

Seismic Performance Assessment of Reinforced Concrete Buildings Using Nonlinear Static Pushover Analysis

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Abstract— Seismic assessment of reinforced-concrete (RC) buildings requires understanding behavior beyond the elastic range. Linear analysis cannot reveal yielding or capacity loss; nonlinear dynamic analysis is costly, and few studies report pushover output for a regular frame and an existing building. This study applies nonlinear static (pushover) analysis to both. Two three-dimensional models were analysed: a regular four-story frame and an existing three-story Libyan building. Models were developed in SAP2000 with nonlinear hinges assigned to beams and columns. Monotonic lateral loading was applied until a mechanism formed. Capacity curves were converted to capacity spectra following ATC-40. Evaluation used base shear, roof displacement, hinge states (Immediate Occupancy, Life Safety, Collapse Prevention) and the performance point. The frame peaked at 483.87 kN, then lost capacity abruptly, whereas the building peaked at 2256.583 kN, degrading gradually, with a performance point at 48.973 mm and 1786.369 kN. It contributes a documented workflow and comparative results.

Keywords— Pushover analysis, nonlinear static analysis, reinforced concrete, plastic hinge, capacity spectrum.

1. Introduction

Buildings are subjected to earthquake demands that regularly go beyond the yield point of their members[1]. To find out if the building would resist these demands without any unacceptable structural damage, it is necessary to use an approach based on the structural behaviour beyond the elastic limit. Such an approach is nonlinear structural analysis that is now an indispensable part of any assessment, while advanced computer programs have made it convenient for engineers in everyday practice[2]. Existing buildings were not designed or checked under the action of earthquakes or wind loads, so their evaluation and rehabilitation would be impossible without finding out their true capacity. The conventional approach to structural design includes linear static or quasi-dynamic analysis[3], [4]. It is approximate and satisfies the requirements of normal design, but it cannot reveal yielding sequence, force redistribution, or ultimate structural capacity[5]. The latter can be revealed using nonlinear dynamic analysis, but this approach is quite complicated and time-consuming, especially for complex buildings.

Nonlinear static analysis, also known as pushover analysis, represents a practical middle way. The structure is subjected to a gradually increasing lateral load either in force control or displacement control modes up to achieving a certain displacement or developing a collapse mechanism[6]. This provides a capacity curve of the overall structure along with the plastic hinge state at every step. During the past two decades, this method has gained popularity in seismic design and assessment as it is rather straightforward and considers post-elastic behaviour of structures, yet some simplifications lead to variability in seismic demand prediction[7]. The goals of this study are to introduce pushover analysis as a practical methodology which provides characteristics not available from elastic analysis; demonstrate its application in SAP2000; perform such an analysis on a regular RC frame structure and on an existing RC building in Libya; and highlight the advantages and disadvantages of the method. The novelties of the study are the described process of modelling and analysis and the capacity curves, hinge states, ATC-40 performance points and their comparative analysis for both cases.

1.1 Problem statement

The linear analysis fails to explain how RC structures deform, distribute forces, and lose their capacity during earthquakes, whereas the nonlinear dynamic analysis is complex and expensive[8]. There is a need for a simple technique to predict the true structural capacity of existing buildings that have not been designed based on earthquake resistance criteria[9]. This study tries to fill this gap using pushover analysis on SAP2000 software.

1.2 Key contributions

- A documented SAP2000 pushover workflow, from model definition and hinge assignment to capacity curves, hinge states and ATC-40 performance-point extraction.
- Numerical pushover findings, capacity curves and hinge-state tables for a regular four-story frame and an existing three-story building, reported unaltered.
- An ATC-40 performance point for the existing building (48.973 mm, 1786.369 kN), showing demand is met before the capacity plateau.
- A comparison showing the frame losing capacity abruptly, while the building spreads yielding over 718 hinges and degrades only gradually.

2. Related Works

Kazemi et al. [10] set out to model seismic response and performance of RC moment-resisting frames. The trained machine learning models include neural networks and extreme gradient boosting. The data set consisted of 92,400 data points from incremental dynamic analyses of 165 two- to twelve-story frames subjected to near-fault excitations. The models provide an accurate estimation of the maximum interstory drift ratio and were tested on a 5-story building. The dataset is, however, limited to near-fault data only, and generalisation is limited.

Rodríguez et al. [11] compared the different techniques used to assess the seismic response of structures, specifically maximum displacement and collapse evaluation. They used modal spectral analysis, non-linear dynamic analysis and incremental pushover analysis for one building. The structure was tested with an earthquake magnitude equal to the one that happened in Lorca in 2011. The observations indicated that time step and ductility have an influence on the accuracy, and that the building would be destroyed by the earthquake. A single structure was studied, however, so findings can only be generalised to that structure.

Nascimbene et al. [12] evaluated the seismic performance of an RC hospital building retrofitted with dissipative bracing throughout the world. Evaluated two traditional solutions and one innovative solution with low activation through nonlinear static pushover analysis according to the Italian Building Code NTC 2018. The case study was based on a single hospital building. Improved stiffness and damping, lowered lateral movements and raised the safety index above 60%. But local optimisation and device optimisation are not covered.

Ahmad et al. [13] explored the seismic response of RC frames with deficient beam-column joints due to joint damage and deterioration. The suggested nonlinear model has decreasing hysteresis and fibre-section elements, which are numerically efficient and validated against a full-scale quasi-static cyclic T-joint test. When applied to a 5-story frame, joint shear was seen to increase the demand for beam chord rotation by 85% and the collapse probability from 4.20% to 29.20%. But validation was based on a single exterior joint test.

Ahmed et al. [14] proposed a framework for assessing the seismic risk, economic, social and environmental losses, resilience and vulnerability of RC buildings under retrofitting alternatives based on their performance. This method is based on nonlinear pushover capacity curves, consequence functions and loss index weights. A five-story hospital with four retrofitting alternatives served as the example. The framework helped in optimising the selection of the retrofit. But it was shown in one building; this limited the generalisation.

The aforementioned papers [10]–[14] include topics such as machine-learning prediction of drifts, comparative study on analysis methods, retrofitting of a hospital structure, joint degradation modeling, and pushover-based seismic risk and retrofit framework. In general, all these studies indicate the crucial importance of pushover and nonlinear analyses in evaluating seismic behavior of buildings; however, most of them rely only on one particular building, joint or set of data; therefore, the results are not applicable. Only few publications provide full pushover results (capacity curves, hinge state development, ATC-40 performance points) for both regular frame and existing buildings analysed by the same method.

2.1 Research gap

Several studies describe how to perform the pushover analysis in a general sense or with tutorials[15]. There have been few instances where the numerical outcomes are provided together for a simple regular structure as well as a real

existing structure analysed using the same technique. This study attempts to fill that void and provides the original software outcomes without any modifications wherever necessary.

3. Proposed Pushover-Based Seismic Evaluation Framework

Four steps are involved. The first is the creation of two three-dimensional RC structures using SAP2000, namely a four-story regular structure and the existing three-story Libyan structure. The geometry, materials, sections, loads and loading patterns are specified, along with the introduction of nonlinear hinges. The second step involves the application of a monotonically increasing lateral load such that hinges are created where the sections have attained their capacity, until either a mechanism develops or the required displacement is attained. The third step includes recording the roof displacement, base load and state of the hinges in order to create the capacity curve, as shown in Fig.1.

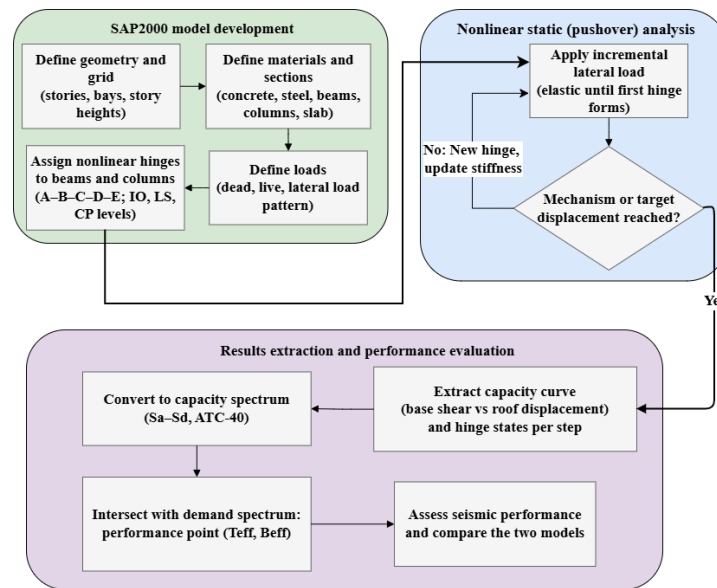


Fig. 1 Structural modelling and pushover analysis workflow

3.1 Structural configurations and material properties

Case 1 (regular frame): 3D moment-resisting frame structure, four stories at 4m height each, 5m bay (one in x-direction and two in y-direction), M-20 concrete ($E_c = 22.36$ GPa), 415MPa steel, 0.0035 strain.

Case 2 (existing RC structure): Three-story structure in Owbary, southern Libya, 3.0 m story height, 13m $\times$ 22.8m plan area using response spectrum and non-linear static method of analysis, lateral load applied to back wall.

The regular four-story RC frame is presented by its floor plan and section elevation. This figure demonstrates the bay arrangement, with 1 5m bay in the x-direction and 2 in the y-direction, with 4m story heights as presented in Fig.2.

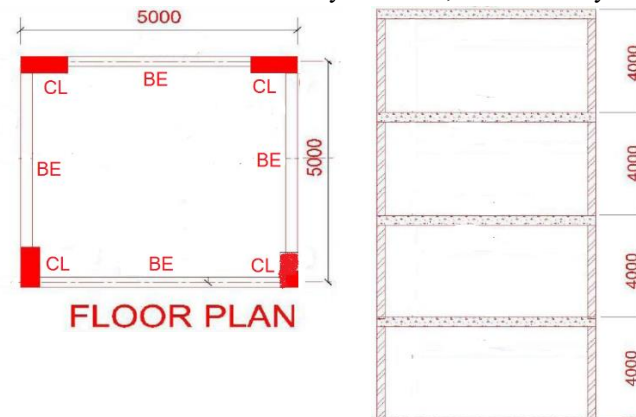


Fig. 2 RC frame configuration: floor plan and sectional elevation

Structural forms and material properties of both structures are summarised, such as number of storeys, storey height, plan/bay dimensions, structural form, grade of concrete, steel yield strength and ultimate concrete strain, as shown in Table I.

Table I. Structural configurations and material properties

Parameter	Case 1: regular frame	Case 2: existing building
Number of stories	4	3
Story height	4 m	3.0 m
Plan/bays	1 bay $\times$ 5 m (x); 2 bays $\times$ 5 m (y)	13 m $\times$ 22.8 m
Structural system	Moment-resisting 3D frame	Existing 3D RC building
Concrete	M-20, $E_c = 22.36$ GPa	As modelled in SAP2000
Steel yield stress	415 MPa	As modelled in SAP2000
Ultimate concrete strain	0.0035	As modelled in SAP2000

The dimensions of the cross-section, cover, steel, and ties of the columns and beams for the two models are tabulated, providing the necessary information about the members of the models, as shown in Table II.

Table II. Member cross-sections and reinforcement

Case	Member	B $\times$ D (mm)	Cover (mm)	Main steel	Tie dia. (mm)	Tie spacing (mm)
1	Column	400 $\times$ 900	54	12 ϕ 28	10	100
1	Beam	230 $\times$ 1000	33	3 ϕ 16 and 2 ϕ 16 (top/bottom)	8	200
2	Column C1	200 $\times$ 500	20	8 ϕ 16	6	200
2	Column C2	200 $\times$ 450	20	6 ϕ 16	6	200
2	Beam B1	200 $\times$ 500	25	2 ϕ 14 and 4 ϕ 16	6	200
2	Beam B2	200 $\times$ 500	25	2 ϕ 14 and 5 ϕ 14	6	200
2	Beam B3	200 $\times$ 400	25	3 ϕ 16 and 3 ϕ 16	6	200
2	Beam B4	200 $\times$ 500	25	2 ϕ 14 and 4 ϕ 16	6	200

3.2 SAP2000 model development

Both models have been created in SAP2000 using a three-step process: input, analysis and output. In the input phase, the basic grid parameters, materials, sections of frames and slabs, dead and live gravity loads, earthquake lateral loading and the pushover load case were defined, and nonlinear hinges were applied for beams and columns.

3.3 Nonlinear hinges and load cases

Pushover hinge properties were selected from the Frame Hinges (Pushover) option. All hinges use the generalised force-displacement relationship, shown in Fig. 3, including points A–B–C–D–E and acceptance levels IO, LS and CP [6]. The state of hinges is reported for each step in terms of the number of hinges in intervals: A–B, B–IO, IO–LS, LS–CP, CP–C, C–D, D–E and beyond E.

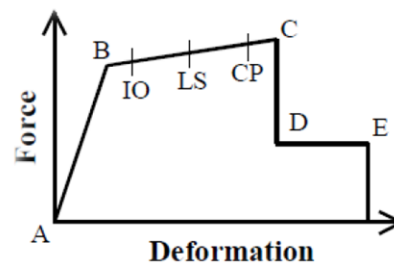


Fig. 3 Force–deformation relation for pushover hinge states

3.4 Pushover procedure

This process starts from the elastic stage that is valid until reaching the capacity moment of the first critical section. At this point, a hinge is put in place, and the updated frame is analysed under a new loading increment that leads to

the formation of hinges at new points, and the findings of every loading increment are summed up to the previous increment. This process continues until the hinges form such that the frame approaches plastic mechanism and a partial collapse mechanism where the frame behaves like a rigid body. Rotations of all members at every stage are monitored.

3.5 Extraction of results

At each stage, SAP2000 gives the roof displacement, base force and number of hinges that are used to generate the capacity curve and Tables III and V. In the process of evaluating the capacity spectrum, the base shear V and roof displacement Δ_{roof} are transformed into spectral coordinates as per ATC-40 given by equations (1) and (2).

$$S_a = \frac{V}{\alpha_1 W} \quad (1)$$

$$S_d = \frac{\Delta_{roof}}{(PF1 \cdot \phi_{1,roof})} \quad (2)$$

Where W represents building weight, α_1 is the modal mass parameter (Alpha), and $PF1 \cdot \phi_{1,roof}$ is the first mode participation factor and the modal displacement at the roof level ϕ . The capacity curve is superimposed on the demand curve; the intersection point is called the performance point, along with the values for T_{eff} and B_{eff} at each step.

4. Results and Discussion

The pushover analysis on both the regular four-story frame and the existing three-story building resulted in the capacity curve, hinge state progression, and the ATC-40 values. This part describes the comparison of both responses.

4.1 Regular RC frame (Case 1)

Regular frame pushover capacity curve displays the base shear vs. roof displacement relationship of the structure. This is almost elastic in nature, having maximum base shear of 483.87 kN at 0.222 m and then sudden drop of capacity, as presented in Fig. 4.

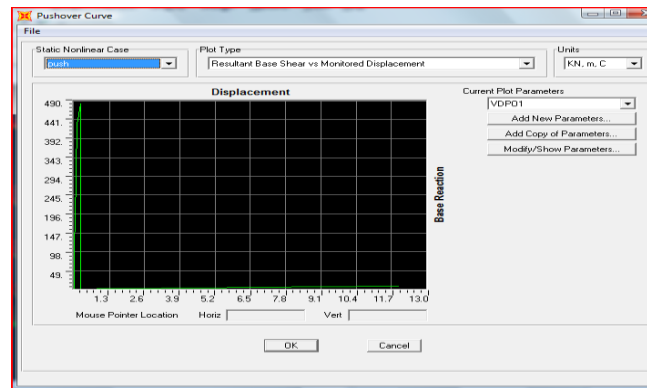


Fig. 4 Nonlinear pushover capacity curve, base shear versus displacement

The pushover output selected for the regular frame gives displacements, base forces, and number of hinges in each mode of failure, from A-B to even beyond E, at stages 0-6 and 16, as shown in Table III.

Table III. Selected pushover output for the regular RC frame (Case 1; 64 hinges)

Step	Displacement (m)	Base force (kN)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	> E
0	0.000001	0	64	0	0	0	0	0	0	0
1	0.011063	119.126	62	2	0	0	0	0	0	0
2	0.019003	175.512	60	4	0	0	0	0	0	0
3	0.115711	429.643	50	12	2	0	0	0	0	0
4	0.122143	440.766	48	14	2	0	0	0	0	0
5	0.222127	483.870	48	8	4	2	0	2	0	0
6	0.222247	0.024	48	8	4	0	0	0	0	4
16	12.000001	8.361	48	8	4	0	0	0	0	4

The order of hinge formation has been tracked through steps 0 to 16. Hinge formation starts from A-B, proceeds through B-IO and IO-LS, and some go on to form LS-CP and C-D, until the point of capacity loss, which occurs at step 6. This is illustrated in Fig. 5.

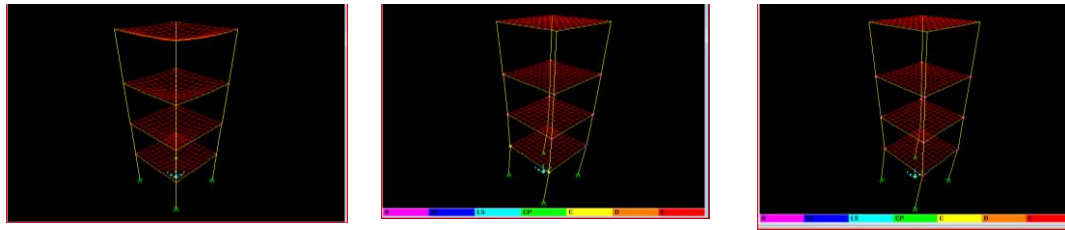


Fig. 5 Plastic-hinge formation at representative stages

The ATC-40 capacity-spectrum data for the regular frame for steps 0 to 5 are provided in Table IV. These data include the effective period, effective damping, and the capacity and demand spectral displacement and acceleration.

Table IV. Capacity-spectrum quantities for the regular RC frame (ATC-40), steps 0–5

Step	Teff (s)	Beff	Sd cap. (mm)	Sa cap. (g)	Sd dem. (mm)	Sa dem. (g)
0	0.637	0.050	0	0	63.279	0.628
1	0.637	0.050	9.553	0.095	63.279	0.628
2	0.691	0.094	16.567	0.140	57.962	0.489
3	1.074	0.153	100.148	0.349	77.100	0.269
4	1.089	0.156	105.546	0.359	77.581	0.264
5	1.448	0.256	199.139	0.382	85.537	0.164

The capacity spectrum of the regular frame is plotted versus the demand spectrum on Sa-Sd coordinates. The point at which the two meet is the performance point for comparison between the structural response and the seismic demand, as shown in Fig. 6.

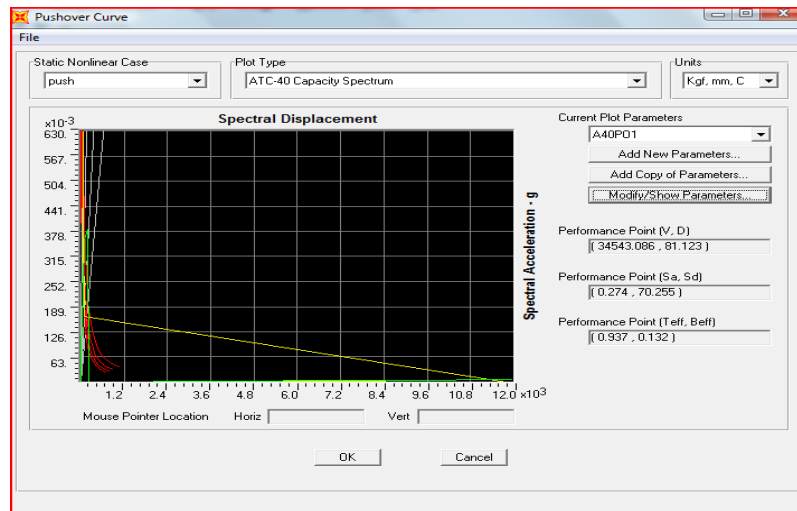


Fig. 6 Capacity spectrum, demand spectrum and performance point for the regular frame

4.2 Existing RC building (Case 2)

The stiffness and strength of the building is greater than that of the frame, as shown by a base force of 535.23 kN at 0.007 m in step 1, four-and-a-half times the value for the frame. The peak is 2256.583 kN at step 5 (0.1147 m), after which it becomes flat between 2096 kN and 2221 kN up to 0.1180 m. Hinging occurs at beam ends and lower-story columns, denoting that moment redistribution is taking place through beams. Out of the total number of hinges (718), step 5 has 497 in A-B, 123 in B-IO, 69 in IO-LS, 28 in LS-CP and 1 in C-D while step 16 has 497, 123, 67, 23, 2 and 6 in D-E. Pushover outputs for the current structure are provided below in Table V for displacements, base forces, and numbers of hinges at states ranging from A-B up to beyond E at steps 0-5, 10, and 16.

Table V. Selected pushover output for the existing RC building (Case 2; 718 hinges)

Step	Displacement (m)	Base force (kN)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	> E
0	0.000002	0	718	0	0	0	0	0	0	0
1	0.007404	535.230	717	1	0	0	0	0	0	0
2	0.017470	1165.311	663	55	0	0	0	0	0	0
3	0.031549	1592.114	601	117	0	0	0	0	0	0
4	0.074120	2066.732	519	139	60	0	0	0	0	0
5	0.114699	2256.583	497	123	69	28	0	1	0	0
10	0.116024	2205.539	497	123	69	26	0	1	2	0
16	0.118030	2096.596	497	123	67	23	0	2	6	0

4.3 Performance point of the building

The capacity spectrum analysis provides the point of performance shown in Table VI. The roof displacement of the point is 48.973 mm, far less than the 115 to 118 mm displacement at the plateau of the capacity curve, with a base shear of 1786.369 kN. The effective period is 0.615 seconds, and the effective damping ratio is 18.1 percent.

Table VI. ATC-40 performance point of the existing RC building

Quantity	Value
Roof displacement	48.973 mm
Base shear	1786.369 kN
Spectral displacement	41.460 mm
Spectral acceleration	0.413 (in g; see note)
Effective period	0.615 s
Effective damping	18.1%

The capacity spectrum of the structure intersects with the demand spectrum at the performance point with the following values for roof displacement (48.973 mm), base shear (1786.369 kN), period (0.615 sec) and damping ratio (18.1%), as shown in Fig. 7.

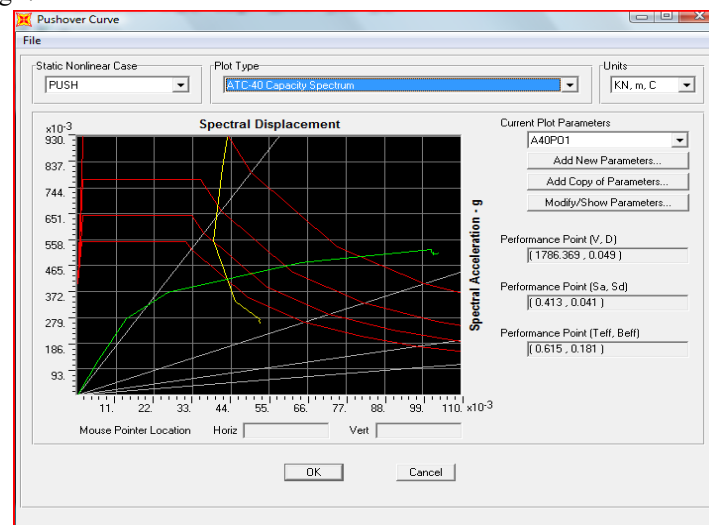


Fig. 7 Building capacity spectrum, demand spectrum and performance point

4.4 Comparison of structural responses

Comparison Summary of the Two Models Is Based on Number of Hinges, Peak Base Force and Displacement, Hinges After Point E, Behaviour After Peak, Performance Point, and Initial Effective Period, As Shown in Table VII.

Table VII. Comparative performance summary

Quantity	Case 1: regular frame	Case 2: existing building
Number of hinges	64	718
Peak tabulated base force	483.870 kN	2256.583 kN
Displacement at peak	0.222 m	0.115 m
Hinges beyond E at last step	4	0
Behaviour after peak	Abrupt loss of capacity at step 6	Plateau, gradual degradation
Performance point	Shown graphically (Fig. 6)	48.973 mm; 1786.369 kN
Effective period at start	0.637 s	0.431 s

The pushover analysis of both the RC models gave capacity curves and hinge state development, and the ATC-40 performance point. The four-story framed structure reached its peak value of 483.87 kN at 0.222 m and experienced a sudden drop in capacity. The three-story existing building reached its peak value of 2256.583 kN at around 0.115 m and continued to maintain its capacity. The performance point (48.973 mm, 1786.369 kN, $T_{eff} = 0.615$ s, $B_{eff} = 18.1\%$) was established before the plateau region.

4.5 Discussion

The base shear force for the building structure is nearly 4.7 times that of the frame and occurs when there is displacement at half the value. This is because of the reduced initial effective period of the building, which is 0.431 sec compared to 0.637 sec for the frame. While the frame fails sharply, the building yields in stages through hundreds of hinges until failure. It has a performance point base shear which is almost 79 % of the peak base force. This implies that demand is achieved even before reaching the strength level.

5. Conclusion and Future Work

Pushover analysis using SAP2000 provides capacity curve, hinge development sequence and performance points that are not achievable through elastic analysis. The existing building was able to achieve a peak value of 2256.583 kN at around 0.115m, where the hinging process took place from the beam ends and the low-story columns' base, while the performance point (48.973mm, 1786.369 kN) was found before reaching the plateau. In contrast, the regular frame was able to reach 483.87kN, after which there is a sharp reduction in capacity. Observations vary depending on the applied load pattern only.

Further study should be conducted on buildings of various heights and irregularities subjected to different intensity ground motion, using refined material nonlinearity and multiple lateral loads or adaptive loads. Comparison between pushover analysis and nonlinear dynamic analysis should also be considered.

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