2D SEISMIC NUMERICAL ANALYSIS OF SEGMENTAL TUNNEL LINING BEHAVIOUR

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ABSTRACT

Segmental tunnel linings are now often used for seismic areas in many countries. Some prescriptions and guidelines specifically address the issue of seismic design. Unfortunately, the behaviour of segmental tunnel lining under seismic loads is still somewhat unclear. The influence of segment joints on tunnel lining behaviour during seismic loading has in fact not been quantitatively estimated in the literature. This paper presents a numerical study in order to investigate the performance of segmental tunnel lining under seismic excitation. Analyses have been carried out using a two-dimensional finite difference element model. The seismic signal obtained from an earthquake in Nice has been adopted as input. The numerical results show that a segmental lining can perform better than a continuous lining during an earthquake. The effect of plasticity of the soil constitutive model on the tunnel lining has also been highlighted. The results have indicated that an elastic analysis is not sufficient to determine the seismic induced response of a soil-tunnel system. Moreover, comparative results have pointed out that equivalent static solutions could result in smaller structural lining forces than those of a true dynamic analysis.

1 INTRODUCTION

While tunnels generally perform better than above ground structures during earthquakes, the damage to some important structures during earthquake events, e.g., the 1995 Kobe earthquake in Japan, the 1999 Chi Chi earthquake in Taiwan, the 1999 Bolu earthquake in Turkey, the 2004 Baladeh earthquake in Iran and recently the 2008 Sichuan earthquake in China has highlighted the need to account for seismic load in the design of underground structures [40].

The behaviour of tunnels under seismic loads has been studied using a variety of approaches: empirical and analytical methods, physical model tests and numerical modelling.

Due to their simplicity, various elastic closed-form solutions have been developed to determine the structural forces induced in a circular tunnel lining due to a seismic load, e.g. [7,12,41,43,44,52]. Hashash *et al.* [23] described discrepancies between the Wang [52] and Penzien [44] methods, and used numerical analyses under the same assumptions to obtain a better understanding of the differences in thrust between the two solutions. The comparisons clearly demonstrated that Wang's solution provides a realistic estimation of the thrust in the tunnel lining for a no-slip condition. It has been recommended that Penzien's solution should not be used for a no-slip condition, Hashash *et al.* [23]. These differences have also been reported by Park *et al.* [41], as well as by Bazaz and Besharat [5]. The works performed by Park *et al.* [41] and Park *et al.* [42] have indicated a good agreement between their solution and the previous solutions of Wang [52] and Bobet [7]. However, closed-form solutions are usually limited to the following assumptions [47]:

- Homogenous soil mass and tunnel lining, which are assumed to be linear elastic and mass-less materials;
- Circular tunnel with uniform thickness and without joints;
- The construction sequence is not considered.

In order to overcome the drawbacks of analytical methods, physical model tests and numerical analysis have been used to obtain a better understanding of the physical problem and in particular of the soil-structure interaction phenomenon.

Physical model tests have been carried out by many researchers to investigate the performance of underground structures and to check current design/analysis methods. Most of them have focused on data for the validation of the design models, e.g. [6,9,11,33] and on tunnel lining behavior, e.g. [25]. Due to their complexity and the high costs of the tests, the results obtained from physical tests are still quite limited.

The recently common trend is to use numerical analysis techniques, e.g. [5,19,23,42,47] in which seismic loads are considered as pseudo-static loads. The main disadvantage of pseudo-static models is that they do not to take into consideration changes in the structure behaviour change in time. In addition, it has been suggested that an equivalent static solution would yield smaller structural lining forces than a true dynamic solution [26,34,46]. Full dynamic analysis, which is also called time-history analysis, and which has been used in this study, is the most complex level of seismic analysis. Therefore it is also the most precise method. This type of analysis is generally numerical. However, this method is not economic, due to the long calculation time that is necessary. This is why full dynamic analysis application is still limited.

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Pakbaz and Yareevand [40] performed 2D numerical analyses using the CA2 software, in order to estimate the effect of an earthquake on circular tunnels, in an elasto-plastic medium. The tunnel was modeled as an elastic beam. The recordings of the Naghan Fars earthquake, which showed a maximum acceleration of 0.7g and an intensity of 7 in the Richter scale, were used. Two sets of parametric analyses were carried out in order to show the variation in the maximum stresses with the peak ground acceleration and with the flexibility ratio F [52]. Kontoe *et al.* [30] presented a case study of the Bolu highway twin tunnels, which experienced extensive damage during the 1999 Duzce earthquake in Turkey. Static and full dynamic plane-strain finite element analyses were undertaken to investigate the seismic response of the tunnel and to compare the results with post-earthquake field observations. The results of the dynamic numerical analyses were also compared with those obtained from pseudo-static elasto-plastic analyses. The pseudo-static analyses, in which prescribed shear strains were assigned to the model boundaries, gave thrusts which were lower than the full dynamic analyses. Conversely, pseudo-static analysis predicts a much higher bending moment. Cao and Yan [8] have conducted a study in which attempts have been made to systematically analyse tunnel responses for different degree of lining rigidity, to an earthquake and to find rules that govern the change in lining rigidity and the seismic response of tunnels. Conclusions have been drawn state that, under the impact of a seismic wave, an increase in rigidity is consistent with an increase in the maximum axial force, the maximum bending moment and the maximum combined stress of a lining structure. Therefore, it is unfeasible and uneconomic to diminish the response of tunnels to seismic motion by increasing lining rigidity. In all the above analyses, segmental linings and the effects of the joints were not considered either.

Segmental tunnel linings are now often used for seismic areas in many countries, such as the United States, Japan, Iran, Taiwan, Turkey, Venezuela, Spain, Portugal, Italy, Greece, India, and elsewhere [14]. As is well known, due to their higher flexibility, which is achieved through the use of joints between the segments, they can accommodate deformations with little or no damage. A segmental lining generally performs better than a continuous lining during an earthquake [14,28,45]. The presence of segment joints in a tunnel lining can reduce the stresses and strains in the lining [24]. An interesting estimation of the use and performance of segmental tunnel linings in seismic areas can be found in Dean *et al.* [14].

However, the design of segmental linings under seismic loads still needs to be discussed. Only a few prescriptions and guidelines specifically address the issue of seismic design, e.g. [1,45]. Unfortunately, the effect of joints on lining behaviour has still not been quantitatively estimated.

Obviously, one of the key issues regarding the simulation of the response of a segmental tunnel lining is the necessity of taking into consideration the influence of the joints. Chen and Gui [10] used an equivalent continuous lining with reduced stiffness (Muir Wood [35]) to consider the presence of joints in a tunnel lining. He and Koizumi [25] performed a series of shaking table model tests, seismic 2D FEM analysis and static analysis based on the seismic deformation method [24,31,39] to study the seismic behaviour and the seismic design method in the transverse direction of shield tunnels. In their static FEM analysis, the segment joints were simulated using short beam elements lowered in tension-compression rigidity and bending rigidity. As for the beam spring model, the segment joints were simulated using a rotational spring constant. Unfortunately, the influence of joint distribution and joint stiffness on tunnel behaviour under seismic loads was not introduced. Naggar *et al.* [37] developed a simplified analytical solution which allows the joints in the tunnel lining to be considered. The segmental joints were simulated through rotational stiffness. However, the method cannot be applied to cases in which the joint distribution is asymmetrical to the vertical axis of the tunnel. As can be seen from the results of their jointed lining analysis, the presence of joints permit the developed moments in a tunnel lining to be suppressed by up to 50% compared to an unjointed lining. However, the effect of the joints on the developed thrusts was not so significant (10% or less). They concluded that the effect of joints on the internal forces needs be considered to achieve an economic design. Naggar and Hinchberger [38] used a non-linear FEM model and a linear elastic solution for jointed tunnel linings to examine the effects of concrete degradation on structural forces in tunnel linings under static and seismic loads. In their analyses, segment joints were simulated using thin zones of linear elastic elements with an equivalent elastic property determined on the basis of the rotational stiffness of the joints. Most of the presented studies that have been considered the presence of joints during the analysis of the response of a tunnel lining under seismic loads are related to immersed tunnels with rectangular cross-sections, e.g. [4,35,51]. This kind of cross-section presents different behaviour, under seismic circumstance, from that of the circular tunnel studied in this paper. It should be noted that in any of the above cited works, the effect of the joints on tunnel behaviour during a time seismic history that is different from that of a continuous lining was not mentioned in detail.

It is well known that soil exhibits non-linear and irreversible behaviour, even at low strains. Under severe earthquake loading, the seismic response of a tunnel may be affected significantly by this non-linear behaviour [48,49]. Shahrour *et al.* [48] conducted an elasto-plastic finite element analysis using the advanced cyclic elasto-plastic model Modsol [29], which is based on the bounding surface concept. This constitutive model reproduces the contracting behaviour of soft soils under cyclic loading, and generally leads to good results in terms of settlements. The results of numerical simulations have shown that seismic-induced plastic deformations lead to a significant reduction in the seismic amplification, and, consequently, to an important decrease in the seismic-induced bending moment in a tunnel. The 3D numerical results obtained by Sliteen [49] and Sliteen *et al.* [50] instead showed that an elastic analysis is not sufficient to determine the seismic induced response of a soil-tunnel system. In their study, the maximum bending obtained with non-linear analysis using Mohr-Coulomb criteria was 24% higher than the one obtained with an elastic analysis. They concluded that elastic analysis is not sufficient to determine the seismic induced response of soil-tunnel interaction. The work of Amorosi and Boldini [3] focused on the role of plasticity in the dynamic response of the soil-tunnel system. Their results indicated the responses of a tunnel lining in terms of bending moment and normal forces under seismic excitation and in undrained conditions. Different tendencies were shown when soft clay and stiff clay were studied. The aforementioned literature indicates that the influence of a soil constitutive model on tunnel behaviour under seismic loads is an important factor that should be considered to obtain a reliable design.

A 2D finite difference model of a segmental tunnel lining exposed to a full seismic excitation in which the segment joints are taken into consideration has been proposed in this study. The joint has been considered as an elastic pin and its stiffness characteristics have been specified by rotational stiffness K_{RO}, axial stiffness K_A, and radial stiffness K_R. The numerical results allow the differences in tunnel behaviour under seismic excitation due to the effect of the segmental joints and of the soil constitutive model to be highlighted. Additionally, significant differences in structural lining forces when pseudo-static solutions and a full dynamic analysis have been used have also been introduced.

2 NUMERICAL MODELLING

2.1 Ground parameters

Parameters from the Bologna-Florence high-speed railway line tunnel project in Bologna have been adopted (see Table 1). This case is named the reference case [13].

Table 1. Details of the reference case [13]

Parameter	Symbol	Value	Unit
Properties of the clayey sand			
Unit weight	γ	17	kN/m^3
Young's modulus	E	450	MPa
Poisson's ratio	ν	0.3	-
Lateral earth pressure factor	K_0	0.5	-
Overburden	H	20	m
Properties of the tunnel lining			
Young's modulus	E_l	35,000	MPa
Poisson's ratio	ν_l	0.15	-
Lining thickness	t_l	0.4	m
External diameter	D	9.4	m

2.2 Numerical model description

Figure 1 illustrates the 2D plane-strain numerical model which has been set up using the finite difference software FLAC^{3D}. It has been assumed that the behaviour of the tunnel structure is linear-elastic. In order to highlight the effect of the soil constitutive model, the behaviour of the soil has been assumed to be linear-elastic and elastic perfectly plastic. The latter constitutive model of the soil is based on the Mohr-Coulomb failure criterion. The effect of the gravity field has also been taken into consideration.

The volume under study is discretized into hexahedral zones. The tunnel lining is modeled using embedded liner elements. No relative movement (no-slip) was allowed at the tunnel lining-soil interface. The segment joint is simulated using double nodes (Figure 2). These include six degrees of freedom, which are represented by six springs: three translational components in the x, y and z directions, and three rotational components around the x, y and z directions. It is possible to assign the stiffness to each spring. One of the following four attachment conditions is used: (1) free; (2) linear spring characterized by a stiffness factor; (3) bi-linear spring characterized by a stiffness factor and yield strength; and (4) rigid.

In this study, the stiffness characteristics of the joint connection are represented by a set composed of a rotational spring (K_{RO}), an axial spring (K_A) and a radial spring (K_R), as depicted in Figure 2 and Figure 3. As in the works of the same authors [16,17,18], the behaviour of axial springs has been approximately represented by a linear relation, using a constant coefficient spring. The radial stiffness and rotational stiffness of a segment joint have instead been modelled by means of a bi-linear relation that is characterized by a stiffness factor and a maximum bearing capacity. The attachment conditions of the translational component in the y direction (parallel to the longitudinal axis of the tunnel) and two rotational components around the x and z directions have been assumed to be rigid for all the investigated cases. The values of the spring stiffness used to simulate the segment joints have been determined on the basis of the simplified procedures presented by Do *et al.* [16].

Embedded liner elements are attached to the zone faces along the tunnel boundary. The liner-zone interface stiffness (normal stiffness k_n and tangential stiffness k_s) is chosen using a rule-of-thumb in which k_n and k_s are set to one hundred times the equivalent stiffness of the stiffest neighboring zone [27].

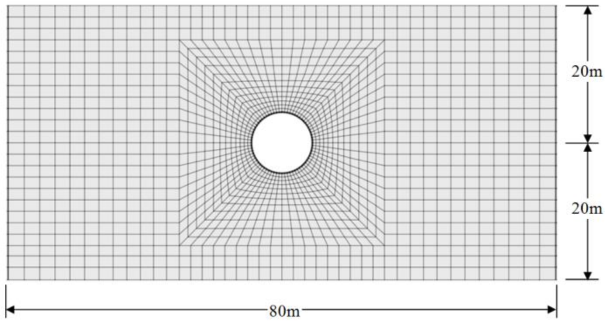


Figure 1: Plane strain model under consideration.

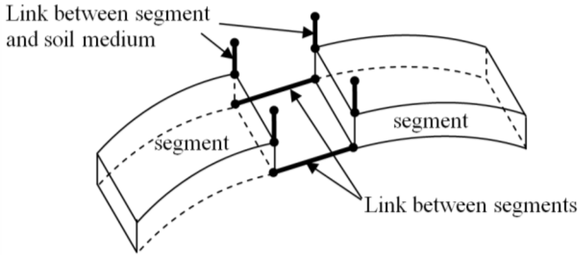


Figure 2: Joint connection scheme.

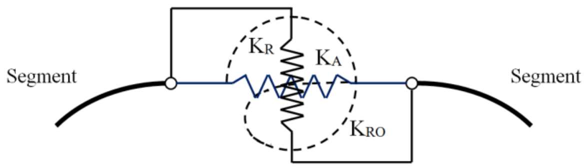


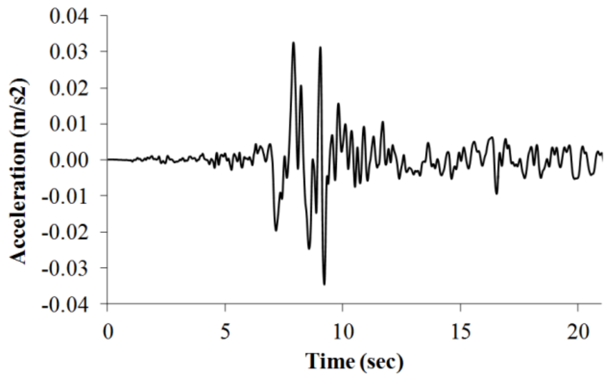
Figure 3: K_A, K_R, K_{RO} stiffness in the axial, radial and rotational directions of a joint.

In the seismic analysis, the width of the model plays an important role in ensuring the development of the free-field deformation far away from the tunnel. A sensitivity study found that the soil-tunnel structure interaction region may be extended up to three diameters from the tunnel centre [20]. In this analysis, the lateral boundaries of the mesh were placed approximately 4.5 diameters from the tunnel centre. The FLAC^{3D} model grid contains a single layer of zones in the y-direction, and the dimension of elements increases as one moves away from the tunnel (see Figure 1). The numerical model is 80 m wide in the x-direction, 40 m high in the z-direction and consists of approximately 2070 zones and 4280 grid points.

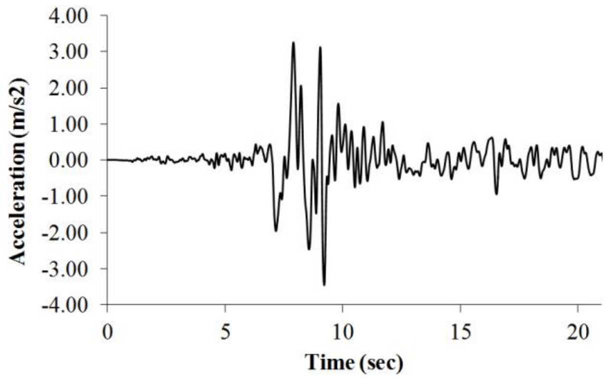
During the static analysis, the nodes were fixed in the directions perpendicular to the x-z and the y-z planes (i.e. y = 0, y = 1, x = -40 and x = 40), while the nodes at the base of the model (z = -20) were fixed in both the horizontal (x) and vertical (z) directions. During the dynamic stage, the nodes along the base of the model were freed in the horizontal direction in order to apply seismic input motion.

The seismic signal from the Nice earthquake case has been adopted in the study. This accelerogram is representative of the French design spectrum [22]. It has a maximum acceleration of 0.35g. The spectrum of the corresponding Fourier amplitude is shown in Figure 5. Moreover, a lower signal (maximum acceleration of 0.0035g) has also been used. The acceleration time-histories of 21 seconds duration are presented in Figure 4. These records were integrated to provide velocity histories which were then applied in the horizontal direction to all the nodes along the bottom boundary of the model.

The dynamic simulations were performed using Rayleigh damping. The centre frequency for Rayleigh damping is 1.99 Hz, which was determined from an undamped analysis. The Rayleigh damping ratio of the soil and of the tunnel lining were fixed at 5% and 2%, respectively. In order to simulate a semi-infinite elastic space, quiet boundaries are placed at the borders of the model in order to avoid wave reflection. Since a dynamic source is applied as a boundary condition at the bottom of the model, a free filed boundary was applied at the sides of the model [27]. In order to accurately represent wave transmission through the model mesh, it is necessary to ensure that the size of the element is small compared to the transmitted wavelength. Therefore, the side length of the element (Δl) was chosen accordingly on the basis of the recommendation by Kuhlemeyer and Lysmer [32]:



a) Low acceleration time history



b) High acceleration time history

Figure 4: Seismic input signals.

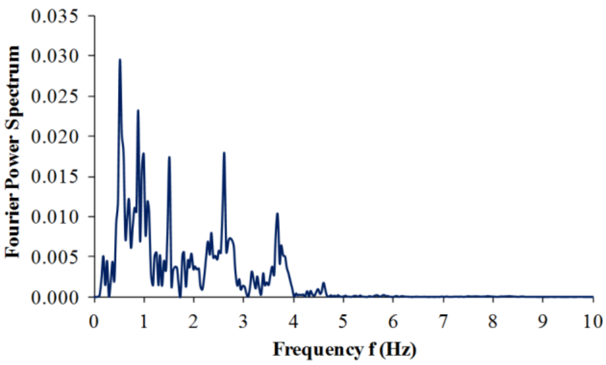


Figure 5: Input acceleration power spectrum (e.g., high signal case).

$$\Delta l \leq \frac{V_{S,\min}}{10 f_{\max}} \tag{1}$$

where $V_{s,min}$ is the lowest shear wave velocity that is of interest in the simulation;
f_{max} is the highest frequency of the input wave.

As far as the higher signal is concerned, the Fourier amplitude of the uncorrected Nice record indicates that the highest frequency f_{max} is less than 5 Hz (Figure 5). The Fourier spectrum obtained in the case in which a low signal was used indicated the same highest frequency. All the parameters of the FLAC^{3D} model applied in the case in which a low signal was used were therefore similar to those applied in the case of a high signal.

2.3 Construction simulation

Prior to the application of the seismic load, it is necessary to establish the steady state of the excavated tunnel under static conditions. When a tunnelling process is performed in a 2D plane strain model, an assumption that takes into account the pre-displacement of the ground surrounding the tunnel boundary prior to installation of the structural elements must be adopted. The convergence confinement method has been chosen in the present study with a relaxation factor, λ_d, of 0.3. The numerical modeling of tunnel ovaling was performed through the following steps:

- *Step 1*: Establishing the in-situ state of stress in the soil prior to tunnel construction.
- *Step 2*: Deactivating the excavated soil inside the tunnel and simultaneously applying the convergence-confinement process using a relaxation factor, λ_d, of 0.3. The concrete lining is then active on the tunnel periphery. The computation process is stopped when the equilibrium state is reached.
- *Step 3*: Assigning a seismic load to the bottom of the model thus allowing a new computation process to be made.

It should be noted that all the values presented in the present study are determined by subtracting the lining forces computed at the end of tunnel construction (step 2) from those at the end of the seismic load (step 3). In addition, all the calculations were performed under drained conditions.

It is important to mention that the average time necessary for the calculations with a continuous lining and a segmental lining is approximately 30 days and 40 days, respectively, when a 2.67 GHz core i7 CPU ram 24Go computer is used.

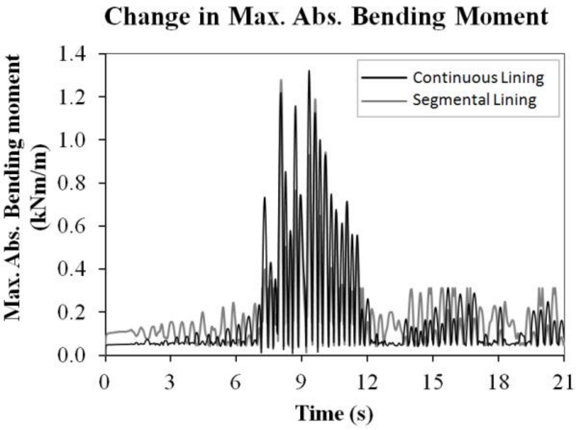
3 NUMERICAL ANALYSES

3.1 Behaviour of a tunnel under a low seismic load

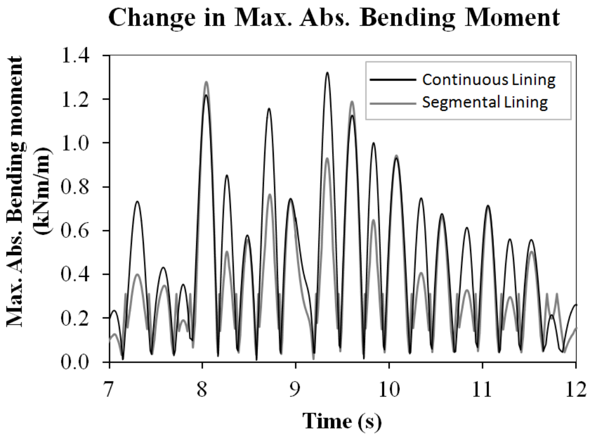
This section highlights the effect of segmental joints and a soil constitutive model on the lining behaviour of a tunnel under low seismic excitation.

The obtained numerical results have pointed out an insignificant influence of the soil constitutive model on the maximum absolute bending moment change developed in a tunnel lining during seismic loads. When the seismic loading is low, only a few of plastic zones occurs in the soil mass. It can be seen that the largest variation in structural lining forces occurs between 7 and 11 s approximately, which corresponds to the most intense part of the seismic excitation (see Figure 6, for example).

In order to highlight the effect of the joints, Figure 6 compares the maximum absolute bending moment change induced in a continuous lining and a segmental lining in the elastic case. During seismic excitation, the change in the maximum absolute bending moment induced in a segmental lining is larger than that induced in a continuous lining when the acceleration of the input motion is small. However, a reverse phenomenon is observed when the acceleration of the input is increased (see Figure 6b).



a)



b)

Figure 6: Change in maximum absolute bending moment during 21 seconds (a) and during the most intense part of seismic excitation (b) - Influence of segmental joints when an elastic constitutive model is used.

As far as the influence of the constitutive model is concerned, the changes in the maximum absolute bending moment induced in segmental and continuous linings when elastic and elastic-perfectly plastic models are used are not similar. In the case in which a continuous lining is used, the obtained value with Mohr-Coulomb is approximately 2% smaller than the one with the elastic soil constitutive model. On the other hand, in the case in which a segmental lining is used, Mohr-Coulomb gives a value which is 15% larger than the elastic one (Table 2).

It can be seen that the joints have a negligible effect on the change in the maximum normal forces induced in a continuous lining and in a segmental lining in the case in which an elastic soil model is used (Table 2). The same conclusion is obtained when the Mohr-Coulomb soil model is used. As far as the influence of the constitutive model on the change in maximum normal forces is concerned, both continuous and segmental linings show a difference of about 10% (Table 2).

As can be seen in Figure 7 and Table 2, the maximum value of the change in the maximum normal displacement induced in a segmental lining is approximately 5% higher than that of a continuous lining for the two constitutive models. It is interesting to note that, during seismic excitation, when the acceleration of the input motion is large enough, the change in maximum normal displacement induced in a segmental lining is usually larger than that of a continuous lining. However, an opposite effect is observed at low accelerations of the input motion. This could be explained by the fact that the movement of the tunnel lining under seismic loading is absorbed by the segmental joints. Nevertheless, when acceleration of the input motion is large enough, the effect of the deformation absorbability of the joint on the overall deformation of the lining is negligible.

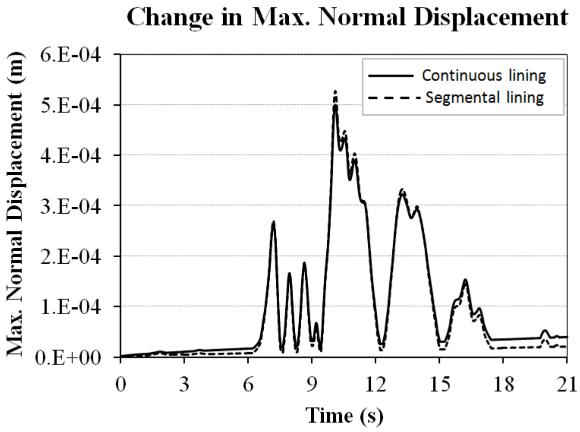


Figure 7: Change in normal displacement - Influence of segmental joints when an elastic soil constitutive model is used.

Table 2. Maximum changes in structural forces, lining deformation and surface settlement (low signal case)

	Elastic model		*Mohr-Coulomb model*	
	Cont. lining	Seg. lining	Cont. lining	Seg. lining
Change in max. absolute M (kNm/m)	1.32	1.28	1.30	1.47
Change in max. N (kN/m)	5.55	5.49	5.14	4.91
Change in max. Normal disp (mm)	0.500	0.527	0.499	0.532
Change in max. Settlement (mm)	0.038	0.020	0.043	0.038

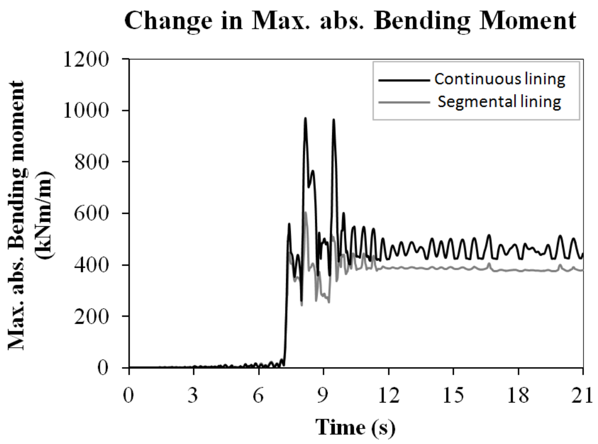
As far as surface settlement during seismic excitation is concerned, the results indicate that the change in surface settlement when a segmental lining is used is less sensitive to seismic loads than that of a continuous lining. Similarly, a less sensitive surface settlement to seismic loads when the elastic soil model is applied compared to that of the Mohr-Coulomb model can also be observed. The change in surface settlement when the Mohr-Coulomb model is used is greater than that of the elastic model. However, the absolute differences are negligible. It should be mentioned that a positive value of the change in surface settlement corresponds to heave of the soil.

3.2 Behaviour of a tunnel under a high seismic load

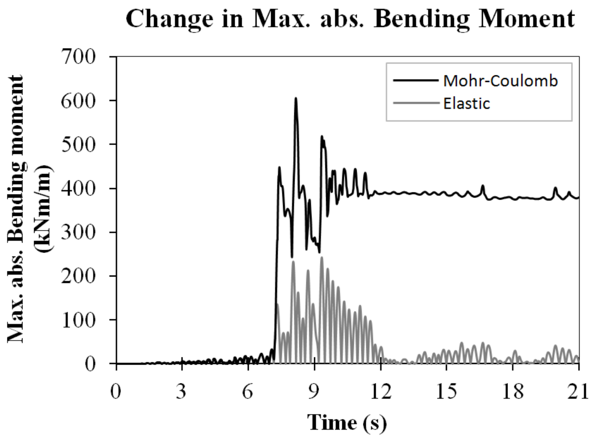
This section highlights the effect of segmental joints and a soil constitutive model on the lining behaviour of a tunnel under high seismic excitation.

In contrast to the results obtained when the low seismic signal presented in the above section was used, Figure 8b shows a significant effect of the soil constitutive model on the change in maximum absolute bending moment developed in a tunnel lining. The changes in maximum bending absolute moment in the case in which the Mohr-Coulomb model is used are usually higher than those determined when elastic material is used and reach a maximum This could be attributed to the lower shear strains induced in the linear elastic case compared to those observed for the Mohr-Coulomb one as can be seen in

Figure 9 differences of about 250% in a segmental lining (Table 3). The same differences in the bending moment were observed in the numerical work of Sliteen [49] and Sliteen *et al.* [50]. However, the present results appear to be in contrast those obtained by Shahrour *et al.* [48], who showed an important decrease of about 50% in the seismic-induced bending moment in a tunnel when a soil constitutive model, which took into consideration plastic deformations, was used. It should be noted that an elastic-perfectly plastic constitutive model with a Mohr-Coulomb failure criterion has been used in the present study and in the work of Sliteen [49] and Sliteen *et al*. [50]. Nevertheless, Shahrour *et al.* [48] used an advanced cyclic elastoplastic model, Modsol [29], which is based on the bounding surface concept. Another reason for the discrepancy could be the effect of the soil type, as found in the work by Amorosi and Boldini [3]. While medium Nevada sand was used in the study by Shahrour *et al*. [48], clayey sand has been used in the present study. It should also be noted that the accumulation of plastic strain during seismic loading could result in the difference in the locations on the tunnel structure at which the induced-maximum bending moment is observed when Mohr-Coulomb and elastic models are used.



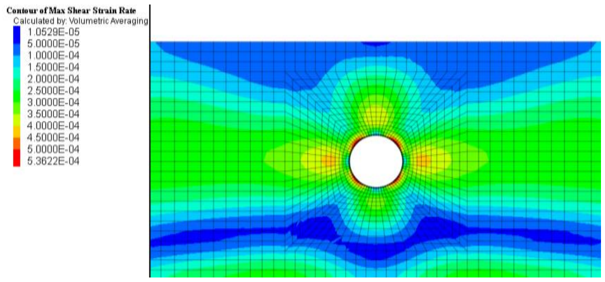
a) Influence of segmental joints when the Mohr-Coulomb constitutive model is used.



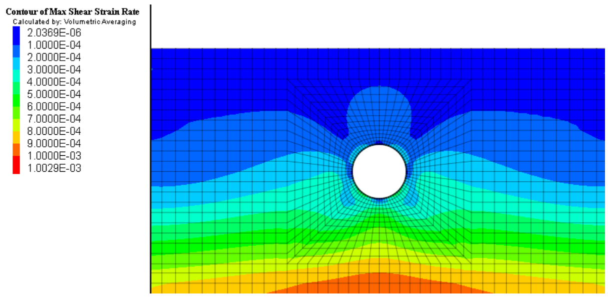
b) Influence of the soil constitutive model when a segmental lining is used.

Figure 8: ***Change in the maximum absolute bending moment.***

It is interesting to note that although the residual bending moment is almost zero in the case in which the elastic soil constitutive model is used, significant residual maximum absolute bending moment of about 58%, compared to the maximum value, can be observed 21 seconds after the end of the seismic load when the Mohr-Coulomb constitutive model is used. The residual bending moment is due to irreversible deformations cumulated by the soil during seismic excitation. This statement can be also confirmed by considering the residual normal displacement of the tunnel lining presented in Figure 11b. It should be mentioned that these residual bending moments are not so obvious when a low seismic signal is used.



a) Elastic soil constitutive model.



b) Mohr-Coulomb soil constitutive model.

Figure 9: ***Shear strain contour in case of using continuous lining at the time of 9.5 seconds.***

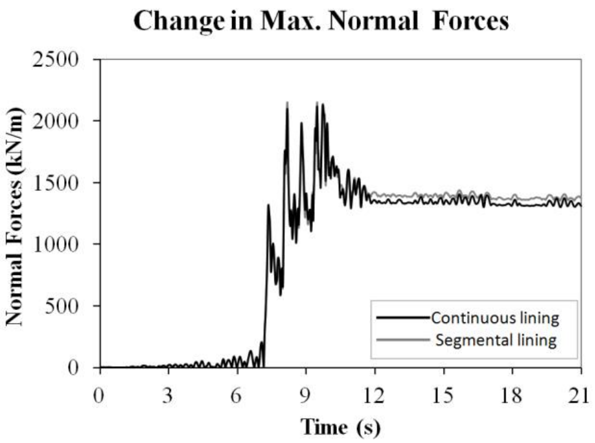
It is reasonable to conclude, on the basis of above analysis, that the effect of the soil constitutive model on the tunnel behaviour depends on the amplitude of the seismic excitation. In addition, the irreversible soil behaviour significantly modifies the tunnel loads both during the seismic excitation and, more importantly, after it. This finding is consistent with what was observed by Amorosi and Boldini [3].

Besides the soil constitutive model, the numerical results also introduce a significant influence of the segmental joints on the tunnel lining behaviour during seismic loading. Like the phenomena observed and explained in the case in which the low seismic signal presented in the above section is used, important decreases in the changes of the maximum absolute bending moment induced in a segmental lining, compared to the corresponding ones of a continuous lining, can be seen (see Figure 8a). The maximum differences in the changes in maximum absolute bending moment are 160%. The same result was found in the analytical analyses by Naggar *et al.* [37]. It is necessary to note that segmental joints perform like yield joints in a concrete lining during the impact of high seismic loads and the concrete segments continue to behave like elastic material. This means that the elastic material assumption applied to the concrete lining mentioned in section 2 can be considered for segmental linings. However, the assumption of elastic material for a continuous lining does not lead to the appearance of yield joints in the lining structure. Consequently, adopting elasticity in a constitutive model for a continuous lining is not realistic. This assumption is too simple and it also represents a limitation of the present study.

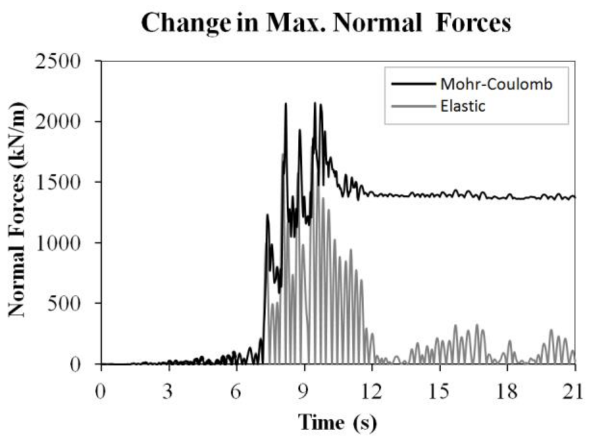
After the end of seismic excitation, residual maximum absolute bending moments exist in both continuous and segmental linings. It should be mentioned that the residual absolute bending moment ratio in a segmental lining is usually higher than that of a continuous lining. For instance, the residual maximum absolute bending moments of a segmental lining and a continuous lining are 57% and 45%, respectively (Figure 8a). This could be attributed to the non-linear (bilinear) rotational stiffness behaviour of the joints that have been applied in this study.

Figure 10 presents the effects of segmental joints and a soil constitutive model on the change in maximum normal forces during seismic excitation. It can be seen in Figure 10a that segmental joints have a negligible influence on the normal forces, in particular at the beginning when the seismic amplitude is small and seismic excitation increases. However, when seismic excitation gradually decreases, the decreases in the residual maximum normal forces induced in continuous and segmental linings are not similar. Consequently, the residual maximum normal forces in a segmental lining are about 5% higher than that of a continuous lining. This difference can also be explained by the nonlinear behaviour of the segmental joints during seismic loads. A limited influence of the segmental joints on the induced normal forces was also observed in the analytical analyses performed by Naggar *et al.* [37].

Like the bending moment, the normal forces are affected to a large extent by the soil constitutive model during seismic excitation. The maximum change in the maximum normal forces in the case in which Mohr-Coulomb model is used is about 19% higher than that determined in the case when the elastic soil model is used. This could be attributed to the fact that the progressive plastification of the soil during seismic excitation increases the load transmitted to the tunnel lining [21]. The same tendency has been observed in the work by Sliteen [49] and Sliteen *et al.* [50]. At the end of the seismic excitation, the change in the normal forces induced in the tunnel lining when elastic material is used decrease to zero.



a) Influence of segmental joint when the Mohr-Coulomb constitutive model is used.



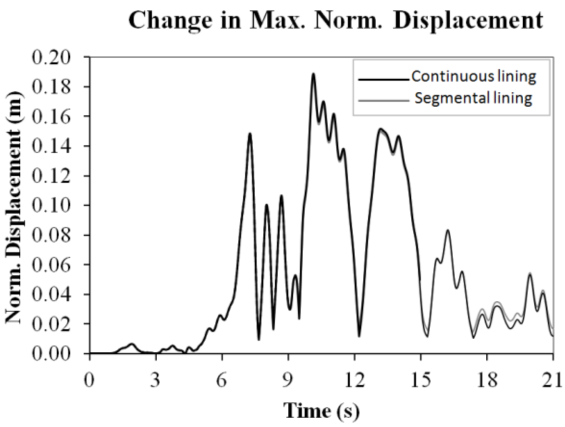
b) Influence of the soil constitutive model when a segmental lining is used.

Figure 10: Change in maximum normal forces.

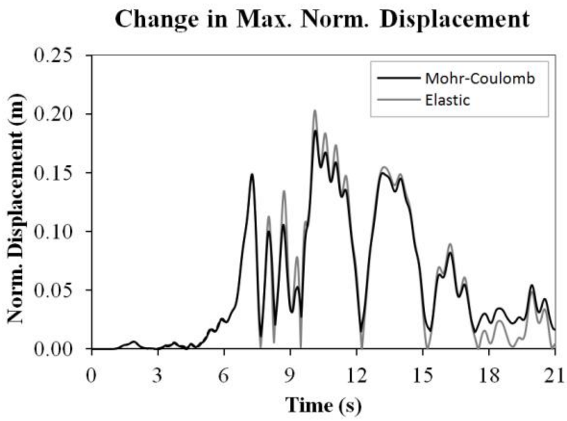
Nevertheless, a change in the residual value of the normal forces of about 61 % can be observed in Figure 10b for the Mohr-Coulomb. This phenomenon might not be seen clearly in the case in which a low seismic signal is used.

As explained in section 3.1 concerning the negligible effect of deformation absorbability of the segmental joints on the overall tunnel lining deformation, Figure 11a shows an insignificant difference in the change in the maximum normal displacement induced in the tunnel lining when both a continuous lining and a segmental lining are used. On the other hand, the maximum change in normal displacement observed in the case in which the Mohr-Coulomb constitutive model is used is about 9% smaller than that obtained when elastic material is used (Figure 11b). A larger residual value of normal displacement can be seen when the Mohr-Coulomb model is applied due to the effect of irreversible deformation.

Figure 12 illustrates an insignificant effect of the joints on the surface settlement. The results indicate that an elastic analysis is inadequate in determining the induced settlement on the ground surface. The values are almost zero (around 2 mm), which is not realistic (Table 3). The same conclusion was also reached in the work by Sliteen *et al.* [50].



a) Influence of segmental joints when the Mohr-Coulomb constitutive model is used.



b) Influence of the soil constitutive model when a segmental lining is used.

Figure 11: Change in normal displacement.

4. COMPARISON WITH SIMPLIFIED METHODS

Due to the complexity and high computational costs of a full dynamic numerical analysis, it is often preferred to use simplified analytical solutions and/or pseudo-static numerical methods to study the seismic response of tunnels.

A pseudo-static numerical analysis usually approximates the seismic-induced inertia forces as a constant horizontal body force applied through a mesh (Debiasi *et al*. [15]). Horizontal accelerations that correspond to the peak acceleration obtained in the cases in which low and high seismic signals are used (Figure 4) have been applied in the present study. The vertical and horizontal displacements were restricted along the base of the model during the application of horizontal acceleration while the horizontal displacements were left free along both sides of the model.

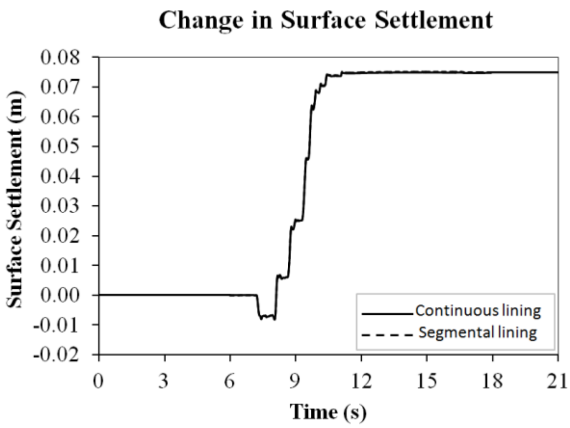


Figure 12: Change in surface settlement - Influence of segmental joints when the Mohr-Coulomb constitutive model is used.

Besides the horizontal acceleration method, Kramer *et al.* [31], Kontoe *et al.* [30], Sederat *et al.* [47] used another pseudo-static method, in terms of ovaling deformations, in which seismic loads are imposed as inverted triangular displacements along the lateral boundaries of the model and uniform lateral displacements are imposed along the top boundary. The magnitude of the prescribed displacement at the top of the model is related to the maximum shear strain γ_{max} and to the height of the model, in which the maximum shear strain γ_{max} is equal to the maximum shear strain determined through the horizontal-acceleration method.

Table 3. Maximum changes in structural forces, lining deformation and surface settlement (high signal case)

	Elastic model		*Mohr-Coulomb model*	
	Cont. lining	Seg. lining	Cont. lining	Seg. lining
Change in max. absolute M (kNm/m)	277.8	242.2	970.3	605.6
Change in max. N (kN/m)	1811.4	1796.6	2133.1	2154.4
Change in max. Normal disp (mm)	203.4	203.4	189.1	186.2
Change in max. Settlement (mm)	2	2	74.91	75.12

4.1 Validation of the pseudo-static models

The parameter set that has been used here was taken from the work performed by Hashash *et al.* [23]. Comparisons of the results obtained using pseudo-static methods with those of Wang's well-known analytical method [52] have been made and are presented in Table 4. All of the calculations have been performed using the assumptions of Wang's solution.

Wang's solution is considered by many authors, e.g. [23], as a realistic way of estimating structural forces induced in a tunnel lining. It should be noted that Wang's well-known analytical method and the prescribed shear strain method are built on the basis of the same assumptions. Unlike Wang's solution, the prescribed shear strain and horizontal acceleration methods both allow one to take the effect of gravity and the soil-structure interface contact into consideration.

It can be seen that both pseudo-static numerical models result in normal forces and in bending moments which are in good agreement with the ones obtained using Wang's well-known analytical solution. The difference between the results of horizontal acceleration method and those of the prescribed shear strain method could be attributed to the difference in shear displacements applied to the tunnel structure in each case, as illustrated in Figure 13.

Due to the fact that three above methods give more or less similar results, only the horizontal acceleration method will be used in the next section for comparison with the results of full dynamic analyses.

Table 4. Summary of pseudo-static methods

	Full Slip			No Slip		
	Wang	Pre. Shear Strain	Hor. Acceleration	Wang	Pre. Shear Strain	Hor. Acceleration
Max. Normal Forces (kN/m)	62.9*	78.7	77.6	1045.4*	1036.8	1154.2
Ratio N (%)	100.0	125.1	123.3	100.0	99.2	110.4
Max. Bending Moment (kNm/m)	188.8*	184.1	201.5	-	156.0	169.1
Ratio M (%)	100.0	97.5	106.7		100	108.4

Note: * Parameter from Table 3 in Hashash *et al*. [22]

Table 5. Summary of pseudo-static and full seismic analyses (no slip condition)

	Low seismic signal				High seismic signal			
	Continuous lining		Segmental lining		Continuous lining		Segmental lining	
	Pseudo-static	Full dynamic	Pseudo-static	Full dynamic	Pseudo-static	Full dynamic	Pseudo-static	Full dynamic
Max. Normal Forces (kN/m)	5.99	5.55	5.85	5.49	703.8	1811.4	703.7	1796.6
Max. Bending moment (kNm/m)	1.53	1.32	1.35	0.75	106.0	277.8	97.0	219.7

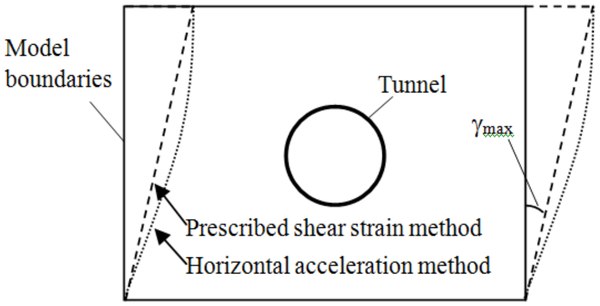


Figure 13: Comparison of shear displacements.

4.2 Comparison between pseudo-static analysis and full dynamic analysis

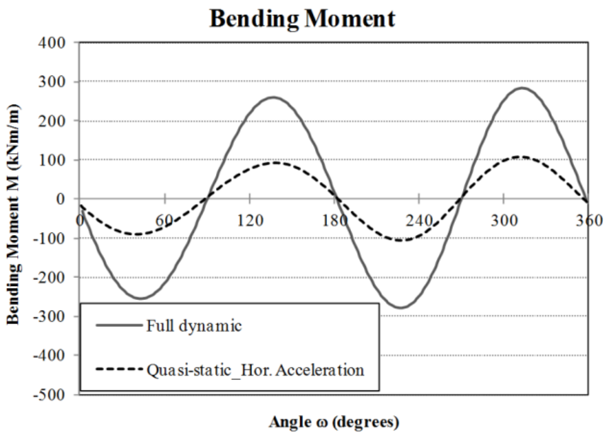
Table 5 illustrates the maximum normal forces and the bending moment induced in a tunnel lining determined using pseudo-static and full seismic analyses. All the calculations have been performed using elastic soil material and applying the no-slip condition at the soil-tunnel interface. It can be seen that the pseudo-static analysis gives results that are in good agreement with those predicted with a full seismic analysis, when a low seismic excitation is considered. However, pseudo-static analysis is inadequate, under the impact of a high seismic excitation, in determining the normal forces and bending moment induced in a tunnel lining (Figure 14 and Table 5). It has been suggested that an equivalent static solution could yield smaller structural lining forces than those of a true dynamic solution. This finding is consistent with the results of Lee *et al.* [34], who pointed out that the bending moment in a segmental lining is about 0.3-0.7 times smaller in a pseudo-dynamic method than in a full dynamic analysis. Similarly, Akhlaghi and Nikkar [2] indicated the maximum values of bending moment and normal forces induced in a circular tunnel determined using analytical method (Wang [52]) are 0.7 and 0.65 times, respectively, smaller than those estimated using numerical analysis. The same conclusion can also be found in the literature, e.g. [26,46].

5. CONCLUSIONS

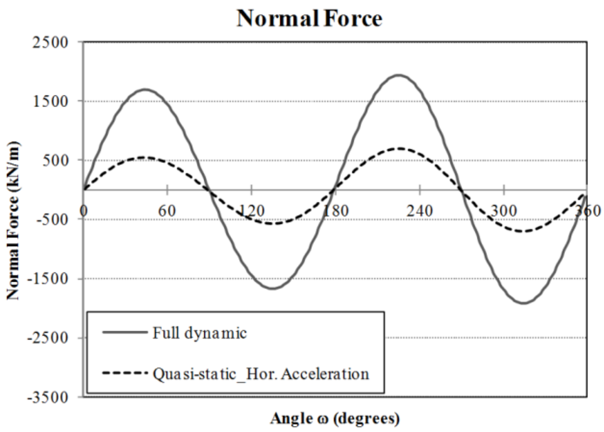
This paper presents a numerical study that investigates the performance of segmental tunnel linings under seismic loads. Analyses have been carried out using a 2D finite difference element model. Two typical seismic signals, obtained using Nice earthquake data, which correspond to a low seismic signal and a high seismic signal, have been adopted as input.

Several conclusions can be drawn from the results of the present study:

- A segmental lining can perform better than a continuous lining during an earthquake. The maximum bending moment induced in a segmental lining is significantly smaller than that developed in a continuous lining;
- Although there are differences between the behaviour of a segmental lining and that of a continuous lining, the effect of the joints under a low seismic excitation could be neglected;
- The effect of the soil constitutive model on the tunnel behaviour depends to a great extent on the amplitude of the seismic excitation and it could be neglected under low seismic excitation. However, this effect must be taken into consideration under a high seismic excitation;
- The irreversible behaviour of the soil significantly modifies the tunnel loads both during seismic excitation and, more importantly, after it. Significant residual structural lining forces should be predicted when plasticity of the soil is taken into account;
- An elastic analysis is not sufficient to determine the seismic induced response of a soil-tunnel system;
- An equivalent static solution would yield smaller structural lining forces than those of a true dynamic solution.



a) Bending moment



b) Normal forces

Figure 14: Comparison between pseudo-static analysis and full dynamic analysis (high seismic signal case).

Due to time calculation, this study has been limited to a linear elastic perfectly plastic constitutive model for the soil behaviour. However, soils do not have this simple type of behaviour under dynamic loads. So the use of a more complex constitutive model is necessary and will probably induce different amplitudes of soil settlements. It should also be mentioned that the presented results are limited to a single geometric configuration and a single and homogeneous soil medium. A more extensive study should be done to generalize the given results.

Further comparisons with experimental data, obtained from real tunnel excavations, should be made in order to improve the quality of the numerical simulation.

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