	0.0033

In order to extend the conclusions of the obtained fragility curves to any SFRC mix, a general relation is established between the fragility curve parameters and a definitive parameter of a SFRC composite. In this regard, the residual flexural strength parameter, f_{r4} , of SFRC composites is related to the fragility function, resulting in f_{r4} -PGA-Probability of Damage relationships. Since the fragility curves have been approximated with 4th degree polynomial equations, the relation between f_{r4} and a_i , $i=0, 1, 2, 3$ and 4, is found using polynomials, $g_i(f_{r4})$ (equation 8), and substituted in equation 7.

$$a_i = g_i(f_{r4}) = b_{4i} \times (f_{r4})^4 + b_{3i} \times (f_{r4})^3 + b_{2i} \times (f_{r4})^2 + b_{1i} \times (f_{r4}) + b_{0i}; \quad (8)$$

$$i = 0.1.2.3.4$$

In Figure 21, f_{r4} vs. a_4 data points for each damage state are plotted together and fitted with suitable polynomial equations. The polynomials used are of degree 3 or 4. This process is repeated for a_i , $i= 0, 1, 2$ and 3 . The corresponding polynomial coefficients, b_i , are calculated from regression analysis for each damage state and presented in Table 16 to 18.

Table 16: Coefficients of $g_i(f_{r4})$, $i=0,1,2,3$ and 4 (minor damage state)

$g_i(f_{r4})$	Minor Damage State				
	b_4	b_3	b_2	b_1	b_0
g_4	0.00	0.00	-0.01	0.02	0.24
g_3	0.00	0.18	-1.63	4.75	-5.59
g_2	0.00	-0.75	6.70	-19.37	19.52
g_1	-7.21	78.78	-319.44	569.18	-375.60
g_0	0.71	-7.78	31.42	-55.71	36.57

Table 17: Coefficients of $g_i(f_{r4})$, $i=0,1,2,3$ and 4 (moderate damage state)

$g_i(f_{r4})$	Moderate Damage State				
	b_4	b_3	b_2	b_1	b_0
g_4	0.00	0.13	-1.16	3.33	-2.94
g_3	0.00	-0.47	4.16	-11.89	10.23
g_2	0.00	0.31	-2.71	7.68	-5.85
g_1	5.39	-61.86	263.02	-490.47	337.36
g_0	0.00	-0.02	0.19	0.53	0.49

Table 18: Coefficients of $g_i(f_{r4})$, $i=0,1,2,3$ and 4 (extensive damage state)

$g_i(f_{r4})$	Extensive Damage State				
	b_4	b_3	b_2	b_1	b_0
g_4	0.00	0.15	-1.30	3.73	-3.35
g_3	0.00	-0.59	5.15	-14.76	13.09
g_2	0.00	0.57	-4.98	14.21	-12.13
g_1	0.05	-0.38	0.82	-0.07	-1.11
g_0	0.00	-0.01	0.12	0.34	0.32

Figure 21: f_{r4} vs. a_4 data points and corresponding polynomial fits

7. Conclusions

The seismic performance of TBM constructed tunnels with segmental linings of different type and content of steel fiber reinforced concrete is investigated. Conventionally reinforced and unreinforced (plain) concrete lining materials are analyzed as a basis for comparison. Analytical fragility curves developed from nonlinear quasi-static seismic analysis of the tunnel section is used as a potent tool to express the tunnel's vulnerability under different levels of seismic intensity.

Based on the specified conditions (tunnel geometry and rebar configuration, soil conditions, etc.) of this research, the main results drawn from the obtained fragility curves are summarized as follows:

1. The reinforced lining materials, i.e. SFRC and conventionally Reinforced Concrete (RC), intuitively show less vulnerability than the plain concrete case at all levels of seismic intensity.
2. The SFRC segmental lined tunnels generally show a lower probability of seismic damage in all damage states than the RC case, up to 15% at certain seismic intensities.
3. Fragility curves for moderate and extensive damage states of SFRC mix 1, nearly match that of conventional reinforcement at all PGA values, being the most vulnerable among the SFRC lining materials.
4. The seismic vulnerability of mixes 2, 3, 4 and 6 are quite close to each other

The results obtained show that, as anticipated, unreinforced concrete lined tunnels generally show a higher probability of conceding damage than conventional and fiber reinforced cases.

However, as the level of damage intensifies, i.e. from minor to extensive damage states, this deficit in seismic performance reduces, from approximately 25% to around 15% at maximum for $PGA=0.8g$. By comparing the fragility curves for different damage states, conventionally reinforced concrete linings show a relatively identical behavior to that of 0.5% macro fiber concrete mix (Mix No. 1) lining, weaker than the other fiber concrete mixes. Linings of concrete mixes 2, 3, 4 and 6 exhibit a close performance with each other and superior to that of concrete mix 1. The concrete mix containing 0.3% and 0.5% volume content of macro and micro steel fibers respectively (Mix No. 5), displays the best performance among other lining materials. This hybrid mix optimizes the advantageous behavior from micro and macro fibers regarding ductility, toughness, and crack-bridging mechanism.

In summary, reinforced concrete segmental linings intuitively show less vulnerability than the unreinforced case under different seismic intensities. Steel fibers, especially micro size fibers and for higher states of damage, display better performance over conventional steel rebar in reinforced concrete linings. For best performance, a hybrid SFRC mix containing both micro and macro fibers is suggested, with a higher content of micro fibers over macro ones. Therefore, steel fiber reinforced concrete can be considered a technically preferable option for the design of segmental lining tunnels in seismically active regions.

Similar to any analytical fragility analysis, the obtained results and proposed fragility functions in this paper is applicable to limited problems. The derived fragility curves are based on the assumptions made for the tunnel geometry, soil profile, definition of damage index, damage states and corresponding threshold values. In particular, the defined damage index considers only flexural damage of the joints and damage induced due to ovaling in the lining. Thus, to characterize other possible forms of damage, e.g. shear failure of the lining, other suitable damage indexes should be derived. Future researches can aim at identifying different seismic damage forms for SFRC segmental tunnels, define corresponding damage indexes and develop fragility curves on this basis. The fragility analysis conducted here comprised of 7 earthquake records, 4 levels of PGA, 8 lining materials (6 SFRC composites + 1 RC + 1 plain concrete) and 3 damage states (minor, moderate, extensive). The fragility analysis can be extended to include more variations of the considered parameters or introduce new relevant ones (e.g. soil type, tunnel diameter, etc.).

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