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Title:

Modelling the transverse behaviour of circular tunnels in structured clayey soils during earthquakes

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Abstract

The paper presents novel results from advanced numerical simulations of the transverse behaviour of shallow circular tunnels in natural clays accounting for soil structure degradation induced by earthquake loading. It combines the calibration of a kinematic hardening model against real laboratory data with outputs from a parametric study with different degradation rates of soil structure to demonstrate the good performance of the model and draw conclusions of significance to both researchers and designers. The sensitivity analysis indicates an increase in the maximum and minimum values of the lining forces attained during the earthquake motions and in the lining hoop force and bending moment increments in response to the seismic events when higher rates of destructuration are accounted for. Hence, the paper highlights for the first time the importance of the initial structure and its degradation in controlling the magnitude of the tunnel lining forces and, consequently, the overall seismic tunnel design.

Keywords:

Tunnels; Earthquakes; Finite element method; Constitutive models; Natural clays; Destructuration.

1 **1. Introduction**

2 The urban population is predicted to rise to more than 6 billion by 2045, with the largest growth
3 occurring in countries of known seismicity, such as China, India and Indonesia [1]. The need for
4 space within cities with growing population has recently seen an increase in urban regeneration
5 projects as well as construction of new underground services and transport links. In this context,
6 urban tunnels are considered “lifeline” utilities as their continued operation is of vital importance
7 during and in the immediate aftermath of an earthquake. It is, therefore, imperative to assess the
8 engineering performance of such important geotechnical structures to ensure their resilience during
9 and after seismic events.

10 Failures of geotechnical structures due to earthquake events, with huge consequences in terms of
11 fatalities and financial costs, have been widely documented in recent years. These failures are often
12 associated with significant deformation of soil deposits which can cause major damage to buildings
13 and surface infrastructure facilities. On the contrary, dynamic effects on underground structures
14 have often been neglected based on the assumption that their response to earthquake loading is
15 relatively safe. Nevertheless, several examples of recorded damage to underground structures for
16 which seismic forces were not considered in the original design can be found in the literature.
17 Hashash *et al.* [2] described the collapse of the Daikai subway station in Kobe during the 1995
18 Hyogoken-Nambu earthquake, the damages to highway tunnels in Central Taiwan during the 1999
19 Chi-Chi earthquake and the collapse of the Bolu tunnel in Turkey during the 1999 Düzce
20 earthquake. Twelve per cent of the mountain tunnels in the epicentral area were heavily damaged
21 during the Kobe earthquake [3], while after the Chi-Chi earthquake 26% of the 50 tunnels located
22 within 25 km of the rupture zone were severely damaged and 22% moderately damaged [4]. More
23 recently, Li [5] reported the investigation of seismic damages to 11 highway tunnels in the Yingxiu
24 Town area during the Wenchuan earthquake occurred on May 2008: 4 were seriously damaged, 3
25 moderately damaged and 4 slightly damaged. The main causes of damage in these case histories

1 were the shallow depth of tunnels, the poor geological conditions (i.e. soft soils with high plasticity,
2 weak rocks), the displacement of active faults crossing the tunnel and pre-existing structural defects
3 in the tunnel lining. During a seismic event, tunnels are subjected to axial compression and
4 extension, longitudinal bending and ovaling or racking of the tunnel lining [6]. Underground
5 facilities constructed in soft soils or weak rocks, such as urban tunnels, can be expected to suffer
6 more damage compared to openings constructed in competent rocks [2]. Ovaling and racking of the
7 tunnel lining are reported to be the most critical sources of damage [7], although longitudinal effects
8 may have a significant impact on the response of long underground structures [2, 8].

9 The seismic design of tunnels has been addressed in the past by a number of researchers who have
10 proposed solutions based on analytical or numerical investigations. The analytical design methods
11 typically rely on elasticity solutions to calculate the dynamic lining forces a tunnel experiences
12 during an earthquake event and generally ignore the inertial effects [e.g. 9-10]. The Soil-Structure
13 Interaction (SSI) approach has, instead, the ability to consider relatively complex conditions in
14 terms of heterogeneity of soil strata, non-regularity of tunnel geometry, pre-existence of surface and
15 sub-surface structures, ground water flow. In such cases, the analysis of SSI can take advantage of
16 the use of numerical two-dimensional (2D) and three-dimensional (3D) approaches, like the
17 boundary element (BE) method and the finite element (FE) method. Most commonly, the problem
18 is modelled in the transverse direction only, assuming plane strain conditions [e.g. 11-17].
19 However, when the direction of wave propagation is arbitrary with respect to the axis of the
20 structure the problem becomes three-dimensional and few contributions studying this aspect can be
21 found in literature [e.g. 18-22]. The capabilities of such numerical approaches in assessing tunnels
22 behaviour under seismic loading have not been fully exploited, as they require the use and
23 calibration of advanced constitutive models to appropriately describe the soil stress-strain behaviour
24 during the dynamic action. In particular, the evolution of microstructure (destruction) induced in
25 natural soil deposits by the seismic action, and its effect on the soil-tunnel dynamic behaviour has

1 never been investigated before.

2 In this paper the dynamic performance of a shallow circular tunnel in a natural clay deposit is
3 analysed by means of a 2D non-linear FE approach. The calibration of an advanced kinematic
4 hardening multi-surface soil model against real laboratory data is firstly presented. Different rates of
5 destructuration of the soil initial structure are assumed in the calibration phase to investigate, for the
6 first time, the effects induced by structure degradation on the soil-tunnel behaviour in dynamic
7 conditions. The geometrical and geotechnical properties of the numerical model are then discussed
8 along with the selection strategies for the definition of the bedrock input motions. The soil-tunnel
9 interaction during the seismic events is, in turn, described by presenting the profile of maximum
10 accelerations within the deposit, the stress-strain soil response, the structure degradation process
11 induced by the seismic motions and the evolution with time of the excess pore pressures throughout
12 the earthquake events. Finally, the distribution and time histories of the lining forces induced in the
13 tunnel by the input motions are presented for the different cases and some conclusions are drawn at
14 the end.

15

16 **2. Soil constitutive model**

17 To accurately predict the behaviour of tunnels during earthquakes through FE analyses, it is crucial
18 to use a constitutive model that can appropriately capture the soil response to seismic loads. In
19 common engineering practice and design, simple constitutive assumptions (e.g. Mohr-Coulomb) are
20 often employed to model the stress-strain behaviour of the soil deposit. However, these constitutive
21 hypotheses are not adequate to simulate the main features of the mechanical behaviour of soils
22 during cyclic loading such as non-linearity, early irreversibility, decrease of nominal stiffness,
23 hysteretic energy dissipation and pore pressure build-up in undrained conditions. Ignoring these
24 effects can lead to an incorrect prediction of the ground deformations, especially in soft soil
25 deposits subjected to dynamic loading, thus resulting in an inappropriate design of the tunnel lining.

1 In this work the kinematic hardening model (*RMW*) developed by Rouainia and Muir Wood [23]
2 has been employed to simulate the cyclic response of natural clay materials. It contains three
3 surfaces: 1) a reference surface, which controls the state of the soil in its reconstituted, structureless
4 form and describes the intrinsic behaviour of the clay; 2) a structure surface, which controls the
5 process of destructuration; 3) a bubble, which encloses the elastic domain of the soil and moves
6 within the structure surface following a kinematic hardening rule. The degree of structure, r , which
7 describes the relative sizes of the structure and reference surfaces, is a monotonically decreasing
8 function of the plastic strain, thus representing the progressive degradation of the material. The
9 model converges to the Modified Cam-Clay model for remoulded structureless soils once the three
10 surfaces are set to be coincident. The *RMW* model has been successfully employed to simulate both
11 static [24-25] and dynamic geotechnical problems [26-27]. It has been implemented in PLAXIS 2D
12 [28] with an explicit stress integration algorithm adopting an automatic sub-stepping and error
13 control scheme [29]. The model has been calibrated against a series of laboratory results reported by
14 Burghignoli *et al.* [30], D'Elia [31] and Burghignoli *et al.* [32] for Avezzano Clay, a structured
15 Italian clay. The Avezzano deposit is in a highly seismic area of central Italy and could potentially
16 be the location of underground infrastructure and transport links in the future. Specifically, the site
17 considered is in the Fucino basin, a large intra-mountain depression located 80km east of Rome and
18 surrounded by the Apennines. The basin has originated from the sedimentation of fluvio-lacustrine
19 sediments during the Pleistocene period and is composed by top layers of clayey and silty soils,
20 with sand and gravel found underneath. The deposit is geologically normally consolidated and the
21 silty clay layers are characterised by very low plasticity (PI of about 10%) and high values of
22 calcium carbonate content (i.e. CaCO_3 content between 60 and 80%). Standard oedometer and
23 undrained triaxial compression tests clearly indicate that the clayey layers are characterised by the
24 typical mechanical behaviour of cemented clays [30, 32]. The response of the deposit in Avezzano
25 to cyclic loads imposed by a silo shallow foundation has been investigated in the past by

1 Burghignoli *et al.* [30], D’Elia *et al.* [33], Burghignoli *et al.* [34] and, more recently, Elia and
2 Rouainia [27] evaluated the performance of the same footing under seismic loading conditions. In
3 this work, the undrained triaxial compression tests on Avezzano Clay natural samples retrieved at
4 depths of 15 and 21m have been considered. Figure 1 shows the comparison in terms of stress
5 paths, stress-strain and pore pressure-strain response between the laboratory data from [32] and the
6 numerical predictions obtained with *RMW*. For the same set of experimental data shown in the
7 figure, the constitutive model has been calibrated following the work by [27, 34], considering an
8 initial degree of structure r_0 equal to 5.2 and two rates of destructuration with damage strain (i.e.
9 two values of the parameter k). The numerical prediction obtained assuming k equal to 1.5,
10 presented in Figure 1, is in very good agreement with the laboratory data, whereas the strain-
11 softening will be more abrupt than in the experiments when a higher rate of destructuration ($k = 5.0$)
12 is adopted. In previous versions of *RMW* a classical hypoelastic formulation was employed for the
13 determination of the bulk and shear moduli, K and G_0 . In this work, the well-known equation
14 proposed by Viggiani and Atkinson [35] for the small-strain shear modulus has been implemented
15 to reproduce the dependency of G_0 on the mean effective stress and overconsolidation ratio. It
16 should be noted that the adopted elastic formulation cannot predict the influence of structure on the
17 initial elastic stiffness, as recently proposed by Elia and Rouainia [36]. Consistently with the
18 Avezzano Clay plasticity index of 10%, the dimensionless stiffness parameters A , n and m in the
19 equation proposed by Viggiani and Atkinson [35] have been set equal to 2150, 0.78 and 0.22,
20 respectively. The small-strain stiffness response and the evolution of the shear modulus and
21 damping ratio of Avezzano Clay has been experimentally investigated and reported by D’Elia [31].
22 In particular, Figure 2 shows the normalised modulus decay and damping curves obtained from
23 double specimen direct simple shear (DSDSS) and combined resonant column/torsional shear
24 (RC/TS) tests performed on natural Avezzano Clay samples, which were retrieved from the top part
25 of the deposit (i.e. at depths between 8 and 11m). Numerical simulations of strain-controlled

1 undrained cyclic simple shear tests have been carried out in order to calibrate the *RMW* parameters,
2 which control the reduction of shear modulus and the evolution of the damping ratio with cyclic
3 shear strain [e.g. 36]. The model predictions, presented in the same Figure 2, are in good agreement
4 with the laboratory data when the lower rate of destructuration is accounted for. Associated with a
5 more pronounced strain-softening response, the normalised stiffness modulus for k equal to 5.0
6 decays quicker than that obtained assuming a destructuration rate equal to 1.5 over the entire strain
7 range and underestimates the experimental data. In contrast, a very small difference can be observed
8 in terms of hysteretic damping predicted by the model in the two cases.

9 A summary of the *RMW* model parameters resulting from the calibration and adopted in the FE
10 non-linear dynamic simulations undertaken in this work is reported in Table 1.

11

12 **3. Numerical model**

13 The case of a shallow circular tunnel, 10m in diameter with 15m soil cover, within a normally
14 consolidated 70m thick deposit of Avezzano Clay overlying a rigid bedrock has been considered,
15 using the soil model parameters derived from the above calibration procedure. The lining has been
16 assumed to be made by precast concrete segments 0.5m thick and it has been modelled as a linear
17 visco-elastic material with Young's modulus equal to 38GPa, Poisson's ratio equal to 0.25 and
18 damping ratio equal to 5%. Standard boundary conditions have been adopted for the static analyses,
19 while the bottom of the model has been assumed rigid and equal displacements have been imposed
20 to the nodes along the vertical sides of the mesh (i.e. tied-nodes lateral boundary conditions). Tied-
21 nodes have been employed for the dynamic simulations in order to avoid spurious wave reflections
22 at the boundaries of the soil deposit. Their effectiveness in absorbing the energy induced by the
23 seismic action has been proved by Zienkiewicz *et al.* [37]. In addition, a parametric study of the FE
24 model length has shown that the adopted horizontal dimension (i.e. 350m), equal to 5 times the
25 deposit depth as suggested by Amorosi *et al.* [38], in conjunction with the tied-node boundaries, is

1 sufficient to properly simulate the free-field conditions at the edges of the model. The water level
2 has been assumed to coincide with the ground surface. Figure 3 shows the geometry of the FE
3 model adopted in PLAXIS 2D [28] along with the dynamic boundary conditions. As in previous
4 works [27, 34], a coefficient of earth pressure at rest K_0 equal to 0.5 has been assumed in the
5 generation of the geostatic stress state, using a unit weight of 18kN/m^3 for the clay. The G_0 profile
6 resulting from the equation proposed by Viggiani and Atkinson [35] is shown in the same figure
7 and compared with RC data on Avezzano Clay presented by Burghignoli *et al.* [34]. The FE model
8 has been discretised with a total number of 5088 15-noded plane strain triangular elements, with the
9 mesh refined in the region around the tunnel to avoid mesh sensitivity due to the softening
10 behaviour predicted by the *RMW* model. To ensure that the seismic wave transmission is
11 represented accurately through the finite element mesh, the vertical distance between adjacent
12 element nodes has been limited to satisfy the condition recommended by Kuhlemeyer and Lysmer
13 [39], even when the reduction of the initial shear modulus during the dynamic excitation produces a
14 reduction in the wave length. A static analysis under undrained conditions has been initially
15 performed to simulate the tunnel excavation and installation of the tunnel lining. A contraction
16 equivalent to 0.8% volume loss, deemed to be acceptable for a satisfactory performance of the
17 tunnel excavation [e.g. 40], has been imposed. The dynamic analyses have been carried out under
18 undrained conditions with a time step corresponding to that of the earthquake input signals. In the
19 dynamic simulations only 2% Rayleigh damping has been added to avoid the propagation of
20 spurious high frequencies and to compensate for the *RMW* underestimation of damping in the small-
21 strain range, using the calibration procedure proposed by Amorosi *et al.* [38].

22 Two earthquake signals, both reasonably matching the response spectrum provided by Eurocode 8
23 (EC8) for soil type A, have been considered in the dynamic simulations, as illustrated in Figure 4.
24 The first signal was recorded at the Assisi-Stallone station during the Umbria-Marche earthquake in
25 September 1997, while the second was recorded at the Ulcinj-Hotel Albatros station during the

1 Montenegro earthquake in April 1979. The Montenegro earthquake has been selected to explicitly
2 match EC8 for the first natural period of the soil deposit, T_1 (equal to 1.03s) calculated according to
3 the viscoelasticity theory [e.g. 41]. This value represents only an initial guess of the first oscillation
4 mode of the soil deposit and it is used to guide the selection process of the input motion and the
5 interpretation of the non-linear dynamic analyses presented in the following. Actually, from Figure
6 4 it is evident that the Montenegro signal is characterised by much higher spectral accelerations at
7 around T_1 with respect to the Umbria-Marche earthquake, thus implying a higher energy content
8 applied to the system at its first oscillation mode. The relevant characteristics of the selected
9 acceleration time histories are listed in Table 2 in terms of magnitude (M_W), Arias Intensity (I_a) as
10 proposed by Arias [42], epicentral distance, effective duration (T_{90}) as defined by Trifunac and
11 Brady [43], maximum acceleration (a_{max}) and maximum velocity (v_{max}). Both input motions have
12 been filtered to prevent frequencies higher than 10Hz and scaled up to the same peak acceleration
13 (PGA) of 0.30g, according to the seismic hazard analysis of the site presented by Elia and Rouainia
14 [27]. They have been applied at the base of the mesh as prescribed horizontal displacement time
15 histories.

16

17 **4. Results and discussion**

18 In this section, the soil-tunnel interaction response during the selected earthquake events has been
19 systematically investigated to highlight the influence of the initial degree of structure and its
20 subsequent degradation induced by the seismic loads. Specifically, two sets of simulations have
21 been considered and compared: the ones performed applying the two input motions and using a rate
22 of destructuration equal to 1.5 and those in which the *RMW* parameter k has been set equal to 5.0. In
23 the following, the results of the dynamic analyses are presented in terms of propagation of the
24 seismic waves within the deposit, mechanical response of the soil surrounding the tunnel and
25 distribution and time histories of the lining forces induced in the tunnel by the earthquake actions.

1 The profiles of maximum horizontal acceleration recorded along the tunnel vertical and in free field
2 conditions (with location shown in Figure 3) during the two selected earthquake events are
3 presented in Figure 5. For both seismic events and along both verticals, the FE analyses predict an
4 overall deamplification of the bedrock motion at surface. It also seems that the degradation rate
5 does not particularly influence the wave propagation in the deposit. Moreover, it appears that the
6 Montenegro input motion induces higher accelerations, especially in the bottom part of the deposit
7 and below the tunnel, but the PGA at surface is similar to the one recorded during the Umbria-
8 Marche event. These findings are confirmed by the response spectra of the accelerations recorded at
9 ground surface along the tunnel vertical and in free field conditions shown in Figure 6 and
10 compared with the response spectra of the input motions applied at bedrock. In general, given the
11 higher energy content of the Montenegro event at T_1 (see Figure 4) significantly higher shear strain
12 levels are induced in the deposit than those by the Umbria-Marche seismic motion.

13 The stress-strain curves recorded in free field conditions during the two seismic events at different
14 depths (i.e. at 10, 15, 25 and 50 m from the ground surface) and assuming two rates of
15 destructuration are presented in Figure 7. The shear strains induced by the Montenegro earthquake,
16 ranging between about 0.5% at the surface and 0.25% at depth, are at least double the
17 corresponding strains associated to the Umbria-Marche event. The figure also shows a softer
18 behaviour of the soil when the rate of destructuration assumed in the simulations is higher (i.e. for k
19 = 5.0), consistently with the normalised stiffness modulus curves presented in Figure 2.

20 For the case of $k = 1.5$, the evolution with time of the shear strains recorded during the two seismic
21 events around the tunnel is shown in Figures 8a and 8d as function of the angle θ (defined positive
22 in the anti-clockwise direction). The non-symmetric response observed in terms of shear strains is
23 the counterpart of an elasto-plastic behaviour induced in the soil deposit by the dynamic excitation.

24 In addition, Figures 8b and 8e show the time histories of excess pore water pressures predicted
25 along the tunnel vertical for the Umbria-Marche and Montenegro events, respectively. The FE non-

1 linear analyses predict the build-up of positive pore pressures above the tunnel and negative
2 pressures below it, associated with the accumulation of permanent soil deformations and structure
3 degradation throughout the two earthquake motions. Higher excess pore pressures are recorded
4 during the Montenegro event, ranging between 100kPa at 25m depth and -27kPa at the bottom of
5 the mesh. The evolution with time of the *RMW* parameter r , describing the degree of soil structure,
6 during the two seismic motions is shown in Figures 8c and 8f for three different locations around
7 the tunnel. More pronounced structure degradation is induced in the soil surrounding the tunnel by
8 the Montenegro seismic event, consistently with the higher shear strain levels and excess pore
9 pressures recorded during this earthquake in the same locations. Figure 9 shows the contours of r
10 obtained at the end of the two seismic events, indicating a more diffused destructuration occurred in
11 the soil deposit during the Montenegro analysis compared to the Umbria-Marche case. Moreover, in
12 both cases higher destructuration is observed in the top part of the model, between the tunnel and
13 ground surface, where the deamplification of the input signal occurs (see Figure 5).

14 Figure 10 presents the time histories of shear strain, excess pore pressure and *RMW* structure
15 parameter r during the two earthquake events when a destructuration rate of 5.0 is adopted in the
16 simulations. The higher degree of destructuration allowed in the simulations leads to the
17 development of higher shear strains around the tunnel (Figures 10a and 10d) and the consistent
18 accumulation of positive pore water pressures in the soil deposit (Figures 10b and 10e), reaching a
19 maximum value of excess pressure almost equal to 200kPa in the Montenegro analysis. Almost a
20 full structure degradation is observed at $\theta = 90^\circ$ when the Montenegro input motion is applied at
21 bedrock (i.e. r approaches a final value of 1.0), as indicated by Figure 10f. This is confirmed by the
22 contours of r obtained at the end of the two seismic events, shown in Figure 11, where full
23 destructuration of the top 5m of the soil deposit can be observed in the Montenegro case.

24 Moving to the tunnel dynamic behaviour, Figure 12 shows the distribution of hoop force N , bending
25 moment M and shear force Q before and after the seismic events as function of the angle θ for a

1 destructuration rate $k = 1.5$. The standard convention of structural analysis (i.e. compression is
2 negative) is adopted here when presenting the results in terms of lining forces. The envelopes of
3 maximum and minimum values of N , M and Q during the earthquake events are also shown in the
4 same figure with dashed lines. For both input motions, the use of elasto-plastic models allows to
5 predict permanent increments of hoop force (ΔN), bending moment (ΔM) and shear force (ΔQ) at
6 the end of the motions due to the accumulation of plastic deformation in the soil deposit during the
7 earthquakes, as already observed by other researchers [14, 16, 17]. The lining forces predicted at the
8 end of the Montenegro analyses (Figures 12d - 12f) are larger than those recorded at the end of the
9 Umbria-Marche simulations (Figures 12a - 12c). This can be attributed to the higher accelerations
10 induced by the Montenegro signal at tunnel location (see Figures 5a and 5b). The corresponding
11 distribution of lining forces obtained for a destructuration rate equal to 5.0 are reported in Figure 13.
12 With respect to the case of $k = 1.5$, higher hoop forces, bending moments and shear forces are
13 induced in the tunnel, especially by the Montenegro earthquake, when a higher degree of
14 destructuration is allowed to occur in the dynamic analyses. Finally, Figure 14 shows, for the two
15 destructuration rates assumed, the time histories of hoop force and bending moment increments
16 during the Umbria-Marche event at θ equal to 0° , 45° , 90° and 135° , whereas the evolution with
17 time of ΔN and ΔM predicted at the same lining locations throughout the Montenegro event is
18 reported in Figure 15. The sensitivity analysis indicates that the assumption of a higher
19 destructuration rate can significantly affect the lining forces, causing a consistent increase of hoop
20 force and bending moment increments accumulated in the tunnel lining during both earthquake
21 scenarios. This rise is due to the softer response of the soil characterised by a higher rate of
22 destructuration (Figure 2), which, in turn, causes the transmission of higher loads to the tunnel
23 lining.

24 Overall, the simulation results highlight the importance of the input motion frequency content in
25 controlling the magnitude of shear strains induced by the earthquake in the tunnel and surrounding

1 deposit. The parametric analysis points out a consistent increase in the hoop force and bending
2 moments increments accumulated in the tunnel lining when a higher degree of destructuration is
3 allowed to occur in the dynamic simulations.

4

5 **5. Conclusion**

6 The paper describes a set of non-linear FE analyses for the simulation of the transverse behaviour of
7 a shallow circular tunnel in a typical natural clay deposit subjected to earthquake loading. Starting
8 from the same set of laboratory test results on a real natural clay, different assumptions on the
9 degradation rate of the initial soil structure are made in the calibration of an advanced constitutive
10 model used to describe the clay dynamic behaviour. Two input motions, scaled to the same peak
11 acceleration but characterised by a different frequency content, are applied at the bedrock of the FE
12 model. The principal findings of the sensitivity analysis are:

- 13 • Simply scaling the ground motion records at the same peak acceleration can induce very
14 different levels of shear strain in the deposit, thus affecting the propagation of the
15 accelerations in the soil and, consequently, the forces in the tunnel lining. This is due to the
16 importance of the spectral shape of the input motion in non-linear soil response, as PGA is
17 not a good indicator of the strength and frequency content of the seismic motion.
- 18 • The implementation of elasto-plastic models allows to predict permanent increments of
19 hoop force, bending moment and shear force at the end of the motions due to the
20 accumulation of plastic deformation in the soil deposit during the earthquakes.
- 21 • The soil structure degradation rate does not particularly affect the wave propagation in the
22 deposit, but leads to higher shear strain levels in the soil deposit due to its softer behaviour.
- 23 • A rise in the maximum and minimum values of the lining forces attained during the
24 earthquake motions is predicted when higher rates of soil structure degradation are
25 accounted for.

- When a higher degree of destructureation is allowed to occur in the dynamic simulations, a consistent increase in the hoop force and bending moment increments accumulated in the tunnel lining can be observed.

Although a natural clay deposit is characterised by high stiffness and peak strength due to its initial degree of structure, the earthquake loading can induce sufficient stiffness degradation in the soil associated to strain-softening processes, which, in turn, facilitate the transmission of higher loads to the tunnel lining. Therefore, the paper highlights for the first time the importance of considering structure degradation in the assessment of the dynamic response of shallow tunnels constructed in structured clayey deposits as it can significantly control the magnitude of the tunnel lining forces and, in consequence, the overall tunnel design.

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Parameter/ symbol	Physical contribution/meaning	Avezzano Clay
M	Critical state stress ratio for triaxial compression	1.42
λ^*	Slope of normal compression line in $\ln v - \ln p$ compression plane	0.11
κ^*	Slope of swelling line in $\ln v - \ln p$ compression plane	0.016
R	Ratio of size of bubble and reference surface	0.4
B	Stiffness interpolation parameter	15.0
ψ	Stiffness interpolation exponent	1.45
η_0	Anisotropy of initial structure	0.0
r_0	Initial degree of structure	5.2
A^*	Parameter controlling relative proportion of distortional and volumetric destructuration	0.2
k	Parameter controlling rate of destructuration with damage strain	1.5 / 5.0

Table 1. *RMW* parameters

Station	Earthquake	Component	Magnitude (M_w)	Arias intensity I_a (m/s)	Epicentral distance (km)	Duration T_{90} (s)	a_{\max} (g)	v_{\max} (m/s)
Assisi-Stallone	Umbria-Marche (1997)	EW	6.0	0.2793	21.6	5.98	0.188	0.102
Ulcinj-Hotel Albatros	Montenegro (1979)	NS	6.9	0.7289	19.7	12.22	0.181	0.176

Table 2. Main characteristics of the selected earthquake signals

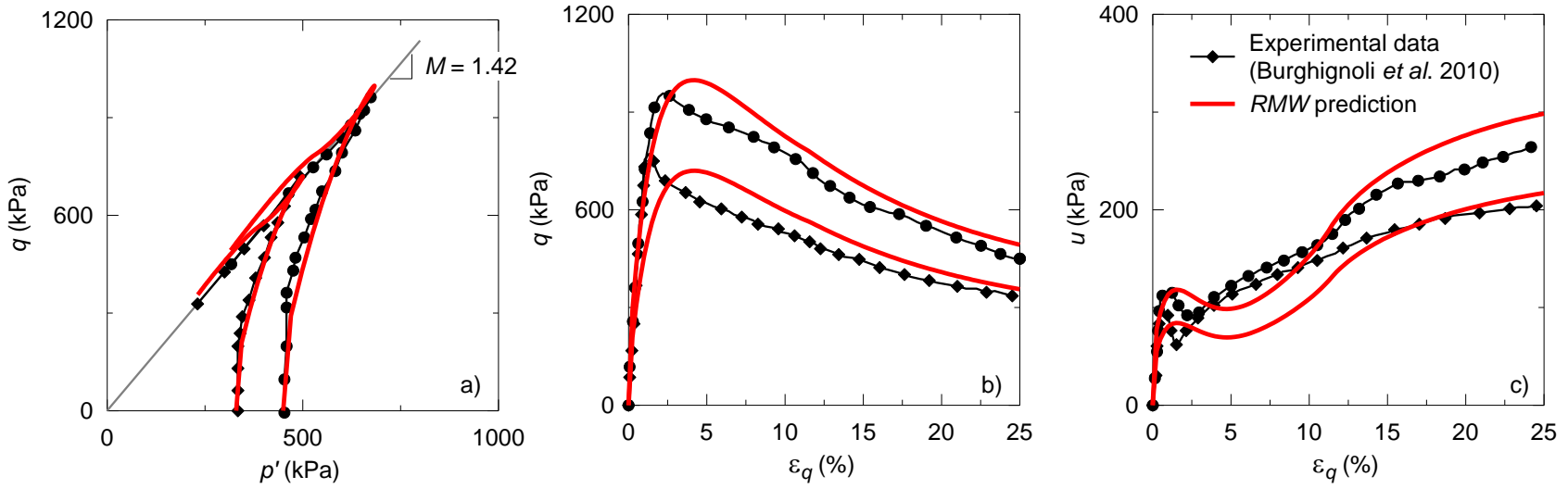


Figure 1. Comparison between *RMW* predictions and laboratory data on Avezzano Clay: (a) stress path; (b) stress-strain response; (c) pore pressure-strain response.

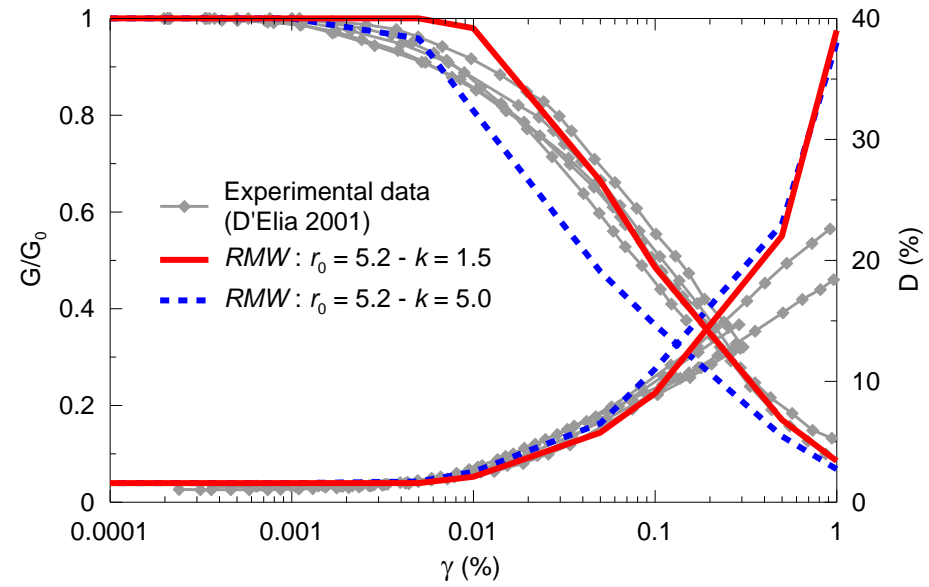


Figure 2. Comparison between *RMW* predictions obtained for two different destructuration rates (i.e. k values) and normalised modulus decay and damping curves for Avezzano Clay.

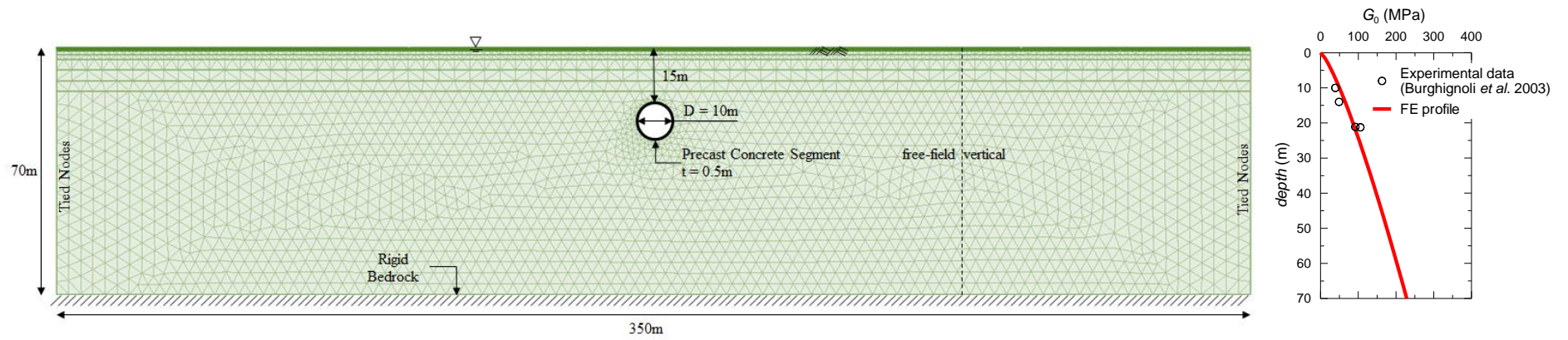


Figure 3. Adopted FE model and boundary conditions for the dynamic simulations.

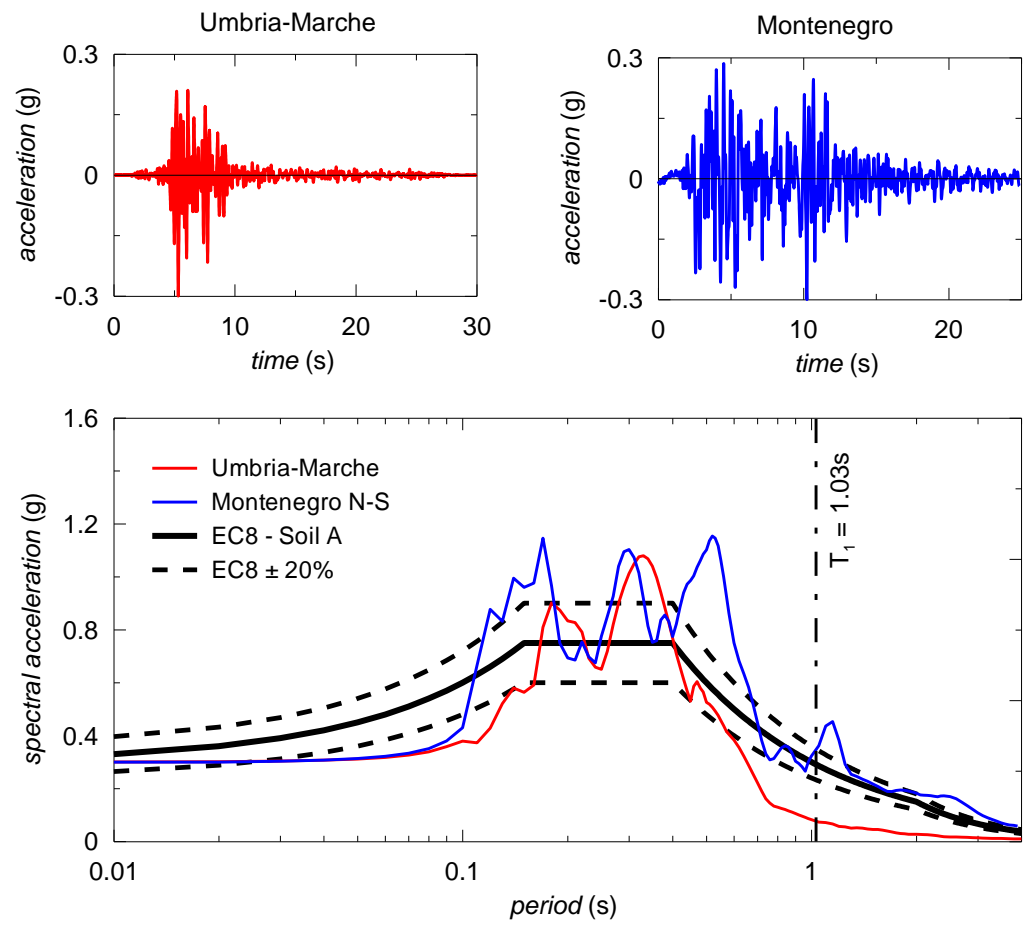


Figure 4. Input motion acceleration time histories and response spectra.

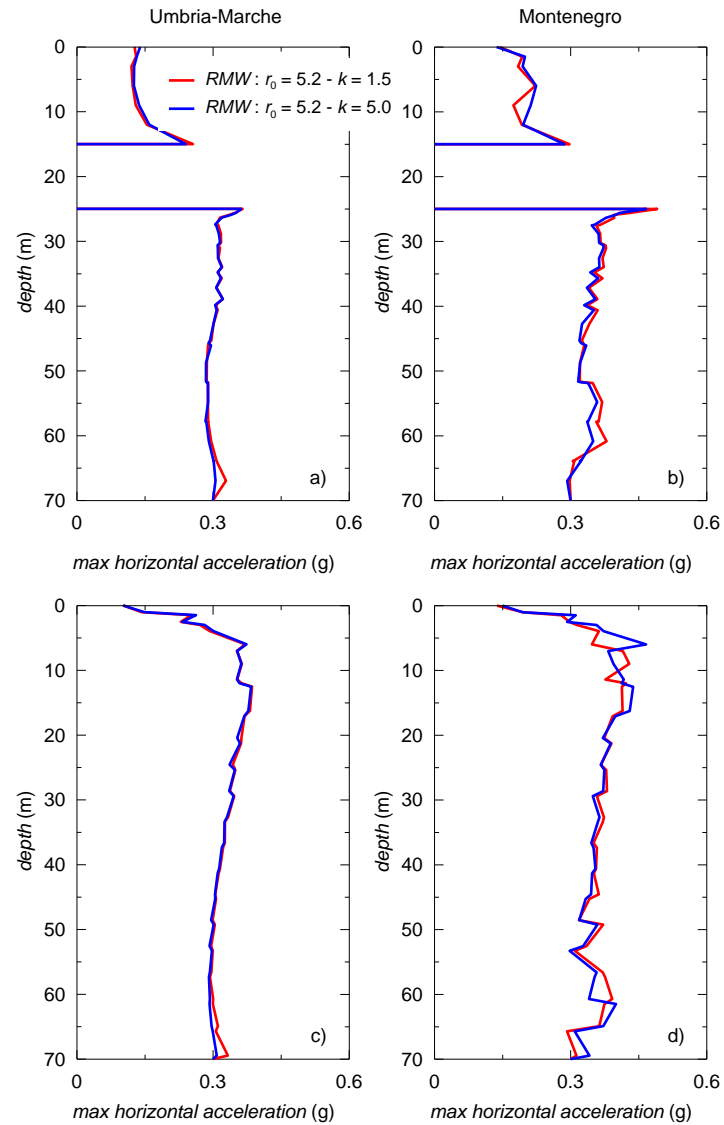


Figure 5. Influence of structure degradation rate on the profiles of max accelerations recorded along the tunnel vertical and in free field conditions during the: (a-c) Umbria-Marche; (b-d) Montenegro event.

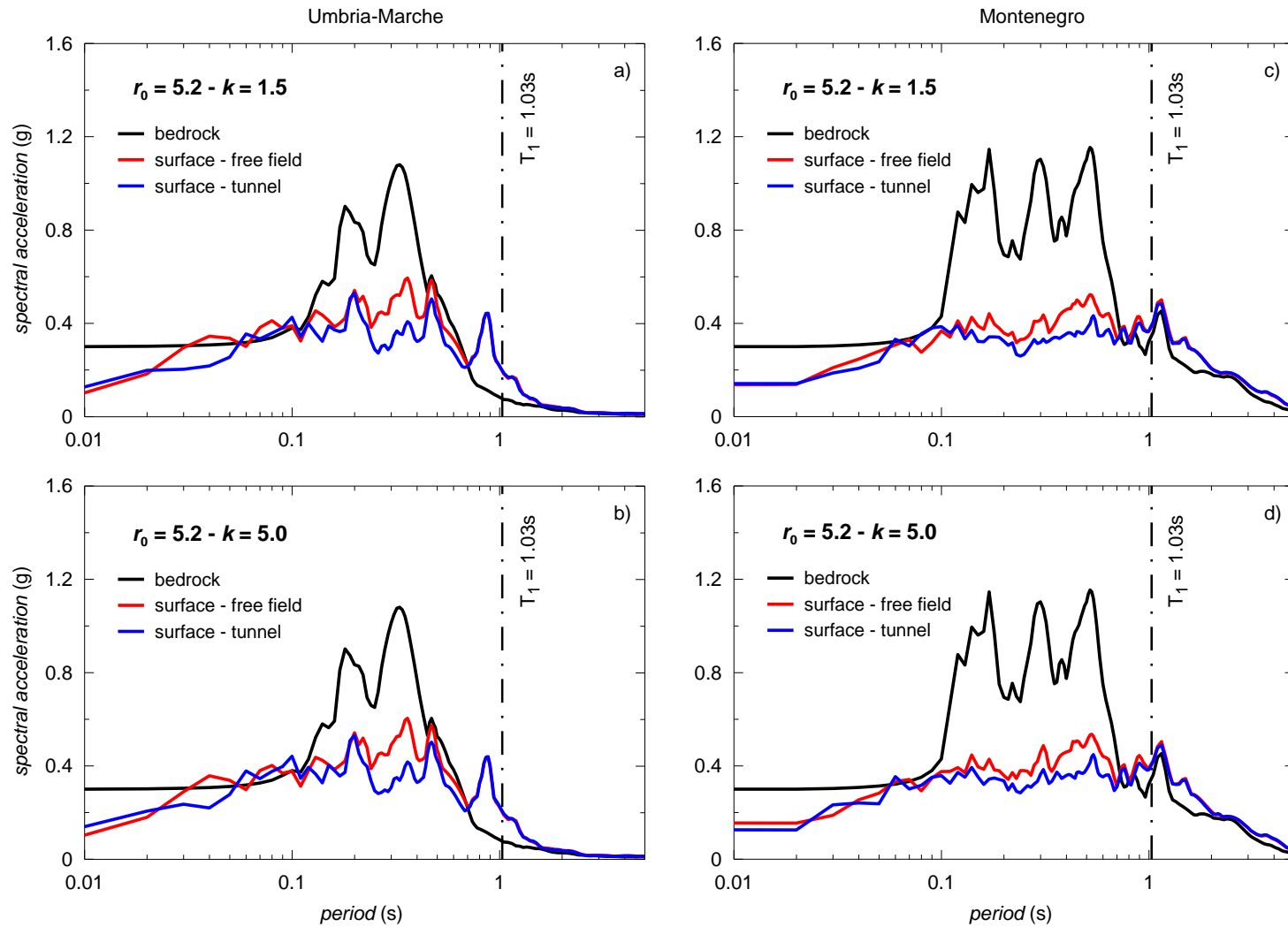


Figure 6. Comparison of response spectra recorded at bedrock and at surface during the: (a) Umbria-Marche; (b) Montenegro event for different destructuration rates.

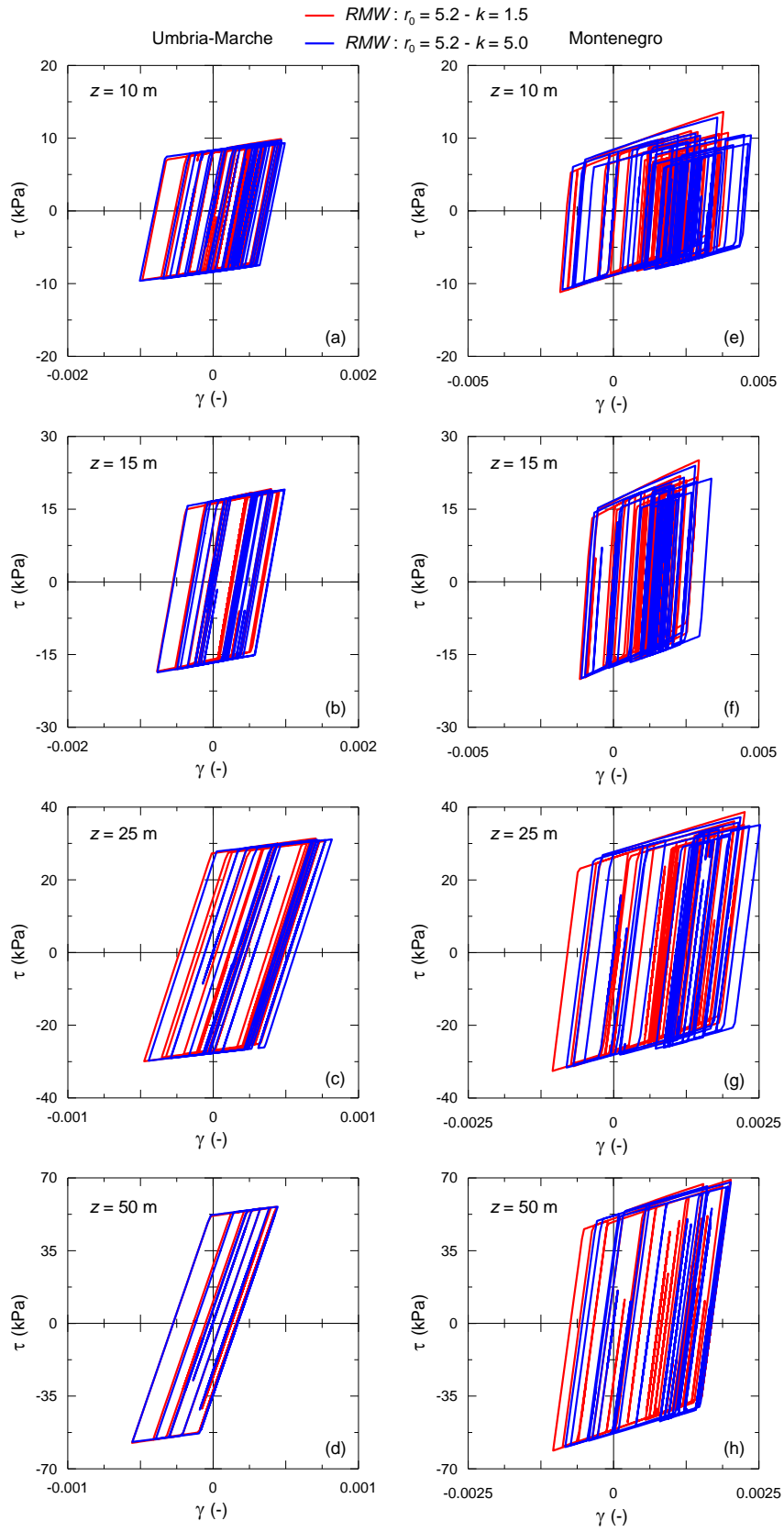


Figure 7. Stress-strain curves during the: (a-d) Umbria-Marche; (e-h) Montenegro event for different destructuration rates.

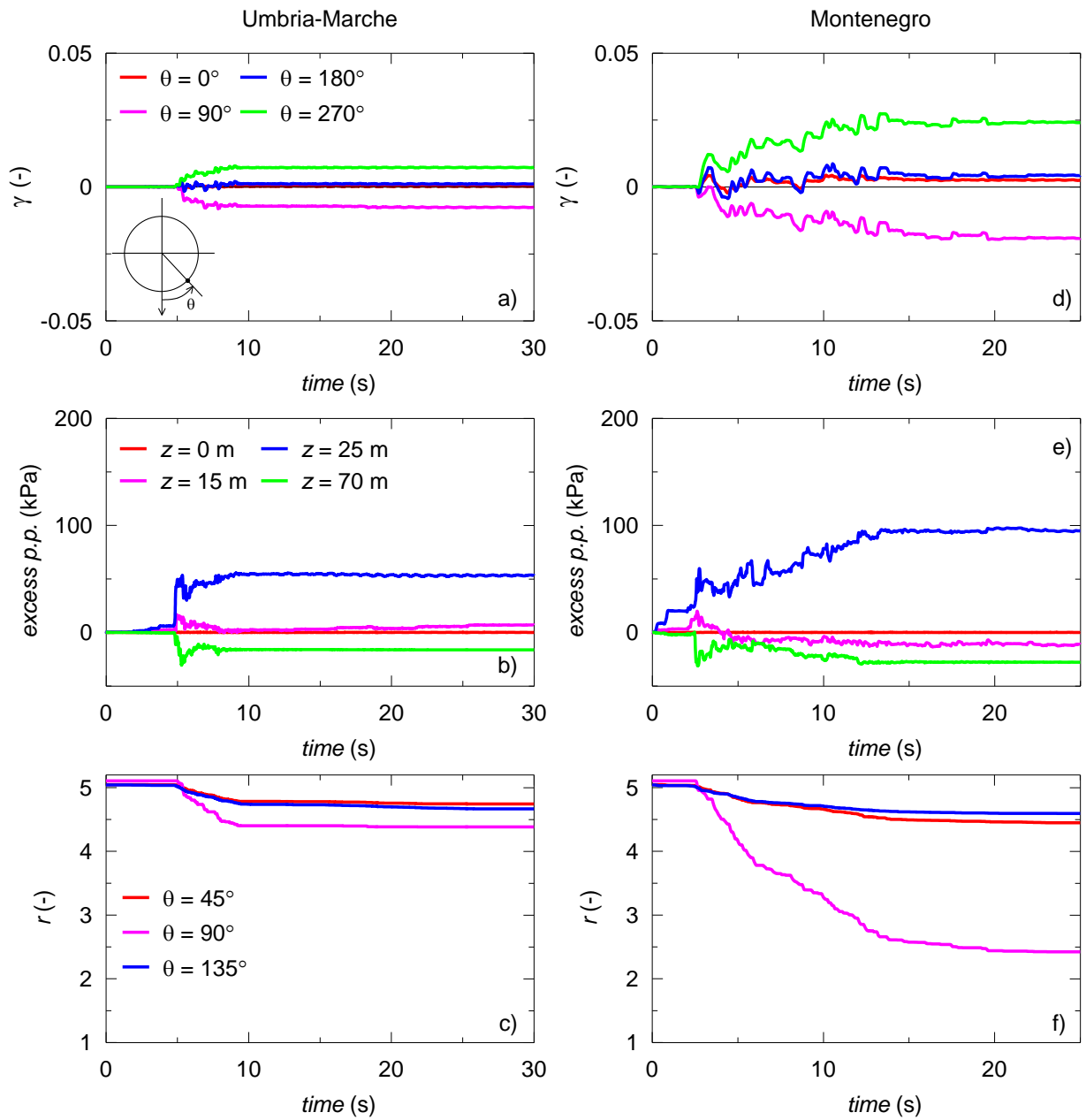


Figure 8. Time histories of shear strain, excess pore pressure and *RMW* soil structure parameter r during the: (a-c) Umbria-Marche; (d-f) Montenegro event for a destructuration rate $k = 1.5$.

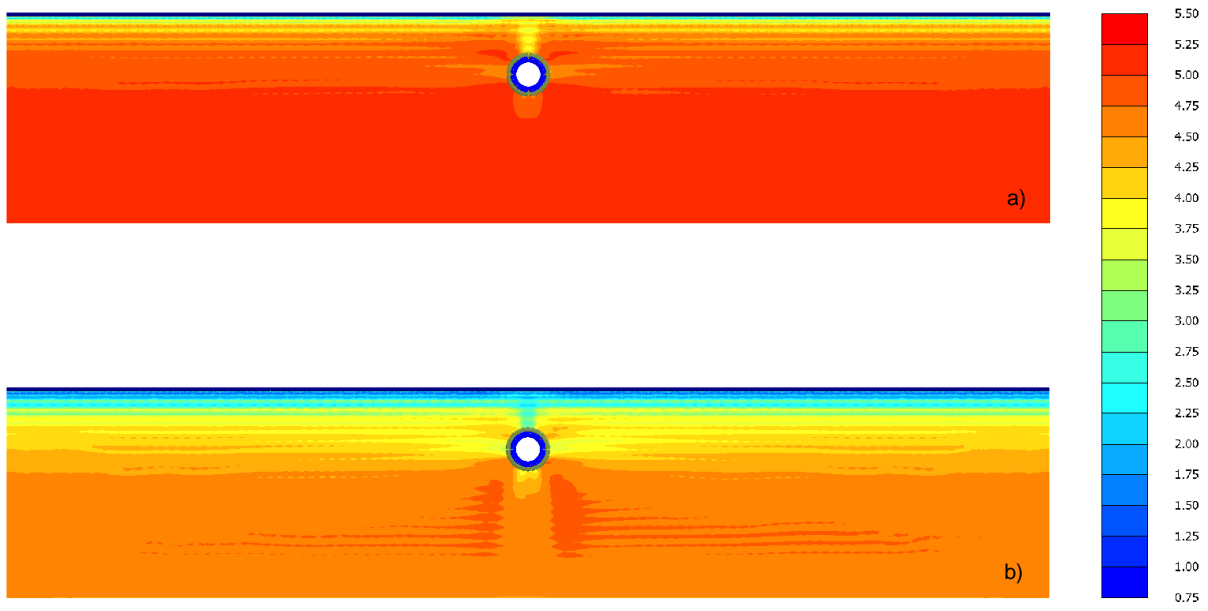


Figure 9. Contours of the *RMW* parameter r at the end of the: (a) Umbria-Marche; (b) Montenegro event for a destructuration rate $k = 1.5$ (note that r_0 is equal to 5.2 in both cases).

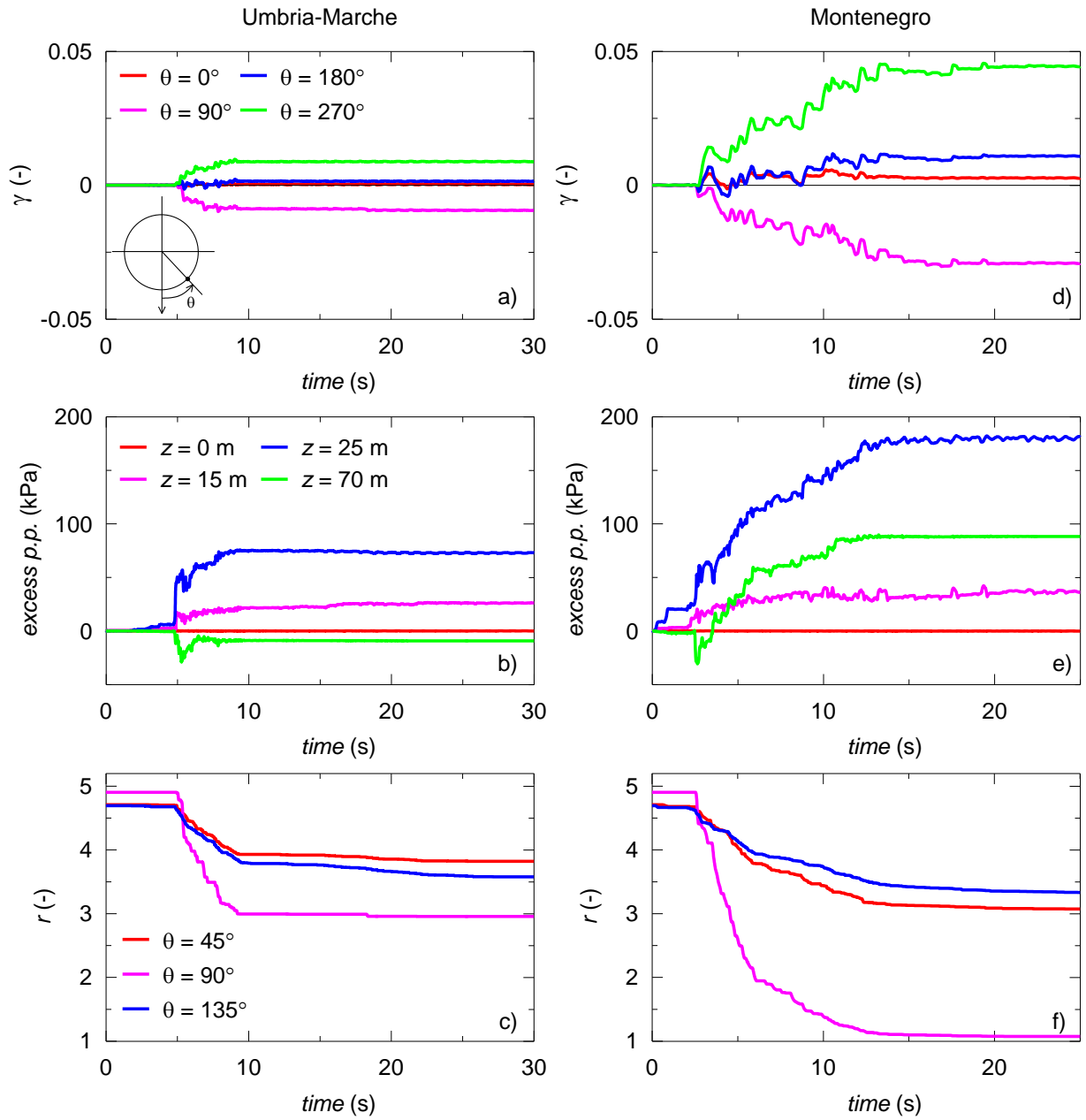


Figure 10. Time histories of shear strain, excess pore pressure and *RMW* soil structure parameter r during the: (a-c) Umbria-Marche; (d-f) Montenegro event for a destructuration rate $k = 5.0$.

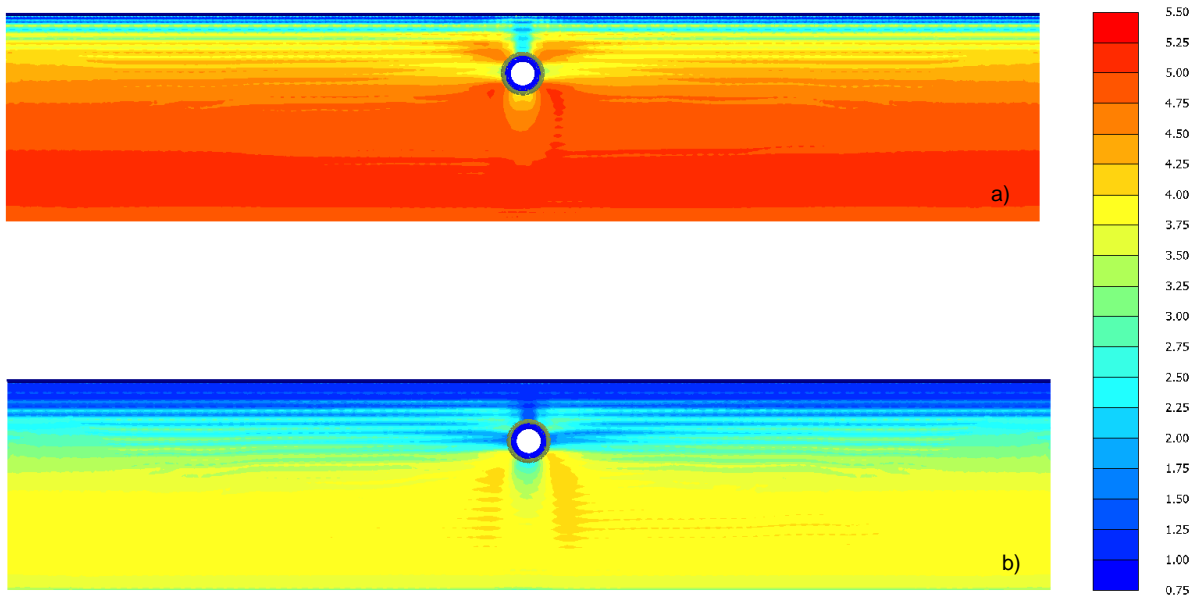


Figure 11. Contours of the *RMW* parameter r at the end of the: (a) Umbria-Marche; (b) Montenegro event for a destructuration rate $k = 5.0$ (note that r_0 is equal to 5.2 in both cases).

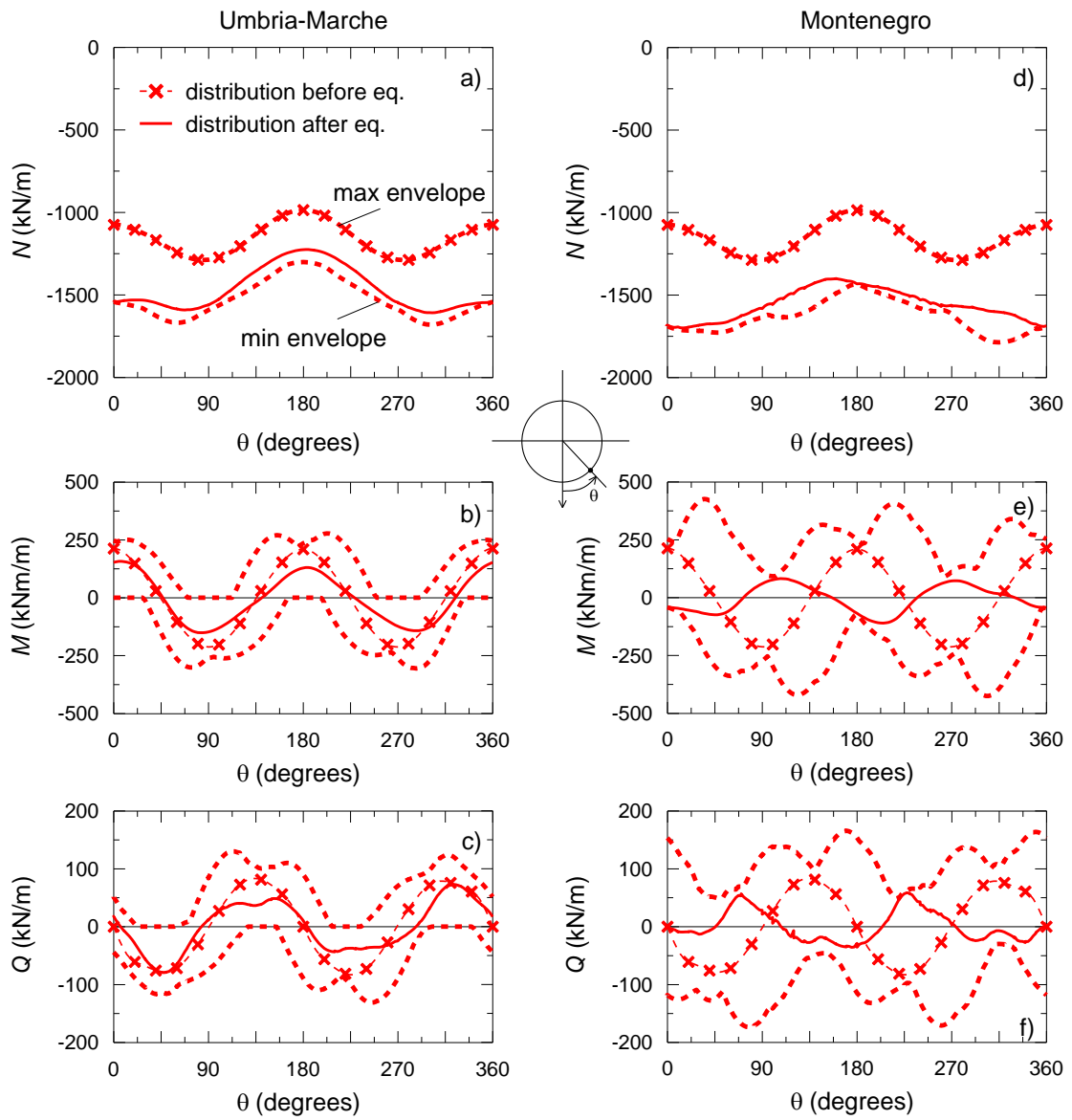


Figure 12. Distribution of hoop force (N), bending moment (M) and shear force (Q) before and after the: (a-c) Umbria-Marche; (d-f) Montenegro event for a destructureation rate $k = 1.5$.

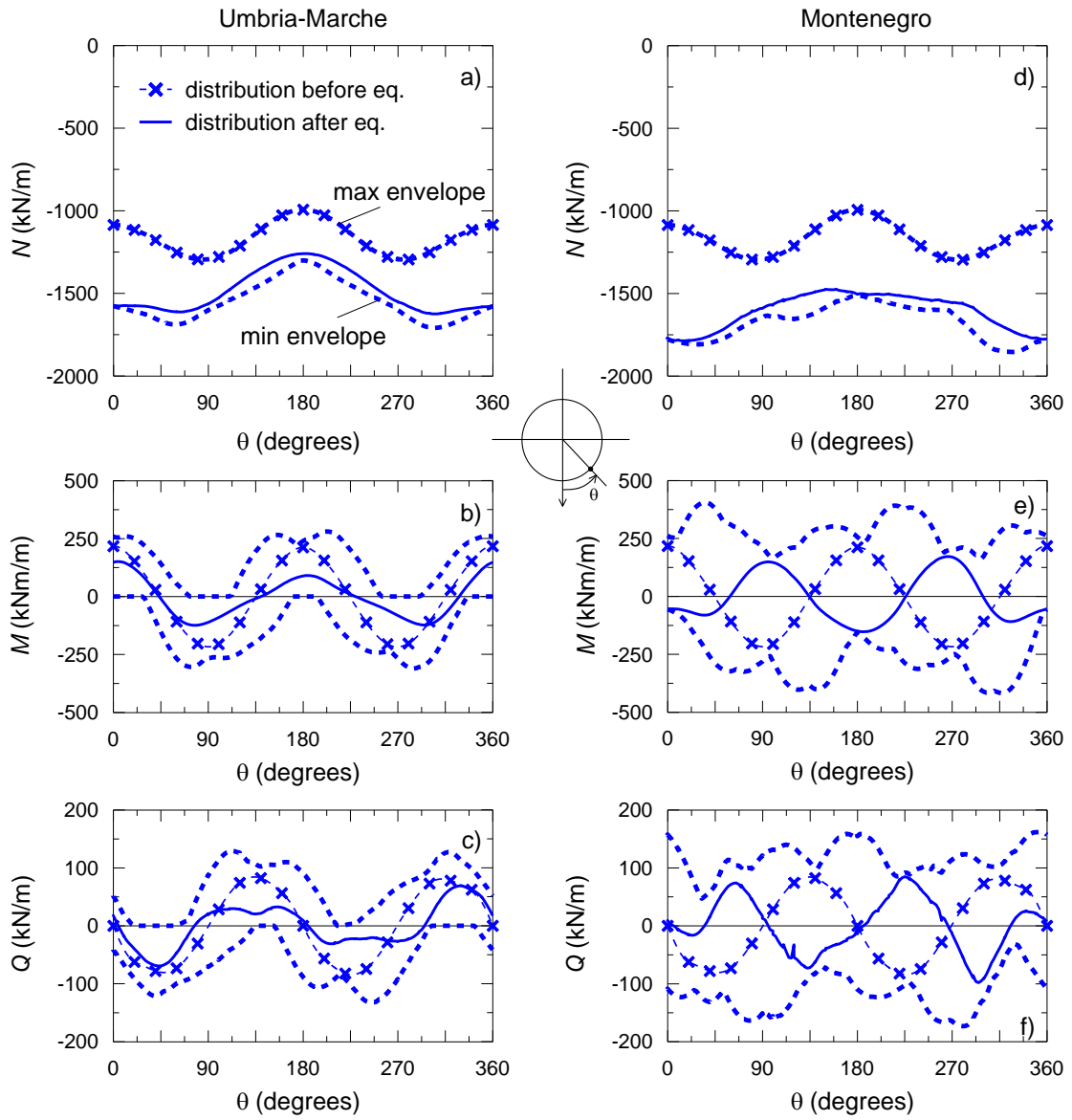


Figure 13. Distribution of hoop force (N), bending moment (M) and shear force (Q) before and after the: (a-c) Umbria-Marche; (d-f) Montenegro event for a destructuration rate $k = 5.0$.

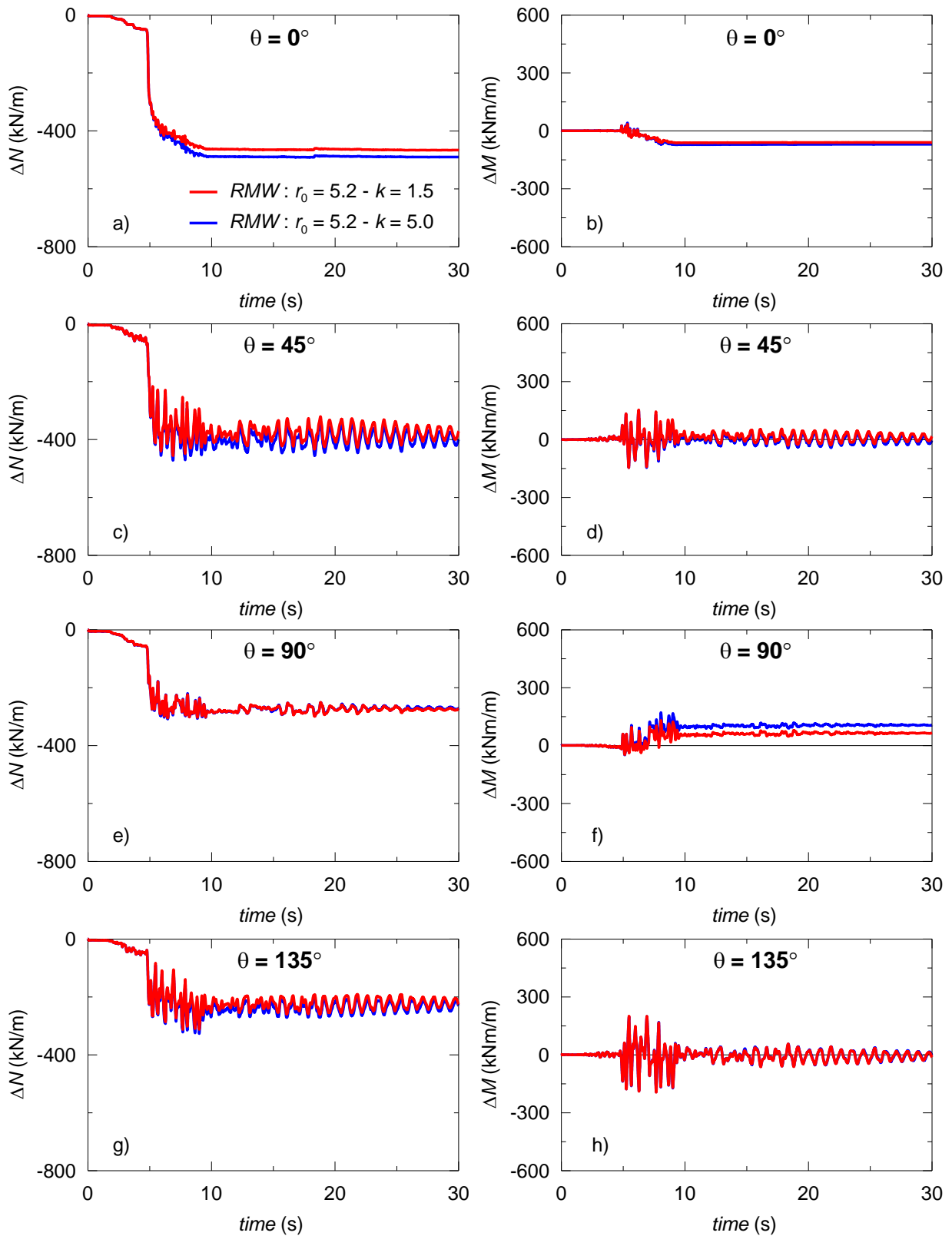


Figure 14. Influence of structure degradation rate on the time histories of hoop force and bending moment increments during the Umbria-Marche event for: (a-b) $\theta = 0^\circ$; (c-d) $\theta = 45^\circ$; (e-f) $\theta = 90^\circ$; (g-h) $\theta = 135^\circ$.

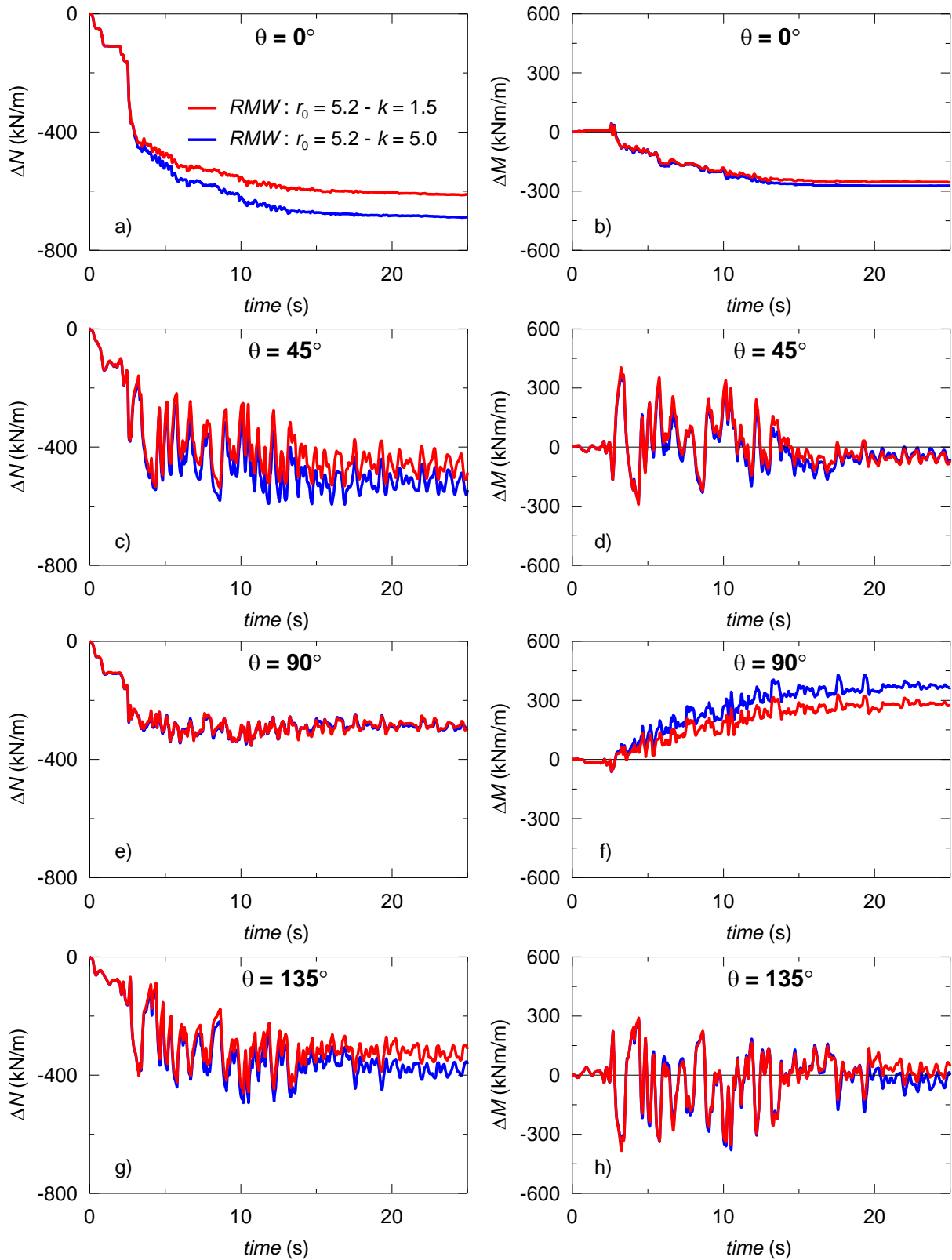


Figure 15. Influence of structure degradation rate on the time histories of hoop force and bending moment increments during the Montenegro event for: (a-b) $\theta = 0^\circ$; (c-d) $\theta = 45^\circ$; (e-f) $\theta = 90^\circ$; (g-h) $\theta = 135^\circ$.