Performance Assessment of RCC Structure Using Nonlinear Analysis

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Abstract. In general, the performance of the structure under earthquake vibrations is not limited to linear analysis. Linear analysis can be done to identify the preliminary parameters like natural frequency and damping ratio of the structure. To understand the structural behaviour beyond the linear elastic region, nonlinear analysis is mandatory. Nonlinear analysis reveals the structural performance under the elasto-plastic state. A large deformation in the plastic region alarms the system sustainability under random motions. The quality of ductility is considered for the ratio of yield strength of the linear system to the maximum strength of the elasto-plastic system. The ductility demand versus response reduction factor of the structure is plotted. Design modifications can be done based on the ductility demands for various response reduction factors. Two structures of uniform and non-uniform varying cross-sections are considered in the study. Modal analysis is performed based on approximate and analytical methods to find the fundamental frequencies of the structures. Both the structures are identified at a linear curve of yielding to the ultimate capacity in the pushover curve. Linear time history analysis is performed to identify the performance point of the structure in the pushover curve. Non-linear dynamic analysis is performed to find the response of the structures using the Newmark method of non-linear time history analysis. The results arrived at the linear response which is validated with the linear time history analysis using ETABS. To have the response decay in the structure, peak strength in the elastoplastic range is assumed as the normalized yield strength factor. Based on the yield strength factors, ductility demand in the structure is identified.

Key words—fundamental frequency; non-linear time history analysis; normalized yield strength factor; ductility demand ratio.

1. Introduction

The response of structure may be linear or nonlinear when subjected to external loading. In the case of linear response, the changes in geometry or material are small during the application of external loads, which is the fundamental assumption made in this analysis where there is no change in stiffness matrix during analysis and the linear relationship holds between applied load and displacement. But in the case of non-linear analysis, there is non-linear relation between applied load and displacement with the geometric non-linearity during the load application, which leads to change in the stiffness matrix. Most of the structures will undergo nonlinear deformation when they subjected to dynamic loads. Non-linear static analysis is done in force-based and displacement-based methods. In the present study displacement-based method is taken to understand the performance levels of the structures (i.e., operation level, Immediate-occupancy level, Life-safety level, Collapse-prevention level). Non-linear static analysis is preferable to identify the deflection capacity of the structure. Non-linear dynamic analysis is used to get the realistic response during the resonance. Priestley [1], proposed three techniques in terms of capacity spectrum approach, N2 method and direct displacement-based
design. Comparisons were done with the conventional force-based seismic design and earlier approaches. Bouchuan et al. [2], summarized the main properties of the theory such as background of event occurrence and development along with the performance-based design concepts. Andreas and Georgios Panagopoulos [3], stated that the performance-based design shows more realistic performance and is economical for lateral reinforcement design than the Eurocode procedure. Analytical tools (time history analysis and pushover analysis) were used for two different earthquake loadings for a 3D frame building. Rehan [7], investigated the effect of different important factors such as input ground motion on performance points, size of columns, changing the percentage of reinforcement in columns and beams individually on the performance-based design using pushover analysis. Dilip et.al [9], checked the assumption of life safety performance level in performance-based design by considering a four-storey RC building designed and modelled as per IS 456-2000 and analyzed in SAP2000 v17. Chetan and Nalamwar [10], analyzed a 5-storey building according to performance-based design for different seismic zones adopting nonlinear static analysis in ETABS. Jonathan Chambers and Trevor Kelly [4], concluded that the nonlinear dynamic procedure (NDP) is only the best method to study the performance of structures without any assumptions i.e., linear and static. Patil and Kumbhar [6], recommended nonlinear time history analysis for RCC structure necessary to ensure safety against earthquake force when is subjected to earthquake intensities like V to X on Mercalli intensity scale. Waseem Khan [8], concluded that the provision of dampers at the fifth floor and ninth floor of a nine-floor RCC building will reduce the maximum base shear, maximum displacement, maximum acceleration when subjected to seismic forces. Gholampour and Ghassemieh [5], proposed a new technique for structural dynamics and compared it with conventional methods (Newmark and Wilson-θ methods). The results obtained from the new method showed accurate results compared with conventional methods. Pravin [12], concluded that the base shear is reduced, and sloshing displacement is increased due to the isolation of the water tank compared to the non-isolated water tank. This is studied by using Newmark's step by step method. He also concluded that as the isolation time period increases base shear reduces significantly. Wael and Amjad [11], proposed a new method is the Modified Newmark method for structural dynamic problems. Results showed that the solution accuracy is more in the proposed method compared to the conventional Newmark method.

There has not been much literature noticed on the non-linear static and dynamic analysis of the structure in conjunction with ductility and response reduction factors. In the present work, both analyses are done on the structures. The non-linear dynamic time history analysis is carried out using numerical (using ETABS) and analytical (Newmark’s Method) approaches to study the effect of normalized yield strength on the response.

2. Methodology

The natural frequency is the fundamental property of the structure required to conduct seismic analysis. This seismic parameter is calculated from three methods (conventional, approximate and numerical methods). The maximum displacement is calculated from linear dynamic analysis using Response spectrum method. There by the performance levels are checked using non-linear static analysis i.e., pushover analysis. The maximum response of a structure is identified by non-linear dynamic analysis through analytical method- Newmark’s method. The results obtained from non-linear dynamic methods are validated using ETABS software tool. The effect of normalized yield strength on the response of the structure is also studied using the step-by-step procedure of Newmark by changing the stiffness at each timestep. The study is conducted on two G+9 RCC structures of uniform and non-uniform geometrical parameters.

A) Description of the structure

An RCC framed structure of total height 30m, length of 12m (3bays) and width of 8m (2bays) as shown in figure.1, each floor height is 3m, is considered. The column size for the bottom first five floors is taken as 600mmx6000mm and 400mmx400mm is taken for the remaining storeys are considered for non-uniform structure. In the case of uniform structure column size of 400mmx400mm and the beams of 300mmx400mm are considered. The slab thickness is 150mm taken for both cases. Live load of 2kN/m2 is taken on floors except for
the roof according to IS: 1893-2002(part 1). The building is considered as a special reinforced concrete moment resisting frame with a response reduction factor of 5 and this building is used as a residential building with less than 200 people with the importance factor as 1. This building is located in zone V and soil type II with 0.05% damping ratio.

Figure. 1. Elevation of the uniform and non-uniform structures

B) Construction of stiffness matrix and mass matrix:
Mass matrix is developed by adopting lumped mass concept in which the seismic weight is lumped at each storey level. The story masses are considered from the shear building idealizations and are shown in Table.1 and Table.2 for uniform and non-uniform structures. Stiffness matrix is constructed by assuming the rotations allowed at each floor level \(K=3EI/L^3\) and for storey one it is taken as fixed at the base level \(K=12EI/L^3\) in both uniform and non-uniform structures as shown in Table. 3 and Table. 4 respectively.

| Table.1 Mass matrix of uniform structure |
|------------------------------------------|
| 88800 | 0   | 0   | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 88800 | 0   | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 88800 | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 88800 | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 88800 | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 88800 | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 88800 | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 0     | 88800 | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 0     | 0     | 63600 |

| Table.2 Mass matrix of non-uniform structure |
|---------------------------------------------|
| 106800 | 0   | 0   | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 106800 | 0   | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 106800 | 0   | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 106800 | 0   | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 97800 | 0   | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 88800 | 0   | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 88800 | 0   | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 0     | 88800 | 0   |
| 0     | 0     | 0     | 0     | 0     | 0     | 0     | 0     | 63600 |
Table.3 Stiffness Matrix of Uniform Structure

|            | 3.89E+08 | -7.81E+07 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
|------------|----------|------------|---|---|---|---|---|---|---|
| 7.81E+07   | 1.56E+08 | -7.81E+07 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0          | -7.8E+07 | 1.57E+08   | -7.81E+07 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0          | 0        | -7.81E+07 | 1.56E+08 | -7.81E+07 | 0 | 0 | 0 | 0 | 0 |
| 0          | 0        | 0          | -7.81E+07 | 1.56E+08 | -7.81E+07 | 0 | 0 | 0 | 0 |
| 0          | 0        | 0          | 0          | -7.81E+07 | 1.56E+08 | -7.81E+07 | 0 | 0 | 0 |
| 0          | 0        | 0          | 0          | 0          | -7.81E+07 | 1.56E+08 | -7.81E+07 | 0 | 0 |
| 0          | 0        | 0          | 0          | 0          | 0          | -7.81E+07 | 1.56E+08 | -7.81E+07 | 0 |
| 0          | 0        | 0          | 0          | 0          | 0          | 0          | -7.81E+07 | 8.22E+07 | 0 |

Table.4 Stiffness matrix of non-uniform structure

|            | 1.98E+09 | -3.94E+08 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
|------------|----------|------------|---|---|---|---|---|---|---|
| 3.94E+08   | 7.88E+08 | -3.94E+08 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0          | -3.94E+08 | 7.89E+08   | -3.94E+08 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0          | 0        | -3.94E+08 | 7.88E+08 | -3.94E+08 | 0 | 0 | 0 | 0 | 0 |
| 0          | 0        | 0          | -3.94E+08 | 4.72E+08 | -7.78E+07 | 0 | 0 | 0 | 0 |
| 0          | 0        | 0          | 0          | -7.78E+07 | 1.6E+08 | -7.78E+07 | 0 | 0 | 0 |
| 0          | 0        | 0          | 0          | 0          | -7.78E+07 | 1.56E+08 | -8.01E+07 | 0 | 0 |
| 0          | 0        | 0          | 0          | 0          | 0          | -7.8E+07 | 1.6E+08 | -7.81E+07 | 0 |
| 0          | 0        | 0          | 0          | 0          | 0          | 0          | -8.01E+07 | 7.81E+07 | 0 |

C) Natural frequency calculation:
Natural frequencies of the structures are obtained from eigenvalue method, Stodola method and numerical analysis. The fundamental natural frequencies of 4.84 rad/sec and 7.7 rad/sec for uniform and non-uniform structures are derived using eigen value problem. Fundamental frequencies calculated from Stodola method for the two types of structures are 4.9 rad/sec and 7.70 rad/sec. Natural frequencies obtained from ETABS are 4.09 rad/sec and 4.76rad/sec for uniform and non-uniform structures.

D) Linear Dynamic Analysis
Peak response of a building subjected to zone V earthquake data is obtained using ETABS. Demand curve is identified by defining response spectrum function, soil type & damping ratio as input. The maximum storey displacement in both the structures is obtained as shown in figure 2.

Figure.2 Maximum Storey displacement in uniform and non-uniform structure
E) Nonlinear Static Analysis
ETABS is used to obtain the performance levels and performance point (shown in figure 3) of each structure. In this present work, the displacement control method is adopted by giving displacement as 300mm at the top 10 storey in both structures which is obtained from linear analysis.

![Figure 3 Performance point in uniform and non-uniform structure](image)

F) Nonlinear Dynamic Analysis
The Non-linear time history analysis is done to study the structural response subjected to past earthquake data. In this present work, Imperial valley earthquake data (shown in Figure 4) is considered as input ground motion. In this analysis, time is divided into small intervals known as time steps and the maximum acceleration at a particular time step is studied. This analysis is done using step by step analytical method i.e., Newmark’s Beta method. Newmark’s linear acceleration method and average acceleration method are considered to determine the dynamic behaviour of the structures.

i. Numerical Analysis using ETABS:
In the case of uniform structure, the time step for output response is taken as 0.01sec whereas in non-uniform structure 0.0078sec is considered (shown in figure 5).

![Figure 4 Imperial Valley time history data](image)

![Figure 5 Acceleration response in X direction of uniform section and non-uniform section](image)
1. Newmark’s Method:

A mathematical model is developed at a incremental timesteps to find the response of the structure when subjected to dynamic loadings. Average acceleration and linear acceleration methods are employed to perform the analysis. As there are huge iterations are involved, MATLAB software is used to find peak accelerations in the structures of uniform and non-uniform where the damping matrix (Equation 1) is considered as the sum of the kinetic energy and potential energy. The response displacements are shown in figure 6 and figure 7.

\[ C = \frac{1}{2} M \ddot{u}^2 + \frac{1}{2} K u^2 \]  

(1)

Figure. 6 Acceleration response in Average acceleration method in uniform and non-uniform structure

Figure. 7 Acceleration response in linear acceleration method in uniform and non-uniform structure

The values of \( \delta t \) in both structures are considered as 0.01sec & 0.0078sec respectively these values are calculated by taking \( \frac{1}{10} \) th of least time period.

Effect of Normalized Yield strength on response:

The normalized yield strength (\( \bar{f_y} \)) can be expressed as the ratio of resisting yield force to the peak value of the resisting force in linear response.

\[ \bar{f}_y = \frac{f_y}{f_0} = \frac{u_y}{u_0} \]  

(2)

\( f_y, f_0 \) are resisting yield force and peak resisting force in linear response of the structure. \( u_y, u_0 \) are yield displacement and peak displacement in linear response of the structure as shown in figure 8.

Figure. 8 Linear displacement response of uniform &Non-uniform Structure
Here indicates the minimum strength required for the structure to be in a linearly elastic state during an earthquake. The normalized yield strength less than the unity indicates the system is in inelastic range (i.e., elastoplastic system) during an earthquake. The ductility factor (equation 3) has defined the ratio of the maximum acceleration in the elastoplastic system and the yield displacement denoted by $\mu$. This factor gives the ductility demand. The ductility capacity should be more to deform beyond the elastic range.

$$\mu = \frac{u_m}{u_y}$$  \hspace{1cm} (3)

Where $\mu$ is ductility ratio, $u_m$ is peak or absolute maximum displacement of the elastoplastic system, and $u_y$ is yield displacement of the linear response.

For the different values of normalized yield strengths 0.125, 0.25 and 0.5, the peak displacements are found adopting the Newmark method (average acceleration method) by multiplying the stiffness value with normalized yield strength using the MATLAB algorithm. The displacement response of the elastoplastic system is shown below figures 9 to 11.

Figure 9 Displacement response of elastoplastic system ($f_y=0.125$) in uniform & Non-uniform Structure

Figure 10 Displacement response of elastoplastic system ($f_y=0.25$) in uniform & Non-uniform Structure

Figure 11 Displacement response of elastoplastic system ($f_y=0.5$) in uniform & Non-uniform Structure

The ductility factor is calculated by developing a relationship among $u_m$, $u_0$ and $f_y$ using equations 2 and 3 as follows

$$\mu = \frac{1}{f_y} \times \frac{u_m}{u_0}$$
3. Results and discussions

The natural frequency obtained from the various methods is shown in the below Table. 5. The natural frequency in the case of the uniform section is almost the same in all methods of analysis but in the case of non-uniform structure, the frequency from a numerical method is different from other methods, this variation in natural frequency is due to the assumptions involved in the construction of stiffness matrix. This can be modified by using the translation method to find out the actual stiffness matrix.

1) The maximum storey displacement of uniform structure is 0.053mm and in non-uniform structure is 0.055mm.

2) In the analysis of non-linear static, the uniform structure is in life safety level i.e., there is no harm to the people who occupied that structure but there are some damages in structural and non-structural components and overall damage is moderate after the earthquake. The non-uniform structure is in immediate occupancy level i.e., one can use that structure immediately after the earthquake. There is huge damage to the structural members as well as non-structural members and overall damage is less.

3) The peak acceleration values from Non-linear dynamic analysis obtained from different approaches are as shown in the Table. 6.

| Method of analysis | Uniform | Non-uniform |
|--------------------|---------|-------------|
| FEM Method (ETABS) | 4.09rad/sec | 4.76rad/sec |
| Eigen value Method | 4.909rad/sec | 7.7rad/sec |
| Stodola Method     | 4.9rad/sec | 7.704rad/sec |

The values of peak acceleration from numerical modelling and analytical modelling are in line for both structures. So, the code developed for both average acceleration and linear acceleration methods can be used to find the peak response of the structure.

4) The ductility factor values in elastoplastic state for different values of normalized yield strength are as shown below Table. 7 in the case of uniform and non-uniform structure.

| Normalized yield strength | Type of structure |
|---------------------------|-------------------|
| Uniform                   | Non-uniform       |
| 0.125                     | 11.46046512       | 20.93617021    |
| 0.25                      | 7.497674419       | 6.921985816    |
| 0.5                       | 3.218604651       | 1.985815603    |

4. Conclusions

1) An attempt has been made to identify the non-linear dynamic behaviour of the structure using analytical procedures. Two structures with uniform cross-section and non-uniform cross-section are considered in the study. Mass matrix and stiffness matrix for both the cases were developed using shear building idealization concept.
2) Natural frequency obtained from classical and approximation methods for the uniform structure are inline (84% accurate) with the FE model developed in the ETABS software tool.

3) The discrepancy in natural frequency (analytical methods to the numerical method) of the non-uniform model is due to the assumed stiffness matrix. For a non-uniform cross-sectioned structure, a stiffness matrix can be developed using the stiffness factors based on the varying cross-sections. This can be considered as the future scope of the work.

4) The level of damage intensity is less in the case of the non-uniform structure compared to the uniform structure. (Pushover curve shows immediate occupancy level for non-uniform structure and life safety level for uniform structure).

5) On the contrary, the non-linear response of a uniform structure is less compared to the non-uniform structure. The high damage point and less response in the case of the structure with uniform stiffness are due to the resonance condition. Energy dissipation curves may be helpful to understand the dynamic behaviour further.

6) The non-linear responses using analytical solutions match the FE analysis using ETABS. Hence the proposed methodology of analysis has the capability of determining the response accelerations using the module developed in MATLAB.

7) For the assumed normalized yield strength ratios of 0.125, 0.25 and 0.5 the ductility ratios are decrementing accordingly. Hence peak strength in the elastoplastic region can be considered as design yield strength multiplied by the response reduction factor. Therefore design modifications can be done. For non-uniform structures, as the normalized yield strength increases, the response displacement of the structure reduces. This is due to the improving ductility in the structure. Hence for this type of structure, a response reduction factor of 8 can be considered to have the design modification. In the case of a uniform structure, a response reduction factor of 4 can be considered while designing the structure.

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