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ASSESSMENT OF THE SEISMIC VULNERABILITY AND MODELLING OF EXISTING BRIDGE WITH DIFFERENT LEVELS OF ACCURACY: STRENGTHS AND WEAKNESSES

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Abstract

The assessment of the seismic vulnerability of existing bridges is a topic of primary relevance for the infrastructure field, as the knowledge of the vulnerability allows to design the management of the aftermath of the earthquake. To appreciate the vulnerability of the structure, different modelling approaches with different levels of accuracy are available and modern seismic design code provide recommendation about their use.
The study aims at assessing the reliability of less complex methods, but with low computational cost and calibrate them against the results of nonlinear dynamic analyses in order to obtain an acceptable level of accuracy.
The study will be developed considering a bridge prototype, representative of features such as the structural scheme (simply supported), the material of the piers (reinforced concrete), and the geometry of the piers (framed piers). The bridge will be analyzed first through nonlinear dynamic analyses, to define the benchmark solution. Then, the same bridges will be analyzed through linear dynamics analysis (Linear Time history and Response Spectrum) and nonlinear static analyses, such as MPA (Modal Pushover Analysis) and, in the end, through simplified Equivalent Static Analysis, and a comparison among the different modeling approaches will be proposed. Strengths and weaknesses of each model will be highlighted and discussed.

Keywords: Existing bridge, Seismic Vulnerability, Earthquake Engineering, Linear Static Analysis, Linear Dynamic Analysis, Modal Pushover Analysis, Nonlinear Dynamic Analysis.

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1 INTRODUCTION

To appreciate the vulnerability of the structure, different modelling approaches with different levels of accuracy are available and modern seismic design code provide recommendation about their use, in this work it's considered the Italian Building Code (IBC) [1] and the Eurocode 8 [2].

The work presents a two different comparison of analyses, taking nonlinear time history analysis (NTHA) [3] as a benchmark. The first compare the result of linear dynamic analysis, which can be Linear Time History Analysis (LTH) and Response Spectrum (RS) [2] with the NTHA; the second compare the benchmark with the Modal Pushover Analysis (MPA) [4,5] and the Equivalent Static Analysis (ESA).

2 CASE STUDY

The work considers a simply supported bridge with geometries reflect typical configurations available in the Italian stock, namely the frame pier composed of three circular columns and the deck with four T-shaped beams. In the work three seismic scenarios corresponding to three different seismic zones according to the Italian Building Code are analyzed, relevant to the municipalities of Reggio Calabria for zone 1(Z1), Sirmione (BS) for zone 2 (Z2) and Pavia for zone 3 (Z3). The selected typology turns out to be among the most widespread ones in the Italian stock, where they represent more than 90% of existing bridges [6,7].

2.1 Bridge characteristics

The considered case study is a simply supported bridge with a 20m span and 5m pier's high. The static scheme was assumed, in the longitudinal direction, a fixed support axis and a moving support axis, while in the transverse direction both axes have fixed supports (the same would be if the presence of transverse restraints were considered) (Figure 1). In terms of seismic mass distribution this means that in the longitudinal direction the seismic mass acting on the pier under consideration will be the mass of the rigidly connected deck, in the transverse direction the seismic mass will be given by half of the seismic masses of both decks' laying on the analyzed pier.

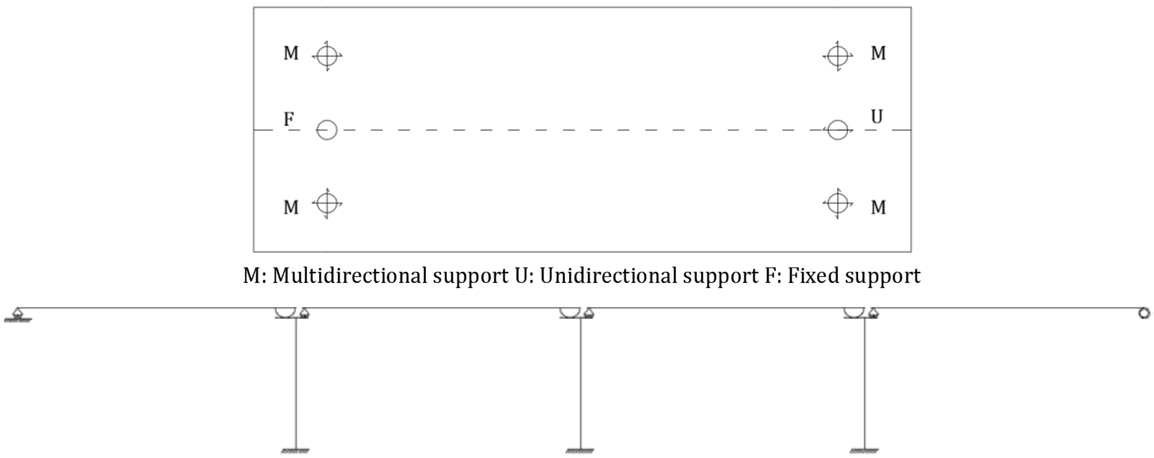

Figure 1: Static scheme for simply supported bridges

For the piers, C25/30 concrete is assumed, with a Young's modulus E_c = 31475,8 MPa and a characteristic cylindrical resistance f_{ck} = 25MPa; steel for the rebar is FeB44k, with Young's modulus E_s = 200GPa, characteristic yield strength f_{yk} = 430 MPa and characteristic tensile strength at break f_{tk} = 540MPa.

Three columns with diameter 1m connected at the top by a pier cap make the frame piers. Volumetric ratios of ρ_s=0,4% and ρ_w =0,09% (defined as the volume of longitudinal bars divided by the volume of concrete, and the volume of stirrups divided by the volume of confined concrete) are assumed for the longitudinal and the transverse reinforcement [8,9].

Each column is reinforced with longitudinal bars arranged circumferentially and consisting of 12φ18 bars, in the case of a 5m high pier (Figure 2). The transverse reinforcement for each column, in both heights, consists of circular φ10 stirrups with 20cm pitch.

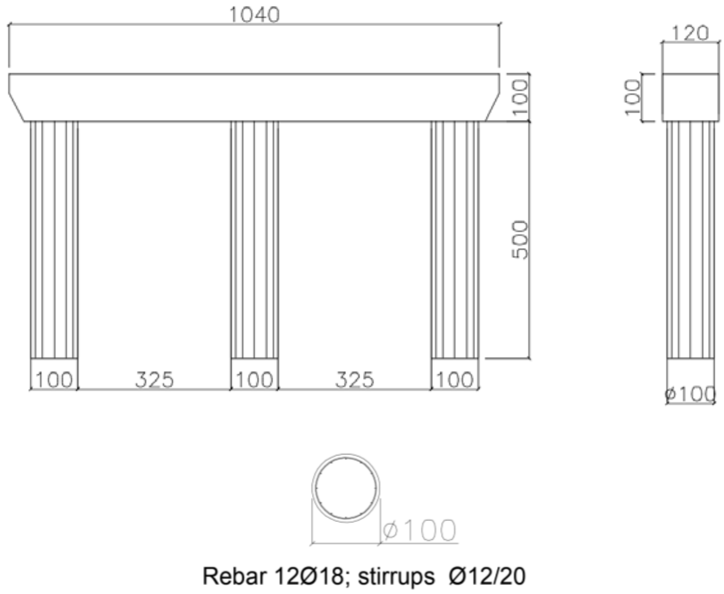

Figure 2 Pier and reinforcement

The type of deck considered have been associated to static layout and span length typical of the Italian scenario. In the case study it was considered the T-shaped beams of 20m span (Figure 3), the considered section is shown below with their characteristics summarized in the Table 1, in terms of geometry and loads.

A	5,434	m2
γ_{cls}	25	kN/m3
G_1	135,85	kN/m
G_2	47	kN/m
L	20	m
G_{deck}	3657	kN
$Q_{0,2}$	276	kN

Table 1 T-shape deck characteristics

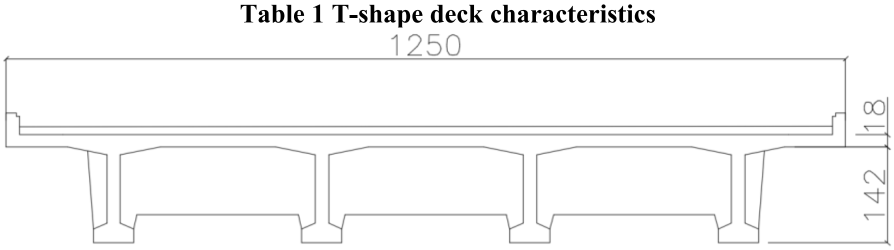

Figure 3 T-shape deck section

2.2 Seismic scenarios

Three scenarios were defined according to IBC [1], assuming a design life V_N = 50 years and use class IV (cu = 2.0) (IBC, § 2.4.3), to which the following structure reference periods (IBC, § 3.2.1) $V_R = V_N c_u$ = 100 years. Only ULS is examined, with a period of 949 years.

The three seismic scenarios correspond to seismic zones 1, 2 and 3 according to the Italian Building Code. The code classifies the entire Italian territory into 4 zones according to the horizontal acceleration with a probability of exceedance equal to 10% in 50 years.

For the first three zones a municipality was chosen to define the seismic scenario: for zone 1, Reggio Calabria, located in Calabria region; for zone 2, Sirmione, located in Lombardy region in the province of Brescia; for zone 3 Pavia, located in Lombardy region. For all zones, topographic class T1 and soil type B were assumed.

In Figure 4, the design spectra considered for the parametric analysis are represented.

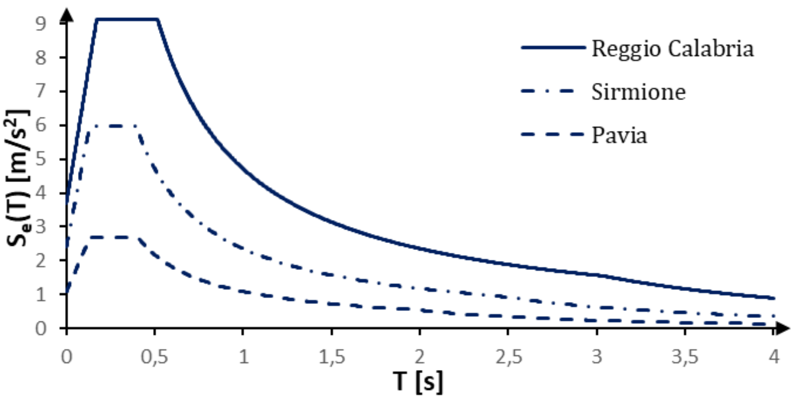

Figure 4 Design spectrum

Starting from the design spectra defined according to IBC, seismic inputs were defined for the different analyses.

The spectra themselves are the seismic input for the response spectrum analysis (RSA) and the equivalent static analysis (ESA).

The input for the nonlinear static (MPA) was derived from the acceleration spectrum and the displacement spectrum, thus obtaining the demand curve in the ADRS plane and are shown in Figure 5.

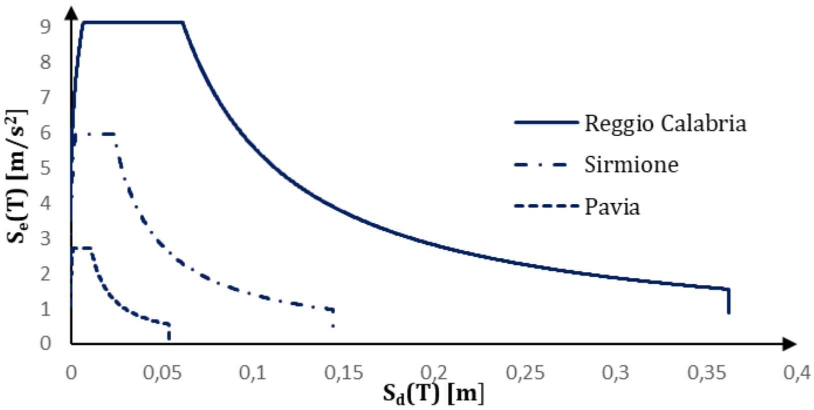

Figure 5 Demand curves for MPA

Three sets of seven bidirectional ground motions consistent with the target spectra were selected one set for each seismic scenario for the dynamics analysis (nonlinear and linear); these are selected with REXEL v3.4 beta [10] software from the European Strong-motion Database [11].

Relevant information on the ground motion data set is reported in Table 2, and the scaled horizontal spectra at 5% damping are shown in Figure 6.

Reggio Calabria

Earthquake Name	Waveform ID	Earthquake ID	Mw	PGA_X [m/s^2]	PGA_Y [m/s^2]	SFx	SFy
Umbria Marche	600	286	6	1,6852	1,0406	2,186	3,5401
Adana	1726	561	6	2,1575	2,6442	1,7074	1,3932
Izmit	1231	472	8	1,5764	2,1922	2,3368	1,6804
Duzce 1	1703	497	7	3,6988	5,0358	0,9959	0,7315
Campano Lucano	290	146	7	2,1206	3,1662	1,7371	1,1635
South Iceland	4674	1635	7	3,1176	3,3109	1,1816	1,1126
Gazli	74	43	7	6,0382	7,065	0,6101	0,5214

Sirmione

Earthquake Name	Waveform ID	Earthquake ID	Mw	PGA_X [m/s^2]	PGA_Y [m/s^2]	SFx	SFy
Campano Lucano	292	146	7	0,5878	0,5876	4,0921	4,0934
Friuli (aftershock)	147	65	6	1,3841	2,3189	1,7377	1,0372
Spitak	439	213	7	1,7932	1,7958	1,3413	1,3394
Tabas	182	87	7	3,316	3,7789	0,7253	0,6365
Montenegro	198	93	7	1,7743	2,1985	1,3556	1,0940
Umbria Marche	594	286	6	5,1383	4,5383	0,4681	0,5300
Campano Lucano	290	146	7	2,1206	3,1662	1,1342	0,7597

Pavia

Earthquake Name	Waveform ID	Earthquake ID	Mw	PGA_X [m/s^2]	PGA_Y [m/s^2]	SFx	SFy
Campano Lucano	292	146	7	0,5878	0,5876	1,8426	1,8432
Montenegro (aftershock)	232	108	6	0,56	0,5426	1,9341	1,9961
Friuli (aftershock)	147	65	6	1,3841	2,3189	0,7825	0,4670
South Iceland (aftershock)	6335	2142	6	1,2481	1,1322	0,8677	0,9566
South Iceland	6263	1635	7	6,1359	5,018	0,1765	0,2158
Umbria Marche	594	286	6	5,1383	4,5383	0,2108	0,2386
Campano Lucano	291	146	7	1,5256	1,7247	0,7099	0,628

Table 2 Selected accelerogram for each scenario

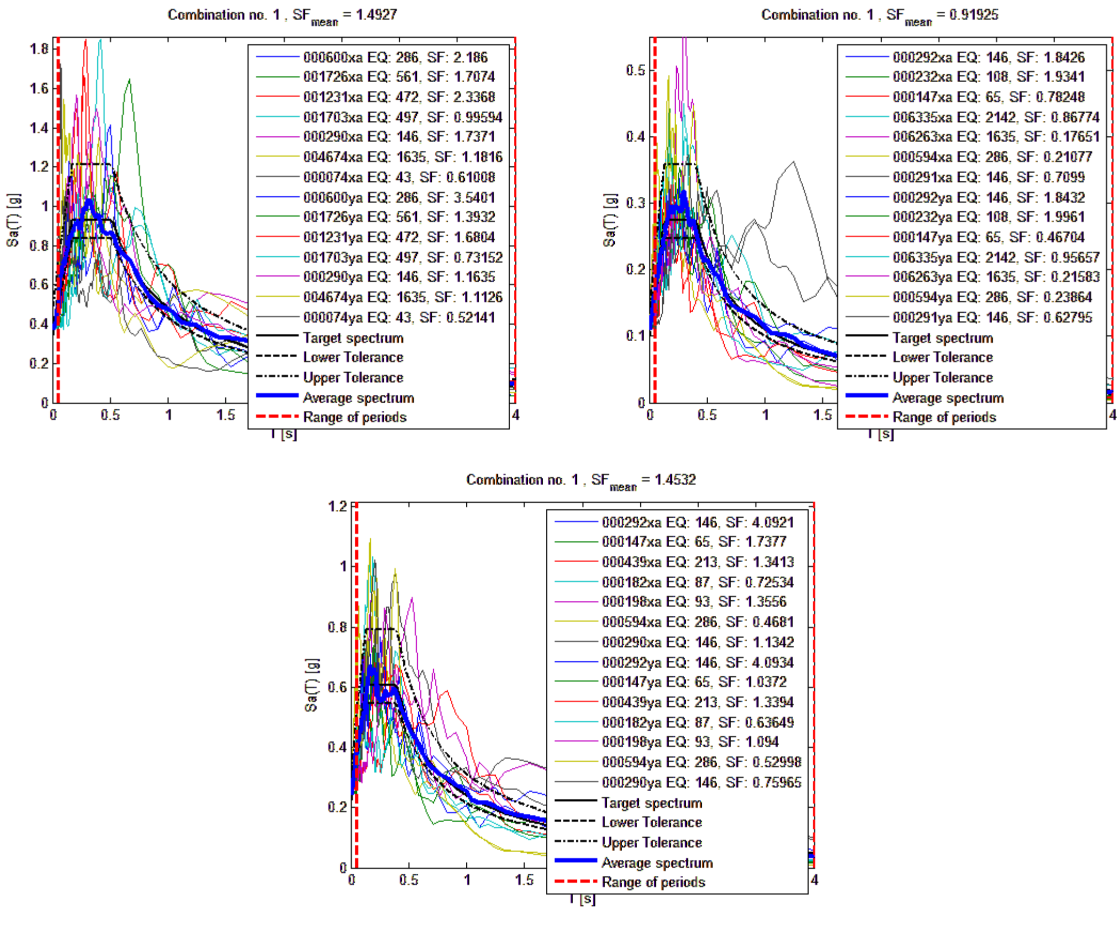

Figure 6 Scaled ground motion acceleration spectra and target spectra

2.3 Numerical model

The structural model was implemented in SAP2000 v23.1.0 software [12]. The geometry represents that expressed in the previous section (Figure 7).

The framed piers were rigidly fixed to the ground and the abutments are one hinged and one longitudinally free to move.

The deck is modelled as linear elastic elements and on this are applied the permanent non-structural load and the variable loads.

The connection between the deck and the pier was modelled through a rigid links [34], the locked degrees of freedom change as the fixed or moving axis of the simply supported bridge changes.

Figure 7 Numerical model

The work presents different types of analysis, for linear analysis the material is purely elastic, while for nonlinear analysis the design values for C25/30 and Fe44k are used.

The material nonlinearity is modeled with concentrated plasticity, the framed piers were rigidly fixed to the ground and modelled as linear elastic elements, with a bidirectional plastic hinge (parametric P-M2-M3) at the basis formulated according to Table 10-8 (concrete columns) of ASCE 41-13 [13] in order to account for inelastic concrete deformation. The plastic hinges backbone is computed in function of the vertical load.

The internal structural damping is modeled as Rayleigh damping [3], with parameters assigned to achieve 5% damping ratio at the first two frequencies.

3 RESULTS

The results shown in this section refer to the NTHA benchmark for each seismic scenario. The percentage change in base reactions and pier head displacement between the linear dynamic analyses (LTH and RS) and the NTHAs, and later between the nonlinear and linear static analyses (MPA and ESA) and the NTHAs, are shown below.

Figure 8 shows the variation in response between NTHAs and linear dynamic analyses (LTH and RS) in terms of base reaction, and there are two interesting aspects to note: first, it can be seen that as seismic intensity varies, the variation in response decreases, this is easily inferred to be due to the capped response induced by modeling nonlinearities in NTHAs; second, it is possible to see that the difference between of result of the two linear methods is minimal, especially as the seismic intensity decreases.

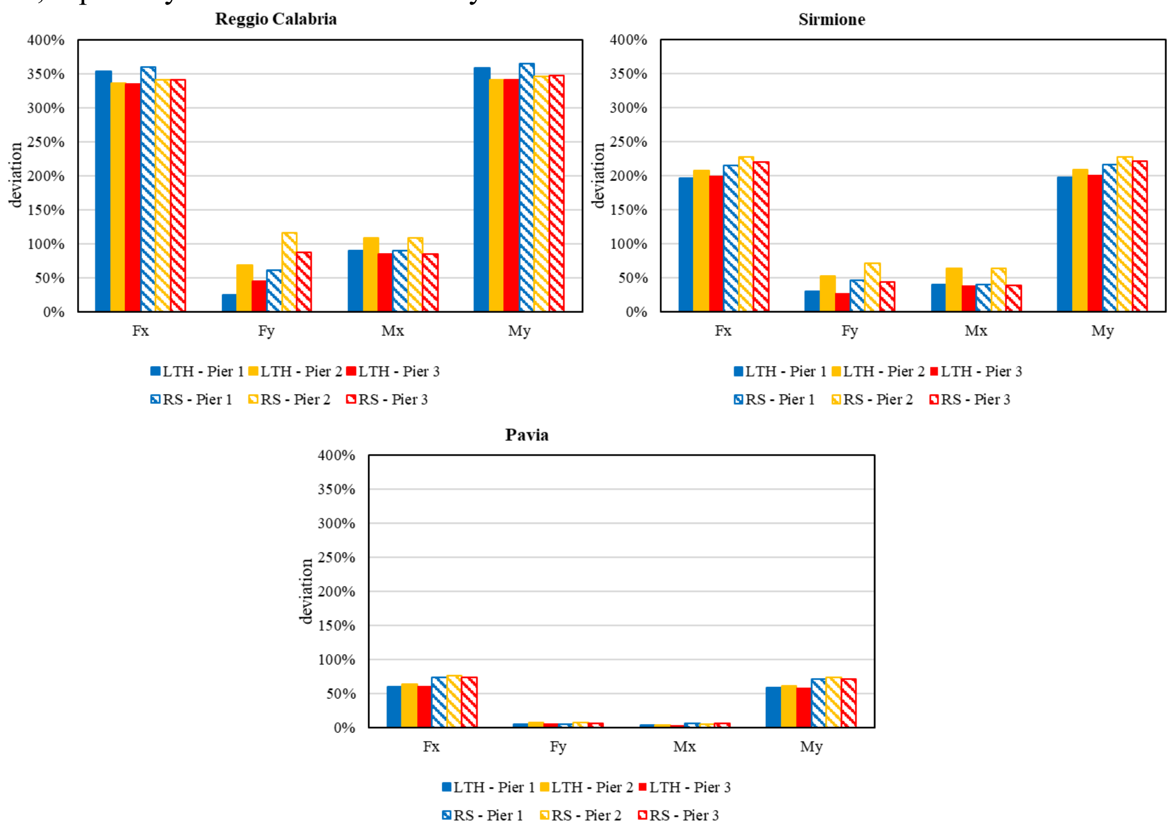

Figure 8 Reactions variation NTHAs vs LTH and RS

Figure 9 shows the variation in terms of displacement, and it can be seen that the percentage variations are generally smaller than those of the reactions. For the medium- and low-intensity seismic scenarios, a larger variation is seen in the x-direction than for the heaviest seismic scenario; this result should be attributed to the large plastic displacement in that scenario with the NTHAs.

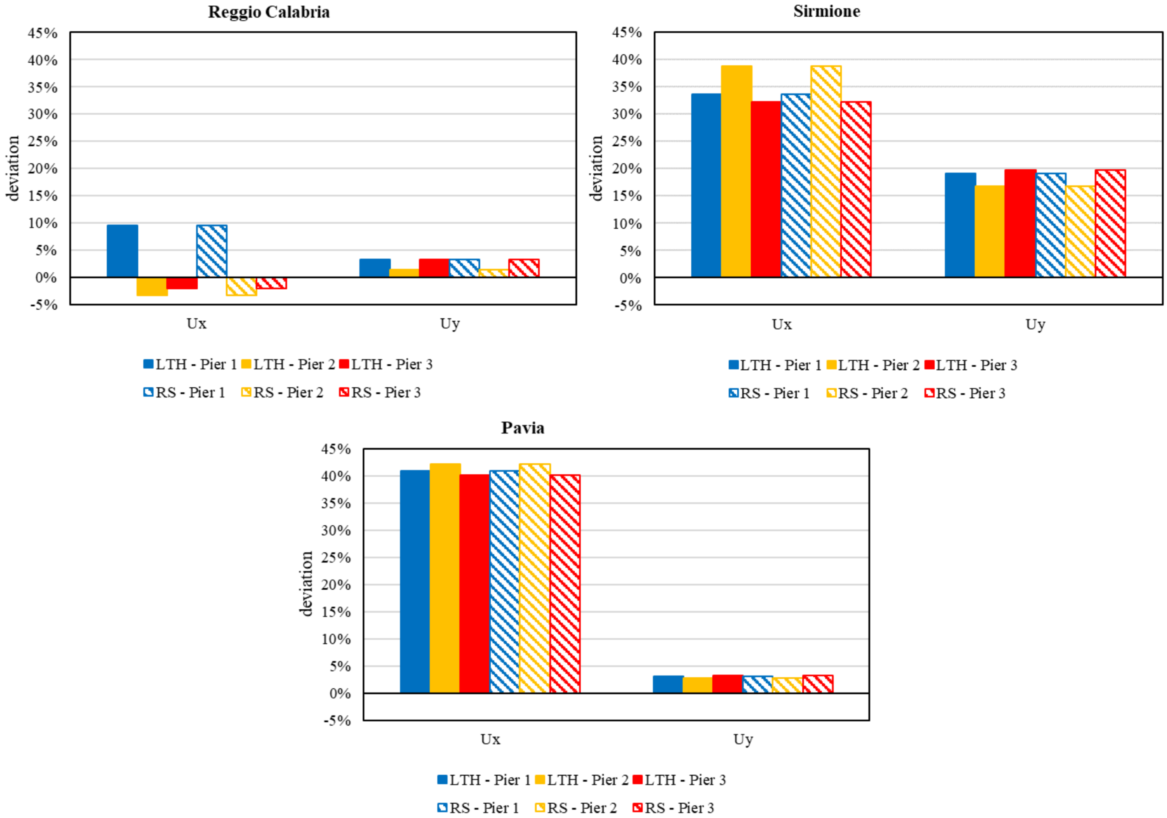

Figure 9 Displacements variation NTHAs vs LTH and RS

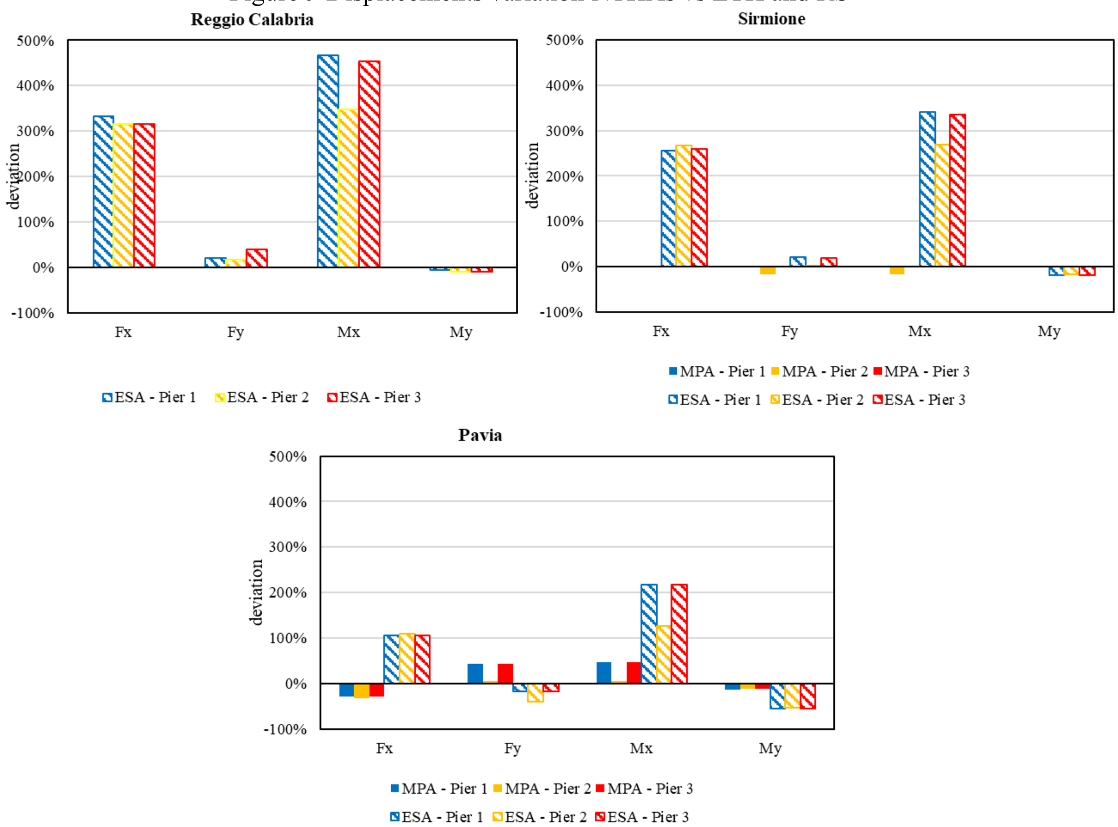

Figure 10 Reactions variation NTHAs vs MPA and ESA

In figure 10 there isn't all the result because in most severe scenario and in x-direction of the medium scenario the performance point doesn't exist. In this comparison, when a performance point exists, the MPA gives a good approximation. Instead, ESAs show a response similar to what we have seen in linear dynamic analyses, with the error decreasing with li decreasing earthquake intensity.

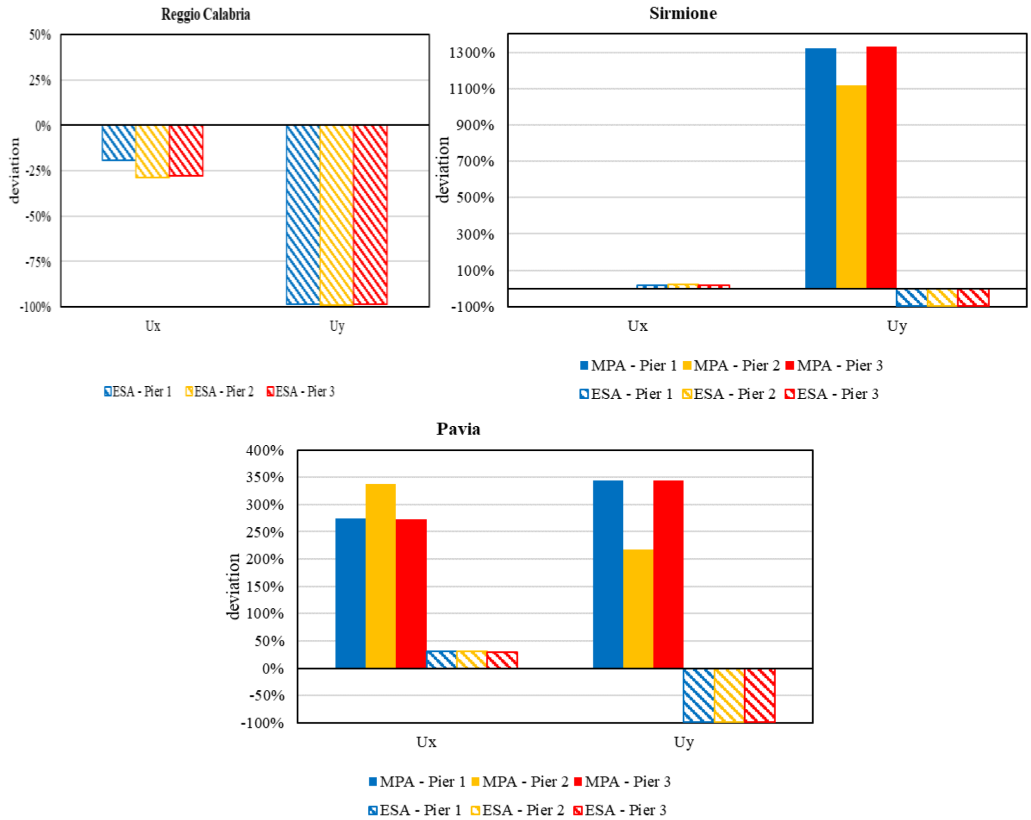

Figure 11 Displacements variation NTHAs vs MPA and ESA

In the last figure there is the results in terms of displacement, the ESAs confirm the previous consideration, a result that has to be commented is the displacement in y-direction for the medium seismic scenario with MPA that is one order on magnitude higher than the benchmark, this is due to the fact that performance point is for higher plastic deformation.

4 CONCLUSIONS

The work presents the results obtained with the different analysis methodologies counted in the current seismic codes. By analyzing the accuracy, it was possible to see that the linear methods for heavier seismic scenarios deviate greatly from the benchmark response, as the modeling for NTHAs has a cap in the strength that cannot be captured of the linear modeling. The accuracy of MPAs comes very close in terms of base reactions, as they capture the resources of the structure.

In terms of computational cost, two cost items can be identified: first, modeling, which in the case of nonlinear analyses (NTHAs and MPAs) involves more effort than in linear dynamic (LTH and RS) and ESAs; second, reading the result, in the case of LTHs is very expensive, as for NTHAs, for MPAs it is less expensive than the former, while for the other analyses (RS and ESAs) it is immediate.

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