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Investigation of Performance Analysis of an Existing Building with Nonlinear Method

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Abstract - In recent years, civil engineers have been turning to nonlinear static analysis or thrust analysis, which directly estimates the amount and location of plastic flow in a building, repairing or strengthening of structures built before the situation in Turkey and analysis may be required, durability, stiffness or assumptions regarding ductility may not be reliable. In an existing building, seismic performance evaluation was made using nonlinear analysis and a number of results were obtained by considering the capacity spectrum method and the displacement coefficient method as a performance-based analysis method.

Keywords: *Pushover Analysis, Capacity Spectrum Method, Displacement Coefficient Method, Seismic Performance*

1. Introduction

In general, a building needs to be modeled and analyzed as a three-dimensional combination of its elements and components. It is important that all elements that are part of the lateral or vertical load system and have significant rigidity or limited deformation capacity are represented in the analytical model. The elastic and inelastic strength and stiffness characteristics of each element should be modeled to the extent that their important influence on the response of the building is reasonably represented.

If the elements have inelastic deformation capacity but the strength attenuation does not necessarily lead to unacceptable performance, force deformation models should include the post-distortion interval as shown in figure 1. In a typical building, almost all elements, including many non-structural components, will contribute to the overall rigidity, mass, damping of the building and, consequently, its response to earthquake ground motion.

The general force and deformation curves used to indicate component modeling and acceptance criteria for deformation controlled actions in any of the four basic material types are shown in figure 1. Elements and components that provide the capacity to withstand collapse under seismic forces induced by ground motion in any direction are

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classified as primary. Other elements and components are classified as secondary. A linear response is depicted between point A (no-load component) and an effective pour point B. The slope from B to C is typically a small percentage (0-10%) of the elastic slope and is included to represent phenomena such as strain stiffening. C has a coordinate representing the strength of the element and an apse value equal to the deformation at which significant strength degradation begins. Beyond point D, the element responds to point E with significantly reduced strength. For deformations larger than point E, the element strength is essentially zero.

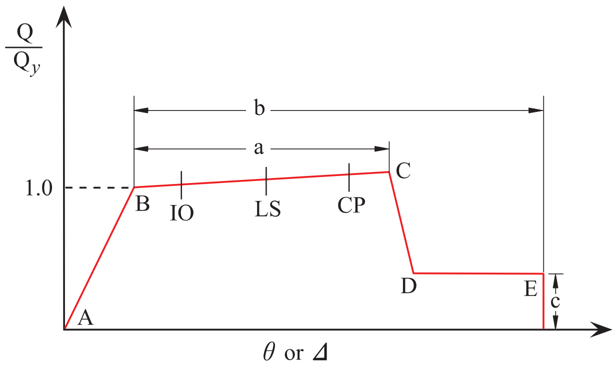


Figure 1: Element Deformation [1]

As shown in the idealized curves between points C and D in Figure 1, sharp transition may result in computational difficulty and convergence failure when used as a modeling input in nonlinear computerized analysis software. To avoid this computational imbalance, a small slope can be provided to the segment of these curves between points C and D.

- CP: Creating Crash Prevention Performance Levels
- LS: Life safety
- IO: Immediate occupancy

As shown in Figure 1, five points labeled A, B, C, D and E are used to define the load deviation behavior of the joint, and three points labeled CP, LS and IO are used to define the acceptance criteria for the joint. Modeling is one of the important steps in the implementation of push analysis. The model should take into account the nonlinear behavior of the structural members, which requires determining the component measured by the strength and deformation capacities in the structure. The final deformation capacity of a component depends on the final curvature and plastic hinge length. Using different criteria for the final curvature and different plastic hinge length may result in different deformation capacities [2].

2. Materials and Methods

In general, nonlinear analysis methods provide more useful and more accurate results than linear methods. If a load irregularity exists and concentrates on a single layer, the nonlinear static method is expected to provide good estimates about the displacement and load demand parameters. an invariant load model is unlikely to detect more than one irregularity. In such cases, nonlinear response history analysis is recommended. The application of nonlinear single mode push analysis is discussed below.

Number of Story :	7
Building Height :	21.42 (m)
Building Height Above Rigid Basement :	18.00 (m)
Rigid of Basement Story Number :	1
Rigid Basement Story :	-1
Maximum Story Height :	3.50 (m)
Maximum Beam Clearance :	7.04 (m)
Planned Usage :	Home + Shop
Number of Rigid Diaphragms :	7

(a)

Columns :	C30 S420
Shear Walls :	C30 S420
Beams :	C30 S420
Slabs :	C30 S420
Foundations :	C30 S420
Concrete Safety Coefficient :	1.50
Reinforcement Safety Coefficient :	1.15
Concrete Unit Volume Weight :	2.50

(b)

Figure 2: (a) Structure geometric information (b) Material information

Building Importance Factor (I) :	1
Building Usage Class (BUC) :	3
Carrier System Behavior Coefficient (Entered) (X/Y) :	1
Carrier System Behavior Coefficient (Selected) (X/Y) :	1
Strength Excess Coefficient (X/Y) :	1
Eccentricity Ratio :	0.05
Ductility Level :	High
Earthquake Ground Motion Level :	DD-2
Earthquake Design Class (EDC) :	1
Building Height Class (BHC) :	5
Normal Performance Destination :	Controlled Damage
Evaluation / Design Approach :	Assessment and Design by Strain

Soil Type :	ZC
Spectrum Characteristic Periods :	Ta : 0.07, Tb : 0.33
Bearing Capacity of Soil:	
Coefficient of Soil Reaction :	
Short Period Map Spectral Acceleration Coefficient (Ss) :	1.198
Map Spectral Acceleration Coefficient for 1.0 Second Period (S1) :	0.312
Short Period Design Spectral Acceleration Coefficient (SDs) :	1.438
Design Spectral Acceleration Coefficient for a 1.0 Second Period (SD1) :	0.468
The Biggest Place Acceleration (g) (PGA) :	0.492
Maximum Place Speed (PGV) :	31.314

Figure 3: Earthquake parameters and soil parameters

Table 1: Loads in the Structure

Load States	Explanation
G	Dead Load
Q	Live Load
EX	Static Earthquake Load
EY	Static Earthquake Load
SPX	Horizontal Elastic Design Spectrum
SPY	Horizontal Elastic Design Spectrum
PUSH0	Nonlinear Vertical Load
PUSHX	Nonlinear Pushover Load
PUSHY	Nonlinear Pushover Load

G	Dead Load
Q	Live Load
EX	Static Earthquake Load

Table 2: Load combinations

Load Combinations
1.4G+1.6Q
G+0.3Q G+Q+EX
G+Q+SPX G+Q+EY
G+Q+SPY 0.9G+EX
0.9G+SPX 0.9G+EY
0.9G+SPY G+0.3Q+EX+0.3EY
G+0.3Q+SPX+0.3SPY G+0.3Q+EX-0.3EY
G+0.3Q+SPX-0.3SPY G+0.3Q-EX-03EY
G+0.3Q-SPX-0.3SPY G+0.3Q-EX+0.3EY
G+0.3Q-SPX+0.3SPY G+0.3Q+EY+0.3EX
G+0.3Q+SPY+0.3SPX G+0.3Q-EY-0.3EX
G+0.3Q-SPY-0.3SPX
1.4G+1.6Q
G+0.3Q G+Q+EX
G+Q+SPX G+Q+EY
G+Q+SPY 0.9G+EX
0.9G+SPX 0.9G+EY
0.9G+SPY G+0.3Q+EX+0.3EY
G+0.3Q+SPX+0.3SPY G+0.3Q+EX-0.3EY
G+0.3Q+SPX-0.3SPY G+0.3Q-EX-03EY
G+0.3Q-SPX-0.3SPY G+0.3Q-EX+0.3EY
G+0.3Q-SPX+0.3SPY G+0.3Q+EY+0.3EX
G+0.3Q+SPY+0.3SPX G+0.3Q-EY-0.3EX
G+0.3Q-SPY-0.3SPX

In this study, TS 500 (February 2000), TBDY 2018, ATC-40 standards and regulations were used.

2.1. Linear Analysis

Structure displacement for modal account coverage is expressed as a combination of displacement in each mode. In each mode, the structure is analyzed as a single-graded system, the resulting force and displacement are combined in proportion to the participation (weight) of the modes. Modal analysis results are given below.

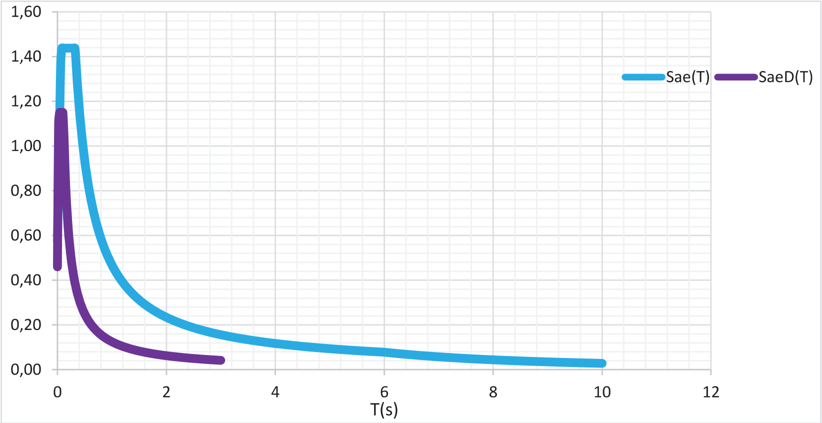


Figure 4: Horizontal spectrum for dynamic loads in the structure

OutputCase	StepType	StepNum	Period	UX	UY	SumUX	SumUY
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	1,100	0,0560	0,0783	0,056	0,078
MODAL Y	**Mode**	**2**	**0,904**	**0,1041**	**0,4623**	**0,160**	**0,541**
MODAL X	**Mode**	**3**	**0,815**	**0,5074**	**0,1541**	**0,668**	**0,695**
MODAL	Mode	4	0,330	0,0060	0,0093	0,674	0,704
MODAL	Mode	5	0,251	0,0119	0,0853	0,685	0,789
MODAL	Mode	6	0,199	0,1176	0,0201	0,803	0,809
MODAL	Mode	7	0,168	0,0032	0,0021	0,806	0,811
MODAL	Mode	8	0,141	0,0000	0,0000	0,806	0,811
MODAL	Mode	9	0,139	0,0000	0,0000	0,806	0,811
MODAL	Mode	10	0,138	0,0000	0,0000	0,806	0,811
MODAL	Mode	11	0,137	0,0000	0,0000	0,806	0,811
MODAL	Mode	12	0,136	0,0000	0,0000	0,806	0,811
MODAL	Mode	13	0,136	0,0000	0,0000	0,806	0,811
MODAL	Mode	14	0,135	0,0000	0,0000	0,806	0,811
MODAL	Mode	15	0,135	0,0000	0,0000	0,806	0,811
MODAL	Mode	16	0,134	0,0000	0,0000	0,806	0,811
MODAL	Mode	17	0,134	0,0000	0,0001	0,806	0,811
MODAL	Mode	18	0,134	0,0000	0,0000	0,806	0,811
MODAL	Mode	19	0,133	0,0000	0,0000	0,806	0,811
MODAL	Mode	20	0,121	0,0000	0,0000	0,806	0,811
MODAL	Mode	21	0,120	0,0000	0,0003	0,806	0,812

Figure 5: Modal participating mass ratios

Here, period in X direction: 0.815 s and period in Y direction: 0.904 s.

OutputCase	StepType	StepNum	Period	UX	UY	ModalMass	ModalStiff
Text	Text	Unitless	Sec	KN-m	KN-m	KN-m-s2	KN-m
MODAL	Mode	1	1,100	-0,00037	-0,00044	0,00001	0,00033
MODAL	Mode	2	0,904	0,00051	0,00107	0,00001	0,00048
MODAL	Mode	3	0,815	0,00112	-0,00062	0,00001	0,00059
MODAL	Mode	4	0,330	0,00012	0,00015	0,00001	0,00363
MODAL	Mode	5	0,251	0,00017	0,00046	0,00001	0,00628
MODAL	Mode	6	0,199	-0,00054	0,00022	0,00001	0,01001
MODAL	Mode	7	0,168	-0,00009	-0,00007	0,00001	0,01404
MODAL	Mode	8	0,141	-0,00001	0,00001	0,00001	0,01988
MODAL	Mode	9	0,139	0,00001	0,00000	0,00001	0,02043
MODAL	Mode	10	0,138	-0,00001	0,00000	0,00001	0,02078
MODAL	Mode	11	0,137	0,00000	0,00000	0,00001	0,02108
MODAL	Mode	12	0,136	0,00000	0,00000	0,00001	0,02125
MODAL	Mode	13	0,136	0,00000	0,00000	0,00001	0,02145
MODAL	Mode	14	0,135	0,00000	0,00000	0,00001	0,02154
MODAL	Mode	15	0,135	0,00000	0,00000	0,00001	0,02181
MODAL	Mode	16	0,134	0,00000	0,00000	0,00001	0,02183
MODAL	Mode	17	0,134	0,00000	-0,00001	0,00001	0,02194
MODAL	Mode	18	0,134	0,00000	0,00000	0,00001	0,02211
MODAL	Mode	19	0,133	0,00000	0,00000	0,00001	0,02219
MODAL	Mode	20	0,121	0,00000	0,00001	0,00001	0,02703
MODAL	Mode	21	0,120	-0,00001	-0,00003	0,00001	0,02735

Figure 6: Modal participation factors

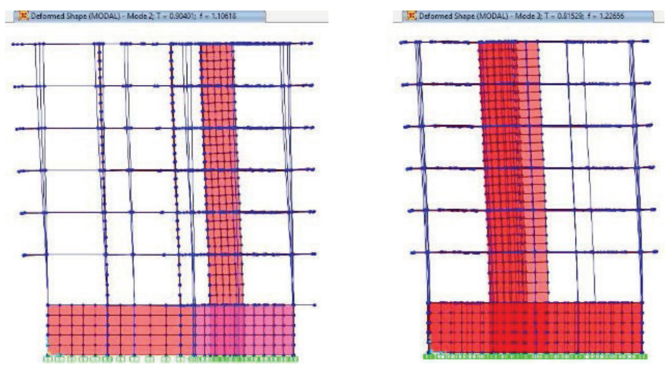


Figure 7: Deformed shapes (Modal)

OutputCase	StepType	StepNum	Period	Frequency	CircFreq	Eigenvalue
Text	Text	Unitless	Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mode	1	1,100	0,909	5,712	32,626
MODAL	Mode	2	0,904	1,106	6,950	48,307
MODAL	Mode	3	0,815	1,227	7,707	59,393
MODAL	Mode	4	0,330	3,030	19,040	362,533
MODAL	Mode	5	0,251	3,988	25,057	627,857
MODAL	Mode	6	0,199	5,035	31,637	1000,881
MODAL	Mode	7	0,168	5,963	37,464	1403,531
MODAL	Mode	8	0,141	7,096	44,587	1987,995
MODAL	Mode	9	0,139	7,194	45,204	2043,392
MODAL	Mode	10	0,138	7,256	45,588	2078,302
MODAL	Mode	11	0,137	7,307	45,908	2107,590
MODAL	Mode	12	0,136	7,336	46,094	2124,660
MODAL	Mode	13	0,136	7,372	46,318	2145,333
MODAL	Mode	14	0,135	7,387	46,414	2154,252
MODAL	Mode	15	0,135	7,432	46,696	2180,510
MODAL	Mode	16	0,134	7,436	46,723	2182,997
MODAL	Mode	17	0,134	7,454	46,836	2193,613
MODAL	Mode	18	0,134	7,484	47,022	2211,049
MODAL	Mode	19	0,133	7,498	47,109	2219,290
MODAL	Mode	20	0,121	8,275	51,994	2703,404
MODAL	Mode	21	0,120	8,323	52,293	2734,556

Figure 8: Modal periods and frequencies

2.2. Nonlinear Single Mode Pushover Analysis

A pushover analysis for determining the behavioral characteristics and performances of the structures under the effect of horizontal loads, is a numerical investigation where the stiffness and strength change are calculated by taking into account the inelastic behavioral characteristics of the building elements in general, and these calculations are defined for certain performance values. Approximate results are produced as the basis of the method is the acceptance of multi degree of freedom systems equivalent to a single degree of freedom system. The analysis method has a suitable procedure for rigid or articulated inelastic frame analysis. This procedure can be considered a different extension of the elastic frame analysis procedure, which is essentially formed with semi-rigid bonds.

Two different calculation methods applied in Pushover analysis; It is displacement-controlled analysis and load-controlled analysis. In the displacement-controlled method, horizontal loading is made until the point where the center of gravity on the top floor (top) of the structure reaches a certain displacement level. In the force-controlled method, the structure is loaded until it reaches a certain horizontal load level. By increasing these loads at certain intervals, the force-displacement relations that occur in the building elements are examined in each step and the level of damage of the structure is determined [3,4,5]. The Pushover analysis shows the change of real (functional) rigidity for loads that reveal inelastic behavior in a frame. Depending on the concept of stiffness factor previously described for the analysis of semi-rigid joints, the use of the plasticity factor to determine changes in the stiffness of the frame members under increased loads will be examined.

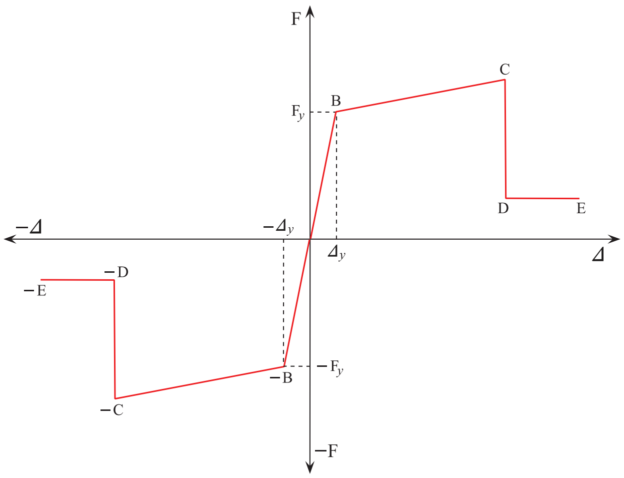


Figure 9: Curve of the F - Δ relationship

Although various methods for nonlinear analysis are determined by regulations, this analysis method, which is still being developed, creates the difficulty of determining the change in the effects against the changing structure stiffness and behavior, and the determination of the rotations in the plastic joints by the procedure for considering various modes of the structure in nonlinear analysis [6, 7, 8]. Thus, different methods are developed and discussed. The procedure given in the FEMA and ATC documents has recently received serious criticism, especially regarding the detection of element joints. In the Pushover analysis, it should be taken into consideration that the error rates are high when examining according to these regulations [9, 10].

Since the values to be determined for floor displacement and floor displacements give more convergent results, it is beneficial to consider these values as a priority in determining the performance of the building [11]. In the general logic of Pushover analysis, there is no cumulative evaluation process in the distribution of flow behavior or plastic hinge formation. It should be remembered that both calculation bases have high error rates in terms of detecting plastic hinge rotations and that results produced especially for irregular structures should therefore be well examined. However, both methods produce good results in regular structures and structural ductility and floor shifts - base shear forces.

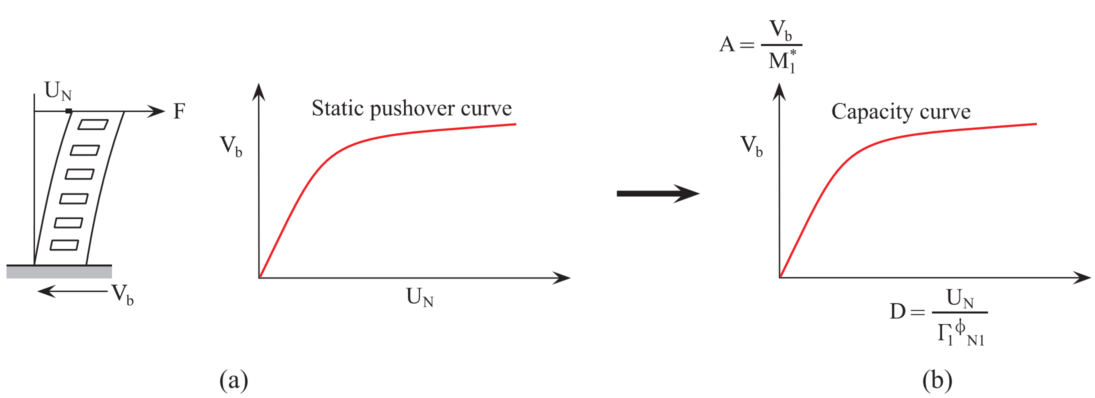


Figure 10: (a) Formation of the pushover curve (b) Converting the pushover curve into a capacity curve

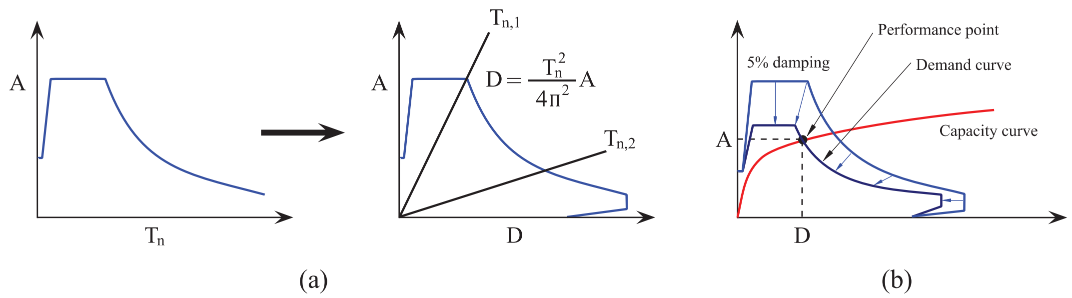


Figure 11: (a) Conversion of the elastic response spectrum from standard format to A-D format (b) Determination of displacement requirement

2.2.1. Determining the Capacity Curve

Creation of two or three dimensional model of the building, determination of the main and secondary carrier elements, ground structure interaction model, defining the second order effects, determining the characteristics of the carrier system elements, defining the cyclical behavior of force-deformation relations, the definition of articulation characters, determining the mutual interaction behavior in columns. [12, 13]. For horizontal load regulation, firstly, determining the vertical loads in the system with appropriate load coefficients, defining horizontal load patterns in both directions positively and negatively, evaluating the effects of torsion and irregularities (here the uniform displacement value for the same displacement value It should be taken into consideration that it will give great base cutting forces).

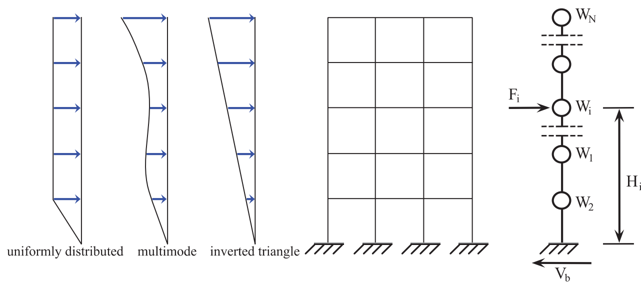
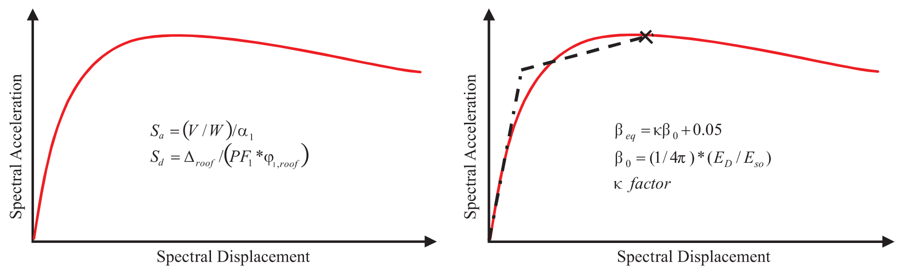


Figure 12: Horizontal load regulation [14]

2.2.2. Determination of Analysis Control Method and Calculations

If we examine this building with the capacity spectrum method ($S_d = S_a T^2/4r^2$); α mass participation (depending on the shear force), the participation factor for the roof displacement of the PF_1 single degree of freedom system is S_a spectral acceleration and S_d spektral displacement.

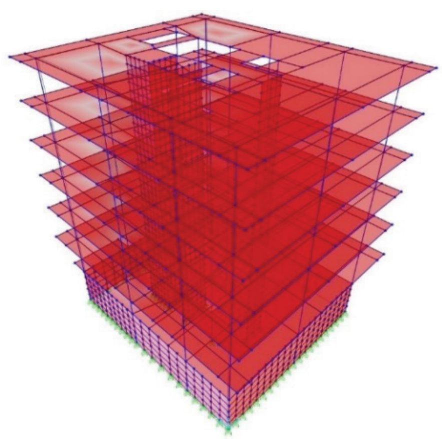


(a) (b)

Figure 13: (a) Constructing capacity spectrum (b) Estimation of equivalent viscous damping

3. Results and Discussion

Analysis results were obtained using the finite element method using the SAP2000 program.



Push Steps (X)		Push Steps (Y)	
Hinge 391		Hinge 391	
Step	Displacement (m)	Step	Displacement (m)
0	-4.98E-01	0	0.00117
1	-0.00167	1	-0.00274
2	-0.01458	2	-0.05484
3	-0.01459	3	-0.1121
4	-0.0217	4	-0.14272
5	-0.0217		
6	-0.05016		
7	-0.05016		
8	-0.10053		
9	-0.1512		
10	-0.15767		

(a) (b)

Figure 14: (a) 3D view of the building (b) Push displacement joint

According to TBDY2018 for the design shear forces taken as the basis for column and beams, if the shear force ratio of 15.7.1.4 - reinforced concrete section with deformation calculation and $V_e / (b_w d f_{ctm}) < 0.65$, the deformation upper limits calculated according to 15.7.1.3 are valid. If the shear force ratio is greater than 1.30, deformation upper limits calculated according to 15.7.1.3 will be reduced by multiplying by 0.50.

Linear interpolation will be applied for intermediate values and V_e will be calculated from equation (1) for beams and equation (2) for columns. In single mode push analysis, let us take the number of pushover steps 10 and G + nQ (KN) as 24300. The modal analysis method we will use is the number 3 in Figure 15 below.

$$V_e = 1.25 f_{yk} \left(A_{s1} + A_{s2}\right) - V_{kol} \quad (1)$$

$$V_e = \frac{M_{ü} + M_a}{l_n} \quad (2)$$

Strain borders were determined in columns and beams and analyzed by making comparisons.

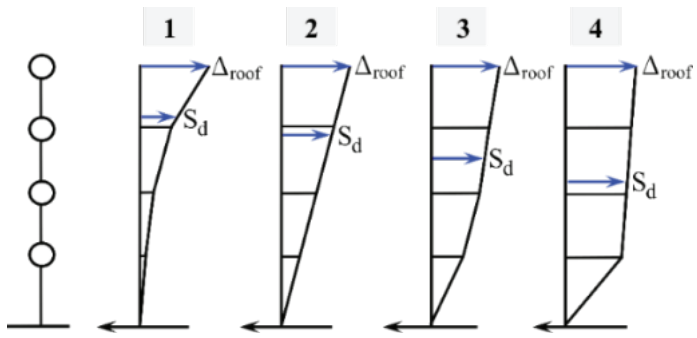


Figure 15: Modal analysis forms [15]

OutputCase	StepType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
G+0.3Q+SPX+0.3SPY	Max	10171,54	6490,51	24427,90	300962,93	-21515,54	124588,05
G+0.3Q+SPX+0.3SPY	Min	-10171,54	-6490,51	24170,17	106814,31	-325073,24	-124588,06
G+0.3Q+SPX-0.3SPY	Max	10171,54	6490,51	24427,90	300962,93	-21515,54	124588,05
G+0.3Q+SPX-0.3SPY	Min	-10171,54	-6490,51	24170,17	106814,31	-325073,24	-124588,06
G+0.3Q-SPX-0.3SPY	Max	10171,54	6490,51	24427,90	300962,93	-21515,54	124588,05
G+0.3Q-SPX-0.3SPY	Min	-10171,54	-6490,51	24170,17	106814,31	-325073,24	-124588,06
G+0.3Q-SPX+0.3SPY	Max	10171,54	6490,51	24427,90	300962,93	-21515,54	124588,05
G+0.3Q-SPX+0.3SPY	Min	-10171,54	-6490,51	24170,17	106814,31	-325073,24	-124588,06
G+0.3Q+SPY+0.3SPX	Max	6765,49	9254,93	24419,41	344036,80	-72075,90	90885,66
G+0.3Q+SPY+0.3SPX	Min	-6765,49	-9254,93	24178,66	63740,44	-274512,89	-90885,66
G+0.3Q-SPY-0.3SPX	Max	6765,49	9254,93	24419,41	344036,80	-72075,90	90885,66
G+0.3Q-SPY-0.3SPX	Min	-6765,49	-9254,93	24178,66	63740,44	-274512,89	-90885,66

Figure 16: Base shear force

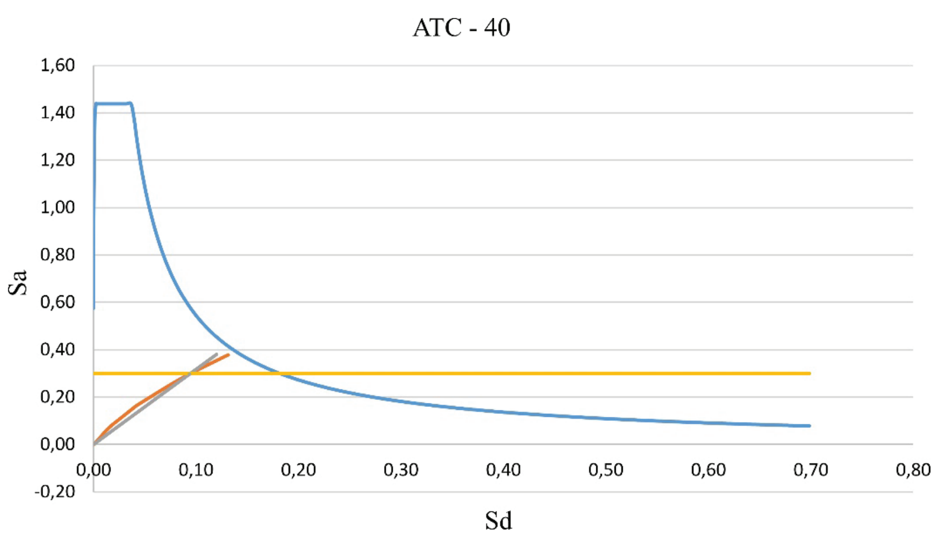


Figure 17: Spectral acceleration and spectral displacement curve

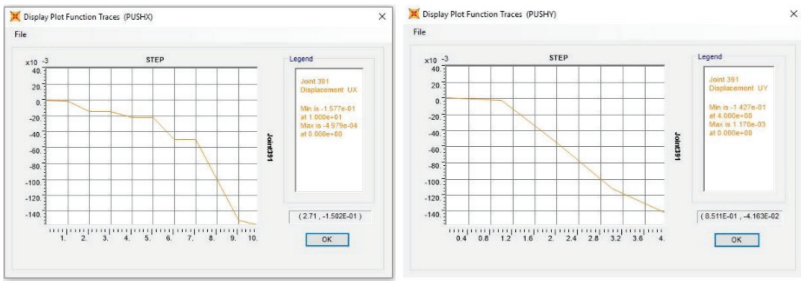


Figure 18: Display plot function traces

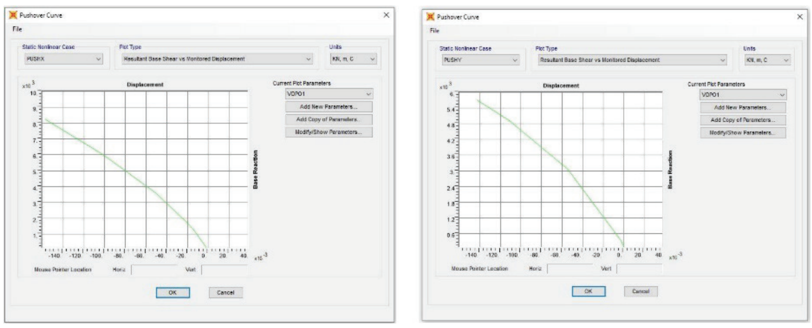


Figure 19: Creating a pushover curve

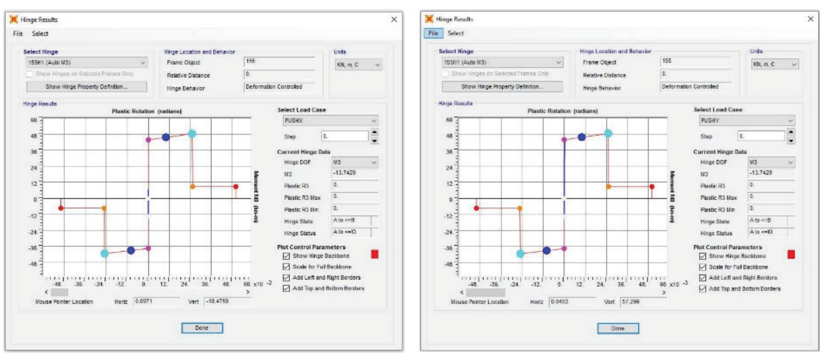


Figure 20: Plastic hinge behavior

LoadCase	Step	Displacement	BaseForce
Text	Unitless	m	KN
PUSHX	0	-0,000498	0
PUSHX	1	-0,001672	115,336
PUSHX	2	-0,014583	1279,592
PUSHX	3	-0,014588	1278,914
PUSHX	4	-0,021699	1793,011
PUSHX	5	-0,021704	1791,54
PUSHX	6	-0,050156	3567,639
PUSHX	7	-0,050161	3576,237
PUSHX	8	-0,100532	5982,031
PUSHX	9	-0,151201	8003,228
PUSHX	10	-0,157666	8248,296

LoadCase	Step	Displacement	BaseForce
Text	Unitless	m	KN
PUSHY	0	0,00117	0
PUSHY	1	-0,002737	325,495
PUSHY	2	-0,054843	3066,572
PUSHY	3	-0,112096	4931,475
PUSHY	4	-0,14272	5708,385

Figure 21: Push capacity diagram

$R_y = 1.9500$ and $C_R = 1$

$d_{1,max} = S_{di}(T_1) = \mathbf{0.16}$ (m) $\qquad d_{imax} =$ 0.145 (m) ✓

$\Downarrow$

Base shear force $V_{max} =$ **13000** (KN) $\qquad V_{max} =$ 10171 (KN) ✓

TA :	0.07
TB :	0.33
TL :	6
T1 :	0.815

Figure 22: d_{imax} - V_{max}

Adım	D (m)	V (KN)	Sd	Sa
0	0.000498	0	0.000415	0
1	0.001672	115.336	0.001393333	0.005274
2	0.014583	1279.592	0.0121525	0.058509
3	0.014588	1278.914	0.012156667	0.058478
4	0.021699	1793.011	0.0180825	0.081985
5	0.021704	1791.54	0.018086667	0.081918
6	0.050156	3567.639	0.041796667	0.163129
7	0.050161	3576.237	0.041800833	0.163522
8	0.100532	5982.031	0.083776667	0.273527
9	0.151201	8003.228	0.126000833	0.365945
10	0.157666	8248.296	0.131388333	0.377151

Figure 23: Pushover capacity diagram

4. Conclusion and Suggestions

Ordinary response spectrum analysis for elastic high-rise buildings can be reformulated as nonlinear push analysis. The peak response of an elastic structure, which is subjected to lateral loading by push analysis, can be estimated and can provide accurate seismic demand prediction in unsymmetrical structures. Analyzing the behavior of the beam column joints and the failure of plastic joints can be an effective and useful approach. Controlled damage performance level in existing buildings is considered to be at the level of controlled damage performance, provided that the brittle damaged elements are reinforced, provided that:

- Up to 35% of beams and vertical elements (columns,curtains and reinforced partition walls) in any floor of reinforced concrete buildings, excluding secondary beams (not included in the horizontal load bearing system) as a result of the calculation made for each earthquake direction applied as many as defined in the paragraph below can proceed to the advanced damage zone.
- The total contribution of the vertical elements in the advanced damage zone to the shear force carried by the vertical elements on each floor should be below 20%. The ratio of the total shear forces of the vertical elements in the forward damage area to the top floor to the sum of the shear forces of all vertical elements on that floor can be at most 40%.

Table 3: Number of beams crossing the border

Stories	Number of beams	Number of Elements Crossing the Border	%	Control
5 st story	61	4	6	✓
4 st story	64	3	5	✓
3 st story	64	4	6	✓
2 st story	64	3	5	✓
1 st story	64	0	0	✓
Ground Floor	64	0	0	✓
1st Basement Floor	29	0	0	✓

Frame	Station	OutputCase	CaseType	StepType	V2	V2
Text	m	Text	Text	Text	KN	Control
20	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	126.517	3.98%
21	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	340.937	10.73%
22	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	126.22	3.97%
23	0.14999	G+0.3Q+SPX+0.3SPY	Combination	Max	179.79	5.66%
24	0.14999	G+0.3Q+SPX+0.3SPY	Combination	Max	219.933	6.92%
25	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	190.478	5.99%
26	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	450.28	14.17%
27	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	487.014	15.32%
28	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	437.389	13.76%
29	0.14999	G+0.3Q+SPX+0.3SPY	Combination	Max	203.866	6.41%
30	0.14999	G+0.3Q+SPX+0.3SPY	Combination	Max	163.664	5.15%
31	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	119.42	3.76%
32	0.15	G+0.3Q+SPX+0.3SPY	Combination	Max	132.88	4.18%
			Total	**Max**	**3178.388**	**100.00%**

Figure 24: Control of ground story columns

Curtain walls with higher rigidity make dynamic analysis easier and simpler. Taking into account the shear effects and applying the unbalanced force approach is essential for a safe and realistic seismic response. If the earthquake loads are large, joints can form in the beams near the middle opening. In such cases, cyclic loads gradually increase the rotation of the joints and cause the beam to sag. Concrete beams are often fragile at shear forces and are therefore designed for flexural strength.

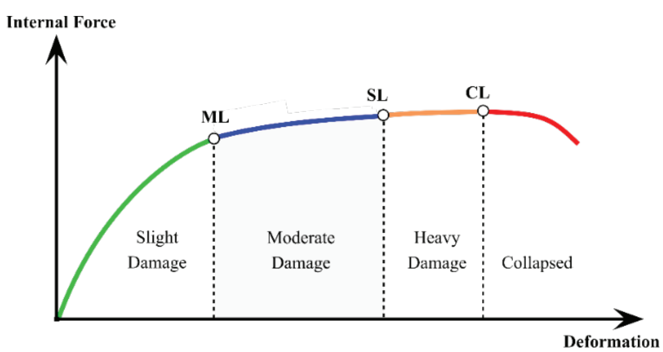


Figure 25: Section damage levels.

Axial force - moment interactions are additionally required for the columns because the flexural strength depends on the axial force and vice versa. In addition, moments and shear forces move along two axes and force - moment values and interactions affect shear strength. Due to these mutual relationships, the hinge behavior is complex. The building we examined, according to the nonlinear analysis (Single mode push analysis), remained on the border of controlled damage (Distinctive damage zone). However, it is possible to say that the capacity spectrum and the displacement coefficient methods both give close results in the analysis of symmetrical structures. But these methods should be used with caution, as they differ in complex and large structures. Nonlinear time history analysis is essential for such cases.

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