Seismic Risk and Vulnerability of A Residential Building

MSc.Eng.Erdit Leka[1], MSc.Eng.Esmerald Filaj[2] , Dr.Eng.Altin Bidaj[3]

[1,2] Department of Civil Constructions, Technology and Transport Infrastructure
[3] Department of Mechanic of Structures
Polytechnic University of Tirana, Tirana, Albania

Abstract: The aim of this work is the study of the seismic risk and vulnerability in a stucture comparing the results from non-linear response of structure and pushover analysis with IDARC 2D program. The theme is devided into three parts. The first part includes theoretical procedure and analysis method. The second part includes some characteristics of the structure, building capacity curves by Non-linear response of structure and by Pushover analysis with IDARC 2D program and the damage and vulnerability assessment. The third part and the last one includes conclusions and recommendations for the study conducted.

Key words: ***seismic vulnerability, damage degree assessment, seismic risk.***

Nomenclature:

γc	Self wight of the element (kN/m^3)
g	Dead load (kN/m^2)
p	Live load (kN/m^2)
K	Seismic coefficient
H	Total building height (m)
d_{max}	Storey drift (cm)
D_{max}	Maximum allowable elastic displacement (cm)
DI	Damage index(cm)
Dm	Maximum displacement (cm)
Dy	Yielding displacement (cm)
Du	Ultimate displacement (cm)
Δ_T	Displacement at the top(cm)
K_0	Coefficient of building category
K_s	Coefficient of seismic intensity
K_d	Coefficient of dynamic response
K_p	Coefficient of ductility and damping

I. INTRODUCTION

Significant damages and disasters caused by major earthquakes indicate the need of evaluation of seismic resistance of the existing structures. Numerous different approaches for nonlinear analysis and approximate methods for seismic assessment of the structures are suggested in the literature. Nonlinear time history analysis is the most accurate and the most reliable method for seismic response determination and assessment of structures under seismic loads [1] . However, there are many difficulties when performing inelastic time history analysis: suitable set of ground motion available, hysteretic behaviour models known, as well as computation time. This leads to proposal of the alternative, simpler nonlinear analysis. Those methods are usually based on reduction of the multiple degrees of freedom system to single degree of freedom system. Nonlinear static pushover analysis is developed in the last 20 years and since then it has become the most widely used method for structural response determination. A pushover analysis is performed by subjecting a structure to a monotonically increasing set of lateral loads [2]. The key phases for conducting pushover analysis are: creating mathematical model and defining the nonlinear characteristics, applying the horizontal forces which would represent the inertial forces experienced by the structure when subjected to ground shaking and increasing the loads until the target displacement is reached. The results of this analysis provide a lot of key information for the structure behaviour: assessment of the story drifts, failure modes and determining the structure capacity under seismic action. Although the procedure is simpler and requires less computational time compared to nonlinear dynamic time history analysis, there are many parameters that influence the accuracy of the results. [3]. Some of those are: the possibilities of modelling the nonlinearity in the computer program, determination of the performance point and lateral load pattern.

II. SOME CHARACTERISTICS OF THE STRUCTURE.

A. Technical characteristic/description of the building.

The considered building is rectangular in plane and symmetric (40.1m x 12.5 m), with basement, ground, first foor and with other 15 floors. Height of basement is 3m; height of ground floor is 5.6m which in part is divided in two floors (each of them of height 2.8m); height of first floor is 4.5m while height of all the stories is 2.86m. Total height of the building is 53.0 m. The structure is a moment resisting frame consisting of frames in both the X and Y directions. Slabs are R/C monolithic type with height of 12cm for all the storeys. Columns are arranged through the axis and have the span of 4.4m in the direction X and 4.6m, 2.5m, 4.6m in direction Y. Column's sections are different (90x100cm, 50x80cm, 50x60cm ect) and change with the high. Beams are of different sections (100x60cm, 70x60cm, 60x50cm, 50x40 ect) and are jointed at column's grid. For the columns till the third floor (quota 15.82m) is used concrete MB40, while for all other structural elements is used concrete MB30. The steel used for all the elements is of type RA 400/500.

B. Building's geometry.

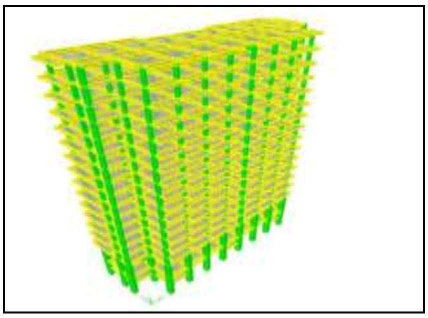


Fig. 1 3D Model (ETABS) of the structure.

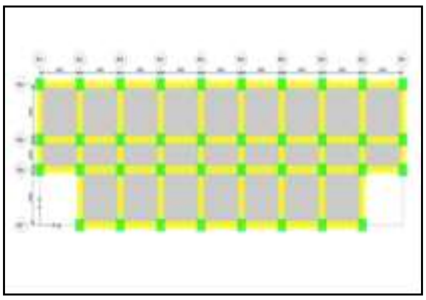


Fig. 2 Plan of the building (1-st storey).

The structural system consists of:

2 frames type R-1
2 frames type R-2 in longitudinal X direction.
6 frames type R-3

1 frame type R-4
1 frame type R-5 in lateral Y direction.
1 frames type R-6
1 frame of type R-7

C. Elastic characteristics of the structure.

For the elastic characteristics of the structure (story stiffness, elastic displacement and story drifts) computer program CSI ETABS is used, performing a 3D linear analysis.

Static load cases

- Dead load

Self weight of the elements $\gamma c = 24.5$ kN/m^3

Finishes and claddings g = 1.5 kN/m^2

Infill walls g = 3.5 kN/m^2

- Live load

Rezidencial areas p = 1.5 kN/m^2

Seismic load

- Location - seismic zone IX MCS scale
- Soil class - B (eurocode classification)
- Building category - II

Seismic forces

$$S = K * G \quad (1)$$

The total seismic coefficient K is defined according to the Code of Technical Regulations for the Design and Construction of Buildings in Seismic Active Regions No.31/81 of former Yougoslavia.

$$K = Ko * Ks * Kd * Kp \quad (2)$$

K_0 = 1 - coefficient of building category,

K_S = 0.1 - coefficient of seismic intensity for IX MSC intesity,

$K_d = \frac{0.7}{T_1} = \frac{0.7}{1.94} = 0.36$ - coefficient of dynamic response,

(T_1 – period of the first mode obtained from elastic analysis from ETABS)

K_p =1 - coefficient of ductility and damping.

K = 1 * 0.1* 0.36 * 1 = **0.036**
G = 120 045 kN
S = 0.036 * 120 045 kN = **4321.62 kN**

For the distribution of the seismic forces at the floors level it is used:
15% *S will applied directly at the top of the building (last floor)
85% *S will be distributed at the stories by formula

$$S_i = 0.85 \cdot S \frac{G_i \cdot h_i}{\Sigma G_i \cdot h_i} \quad (3)$$

0.15 * S = 648.2 kN
0.85 * S = 3673.4 kN

Total building height H=53.0 m
Maximum allowable elastic displacement is:
D_{max} = H/600 = 5300/600 = 8.83cm (4)
The maximum elastic displacement of the top storey from the analysis is **9.08cm**
9.08 > D_{max}=8.83
Maximum allowable relative displacement (storey drift) with h=2.86m is:
d_{max} = h/350 = 286/350 = 0.81cm (5)
The maximum elastic relative displacement (storey drift) from the analysis is **0.69cm**
0.69 < d_{max}=0.81

III. BUILDING CAPACITY CURVES.

A. Non-linear response of structure.

Non-linear analysis was performed by computer program NRES for seven earthquakes records (Petrovac, Ulcinj-Albatros, Ulcinj-Olimpik, Bar, Bitola94, El Centro and Parkfield) with varying of PGA’s from 0.02g to 0.55g.

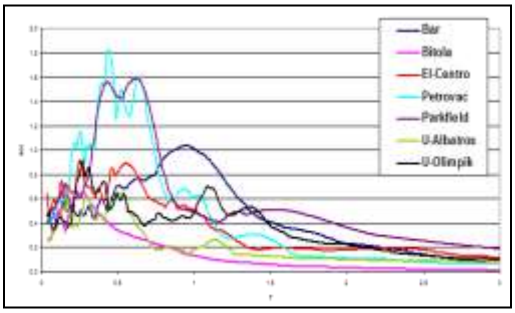


Fig. 3 Normalized specters of selected earthquakes records.

Shear-type-lumped mass model of the structure was adopted with the ductility capacity 8. Bilinear 5% strain-hardening diagram was adopted for modeling of non-linear behavior of elements.

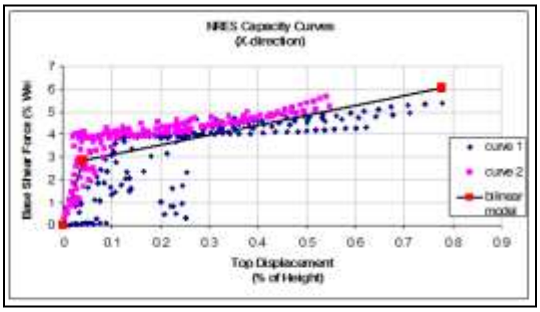


Fig. 4 NRES Capacity Curves(x direction).

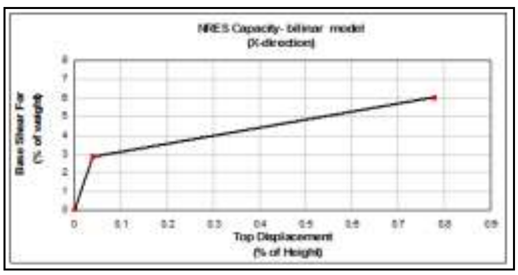
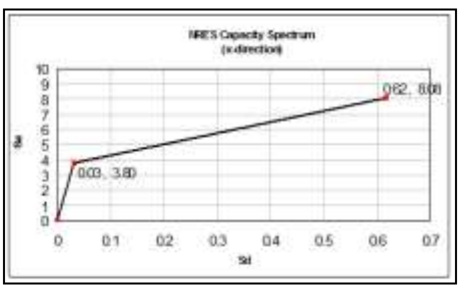


Fig. 5 NRES Bilinar Model(x direction).



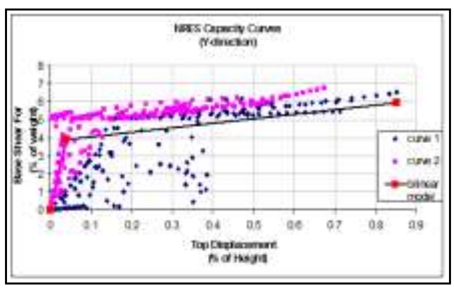


Fig. 6 NRES Capacity Spectrum(x direction).

Fig. 7 NRES Capacity Curves(y direction).

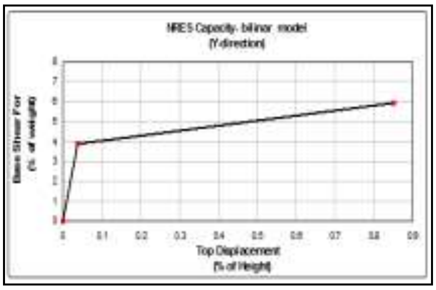


Fig. 8 NRES Bilinar Model(y direction).

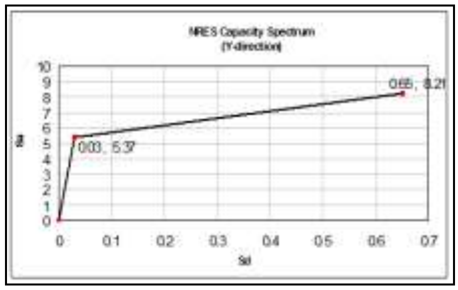


Fig. 9 NRES Capacity Spectrum(y direction).

B. Pushover analysis with IDARC2D program.

The results are presented in graphs as below:

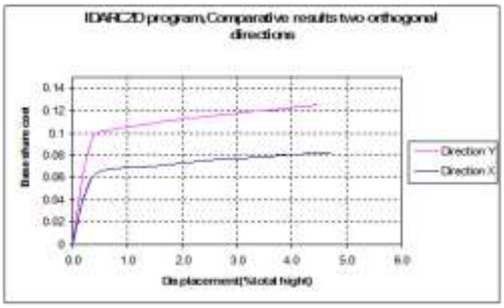


Fig. 10 Comperative results for different lateral load distribution direction Y.

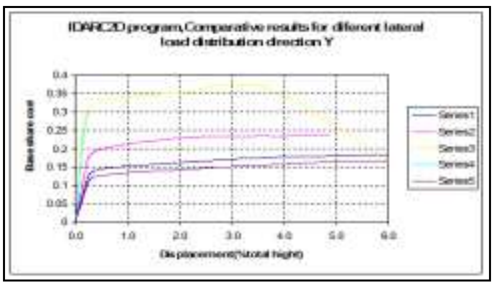


Fig. 11 Comperative results for two orthogonal direction.

Comparative results of the capacities from IDARC2D and NRES program on the direction X and Y are given below:

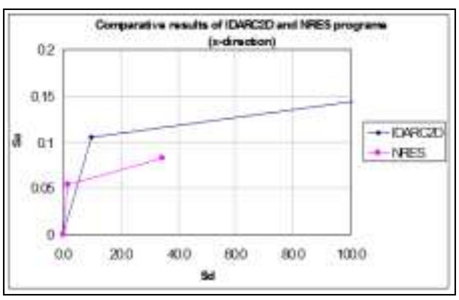


Fig. 12 Comperative results of IDARC2D and NRES program (x direction).

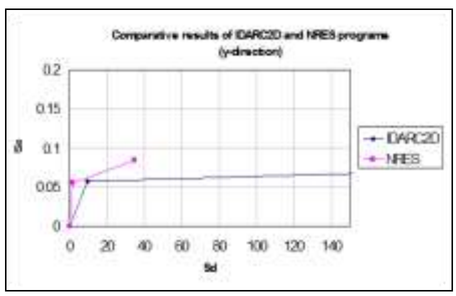


Fig. 13 Comperative results of IDARC2D and NRES program (y direction)

IV. DAMAGE AND VULNERABILITY ASSESSMENT.

Park & Ang damage index was adopted far damage assessment:

$$DI = \frac{D_m - D_y}{D_u - D_y} + \beta_e \frac{\int dE}{F_y \cdot D_u} \quad (6)$$

Computer program DAMAGE used this model for calculating damage index (DI). Damage index is a function of maximum displacement (Dm), yielding displacement (Dy) and ultimate displacement (Du) and hysteretic energy. Computer program NRES, as a result, gives maximum displacements at story level. It is necessary to develop correspond single-degree-of-freedom system which spectral displacement will be input for calculation of damage index. Spectral displacement can be calculated using further formula:

$$S_D = \frac{\Delta_T}{PF_1 \cdot \phi} \quad (7)$$

$$PF_1 = \frac{\sum m_i \cdot \phi_i}{\sum m_i \cdot \phi_i^2} \quad (8)$$

Δ_T – displacement at the top.

After that, a normal probability function Φ(σ) was used for calculation of conditional probability for exceeding certain level of damage or achieving certain level of damage.
For exceeding certain level of damage:

$$P(DI \geq DI_k) = 1 - \phi\left(\frac{DI_k - DI}{\sigma}\right) \quad (9)$$

$0.10 < DI \leq 0.20$ - minor damage
$0.20 < DI \leq 0.40$ - moderate damage
$0.40 < DI \leq 1.00$ - severe damage
$DI \geq 1.00$ - collapse

For achieving certain level of damage:

$$P(D_s) = \frac{\phi\left(\frac{DI_{k+1} - DI}{\sigma}\right) - \phi\left(\frac{DI_k - DI}{\sigma}\right)}{\phi\left(\frac{DI_n - DI}{\sigma}\right) - \phi\left(\frac{DI_i - DI}{\sigma}\right)} \quad (10)$$

Two computer programs VULN and VULNM were used for these calculations. VULN was used for obtaining Fragility Curve and VULNM for obtaining Damage Probability Matrix.Results of analysis from VULN and VULNM are shown below:

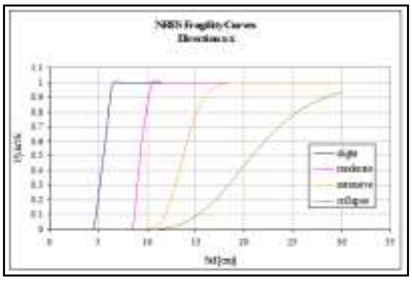


Fig. 14 NRES Fragility Curves (x-xdirection).

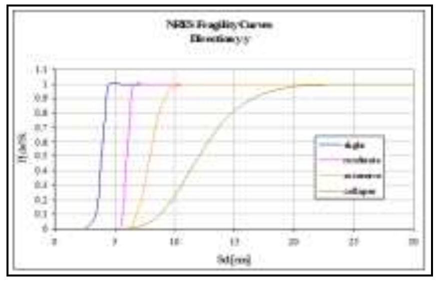


Fig. 15 NRES Fragility Curves (y-ydirection).

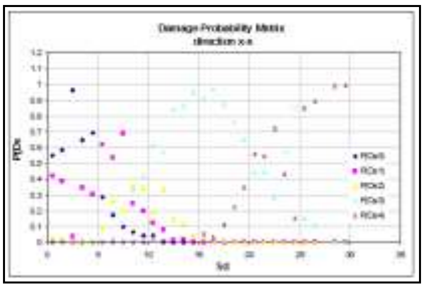


Fig. 16 Damage Probability Mtrix (x-x direction).

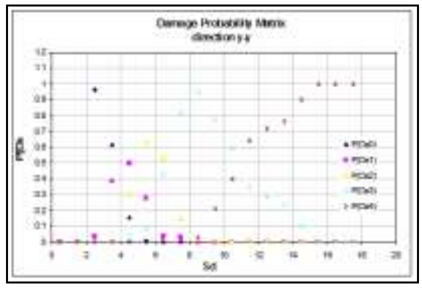


Fig. 17 Damage Probability Mtrix (y-y direction).

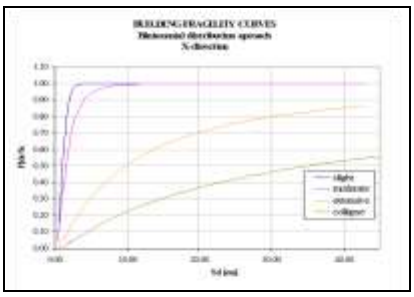


Fig. 18 Bininomial distribution approach (x direction).

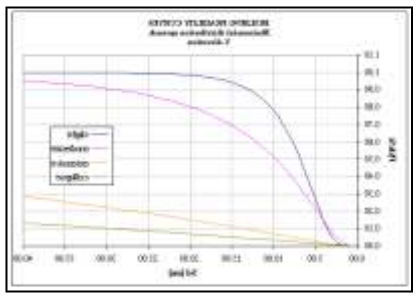


Fig. 19 Bininomial distribution approach (y direction).

V. CONCLUSIONS

Considered structure presented and analyzed in this work it was designed according to YU code. Elastic characteristics of the structure (maximum displacements, interstory drifts) have been calculated using CSI ETABS computer program. The analysis is performed by Response Spectrum Analysis.

From elastic analysis it can be noted that the second mode shape of vibration is twisting, that is not recommended according EuroCod 8. This is a fact that increase the need of careful analysis in three-dimensional models.

From the results of elastic analysis it is mentioned that the base shear forces on the structure are of a small order due to dynamics properties, especially the first periods of vibrations. High values of first modes of vibrations corresponds to small values in the design response spectra.

Control of the structure is done by performing two type of analysis:

Nonlinear Static Analysis (Push over Analysis performed by IDARC2D)

Nonlinear Dynamic Analysis (Nonlinear Time History Analysis performed by NRES)

(The input data for both programs are given in digital form)

The structure is analyzed in both X and Y direction separately.

From NRES results it can be seen in the corresponding graph that the capacity of the building have a small differences in directions X and Y.

The maximum top displacement (as % of the H) for X direction is 0.779 and for Y direction is 0.85.

By using IDARC 2D program we can see the type of the failure of the structure. Separately for each frame type this program gives the failure moment. By controlling the shear force-story drift relationships for each floor separately it can be seen that the floors with the biggest nonlinearity behavior that initiate the failure of the building.

As an example, this relationship are presented for the first, eight and top story in the same graph. From the given graph the top story will remain in elastic range and the plasticity will be concentrated on the first floors [4] .

With IDARC 2D program it can be obtained different pushover capacity curves depending on 5 (five) types of seismic loads distribution along the height of the building [5] .

These results are given on the corresponding graph.

Comparing the results from both programs IDARC2D and NRES in Spectral coordinates it can be seen some discrepancy. Normally this discrepancy should be of a small order due to the dynamics type of nonlinear analysis in NRES program.(IDARC 2D is a nonlinear static analyze).

From using the program VULN and VULNM it has been constructed the fragility function and damage probability matrix for both direction X and Y.

For each step of S_d it is seen probability of exceeding of damages [6] . Probability for exeeding collapse P(collapse)=0.5 is reached for S_d=21cm in x direction.

From plotting the demand spectra and the capacity spectra from Pushover analyses on the same spectral coordinate, it is defined the performance point. It is seen that the structure as one single degree of freedom can be in elastic range under this excitation.

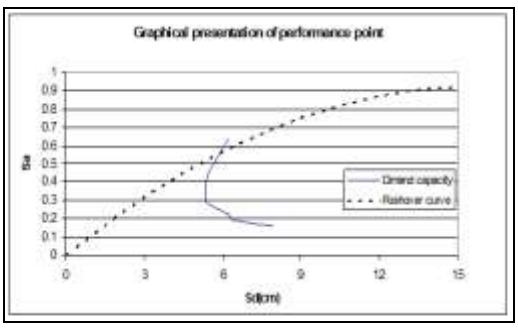


Fig. 20 Graphical presentation of performance point.

It is said that some more deeper analyses have to be made in order to see the influence of the second mode of vibration (the second mode is twisting).

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