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SEISMIC PERFORMANCE EVALUATION OF EXISTING RC HIGH-RISE BUILDING IN MONTENEGRO

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Abstract

Seismic performance evaluation of buildings represents a basis for seismic vulnerability and risk assessment. Considering the importance of seismic risk assessment nowadays, as one of the most actual topics in earthquake engineering, in this paper, a seismic performance evaluation of an existing RC high-rise building with walls on the Montenegrin coast, an area with high seismic risk, is conducted. The aim of the paper is to evaluate the performance of this type of buildings built on the Montenegrin coast and thus contribute to their vulnerability assessment. Two EN 1998-1 fundamental requirements are examined for seismic performance evaluation: non-collapse and damage limitation requirements.

A non-linear 3D building model is defined in the PERFORM 3D software. Modelling is performed by taking into account realistic characteristics of used materials, constructed geometry of structural elements and realistic loads. Non-linear time history analyses are performed using 112 ground motion records, from which 56 are used for evaluation of non-collapse requirement and the other 56 for evaluation of damage limitation requirement. Ground motion records were selected and scaled according to EN 1998-1. Peak horizontal ground acceleration of 0.34g for ground motions with a return period of 475 years for checking non-collapse requirement is adopted, and 0.15g for ground motions with a return period of 95 years for checking damage limitation requirement. Inter-storey drift, wall rotations, dilatations in structural elements and shear capacity are the main parameters that are analysed to obtain conclusions on acceptable seismic performance.

Useful conclusions related to the seismic performance of existing RC high-rise buildings of structural systems with ductile walls built on the Montenegrin coast are pointed out.

Keywords: non-linear time history analyses, non-collapse requirement, damage limitation requirement, seismic performance, damage state, EN 1998-1

1. Introduction

Montenegrin coastline is a seismically active area, with the highest earthquake potential of IX degrees on the Mercalli scale. The rapid development of coastal tourism caused the construction of many multi-storey residential buildings and hotels, which now represent the characteristic urban scenery of that part of the country. A very dense built-up urban environment warns of significant seismic risk and consequences of seismic hazard. For that reason, it is imperative to assess the seismic performance and safety of existing high-rise buildings on the Montenegrin coast, improve vulnerability assessment, and reduce seismic risk.

EN 1998-1 [1] defines two fundamental requirements prescribing acceptable seismic performance of the buildings: non-collapse requirement and damage limitation requirement. Performance assessment and fulfilment of EN 1998-1 [1] basic requirements can be reliably assessed using non-linear time history analysis (NTHA). Applying NTHA gives insight into the intensity of damage and damage locations, which creates the possibility for qualitative and quantitative damage assessment. In the existing literature, there is a large number of papers where NTHA has been used to evaluate building's vulnerability. Luco, Bazzuro and Cornell [2] implemented NTHA on case study building to assess the damage state and remaining lateral capacity. Aghagholizadeh and Massumi [3] examined the behaviour of RC frames using NTHA, where they evaluated damage grade and period elongation. Reuland, Lestuzzi and Smith [4] examined the vulnerability of

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constructions after the main shock for buildings with limited data on previous damages. Orlacchio, Baltzopoulos and Iervolino [5] examined the possibility of creating an SDOF model to evaluate residual displacements. Trevlopoulos et al. [6] analysed the damage state after the main shock, where they simulated the impact of several aftershocks after the damage made by the main shock.

In this paper, the existing high-rise RC building (Fig. 1a) built in Budva, part of the Montenegrin coastline with high seismic risk, is selected as a case study. Construction of the building took place in the 2010-2015 period, and the design was done according to EN 1992-1-1 [7] and EN 1998-1 [1] provisions.

Performance assessment of case study building and fulfilment of the basic requirements prescribed in EN 1998-1 [1] was obtained using NTHA on 3D non-linear building model. CSI PERFORM 3D software was used for non-linear modelling. One hundred twelve ground motion time histories were used for NTHA, 56 for the evaluation of non-collapse requirement and the other 56 for the damage limitation requirement. Ground motion time histories have been scaled according to EN 1998-1, to peak horizontal acceleration of 0.34g for earthquakes with a return period of 475 years and 0.15g for earthquakes with a return period of 95 years, appropriate for the Budva's location. Inter-storey drifts, dilatations in structural elements, wall rotations and shear strength have been the main parameters for seismic performance evaluation.

2. Basic information on case-study building

The building consists of 19 storeys, i.e., two underground garage storeys, a basement, ground level, a mezzanine, thirteen residential floors and a roof. MEST EN 1998-1 [8] provides that in the location of Budva, earthquakes with a return period of 475 years have a peak horizontal acceleration of 0.34g, while earthquakes with a return period of 95 years have a peak horizontal acceleration of 0.15g. The height of the building is 55.9m, out of which 13.4m is below ground level. Non-structural walls are made out of bricks and gypsum boards. Plans of the characteristic floor below and above ground level are shown in Fig. 1b and Fig. 1c. The thicknesses of the designed walls are 20cm, 25cm, 30cm and 40cm. Structural elements are founded in the foundation slab, which thickness is 120cm. Frames have negligible stiffness compared to walls, which makes the structural system of the building a system with ductile walls. The ductility class is medium (DCM). All slabs have a thickness of 15cm. Walls ZP1, ZP2, ZP3 and ZP4, are passing all floors consistently from garage walls up to the roof, while ZP5 and ZP6 are passing all floors from the foundation slab up to the roof.

The used concrete class is C30/37, and the rebar reinforcement class is B500B. Concrete class C30/37 has Young's modulus E=3300 $kN/cm2$, with characteristic unconfined strength f_{ck}=3.00 kN/cm^2. The designed yield stress of B500B is f_{yd}=50.00 kN/cm^2 and Young's modulus E=20500 kN/cm^2. Reinforcement yield dilatation is 0.000234, while maximum dilatation is 0.05. Structural elements have been designed on dead, live and seismic loads. Seismic analysis has been conducted using multimodal spectral analysis according to EN 1998-1 [1], where masses of dead loads and 30% live loads with an eccentricity of 5% have been used.

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b)

a) c)

Figure 1. a) existing RC high-rise building; b) floor plan for floors above ground level; c) floor plan for floors below ground level.

3. Non-linear model of existing RC high-rise building

Modelling of the building is conducted in CSI PERFORM 3D [9] software. Modelled structural elements are walls, floor slabs and foundation slab. The foundation slab is modelled with restraints in the base. Slabs are modelled as rigid diaphragms, with accompanying mass of dead loads and 30% of live loads. According to Powell's recommendations [10], walls are modelled as fibre elements, except garage walls, which are modelled as elastic elements. The walls are modelled using two components which are acting in parallel. The first component contains vertical reinforcement fibres and concrete fibre with negligible strength and Young's modulus, in order not to interfere with the behaviour of the concrete fibres from the second component. The second component contains bending reinforcement and concrete fibres. Concrete of the boundary elements is modelled with confined concrete characteristics, while web concrete is modelled with unconfined concrete characteristics. The area and coordinates of each reinforcement or concrete fibre are separately defined. Shear behaviour is considered elastic, where the shear D/C ratio is examined. Out of plane behaviour of the walls is modelled as elastic without considering P-Δ effects. Columns and beams are not modelled since frames stiffness compared to walls are negligible. Fibres of both reinforcement and concrete are modelled with bilinear stress-strain relationships.

In order to evaluate the non-collapse requirement and damage limitation requirement, a few limit states have been defined and analysed: the strength limit state, the limit state of inter-storey drifts and limit

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states for deformations. The strength limit state is analysed for shear loads. Shear strength has been assigned as the elastic strength of concrete. Inter-storey drifts have been analysed for both planar directions. Capacity for inter-storey drifts has been assigned according to EN 1998-1 [1] from the damage limitation requirement for the non-structural brittle elements. Deformation limit states have been examined in two ways. The first way is the evaluation of the dilatations in the fibres of boundary elements on tension and compression. The examination is conducted using axial strain gauge elements in the software. These elements measure dilatations on node spans where they have been assigned. It has been examined if there was concrete cracking in the wall, i.e., was there any yielding of the reinforcement. The second way of evaluation is the examination of the rotations in the walls. On facade walls, plastic hinges have been modelled on the ground floor and above garage walls, while on the inside walls, plastic hinges have been modelled on the base and ground levels. The plastic hinge's height has been assigned according to recommendations of FEMA 356 [11], i.e., depending on the planar length, they have been assigned as one-half of the length. In cases when the plastic hinge length has been greater than the floor height, then the floor height has been assigned as plastic hinge length. Rotations have been checked using rotation gauge elements from software. Rotation capacities have been assigned according to FEMA 356 [11].

4. Analysis of non-collapse requirement

EN 1998-1 [1] prescribes that „Construction has to be designed and constructed, so that resists to seismic loads without local or global collapse, i.e., to hold its capacity, construction integrity and remaining lateral capacity after seismic events “. In order to define precise criteria based on which would be evaluated acceptable seismic behaviour, damage locations and damage states have been analysed. Following this provision, D/C of rotations, dilatations and shear strength have been evaluated. For non-collapse requirement, 56 NTHA analyses have been performed, 7 for each planar direction, and soil types A, B, C and D. 7 ground motions by the group for each direction and soil type have been chosen because it is the minimum number of analyses prescribed in EN 1998-1 [1] for using mean values. REXEL v 3.5 software [12] has been used for scaling ground motions to the referent peak horizontal acceleration of 0.34g. In Figs. 2-9, the D/C ratios of all deformations are shown. White represents the D/C ratio from 0 to 40%, blue from 40% to 60%, green from 60% to 80%, yellow from 80% to 100%, and red is the state when the ratio goes beyond 100%. D/C ratios have been examined through the following parameters: $R_M/R_{C,M}$, which represents the D/C ratio for rotations, $D_{CO}/D_{C,CO}$, which represents the D/C ratio for compression dilatations, $D_T/D_{C,T}$ which represents the D/C ratio for tension dilatations and $V_S/V_{C,S}$ which represent shear capacity D/C ratio. In Table 1, results for deformations D/C ratios are presented. Maximum wall rotations are defined for LS damage state according to FEMA 356 [11], in the amount of 0.006 radiations.

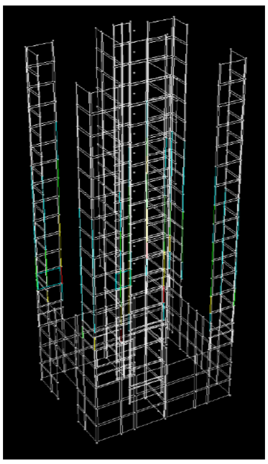

Figure 2. D/C deformation ratio for soil

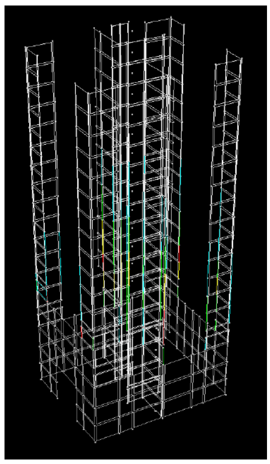

Figure 3. D/C deformation ratio for soil

type A in the X direction.

type A in the Y direction.

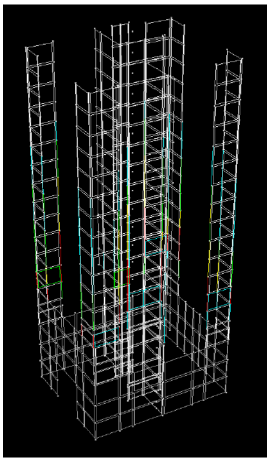

Figure 4. D/C deformation ratio on soil type B in the X direction.

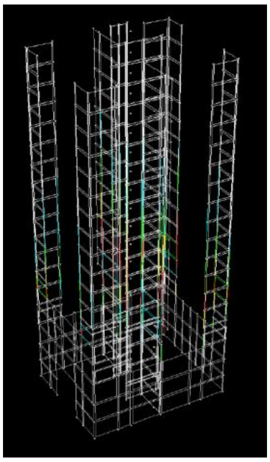

Figure 5. D/C deformation ratio on soil type B in the Y direction.

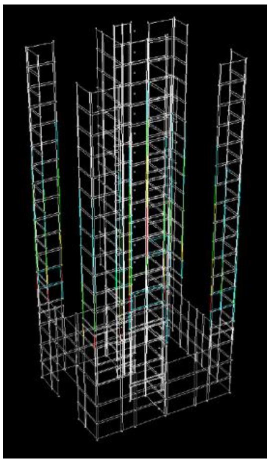

Figure 6. D/C deformation ratio on soil type C in the X direction.

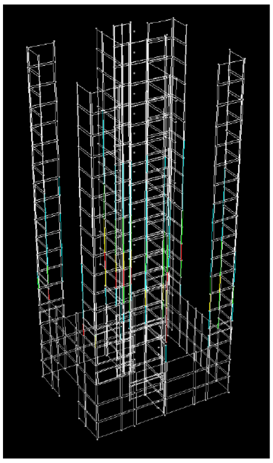

Figure 7. D/C deformation ratio on soil type C in the Y direction.

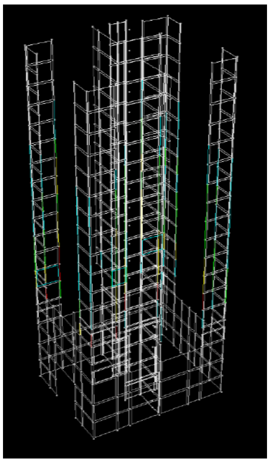

Figure 8. D/C deformation ratio on soil type D in X direction.

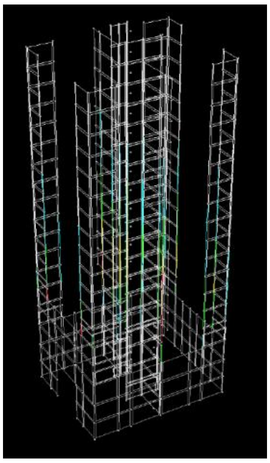

Figure 9. D/C deformation ratio on soil type D in the Y direction.

Table 1 – Demand capacity ratios of deformations for non-collapse requirement analysis

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D/C ratios for soil type A in the X direction				**D/C ratios for soil type A in the Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	**Ground motion**	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$
Montenegro	1,56	0,39	0,50	South Iceland 2	0,94	0,25	0,28
Vrancea	0,48	0,17	0,16	Campano Lucano	1,57	0,32	0,43
South Iceland	1,89	0,43	0,54	Montenegro	1,5	0,31	0,41
Tabas	0,55	0,16	0,16	Vrancea	1,89	0,33	0,51
South Iceland 2	1,58	0,43	0,47	South Iceland	1,07	0,25	0,28
South Iceland 1	2,87	0,59	0,85	Tabas	0,76	0,23	0,23
Campano Lucano	0,89	0,28	0,30	South Iceland 1	2,88	0,39	0,77
D/C ratios for soil type B in X direction				**D/C ratios for soil type B in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	**Ground motion**	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$
Tabas	1,89	0,46	0,56	Montenegro	1,4	0,3	0,38
Montenegro	0,77	0,2	0,24	Campano Lucano	0,88	0,19	0,24
Campano Lucano	1,08	0,31	0,30	Biga	3,4	0,46	0,91
Biga	5,06	1,39	1,49	Racha aftershock	0,92	0,26	0,31
Racha aftershock	0,79	0,27	0,27	Strofades aftershock	0,79	0,2	0,25
Strofades aftershock	1,04	0,31	0,35	Aigion	5,35	0,9	1,30
Aigion	4,82	1,16	1,39	Tabas	6,22	1,02	1,50
D/C ratios for soil type C in X direction				**D/C ratios for soil type C in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	**Ground motion**	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$
Azores	3,45	0,68	1,04	Kefallinia aftershock	0,7	0,19	0,20
Kefallinia aftershock	0,83	0,26	0,25	Umbria Marche 1	0,86	0,22	0,26
Umbria Marche 1	1,04	0,31	0,37	Umbria Marche 2	0,45	0,12	0,13
Umbria Marche 2	1,09	0,28	0,33	Dinar	3,24	0,54	0,86
Dinar	1,74	0,35	0,50	Izmit	4,31	0,74	1,04
Izmit	3,34	0,88	1,04	Strofades aftershock	1,13	0,31	0,39
Strofades aftershock	0,79	0,25	0,26	Azores	0,91	0,22	0,25
D/C ratios for soil type D in X direction				**D/C ratios for soil type D in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	**Ground motion**	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$
Umbria Marche	1,31	0,35	0,42	Umbria Marche	1,38	0,31	0,38
Izmit aftershock 1 x	0,92	0,28	0,31	Izmit aftershock 1 x	1,29	0,23	0,35
Izmit aftershock 1 y	0,86	0,26	0,24	Izmit aftershock 1 y	1,36	0,26	0,37
Izmit aftershock 2	4,11	0,8	1,24	Izmit aftershock 2	1,83	0,37	0,49
Izmit aftershock 3	1,05	0,28	0,35	Izmit aftershock 3	1,11	0,28	0,31
Duzce aftershock	1,48	0,4	0,43	Duzce aftershock	2,28	0,37	0,61
Duzce aftershock	1,15	0,32	0,39	Duzce aftershock	1,76	0,31	0,47

From D/C rotation ratios can be seen that on 9 from 56 ground motions, the D/C ratio has exceeded value 1, i.e. that the probability of overcoming LS state is 9/56=16.7%. However, considering mean values for each group then can be concluded that obtained rotation haven't exceeded the LS damage state. Considering that rotation capacities have been assigned according to FEMA 356 [11], in Table 2 are shown rotation capacities prescribed with EN 1998-3 [13]. Moment and shear forces for calculating rotation values have been taken from a linear model made in the software CSI ETABS [14], based on seismic forces for referent horizontal peak acceleration 0.34g and elastic response spectrum. Rotation capacities have been calculated for the most vulnerable cross sections from Figs. 2-9. θ_{SD} represents rotation capacity for significant damage limit state, which is the counterpart for LS damage state rotation according to FEMA 356 [11].

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Table 2 – Rotation capacity for limit state significant damage

Rotation capacity	Z2x	Z3x	Z4x	Z5x	Z6x	Z1y	Z2y	Z3y	Z4y	Z5y	Z6y
θ_{SD}	0,0072	0,0079	0,0072	0,0088	0,0109	0,009	0,0076	0,0065	0,0067	0,0058	0,0082

From Table 2, rotations obtained according to EN 1998-3 [13] are somewhat greater than those obtained by FEMA 356 [11], which means that D/C ratios for rotations are acceptable by EN 1998-3 [13].

Compression dilatations have exceeded limit values for certain load cases, but according to mean values for each group, compression dilatations are below limit values.

For most load cases, mean tension dilatations have exceeded the yield point. That kind of behaviour is desirable and expectable because it represents the ductile non-linear behaviour of structural walls. The desirable non-linear behaviour of the structural system with ductile walls, which is the case here, is when energy dissipation occurs in the plastic hinges zone above the foundations. From Figs. 2-9 can be observed that reinforcement yielding mainly occurred on higher floors.

Mean values for shear D/C ratios are presented in Table 3 for each wall and planar direction. It can be seen that all D/C ratios are below 1, except on wall 4 where it is 1.00. Since shear capacity is defined in software as concrete shear capacity and that horizontal reinforcement also takes a certain percentage of shear loads, the non-collapse requirement is also fulfilled in this wall.

Table 3 – Demand capacity ratios of shear forces for non-collapse analysis

Shear mean value	**Wall 1**	**Wall 2**	**Wall 3**	**Wall 4**	**Wall 5**	**Wall 6**	**Ground motion**	**Wall 1**	**Wall 2**	**Wall 3**	**Wall 4**	**Wall 5**	**Wall 6**
D/C ratios for soil type A in X direction							D/C ratios for soil type A in Y direction						
V_{mean}	0,32	0,71	0,82	0,90	0,85	0,47	V_{mean}	0,39	0,4	0,48	0,42	0,61	0,55
D/C ratios for soil type B in X direction							D/C ratios for soil type B in Y direction						
V_{mean}	0,38	0,8	0,95	1	0,93	0,56	V_{mean}	0,42	0,43	0,62	0,56	0,8	0,25
D/C ratios for soil type C in X direction							D/C ratios for soil type C in Y direction						
V_{mean}	0,34	0,76	0,88	0,93	0,91	0,52	V_{mean}	0,37	0,34	0,48	0,44	0,67	0,54
D/C ratios for soil type D in X direction							D/C ratios for soil type D in Y direction						
V_{mean}	0,39	0,73	0,83	0,90	0,88	0,49	V_{mean}	0,23	0,24	0,31	0,33	0,30	0,22

5. Analysis of damage limitation requirement

For damage limitation requirement, EN 1998-1 [1] prescribes that "construction shall be designed and constructed to withstand a seismic action having the larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself". The return period of considered earthquakes to examine damage limitation requirement is 95 years. Therefore, D/C ratios have been examined through the following ratios: $R_M/R_{C,M}$, which represents the D/C ratio for rotations, $D_{CO}/D_{C,CO}$, which represents the D/C ratio for compression dilatations $D_T/D_{C,T}$ which represents the D/C ratio for tension dilatations and $D_R/D_{C,R}$ which represents D/C ratio for inters-storey drifts (Table 4, Figs. 10-17).

Capacity rotations for the walls have been taken for the state IO according to FEMA 356 [11], in the amount of 0.003 rad. Damage limitation requirement evaluation for dilatations implies that no reinforcement yielding should be present in the wall, nor the concrete cracking.

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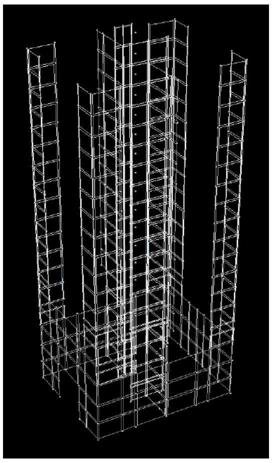

Figure 10. D/C deformation ratio on soil type A in X direction.

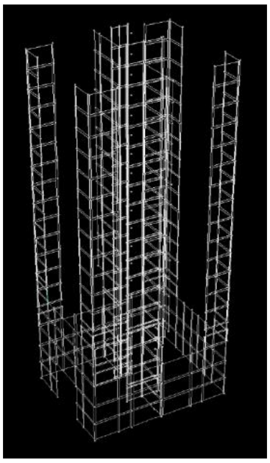

Figure 11. D/C deformation ratio on soil type A in Y direction.

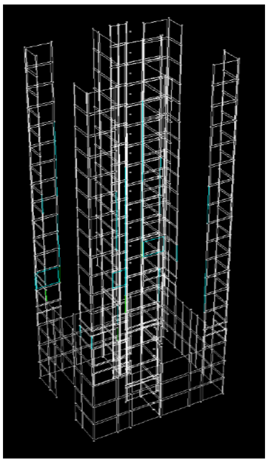

Figure 12. D/C deformation ratio on soil type B in X direction.

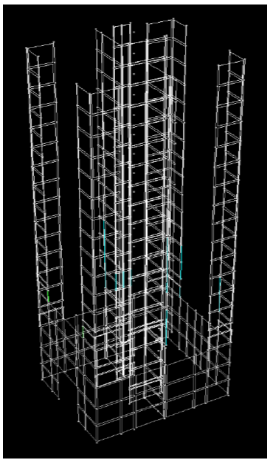

Figure 13. D/C deformation ratio on soil type B in Y direction.

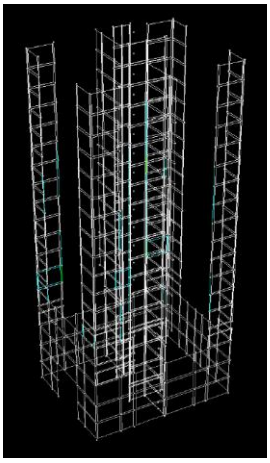

Figure 14. D/C deformation ratio on soil type C in X direction.

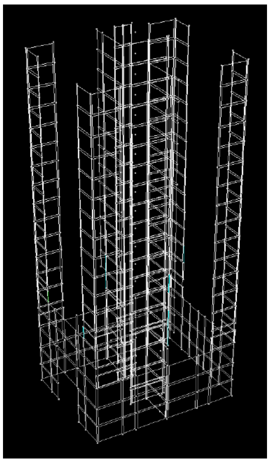

Figure 15. D/C deformation ratio on soil type C in Y direction.

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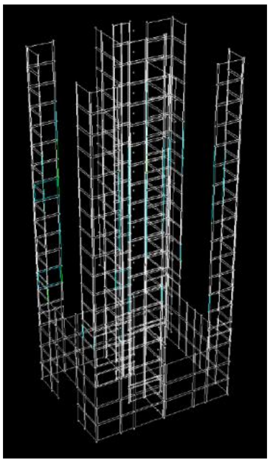

Figure 16. D/C deformation ratio on soil type D in X direction.

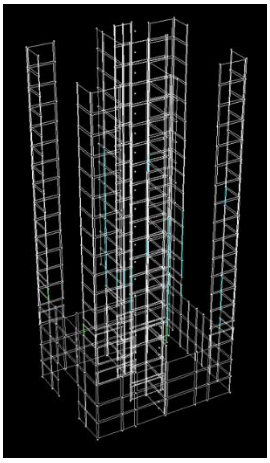

Figure 17. D/C deformation ratio on soil type D in Y direction.

The limit state for inter-storey drift has been adopted in value 0.05 from design criteria for damage limitation when non-structural elements on the building are brittle.

Table 4 – Demand capacity ratios of deformations for damage limitation requirement analysis

	D/C ratios on soil type A in X direction				**D/C ratios on soil type A in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$
Campano Lucano 1	0,54	0,17	0,32	0,55	0,66	0,14	0,36	0,75
Campano Lucano 2	0,48	0,17	0,32	0,41	0,59	0,16	0,33	0,65
Izmit	0,46	0,16	0,34	0,50	0,59	0,14	0,33	0,50
South Iceland 1	0,74	0,23	0,5	0,95	0,48	0,09	0,26	0,45
Mt. Vatnafjoll	0,39	0,13	0,29	0,4	0,46	0,12	0,25	0,60
Kalamata	0,42	0,13	0,25	0,35	0,47	0,10	0,26	0,75
South Iceland 2	0,49	0,18	0,35	0,55	0,55	0,14	0,30	0,65
	D/C ratios on soil type B in X direction				**D/C ratios on soil type B in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$
Friuli	0,49	0,16	0,29	0,55	0,48	0,14	0,31	0,8
Montenegro	0,41	0,14	0,30	0,45	0,41	0,12	0,25	0,75
Campano Lucano	0,56	0,16	0,40	0,60	0,50	0,11	0,27	0,70
Izmir	0,57	0,18	0,38	0,65	0,43	0,11	0,24	0,70
Biga	0,36	0,11	0,25	0,3	1,13	0,25	0,62	0,65
Aigion 1	1,44	0,37	0,85	1,25	1,43	0,30	0,78	1,05
Aigion 2	2,04	0,50	1,19	2,05	0,68	0,20	0,45	1,55
	D/C ratios on soil type C in X direction				**D/C ratios on soil type C in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$
Friuli aftershock	0,45	0,16	0,31	0,55	0,40	0,11	0,21	0,55
Azores	0,76	0,21	0,52	0,85	0,44	0,11	0,24	0,50
Kefallinia aftershock	0,73	0,22	0,42	0,55	0,45	0,10	0,25	0,90
Umbria Marche	0,39	0,14	0,23	0,35	0,45	0,10	0,25	1,30

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Dinar	0,62	0,18	0,41	0,60	0,89	0,23	0,51	0,55
Izmit	1,49	0,42	0,94	1,85	1,57	0,27	0,85	0,60
Griva	0,78	0,24	0,54	1,00	0,59	0,16	0,33	0,55
	D/C ratios on soil type D in X direction				**D/C ratios on soil type D in Y direction**			
Ground motion	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$	$D_T/D_{C,T}$	$D_{CO}/D_{C,CO}$	$R_M/R_{C,M}$	$D_R/D_{C,R}$
Izmit aftershock 1	0,49	0,15	0,30	0,50	0,43	0,11	0,25	0,90
Izmit aftershock 2	0,51	0,17	0,30	0,65	0,47	0,13	0,28	0,70
Izmit aftershock 3	0,55	0,15	0,30	0,50	0,44	0,14	0,30	1,40
Izmit aftershock 4	1,37	0,40	0,83	1,50	1,65	0,33	0,90	1,00
Izmit aftershock 5	1,11	0,28	0,2	1,00	0,96	0,18	0,48	0,60
Izmit aftershock 6	0,61	0,17	0,35	0,50	0,54	0,14	0,30	1,25
Duzce aftershock	0,87	0,25	0,52	1,00	0,94	0,20	0,51	0,70

The mean values of inter-storey D/C ratios are below 1.00 for each group, which means that the damage limitation requirement is fulfilled from this point of view.

Considering the tension dilatations D/C ratios, it can be seen that for each load case, those values are below 1, except on 5 load cases. That means that the probability of reinforcement yielding is 5/56=8.9%. On the other hand, considering the mean values for each group, yielding did not occur.

Limit values haven't exceeded state IO. In Figs. 10-17, several cross-section rotations are between 40%-60% (cross-sections marked with blue colour). For those cross-sections, rotation capacities have been calculated according to EN 1998-3 [13], as in chapter 4, for damage state damage limitation θ_y. Results are presented in Table 5. Based on the ratio $\theta_y/\theta(IO)$ and the exact value of rotations, rotation capacity according to EN 1998-3 [13] is not exceeded.

Table 5 - Rotation D/C ratios according to EN 1998-3 [14]

Cross-section	$\theta(IO)$	θ_y	$\theta y/\theta(IO)$
Z2x	0,003	0,0017	56,7%
Z4x	0,003	0,0017	56,7%
Z6x	0,003	0,0021	70%

6. Conclusion

Based on the conducted analysis, it can be concluded following:

- The case study building has shown satisfactory performance for seismic loads with a return period of 475 years. Mean values of rotations did not exceed capacities for significant damage limit state. In addition, compression dilations did not exceed the capacity of concrete deformations, and shear capacity was not exceeded. Therefore, the non-collapse requirement is fulfilled.
- Yielding of the reinforcement in the higher floors occurred for seismic loads with a return period of 475 years, which did not represent desirable building performance.
- The damage limitation requirement is fulfilled since construction damage did not occur for seismic loads with a return period of 95 years.

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