SEISMIC PERFORMANCE EVALUATION OF ESSENTIAL FACILITIES BUILDING IN DHAKA CITY BY PUSHOVER ANALYSIS

A Thesis Submitted in Partial Fulfilment of the Requirements
for the Bachelor of Science Degree in

Civil and Environmental Engineering

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The Thesis entitled “Seismic Performance Evaluation of Essential Facilities Building in Dhaka City by Pushover Analysis”, by Md. Noyim Uddin, Student ID: 085403 and Md. Tanvir Ehsan Amin, Student ID: 085422 have been approved in partial fulfilment of the requirements for the Bachelor of Science Degree in Civil and Environmental Engineering.

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ABSTRACT

Bangladesh is situated in moderate earthquake prone region. Major metropolitan cities of our country are under serious threat because of faulty design and construction of structures. Buildings designed without seismic consideration could be vulnerable to damage even under low levels of ground shaking from distant earthquakes. So the structural engineers now-a-days are more concerned about the different earthquake analysis procedures. According to Bangladesh National Building Code (BNBC 1993), the buildings are designed according to equivalent static force method, response spectrum method and time history analysis. But the actual performance of a structure can be hardly found by these methods. Nonlinear inelastic pushover analysis provides a better understanding about the actual behaviour of the structures during earthquake and hence, the application of pushover analysis to evaluate the seismic performance of Secretariat Clinic Building located at Secretarial of Government of Bangladesh, Dhaka is the focus of this thesis. The analysis has been performed in two orthogonal direction of the building based on ATC-40 procedure and it was found that building base shear capacity in shorter direction is about 30% higher than the capacity in longer direction. The inter-storey drift in two orthogonal directions is below 1% (0.38% and 0.74%) at performance point, which corresponds to Immediate Occupancy performance level as per ATC-40 guideline of global acceptability of buildings.

Keywords: Seismic performance evaluation, Pushover analysis, Inter-storey drift, ATC-40.

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1 INTRODUCTION

1.1 Background

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post-elastic behaviour. However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis.

Although, in literature, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and the reliability of pushover analysis in predicting global and local seismic demands for all structures have been a subject of discussion and improved pushover procedures have been proposed to overcome the certain limitations of traditional pushover procedures. However, the improved procedures are mostly computationally demanding and conceptually complex that uses of such procedures are impractical in engineering profession and codes.

As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues such as modelling nonlinear member behaviour, computational scheme of the procedure, variations in the predictions of various lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed.

1.2 Method of Analysis

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structure.

1.2.1 Elastic Methods of Analysis

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes.

In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding.

In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis.

Any effects of higher modes are automatically included in time history analysis. In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies.

Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behaviour of structures could not be identified by an elastic analysis. However, post-elastic behaviour should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behaviour indirectly by reducing elastic forces to inelastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well [26].

Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair.

The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly [16].

1.2.2 Inelastic Methods of Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behaviour of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modelling and ground motion characteristics. It requires proper modelling of cyclic load-deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method [3], Displacement Coefficient Method [12] and the Secant Method [9].

The theoretical background, reliability and the accuracy of inelastic static analysis procedure is discussed in detail in the following sections.

1.2.3 Summary

The uncertainties involved in accurate determination of material properties, element and structure capacities, the limited prediction of ground motions that the structure is going to experience and the limitations in accurate modelling of structural behaviour make the seismic performance evaluation of structures a complex and difficult process.

Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

1.3 Description of Pushover Analysis

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached.

Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve (Figure 1.1).

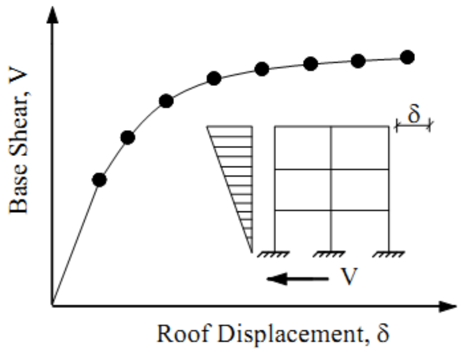


Figure 1.1. Global Capacity (Pushover) Curve of a Structure

Pushover analysis can be performed as force-controlled or displacement-controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e., force-controlled procedure should be used when the load is known (such as gravity loading). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Generally, pushover analysis is performed as displacement-controlled proposed by Allahabadi [1] to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the centre of mass of structure is chosen as the control displacement.

The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

1.3.1 Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provides information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis.

These are [21]:

- Estimates of inter story drifts and its distribution along the height
- Determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections
- Determination of deformation demands for ductile members
- Identification of location of weak points in the structure (or potential failure modes)
- Consequences of strength deterioration of individual members on the behaviour of structural system
- Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range
- Verification of the completeness and adequacy of load path

Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

1.3.2 Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

Target displacement is the global displacement expected in a design earthquake. The roof displacement at mass centre of the structure is used as target displacement. The accurate estimation of target displacement associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis.

In pushover analysis, the target displacement for a multi degree of freedom (MDOF) system is usually estimated as the displacement demand for the corresponding equivalent single degree of freedom (SDOF) system. The basic properties of an equivalent SDOF system are obtained by using a shape vector which represents the deflected shape of the MDOF system. Most of the researchers recommend the use of normalized displacement profile at the target displacement level as a shape vector but iteration is needed since this displacement is not known a priori. Thus, a fixed shape vector, elastic first mode, is used for simplicity without regards to higher modes by most of the approaches.

Moreover, hysteretic characteristics of MDOF should be incorporated into the equivalent SDOF model, if displacement demand is affected from stiffness degradation or pinching, strength deterioration, P-Δ effects. Foundation uplift, torsional effects and semi-rigid diaphragms are also expected to affect the target displacement [21].

Lateral loads represent the likely distribution of inertia forces imposed on structure during an earthquake. The distribution of inertia forces vary with the severity of earthquake and with time during earthquake since.

$$F_{k,i} = \frac{W_k}{g}\ddot{u}_i$$

$F_{k,i}$: Inertia force at kth story at time i

W_k : Weight of kth story

$\ddot{u}_i$: Instantaneous story acceleration

However, in pushover analysis, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. As the response of structure, thus the capacity curve is very sensitive to the choice of lateral load distribution [23], selection of lateral load pattern is more critical than the accurate estimation of target displacement.

The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions and a single concentrated horizontal force at the top of structure.

Multi-modal load pattern derived from Square Root of Sum of Squares (SRSS) story shears is also used to consider at least elastic higher mode effects for long period structures. These loading patterns usually favour certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure [21].

Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if the structural response is not severely affected by higher modes and the structure has only a single load yielding mechanism that can be captured by an invariant load pattern.

FEMA-273 [16] recommends utilizing at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behaviour and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is allowed if more than 75% of the total mass participates in the fundamental load.

The invariant load patterns cannot account for the redistribution of inertia forces due to progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Although some improved predictions have been obtained from adaptive load patterns [26], they make pushover analysis computationally demanding and conceptually complicated. The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy.

Whether lateral loading is invariant or adaptive, it is applied to the structure statically that a static loading cannot represent inelastic dynamic response with a large degree of accuracy.

The above discussion on target displacement and lateral load pattern reveals that pushover analysis assumes that response of structure can be related to that of an equivalent SDOF system. In other words, the response is controlled by fundamental mode which remains constant throughout the response history without considering progressive yielding. Although this assumption is incorrect, some researchers obtained satisfactory local and global pushover predictions on low to mid-rise structures in which response is dominated by fundamental mode and inelasticity is distributed throughout the height of the structure [21].

1.3.3 Summary

Pushover analysis yields insight into elastic and inelastic response of structures under earthquakes provided that adequate modelling of structure, careful selection of lateral load pattern and careful interpretation of results are performed. However, pushover analysis is more appropriate for low to mid-rise buildings with dominant fundamental mode response. For special and high-rise buildings, pushover analysis should be complemented with other evaluation procedures since higher modes could certainly affect the response.

1.4 Objective

The Objective of this thesis work is to assess the seismic performance of the five storey Secretariat Clinic Building by pushover analysis.

1.5 Outline of the Thesis

This thesis is composed of six main chapters. Chapter 1 includes a discussion of analysis methods used for seismic performance evaluation and brief information about pushover analysis and its limitations. Chapter 2 reviews the previous research on simplified nonlinear analysis procedures and on pushover analysis. Chapter 3 describes pushover analysis procedure with element description of SAP2000 [11]. In chapter 4 the basic modelling and analysis parameters used in the study is described. Evaluation of seismic performance and the findings of the research work are presented in chapter 5. And the summary, conclusions and future recommendations is described in chapter 6.

2 REVIEW OF PREVIOUS RESEARCH

2.1 General

Structures are expected to deform in elastically when subjected to severe earthquakes, so seismic performance evaluation of structures should be conducted considering post-elastic behaviour. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behaviour cannot be determined directly by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behaviour. Various simplified nonlinear analysis procedures and approximate methods to estimate maximum inelastic displacement demand of structures are proposed in literature. The widely used simplified nonlinear analysis procedure, pushover analysis has also been an attractive subject of study.

2.2 Past Studies on pushover Analysis

Most of the simplified nonlinear analysis procedures utilized for seismic performance evaluation make use of pushover analysis and/or equivalent SDOF representation of actual structure. However, pushover analysis involves certain approximations that the reliability and the accuracy of the procedure should be identified. For this purpose, researchers investigated various aspects of pushover analysis to identify the limitations and weaknesses of the procedure and proposed improved pushover procedures that consider the effects of lateral load patterns, higher modes, failure mechanisms, etc.

The use of inelastic static analysis in earthquake engineering is traced to the work of Gulkan and Sozen (1974) or earlier [17], where a single degree of freedom system is derived to represent the multi-degree of freedom structure via an equivalent or ‘substitute’ structure. The load-displacement curve of this substitute to the real structure is evaluated by either finite element analysis or hand calculation to obtain the initial and post-yield stiffness, the yield strength and the ultimate strength. Simplified inelastic analysis procedures for multi-degree of freedom systems have also been proposed by Saiidi and Sozen (1981) [30] and Fajfar and Fischinger (1988) [15]. Therefore, pushover analysis is not a recent development.

There are several publications that review the advantages and disadvantages of pushover analysis, with varying degrees of success. They all, however, utilize global response parameters, namely top displacement versus base shear. Lawson et al. (1994) [23] discuss in some detail range of applicability, the expected realism for various structural systems and highlight the difficulties encountered. Krawinkler (1995) [20] discusses pushover analysis as a prelude to capacity spectrum applications. The author mentions a contentious point, which is that 'in most cases the normalized displacement profile at a first estimate of the target displacement level is utilized for these (defining the shape vector) purposes'. Another important issue raised in the latter review is that 'it must be emphasized that the pushover analysis cannot disclose performance problems caused by changes in the inelastic dynamic characteristics due to higher mode effects'. This is true if the current (as in 1995) techniques are assessed, but is not an insurmountable problem, as discussed in the current report and the earlier of Bracci et al. (1995) [7] and the authors of this report. Several cases of success of the pushover method were reported by Faella (1996) [14], who also pointed towards difficulties with static-dynamic comparisons when the strong-motion input is rich in long period amplifications.

Attempts at improving the procedure have been made, with varying degrees of rigor and success. The simplest and most pragmatic of which is the work of Sasaki et al. (1998) [31]. This comprises running several pushover analyses under forcing vectors representing the various modes deemed to be excited in the dynamic response. If the individual pushover curves, converted to spectral displacement-spectral acceleration space using the dynamic characteristics of the individual modes are plotted alongside the composite spectra, it becomes apparent which mode would be the cause of more damage and where is the damage likely to occur. The procedure is intuitive, and does indeed identify potential problems that conventional single mode pushover analysis fails to point out. It, however, falls short of the work of Bracci et al. (1997) [6], which is the most recent in-depth study of pushover analysis.

The pros and cons of the procedure were also discussed by Krawinkler and Seneviratna (1998) [21]. Amongst many other interesting comments, the authors stress that the most important shortcoming of the procedure is the definition and invariance of the applied load vector. The significance of defining the target displacement (required for evaluating structural adequacy under a specific earthquake) is considered secondary to the load vector definition and control. Whereas the authors report on successful pushover cases, they emphasize problem areas by discussing the response of a tall (20 story) structure, and a structure with a full-height wall. In the former case, the errors are due to the omission of higher mode effects, whilst the latter demonstrates the difficulties encountered with effect of concentrated local demand (base of the wall) on force distribution. A short review by Tso and Moghadam (1998) [33] concluded that fixed load patterns in pushover analysis are limiting, but newly proposed variable load patterns are not sufficiently verified as a superior option.

Kim and D'Amore (1999) [19] set out to assess pushover analysis in comparison with inelastic time-history procedures. They concluded that not all analyses of the same structure under a set of distinct earthquake records are predicted by pushover analysis, a rather obvious conclusion that did not require inelastic dynamic analysis to prove. The interaction between the continuously-changing dynamic characteristics of the inelastic multi-degree of freedom structure with the various frequencies of a set of natural records cannot possibly be duplicated by a single pushover analysis under a predefined and fixed transverse load or displacement vector. But, again, this problem may have a solution.

An adaptive procedure is described in the paper by Bracci et al. (1997) [6] and attributed to a previous publication by Reinhorn and Vladescu. This comprises starting the analysis assuming a certain force distribution, usually triangular. Loads imposed in subsequent increments are calculated from the instantaneous story resistance and the base shear in the previous step.

This procedure is applied in the context of defining the moment curvature relationship of the various members as an input parameter, and is intended, to capture the effect of local mechanisms. It does not account for higher mode contribution. The procedure was implemented in the dynamic analysis package IDARC [22] and demonstrated in the paper to give accurate results for the structure considered. However, numerical tests conducted by Lefort (2000) [24] showed that the above procedure grossly under-estimate the strength, compared to inelastic dynamic analysis using IDARC [22], by up to 60% for a regular 10 story structure. Peculiarly, conventional pushover with triangular or uniform load distribution gave results far superior to the above adaptive method.

Work undertaken by the V.K. Papanikolaou, A.S. Elnashai, J.F. Pareja and their co-workers has developed a robust procedure for adaptive pushover analysis that is shown to be superior to, or at worst as good as, conventional pushover. Formulations given by Papanikolaou (2000) [28], subsequent developments by Antoniou (2003) [3] and an overview by Elnashai (2002) [13] detail this fiber-based, self-adjusting adaptive approach. This is described in the subsequent sections and previous results obtained from idealized structures are highlighted.

A similar study by Antoniou and Pinho (2004) [3], also question the applicability of conventional and adaptive pushover methods in predicting the horizontal capacity of reinforced concrete structures, compared to inelastic dynamic analysis. These three analysis approaches were applied on a set of different concrete structures with varying structural properties under various strong motion records. It was mainly concluded that the estimation of structural deformation patterns were poorly predicted by both types of pushover analysis. Several other aspects of this study are met and discussed in the present report in subsequent chapters .

The term ‘pushover analysis’ describes a modern variation of the classical ‘collapse analysis’ method, as fittingly described by Kunnath. It refers to an analysis procedure whereby an incremental-iterative solution of the static equilibrium equations has been carried out to obtain the response of a structure subjected to monotonically increasing lateral load patterns. Whilst the application of pushover methods in the assessment of building frames has been extensively verified in the recent past, nonlinear static analysis of bridge structures has been the subject of only limited scrutiny.

Recent years have also witnessed the development and introduction of an alternative type of nonlinear static analysis, which involve running multiple pushover analyses separately, each of which corresponding to a given modal distribution, and then estimating the structural response by combining the action effects derived from each of the modal responses.

3 PUSHOVER ANALYSIS WITH SAP2000

3.1 General

Nonlinear static analysis, or pushover analysis, could be performed directly by a computer program which can model nonlinear behaviour of lateral load resisting members of a structure. However, the computational scheme and the assumptions involved in modelling nonlinear member behaviour could be different that there may be variations in the pushover results obtained from different software. Therefore, the underlying principles of any software utilized for pushover analysis should be well understood to interpret the results of pushover analysis.

In this study, pushover analyses were performed on steel and reinforced concrete moment resisting frames SAP2000 [11] using various lateral load patterns to identify the basic principles of each software utilized in the implementation of pushover analysis. The approach of each software to model nonlinear force-displacement relationships was investigated. The pushover analysis results obtained from each software were compared to evaluate the ability of this software to perform pushover analysis on frame structures.

3.2 Pushover Analysis Procedure

Pushover analysis can be performed as either force-controlled or displacement-controlled depending on the physical nature of the load and the behaviour expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement-controlled procedure should be used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known in advance, or when the structure can be expected to lose strength or become unstable.

Some computer programs (Nonlinear version of SAP2000) [11] can model nonlinear behaviour and perform pushover analysis directly to obtain capacity curve for two and/or three dimensional models of the structure. When such programs are not available or the available computer programs could not sequential elastic analyses are performed and superimposed to determine a force-displacement curve of the overall structure. A displacement-controlled pushover analysis is basically composed of the following steps:

1. A two or three dimensional model that represents the overall structural behaviour is created.

2. Bilinear or Tri-linear load-deformation diagrams of all important members that affect lateral response are defined.

3. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.

4. A predefined lateral load pattern which is distributed along the building height is then applied.

5. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.

6. Base shear and roof displacement are recorded at first yielding.

7. The structural model is modified to account for the reduced stiffness of yielded member(s).

8. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.

9. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.

10. Steps 7, 8 and 9 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.

11. The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure (Figure 3.1).

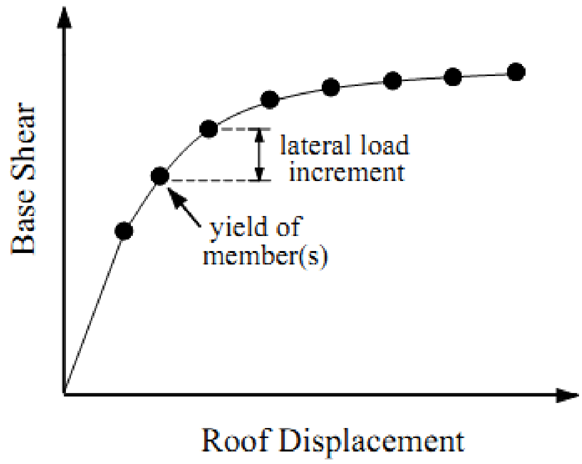


Figure 3.1. Global Capacity (Pushover) Curve of Structure

3.3 Pushover Analysis with SAP2000

Nonlinear static pushover analysis is a very powerful feature offered in the nonlinear version of SAP2000. Pushover analysis can be performed on both two and three dimensional structural models.

SAP2000 can also perform pushover analysis as either force-controlled or displacement-controlled. The "Push to Load Level Defined by Pattern" option button is used to perform a force-controlled analysis (Figure 3.2). The pushover typically proceeds to the full load value defined by the sum of all loads included in the "Load Pattern" box (unless it fails to converge at a lower force value). "The Push to Displacement Magnitude" option button is used to perform a displacement-controlled analysis. The pushover typically proceeds to the specified displacement in the specified control direction at the specified control joint (unless it fails to converge at a lower displacement value) [11].

An event-to-event solution strategy is utilized by SAP2000 pushover analysis and the parameters in the right-hand side of the "Options" area (Figure 3.2) control the pushover analysis. The "Minimum Saved Steps" and "Maximum Total Steps" provide control over the number of points actually saved in the pushover analysis. Only steps resulting in significant changes in the shape of the pushover curve are saved for output. "The Maximum Null Steps" is a cumulative counter through the entire analysis to account for the non-convergence in a step due to numerical sensitivity in the solution or a catastrophic failure in the structure. "Iteration Tolerance" and "Maximum Iteration/Step" are control parameters to check static equilibrium at the end of each step in a pushover analysis. If the ratio of the unbalanced-load to the applied-load exceeds the "Iteration Tolerance", the unbalanced load is applied to the structure in a second iteration for that step. These iterations continue until the unbalanced load satisfies the "Iteration Tolerance" or the "Maximum Iterations/Step" is reached [11]. A constant "Event Tolerance" for all elements is used to determine when an event actually occurs for a hinge.

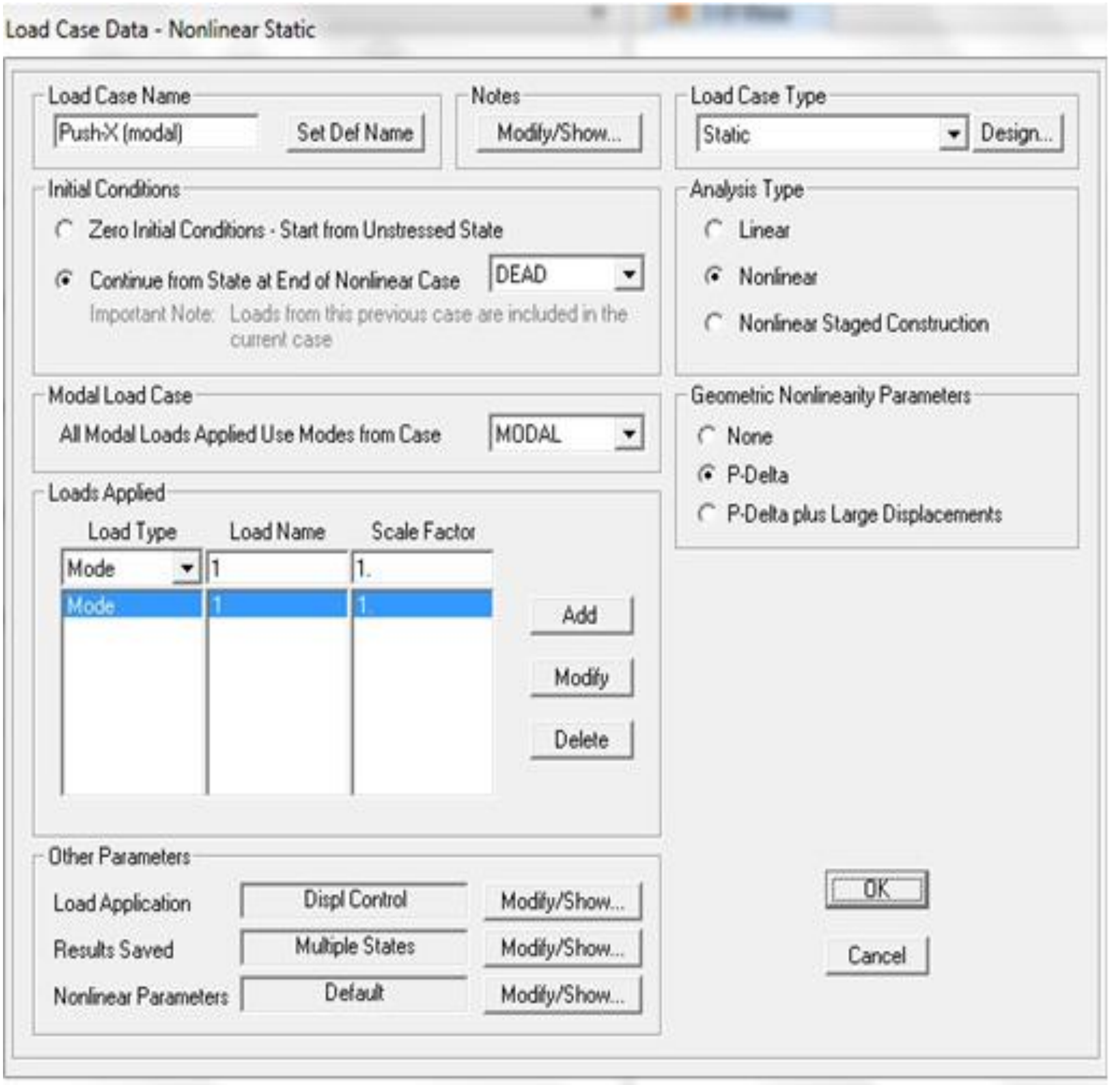


Figure 3.2. Load Case Data Dialog Box (SAP2000)

Geometric nonlinearity can be considered through P-delta effects or P-delta effects plus large displacements (Figure 3.2).

Modal and uniform lateral load patterns can be directly defined by SAP2000 in addition to any user-defined static lateral load case. Modal load pattern is defined for any Eigen or Ritz mode while uniform load pattern is defined by uniform acceleration acting in any of the three global directions (acc dir X, acc dir Y and acc dir Z).

Nonlinear behaviour of a frame element is represented by specified hinges in SAP2000 and a capacity drop occurs for a hinge when the hinge reaches a negative-sloped portion of its force-displacement curve during pushover analysis (Figure 3.3).

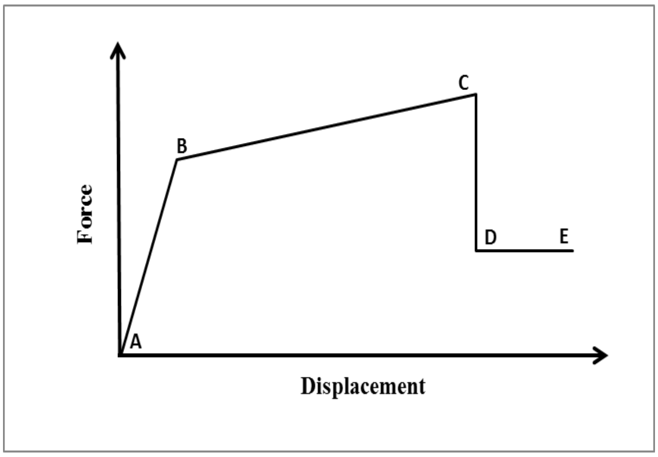


Figure 3.3. Generalized Force-Displacement Characteristic of a Non-Degrading Frame Element of SAP2000

Such unloading along a negative slope is unstable in a static analysis and SAP2000 provides three different member unloading methods to remove the load that the hinge was carrying and redistribute it to the rest of the structure (Figure 3.3). In the "Unload Entire Structure" option, when the hinge reaches point C on its force-displacement curve (Figure 3.3) the program continues to try to increase the base shear. If this results in increased lateral deformation the analysis proceeds. If not, base shear is reduced by reversing the lateral load on the whole structure until the force in that hinge is consistent with the value at point D on its force-displacement curve (Figure 3.3). All elements unload and lateral displacement is reduced since the base shear is reduced. After the hinge is fully unloaded, base shear is again increased, lateral displacement begins to increase and other elements of the structure pick up the load that was removed from the unloaded hinge. If hinge unloading requires large reductions in the applied lateral load and two hinges compete to unload, i.e., where one hinge requires the applied load to increase while the other requires the load to decrease, the method fails.

In the "Apply Local Redistribution" option, only the element containing the hinge is unloaded instead of unloading the entire structure. If the program precedes reducing the base shear when a hinge reaches point C, the hinge unloading is performed by applying a temporary, localized, self-equilibrating, internal load that unloads the element [11]. Once the hinge is unloaded, the temporary load is reversed, transferring the removed load to neighbouring elements. This method will fail if two hinges in the same element compete to unload, i.e., where one hinge requires the temporary load to increase while the other requires the load to decrease.

In the "Restart Using Secant Stiffness" option, whenever any hinge reaches point C on force-displacement curve, all hinges that have become nonlinear are reformed using secant stiffness properties, and the analysis is restarted. This method may fail when the stress in a hinge under gravity load is large enough that the secant stiffness is negative. On the other hand, this method may also give solutions where the other two methods fail due to hinges with small (nearly horizontal) negative slopes [11].

If "Save Positive Increments Only" option box (Figure 3.3) is not checked in a pushover analysis, steps in which hinge unloading occur are also saved to represent the characteristics of member unloading method on pushover curve. However, pushover curve will become an envelope curve of all saved points if "Save Positive Increments Only" option box is checked.

3.4 Element Description of SAP2000

In SAP2000, a frame element is modelled as a line element having linearly elastic properties and nonlinear force-displacement characteristics of individual frame elements are modelled as hinges represented by a series of straight line segments. A generalized force-displacement characteristic of a non-degrading frame element (or hinge properties) in SAP2000 is shown in Figure 3.3.

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element.

There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements. When these hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for each and every hinge.

Default hinge properties could not be modified and they are section dependent. When default hinge properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties. The built-in default hinge properties for steel and concrete members are based on ATC-40 [4] and FEMA-273 criteria.

User-defined hinge properties can be based on default properties or they can be fully user-defined. When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified.

4 Seismic Demand and the Basic Modelling Parameters

4.1 Introduction

In order to find a building's performance against some specified seismic demand, response quantities from a static non-linear analysis is compared with the global deformation for appropriate performance limits. The response limits falls into two categories:

- Global acceptable limits: These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift.

- Element and component acceptability limits: Each element (frame, wall, diaphragm, or foundation) must be checked to determine if its components respond within acceptable limits.

Building performance objectives are checked against some predefined seismic demand. Seismic demand for a structure is totally site dependent. For analysis development of site dependent elastic response spectrum is needed. But unfortunately Bangladesh National Building Code [BNBC, 1993] does not have any guideline to develop such site dependent response spectra. The Federal Emergency Management Agency [FEMA-356, 2002] has recommended standard procedure to establish seismic demand at a site. This procedure is discussed next.

4.2 Seismic Design

Earthquake is an uncertain phenomenon. It is not possible to predict the time and what intensity of earthquake that may hit in some specific regions. For example, large devastating earthquake that hit near our country was the Great Indian Earthquake in 12 June, 1897. It is estimated that such an earthquake in that fault may occur in 3000-4000 yrs. [Bilharn, Rand P. England, Plateau pop-up during the great 1897 Assam earthquake- 2001]. It is possible to design a structure that will withstand such a major devastating earthquake but this huge investment is not always feasible economically for such an uncertain event. Thus earthquake design philosophy accepts that:

- Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage

- Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake

- Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse

Thus for the purpose of the analysis, severity of earthquakes may be classified as follows [4].

4.2.1 The Serviceability Earthquake (SE)

The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50 per cent chance of being exceeded in 50-year period; this level of earthquake ground shaking is typically about 0.5 times of the level of ground shaking of the Design Earthquake. The SE has a mean return period of approximately 75 years. Damage in 1he non-structural elements is expected during Serviceability Earthquake.

4.2.2 The Design Earthquake (DE)

The design Earthquake (DE) is defined probabilistically as the level ground shaking that has a 10 percent chance of being exceeded in a 50-year period. The DE represents an infrequent level of ground shaking that can occur during the life of the building. The DE has a mean return period of approximately 500 years. Minor repairable damage in the primary lateral load carrying system is expected during Design Earthquake. For secondary elements, the damage may be such that they require replacement.

4.2.3 The Maximum Earthquake (ME)

The Maximum Earthquake (ME) is defined deterministically as the maximum level of earthquake ground shaking which may ever be accepted at the building site within the known geologic frame work. In probabilistic terms, the ME has a return period of about 1,000 years. During Maximum Earthquake, buildings will be damaged beyond repairable limit but will not collapse.

4.2.4 Development of Elastic Site Response Spectra

Elastic response spectra for a site are based on estimate of Seismic Coefficient, CA which represents the effective peak acceleration (EPA) of the ground and Ce which represents 5 percent-damped response of a I-second system. These coefficients are depends on the seismic zone, the proximity of the site to active seismic sources, and site soil profile characteristics.

4.2.4.1 Seismic zone

Bangladesh is divided into three seismic zones as per Code BNBC 1993. The table below shows the values of zone coefficients of Bangladesh.

Table 4.1. Seismic Zone Factor Z

Zone	1	2	3
Z	0.075	0.15	0.25

4.2.4.2 Seismic Source Type:

Three types of Seismic Source may be defined [4].

Table 4.2. Seismic Source Type

		Seismic Source Definition	
Seismic Source Type	**Seismic Source Description**	**Maximum Moment Magnitude, M**	**Slip Rate, SR(mm/yr)**
A	Faults that are capable to produce large magnitude events and which have a high rate of seismic activity	M>=7.0	SR>=5
B	All faults other than types A and C	Not Applicable	Not Applicable
C	Faults that are capable to produce large magnitude events and which have a high rate of seismic activity	M<6.5	SR<2

4.2.4.3 Near-Source Factor

Bangladesh does not have any active fault map. It is not possible to estimate the seismic source distance from a specific site. But it may be safely assumed that all the sources are more than 15 km distance and the Table 4.3 from ATC [4] may be used to quantify Near-Source effects.

Table 4.3. Seismic Source Factor

Seismic Source Type	Closed Distance to known Source							
	<= 2 km		5 km		10 km		>= 15 km	
	N_A	N_V	N_A	N_V	N_A	N_V	N_A	N_V
A	1.5	2.0	1.2	1.6	1.0	1.2	1.0	1.0
B	1.3	1.6	1.0	1.2	1.0	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

1. The near-source factor may be used on the linear interpolation of values for distance other than those shown in the table.

2. The closest distance of the seismic source shall be taken as the minimum distance between the site and the area described by the vertical projecting of source on the surface (i.e. surface projection of fault plane). The surface projecting need not include portions of the source a depths of 10km or greater. The largest value of the near-source factor considering all sources shall be used for design.

4.2.4.4 Seismic Coefficients

For each earthquake hazard level, the structure is assigned a seismic coefficient C_A in accordance Table 4.4 (ATC-40, 1996) and a seismic coefficient C_V in accordance with Table 4.5[4]. Seismic coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5 percent-damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5 percent-damped response of a I-second system and divided by period defines acceleration response in the velocity domain. These coefficients are dependent on soil profile type and the product of earthquake zoning coefficient-Z, severity of earthquake-E and near source factor-N.

Table 4.4. Seismic Coefficients C_A

Soil Profile Type	**Shaking Intensity, ZEN[1,2]**			
	=0.075	**=0.15**	**=0.20**	**=0.30**
SB	0.08	0.15	0.20	0.30
SC	0.09	0.18	0.24	0.33
SD	0.12	0.22	0.28	0.36
SE	0.19	0.30	0.34	0.36
SF	Site-specific geo-technical investigation required to determine C_A			

1. The value of E used to determine the product. ZEN should be taken to be equal to 0.5 for the serviceability Earthquake. 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.

2. Seismic coefficient C_V should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4.5. Seismic Coefficient C_V

Soil Profile Type	**Shaking Intensity, ZEN**			
	=0.075	**=0.15**	**=0.20**	**=0.30**
S_B	0.08	0.15	0.20	0.30
S_C	0.13	0.25	0.32	0.45
S_D	0.18	0.32	0.40	0.54
S_E	0.26	0.50	0.64	0.84
S_F	Site-specific geo-technical investigation required to determine C_v			

1. The value of E used to determine the product. ZEN should be taken to be equal to 0.5 for the serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.

2. Seismic coefficient C_V should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4.6. Soil Profile Types

		Average Soil Properties for Top 100 ft of Soil Profile		
Soil Profile Type	**Soil Profile Name/Generic Description**	**Share Wave Velocity, V_S(ft/sec)**	**Standard Penetration Test, N or N_{CH} for cohesion less soil layers(blow/ft)**	**Undrained shear Strength, S_U(psf)**
S_A	Hard Rock	$V_S>5,000$	Not Applicable	
S_B	Rock	$2,500<V_S\leq 5,000$	Not Applicable	
S_C	Very Dense Soil and Rock	$1,200<V_S\leq 2,500$	N>50	$S_U>2,000$
S_D	Stiff Soil Profile	600 $V_S\leq 1,200$	$15\leq N\leq 50$	$1,000\leq S_U \leq 2,000$
S_E	Soft Soil Profile	$V_S<600$	N<50	$S_U<1,000$
S_F	Soil Requiring Site-Specific Evaluation			

4.3 Global Building Acceptability Limits

These response limits include requirement for the vertical load capacity, lateral load resistance, and lateral drift.

Table 4.7. Deformation Limit (ACT-40, 1996)

	Performance level			
Inter-story Drift Limit	**Immediate Occupancy**	**Damage Control**	**Life Safety**	**Structural Stability**
Maximum Total Drift	0.01	0.01~0.02	0.02	$0.33\ V_i / P_i$
Maximum inelastic drift	0.005	0.005~0.015	No Limit	No Limit

4.4 Establishing Demand Spectra

For the purpose of subsequent analysis to be made in this thesis, it is necessary to establish an earthquake demand spectra against which building performance will be evaluated. The following controlling parameters are considered:

Location of the site	:	Dhaka City
Soil profile at the site	:	Soil type SC as per Table 6.5, soft soil with shear wave velocity V s<600 ft/sec
Earthquake source type	:	A - considering the great Indian Earthquake in Assam in 12 June, 1897
Near Source Factor	:	> 15km

Table 4.8. Calculation of C_V

Factors		Value	Reference
Seismic Zone Factor, Z	=	0.15	as per BNBC/93
Earthquake Hazard Level, E	=	1	Design Earthquake
Near-Source Factor, N	=	1	>15km. table 4.3
Shaking Intensity, ZEN	=	0.15	
For Soil Type S_E, C_V	=	0.5	From Table 4.5

Table 4.9. Calculation of C_A

Factors		Value	***Reference***
Seismic Zone Factor, Z	=	0.15	*as per BNBC/93*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor, N	=	1	*>15km. Table 4.3*
Shaking Intensity, ZEN	=	0.15	
For Soil Type S_E, C_A	=	0.3	*From Table 4.5*

An elastic response spectrum, for each earthquake hazard level of interest at a site, is based on the site seismic coefficients C_A and C_V calculated above. The coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5

times C_A represents the average value of peak response of a 5% damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5% damped response of a I-second system and when divided by period defines acceleration response in velocity domain.

Effective peak ground acceleration (EPA)	=	0.3	G	C_A
Average value of peak response	=	0.750	G	$2.5C_A$
Seismic coefficient, C_V	=	0.5	G	C_V
T_S	=	0.667	See	$T_S = C_V / 2.5C_A$
T_A	=	0.133	See	$T_A=0.2T_S$

Calculation of Period and Spectral Acceleration

Period	Spectral Acceleration (g's)
0.000	0.300
0.133	0.750
0.667	0.750
0.800	0.625
1.000	0.500
1.200	0.417
1.400	0.357
1.600	0.313
1.800	0.278
2.000	0.250
2.200	0.227
2.400	0.208
2.600	0.192
2.800	0.179
3.000	0.167

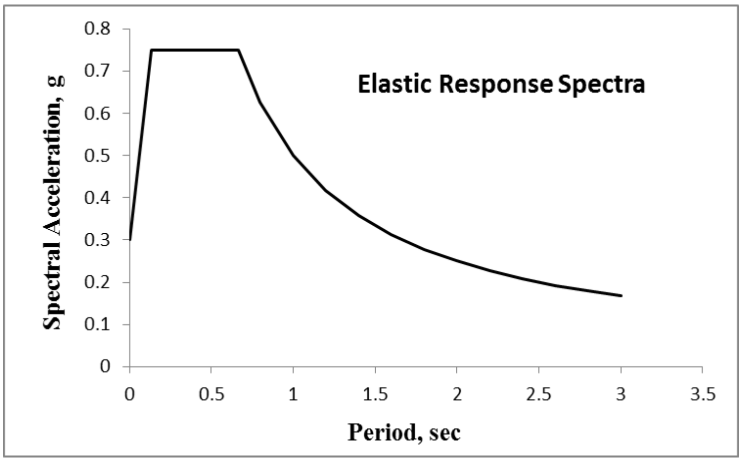


Figure 4.1. Construction of a 5% Damped Elastic Response Spectrum

For seismic performance evaluation purpose, this newly constructed site specific response spectra need to be converted in to ADRS format using relation,

$$S_d = \frac{T^2}{4\pi^2} S_a g$$

T, Sec	S_a,g	S_d,m
0.000	0.300	0.00
0.1333	0.750	0.00
0.667	0.750	0.08
0.800	0.625	0.10
1.000	0.500	0.12
1.200	0.417	0.15
1.400	0.357	0.17
1.600	0.313	0.20
1.800	0.278	0.22
2.000	0.250	0.25
2.200	0.227	0.27
2.400	0.208	0.30
2.600	0.192	0.32
2.800	0.179	0.35
3.00	0.167	0.37

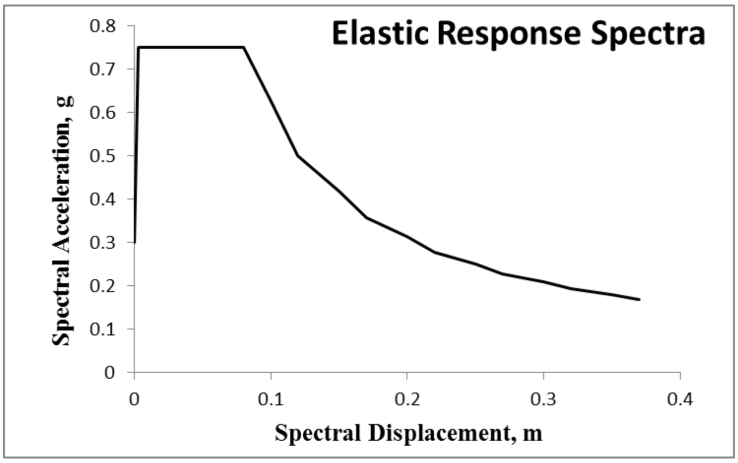


Figure 4.2. Site-specific elastic response spectra in ADRS format

4.5 Elementary Hinge Property

It is known that reinforced concrete does not respond elastically to load level about half the ultimate value. When an element is stressed beyond its elastic limit, due to inelastic deformation of the materials, the element will continue to deform disproportionate to its load, this process is called formation of plastic hinge.

4.5.1 Concrete Axial Hinge

Concrete axial hinge is formed when the axial load carrying capacity of a section exceeds its elastic limit. The elastic limit for axial capacity is different for tension and compression. The limits are explained in Fig. 4.3.

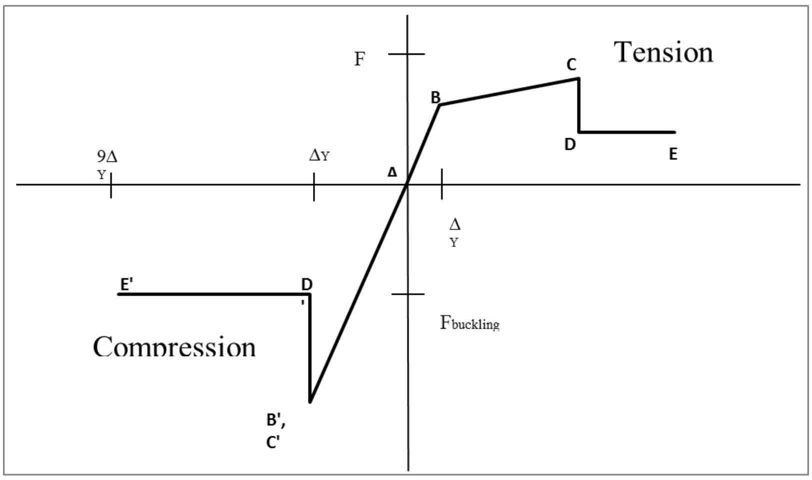


Figure 4.3. Concrete axial hinge property

Axial hinge features used in analysis:

- $p_y = A_S F_y$
- $P_C = 0.85 A_C f'_C$
- Slope between points Band C is taken as 10% total strain hardening for steel
- Hinge length assumption for .Δy is based on the full length
- Point B, C, D and E based on recommendation of Federal Emergency Management Agency [Pre-standard and Commentary/or the Seismic Rehabilitation 0f Buildings]
- Point B' = P_C
- Point E' taken as 9Δy

4.5.2 Concrete Moment Hinge and Concrete P-M-M Hinge

Concrete moment hinge is formed when the flexural moment carrying capacity of a section exceeds its elastic limit. The limits of flexural moment capacity and bi-axial moment with axial load are explained in the Figure. 4.4.

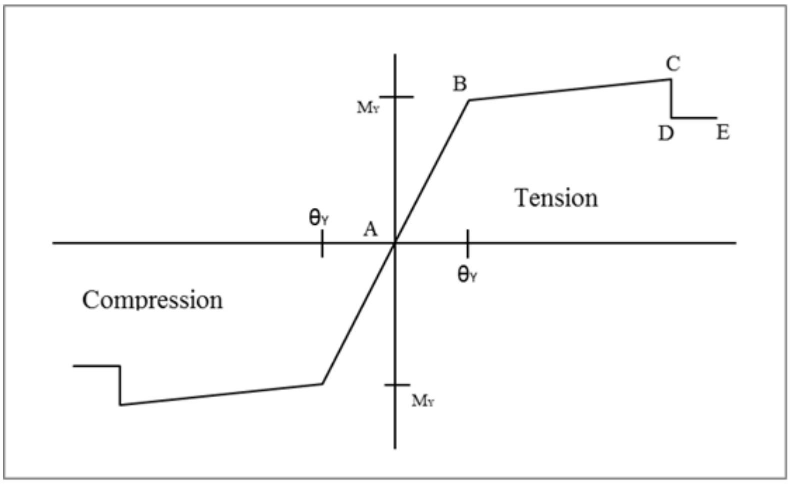


Figure 4.4. Concrete moment and P-M-M hinge property

P-M-M hinge Features used in analysis:

- Slope between points Band C is taken as 10% total strain hardening for steel
- θ_y = 0, since it is not needed
- Points C, D and E based on the recommendation of Advance Technology Council [4] (see Table 4.3).
- M_y based on reinforcement provided,

- P-M-M curve is for major axis moment and is taken to be the same as the Moment curve in conjunction with the definition of Axial-Moment interaction curves.

4.5.3 Concrete Shear Hinge

Concrete shear hinge is formed when the flexural carrying capacity of a section exceeds its elastic limit. The elastic limit for flexural shear capacity for coupling beams controlled by flexure and controlled by shear is explained in Figure. 4.5.

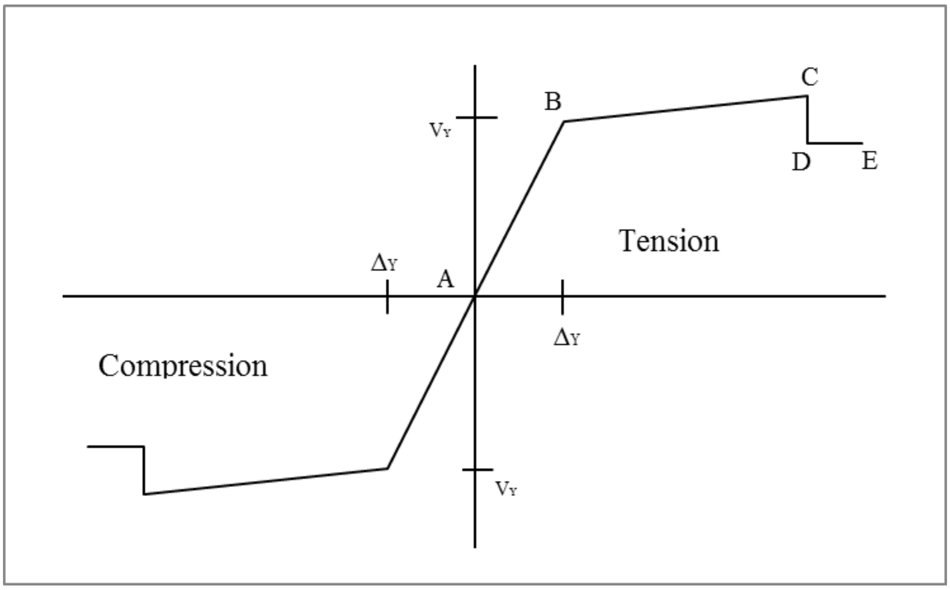


Figure 4.5. Concrete shear hinge property

Shear hinge features used in analysis:

- Slope between points Band C is taken as 10% total strain hardening for steel
- $V_y = 2A_S\sqrt{(f'c)} + f_y A_s vd$

Points C, D and E based on the recommendation of Advance Technology Council [4].

4.6 Concrete Frame Acceptability

To determine the performance objective of a structure, response quantities from a nonlinear static analysis are compared with limits for appropriate performance levels. Following tables define the modelling parameter for beam and column in terms of plastic angles within the yielding plastic hinge.

Table 4.10. Modelling Parameters for Nonlinear Procedures - Reinforced Concrete Beams [4]

			Modelling Parameters[3]		
			Plastic Potation Angle, rad		**Residential Strength Ratio**
Component Type			a	b	c
1. Beam Controlled by Flexure[1]					
$\rho - \rho' / \rho_{bal}$	Transverse Reinforcement[2]	$V/b_w d\sqrt{f'c}$[4]			
≤0.0	C	≤3	0.025	0.05	0.2
≤0.0	C	≥6	0.02	0.04	0.2
≥0.5	C	≤3	0.02	0.03	0.2
≥0.0	C	≥6	0.015	0.02	0.2
≤0.0	NC	≤3	0.02	0.03	0.2
≤0.0	NC	≥6	0.01	0.015	0.2
≥0.5	NC	≤3	0.01	0.015	0.2
≥0.5	NC	≥6	0.005	0.01	0.2
2. Beams controlled by shear[1]					
Stirrup spacing ≤ d/2			0.0	0.02	0.2
Stirrup spacing < d/2			0.0	0.01	0.0
3. Beams controlled by inadequate development or splicing along the span[1]					
Stirrup spacing ≤ d/2			0.0	0.02	0.0
Stirrup spacing < d/2			0.0	0.01	0.0
4. Beams controlled by inadequate embedment into beam-column joint[1]					
			0.015	0.03	0.2

1. When more than one of the conditions 1.2.3 and 4 occur for a given component use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement". 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: (1) closed stirrups are spaced at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. Linear interpolation between values listed in the table is permitted

4. V = design shear force

Table 4.11. Modelling Parameters for Nonlinear Procedures – Reinforced Concrete Column [4]

			Modelling Parameters[4]		
			Plastic Potation Angle, rad		**Residential Strength Ratio**
Component Type			a	b	c
1. Column Controlled by Flexure[1]					
$P/A_g f'c$[5]	Transverse Reinforcement[2]	$V/b_w d\sqrt{f'c}$[6]			
≤0.1	C	≤3	0.02	0.03	0.2
≤0.1	C	≥6	0.015	0.025	0.2
≥0.4	C	≤3	0.015	0.025	0.2
≥0.4	C	≥6	0.01	0.015	0.2
≤0.1	NC	≤3	0.01	0.015	0.2
≤0.1	NC	≥6	0.005	0.005	-
≥0.4	NC	≤3	0.005	0.005	-
≥0.4	NC	≥6	0.0	0.00	-
2. Column controlled by shear[1]					
Hoop spacing ≤ d/2 or $P/A_g f'c$[5] ≤ 0.1			0.0	0.015	0.2
Other cases			0.0	0.0	0.0
3. Columns controlled by inadequate development or splicing along the clear Height[1.3]					
Hoop spacing ≤ d/2			0.01	0.02	0.4
Hoop spacing < d/2			0.0	0.01	0.2
4. Column with axial loads exceeding 0.40 Po[1.3]					
Conforming reinforcement over the entire length			0.015	0.025	0.02
All other cases			0.0	0.0	0.0

1. When more than one of the conditions 1, 2, 3 and 4 occur for a given component. use the minimum appropriate numerical value from the table.

2. Under the heading “transverse reinforcement". 'C’ and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: (1) closed hoops are spaced at ≤d/3 and (2) for components of moderate and high ductility demand the strength provided by the stirrup (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. To quality. (I) hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spelling of cover concrete.

4. Linear interpolation between values listed in the table is permitted.

5. P = Design axial load

6. V = design shear force

4.7 Hinge Properties for Modelling

Different hinge properties may be modelled based on the modelling parameter defined through Table 4.10 and 4.11 depending upon the longitudinal reinforcement, transverse reinforcement etc.

4.7.1 Reinforced Concrete Beams – M3 Hinge

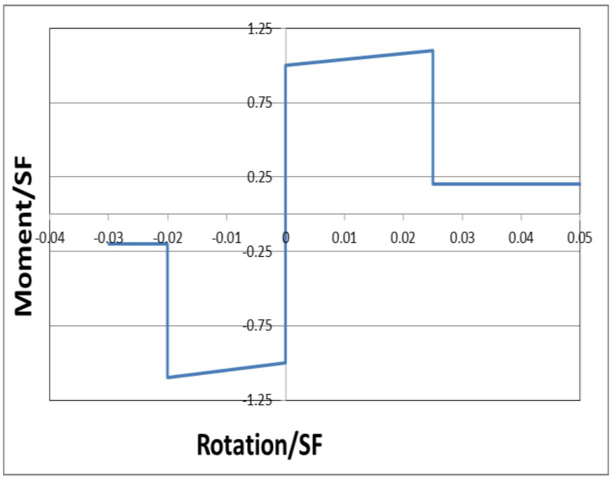
Beams controlled by flexure

Conforming transverse reinforcement

Table 4.12. Beam-M3 Hinge Properties

Point	Moment/SF	Rotation/SF
E-	-0.2	-0.03
D-	-0.2	-0.02
C-	-1.1	-0.02
B-	-1	0
A	0	0
B	1	0
C	1.1	0.025
D	0.2	0.025
E	0.2	0.05

Acceptance Criteria	Column1	Column2
IO	LS	CP
0.01	0.02	0.025



4.7.2 Reinforced Concrete Column - M2/M3 Hinge

Columns controlled by flexure

Conforming transverse reinforcement

Table 4.13. Column-M2/M3 Hinge Properties (Axial force 192)

Point	Moment/Yield Mom	Rotation/SF
A	0	0
B	1	0
C	1.1	0.015
D	0.2	0.015
E	0.2	0.025

Acceptance Criteria	Column1	Column2
IO	LS	CP
0.002	0.002	0.003

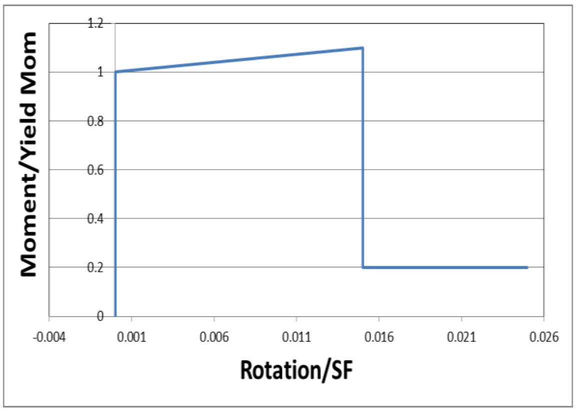
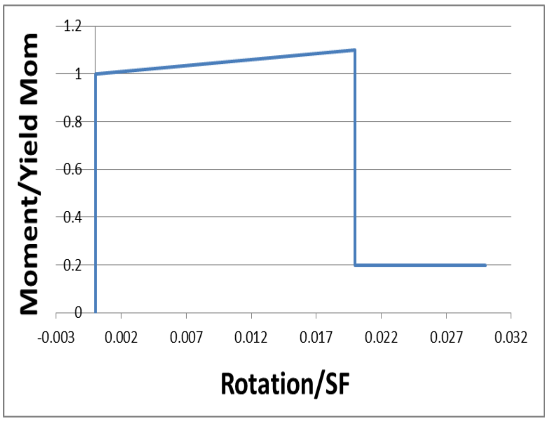


Table 4.14. Column-M2/M3 Hinge Properties (Axial force 48)

Point	Moment/Yield Mom	Rotation/SF
A	0	0
B	1	0
C	1.1	0.02
D	0.2	0.02
E	0.2	0.03

Acceptance Criteria	Column1	Column2
IO	LS	CP
0.002	0.002	0.003



5 SEISMIC PERFORMACE EVALUATION OF BUILDING STRUCTURE BY PUSHOVER ANALYSIS

5.1 General

Evaluation of seismic deficiencies is the most complicated tasks and a great important tool for a design engineer to comment on the structure acceptable limit. Especially the important structures like hospitals or clinic buildings are needed to be evaluated from the perspective of seismic resistance requirements as per code. To this end, this present study is aimed to determine deficiencies focusing seismic conceptual design requirement of building as per codes and also to identify the present situation of the Secretariat Clinic Building located at Secretarial of Government of Bangladesh, Dhaka for which drawings and necessary information are available. The details data of the case study structure are presented below.

5.2 Structure: Secretariat Clinic

Location: 5 storied building at Dhaka City

This structure is a 5 story 4x3 bay building as shown in Figure 5.1. Finite element modelling of the structure has been developed as shown in Figure 5.2. It has been found from the available drawing that the building was designed as frame structure with required parameters as specified. There are two types of column i,e 12x20 in and 15x15 in. The floors beam dimensions are 24x12 and 22x10 in and the thickness of the slab is 5 in. The building Ground floor height is 11ft, typical floor height 11ft and that from base to plinth 6 ft. Other geometric parameters are described in Table 5.1.

Table 5.1. Geometric parameters of Secretariat Clinic

Structure	**Roof height, ft**	**Seismic dead weight, kip**	**Geometric Parameters**					
			L,ft	*B,ft*	*Bay*	*L/H*	*B/H*	*L/B*
Secretariat Clinic	55	3226	60	46	4X3	1.09	0.83	1.30

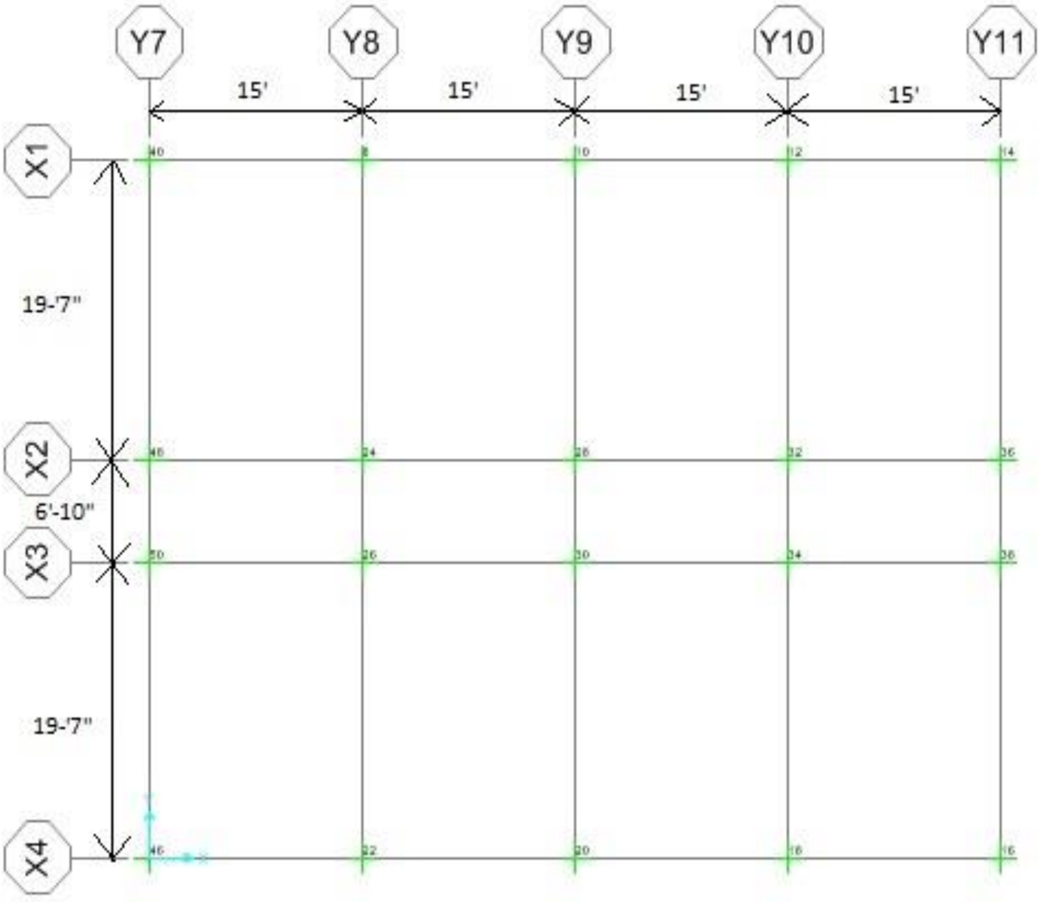


Figure 5.1. Typical floor plan of model

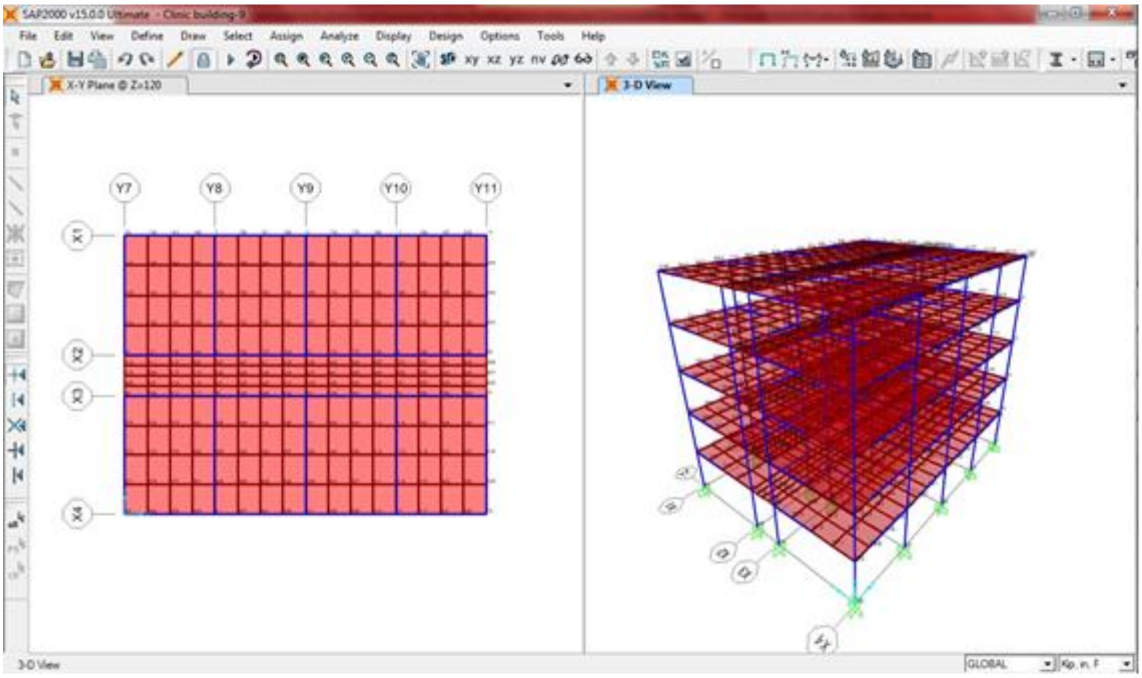


Figure 5.2. Typical 3D finite element model

5.3 Seismic Performance Evaluation of Structural through Pushover Analysis

In the previous chapters an outline of the procedure for structural performance evaluation in the light of ATC-40, 1996 has been described. For the performance evaluation purpose, Dhaka is selected as the site and seismic demand has been estimated as per guideline of ATC-40.

The performance of the structure as evaluated through pushover analysis have been presented through capacity curves and capacity spectrum described in the Section that follow.

5.4 Structural and other Parameters

For basic design and evaluation of the structure the following loading conditions have been considered.

5.4.1 Loading

Self-weight of the structure has been assumed as per geometric dimension of the structural elements with the unit weight of the concrete has been taken as 150 pcf. Other loading due to partition wall, floor finish, claddings loading etc. are considered as per available drawing of the selected structure. Code specified floor finish 25 psf has been considered on the floors and live load considered as 60 psf. Equivalent Static Load method has been used with response modification factor, R = 12. Earthquake load at any level equally distributed among all the nodes in that level.

Table 5.2. Load Distribution in the Structure

a) Load from Slab	
i) Slab thickness (5’’)	= 5*120/12 =62.5 psf sd
ii) Plaster of Ceiling (1/2’’)	= 0.5*120/12 = 5 psf
iii) Floor finish	= 25 psf
b) Load from Wall	
i) 5’’ Brick Wall + Plastering	= 80 psf
iii) 10’’ Brick Wall + Plastering	= 10 psf
c) Live Load	= 60 psf

The material properties and relevant features are as follows:

- The structure was designed with load combination defined in the SAP2000 with
    - Cylinder strength of concrete, f’c = 2 ksi (as per drawing)
    - Yield strength of steel, fy = 36 ksi (as per drawing)

  Sections of the column and beams as per drawing have been chosen.

- All supports are considered as fixed support.

The following assumptions are considered for the pushover analysis of the structure.

- Moment (M3) hinges are considered at the end of beam members and M2-M3 hinges are considered at the end of the column members. All hinges are according to as per ATC-40 document which is described in Section 4.7 of Chapter along with the performance limits.

- Pushover analysis has been done using load pattern Equivalent Static Load of BNBC 1993. Load intensities have been normalized with the base shear. Geometric non-linearity (P-Δ effects) of the structure was considered with full dead loads and 25% of the live load.

- In each case, the horizontal displacement of the left top most node of the structure has been selected for performance monitoring of roof displacement.

5.5 Performance Evaluation of Structure

The structure described in section 5.2 has been modelled and analysed using SAP2000. After analysing the structure, hinges defined in this chapter have been assigned to the respective members and pushover analysis has been performed to develop capacity curves for of the structure. The capacity curves such as base shear – displacement and capacity spectrums can be obtained after push over analysis. Accordingly performance points of the structure for the estimated seismic demand have been determined from the curves. Resulting outputs for the structure presented next. Hinge states near the performance point have been shown in colour code. A general Graphical representation of the performance point is given in Figure 5.3.

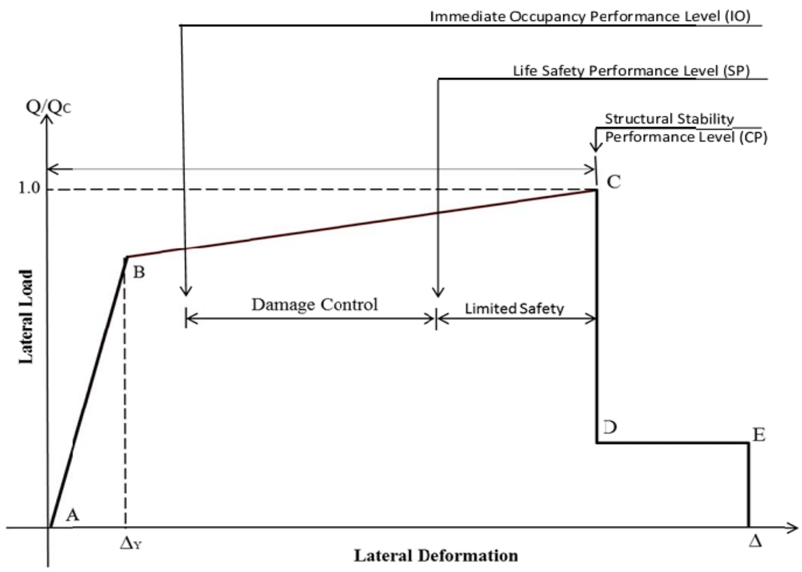


Figure 5.3. Typical Load-Deformation Acceptance Criteria

5.6 Calculation & Selection of Seismic Coefficient as per ATC-40, 1996

5.6.1 Structure: Secretariat Clinic

Location of the site = Dhaka city

Zone factor = 0.15

Type of Soil Profile = SC (Pile foundation)

Near source factor : > 15Km, N = 1.0

For serviceability Earthquake, E = 0.50

For design Earthquake, E = 1.00

For Maximum Earthquake, E = 1.25

Shaking Intensity, ZEN = 0.15 X 0.5 X 1 = 0.075 When E = 0.5

= 0.15 X 1 X 1 = 0.15 When E = 1.0

= 0.15 X 1.25 X 1 = 0.1875 When E = 1.25

Table 5.3. Summary of CA & CV for Secretariat Clinic

Name Of Structure	**E=0.5**		**E=1.0**		**E=1.25**	
	C_A	C_V	C_A	C_V	C_A	C_V
Secretariat Clinic Building	0.12	0.18	0.22	0.32	0.265	0.38

5.6.2 Results of Analysis

The secretariat clinic building is a 5-storey building. Detailed configuration of this structure is described in section. Well defined capacity curve have been found in two orthogonal directions which are shown in Fig. 5.4 (X-direction) and Fig.5.5 (Y-direction) and the capacity in Y-direction was found to be slightly greater than that in the X-direction.

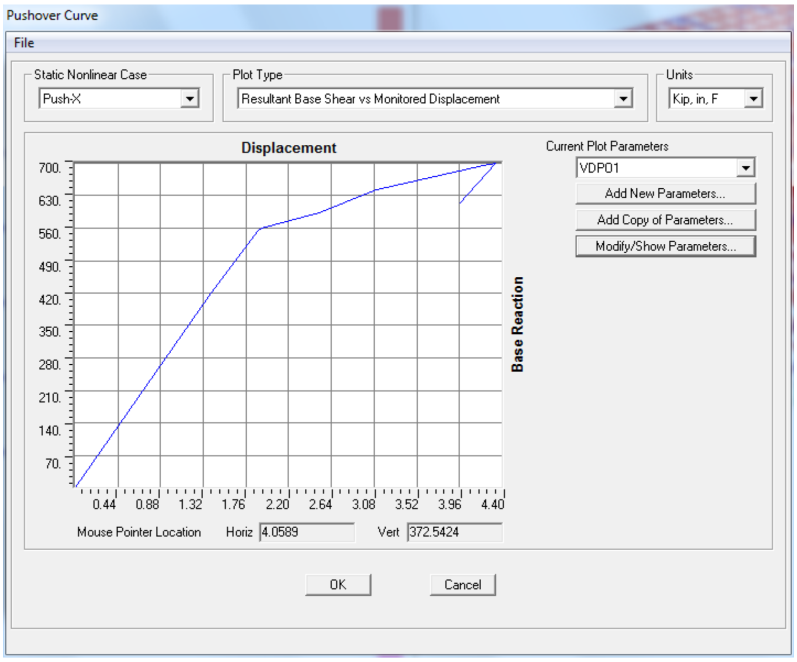


Figure 5.4. Capacity curves in X-direction

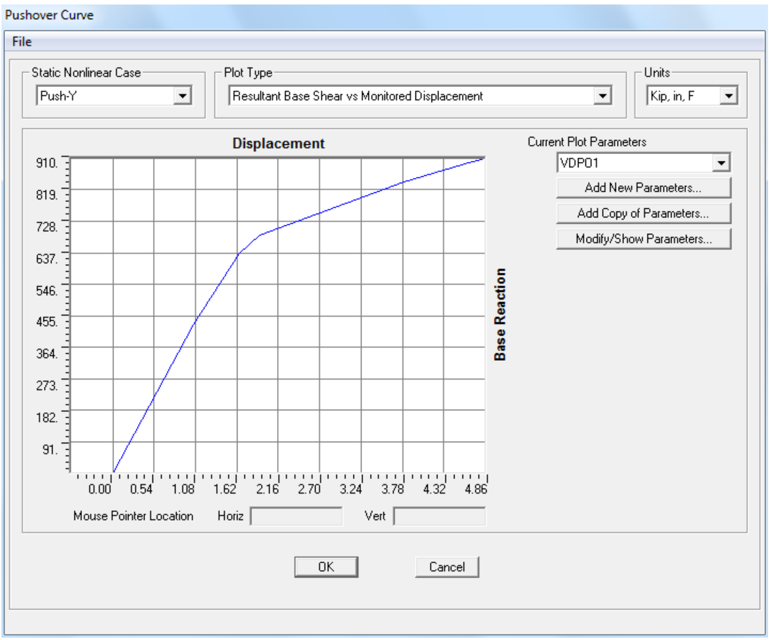


Figure 5.5. Capacity curves in Y-direction

The capacity curve has been converted to capacity spectrum as per ATC-40 procedure using SAP2000 which are shown in Fig.5.3 and Fig.5.4. It is observed that the capacity curves do not intersect the 5% damped elastic spectra in their elastic range. This means that some inelastic deformation will take place if the structure is subjected to the design level earthquake.

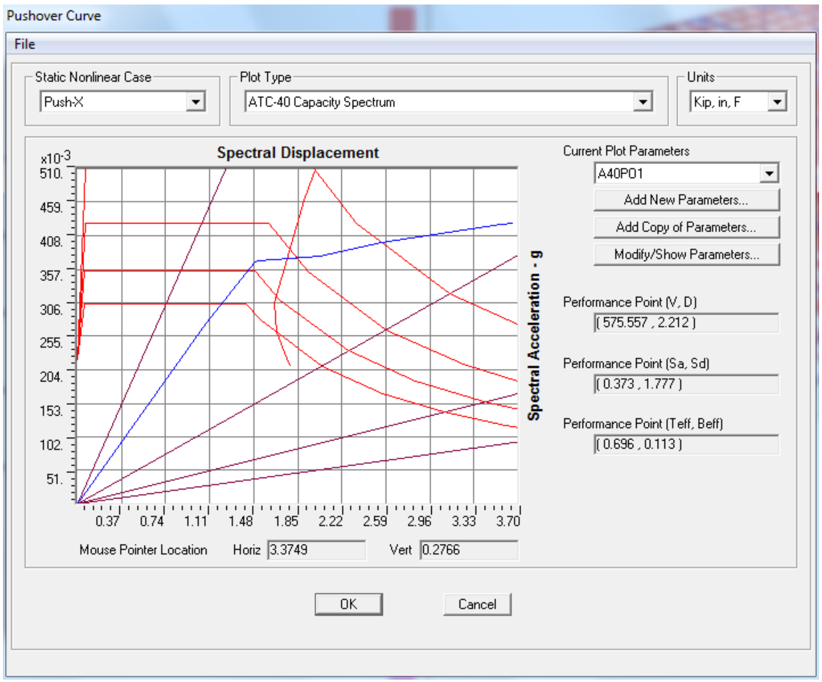


Figure 5.6. Capacity spectrum curves in X-direction

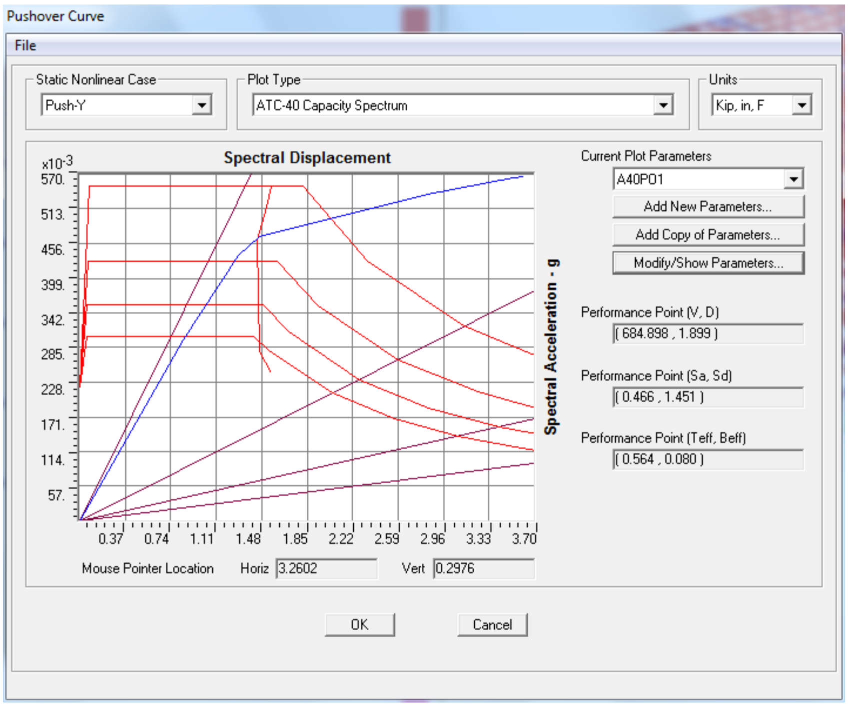


Figure 5.7. Capacity spectrum curves in Y-direction

Performance of the structure as calculated has been presented in Table 5.4 and Table 5.5 from the data retrieved from Fig 5.3 and Fig. 5.4 respectively.

Table 5.4. Performance of Structure in X-direction

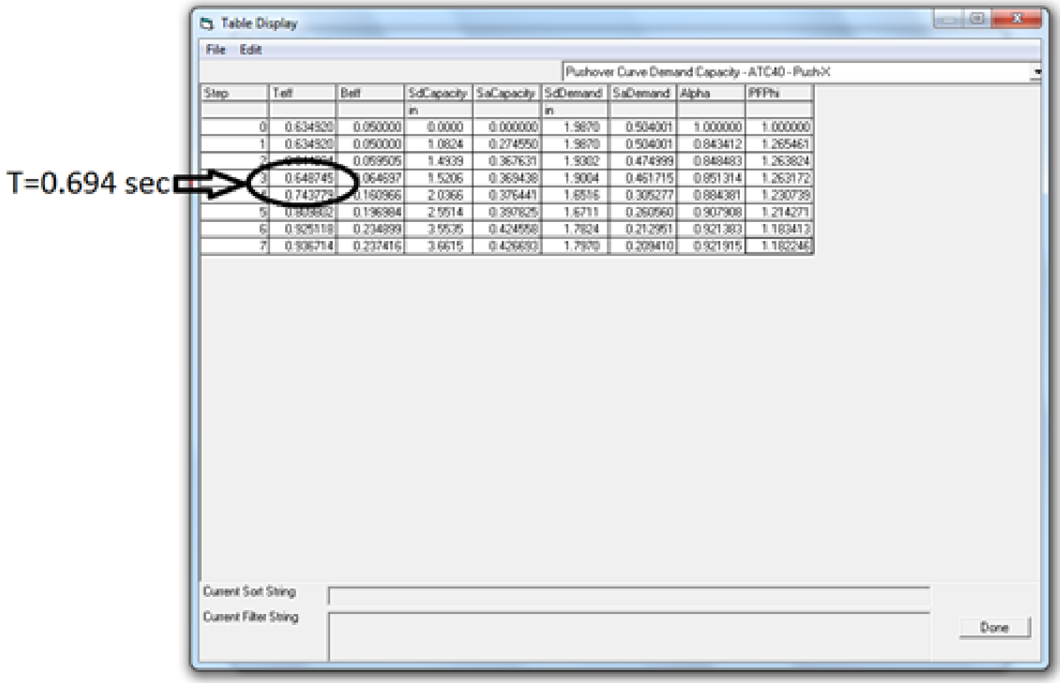
At performance in Point				
X-direction				
T_{eff} (sec)	**V_x(k)**	**Δ_x (in)**	**Max total drift ratio**	
0.694	576	2.2	0.0033	Immediate Occupancy

Table 5.5. Performance of Structure in Y-direction

At performance in Point				
Y-direction				
T_{eff} (sec)	**V_y(k)**	**Δ_y (in)**	**Max total drift ratio**	
0.564	685	1.9	0.0029	Immediate Occupancy

Hinge states of the yielded member at pushover step-3 and pushover step-4, which are the steps before and after the performance point in X-direction are shown in Fig.5.8 and Fig.5.9 respectively. These pushover steps are selected as the performance time period, Teff =0.694 sec lies in between step-3 and step-4 as shown in Table 5.6

Table 5.6. Demand Capacity table for X-direction



Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			in		in			
0	0.634920	0.050000	0.0000	0.000000	1.9870	0.504001	1.000000	1.000000
1	0.634920	0.050000	1.0824	0.274550	1.9870	0.504001	0.843412	1.265461
2	[illegible]	0.059505	1.4939	0.367631	1.9302	0.474999	0.848483	1.263824
3	0.648745	.064697	1.5206	0.369438	1.9004	0.461715	0.851314	1.263172
4	0.743779	.160966	2.0366	0.376441	1.6516	0.305277	0.884381	1.230739
5	0.809802	0.196984	2.5514	0.397825	1.6711	0.260560	0.907908	1.214271
6	0.925118	0.234899	3.5535	0.424558	1.7824	0.212951	0.921383	1.183413
7	0.936714	0.237416	3.6615	0.426693	1.7970	0.209410	0.921915	1.182246

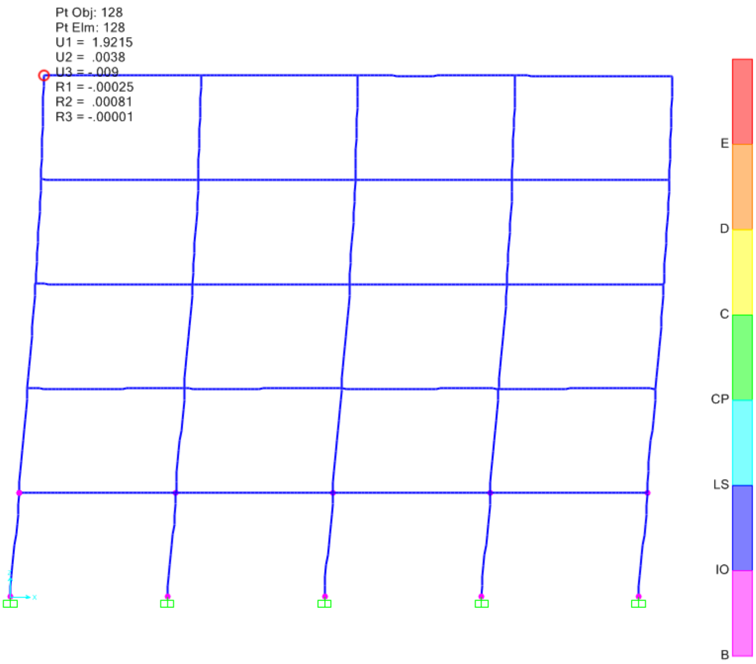


Figure 5.8. Hinge state of frame in X4 grid line during step-3 pushover

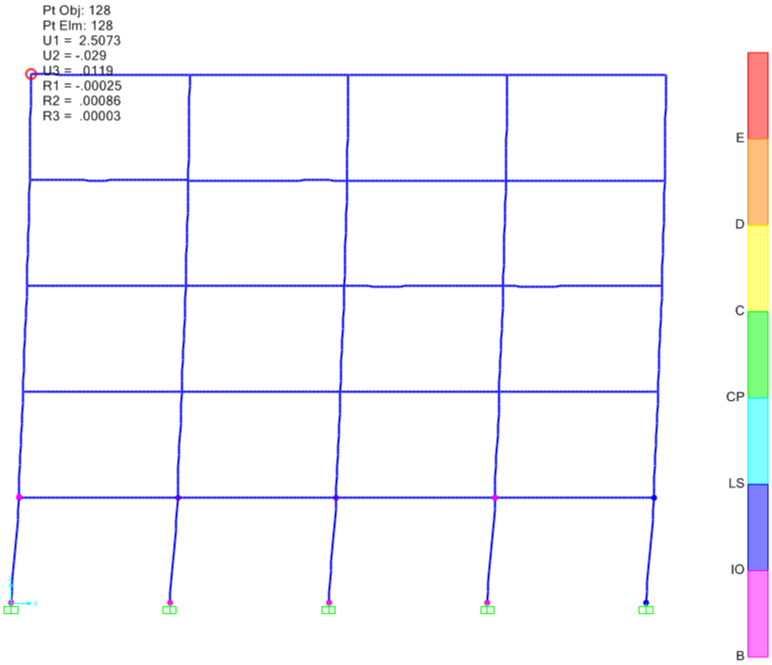


Figure 5.9. Hinge state of frame in X4 grid line during step-4 pushover

The inter-storey drift ration calculation corresponding to the two pushover steps are presented in Table 5.7 and Table 5.8. The maximum inter-storey drift from two tables (0.38% and 0.74%) fall well below the 1 % drift ratio limit corresponding to immediate occupancy performance limit.

Table 5.7. Inter-storey drift of frame in X4 grid line for pushover step-3

Storey	Displacement (in)	Drift (%)
5	1.92	0.12
4	1.76	0.24
3	1.44	0.35
2	0.98	0.36
1	0.5	0.38

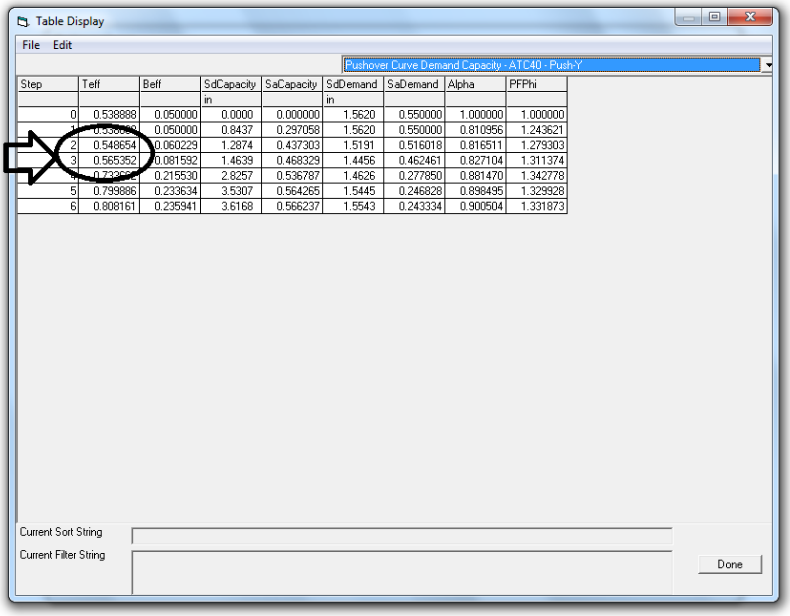
Table 5.8. Inter-storey drift of frame in X4 grid line for pushover step-4

Storey	Displacement (in)	Drift (%)
5	2.5	0.13
4	2.33	0.26
3	1.99	0.37
2	1.5	0.39
1	0.98	0.74

Hinge states of the yielded member at pushover step-3 and pushover step-4, which are the steps before and after the performance point in Y-direction are shown in Fig.5.10 and Fig.5.11 respectively. These pushover steps are selected as the performance time period, Teff =0.564 sec lies in between step-2 and step-3 as shown in Table 5.9.

Table 5.9. Demand Capacity table for Y-direction

Teff=0.564 sec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Pushover Curve Demand Capacity - ATC40 - Push-Y

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			in		in			
0	0.538888	0.050000	0.0000	0.000000	1.5620	0.550000	1.000000	1.000000
1	[illegible]	0.050000	0.8437	0.297058	1.5620	0.550000	0.810956	1.243621
2	0.548654	0.060229	1.2874	0.437303	1.5191	0.516018	0.816511	1.279303
3	0.565352	0.081592	1.4639	0.468329	1.4456	0.462461	0.827104	1.311374
4	[illegible]	0.215530	2.8257	0.536787	1.4626	0.277850	0.881470	1.342778
5	0.799886	0.233634	3.5307	0.564265	1.5445	0.246828	0.898495	1.329928
6	0.808161	0.235941	3.6168	0.566237	1.5543	0.243334	0.900504	1.331873

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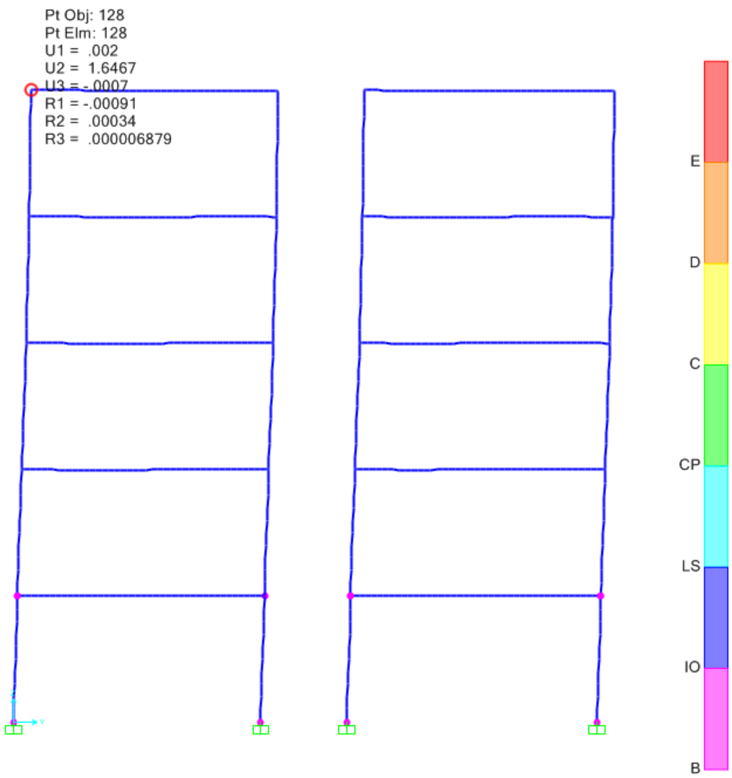

Figure 5.10. Hinge state of frame in Y7 grid line during pushover step-2

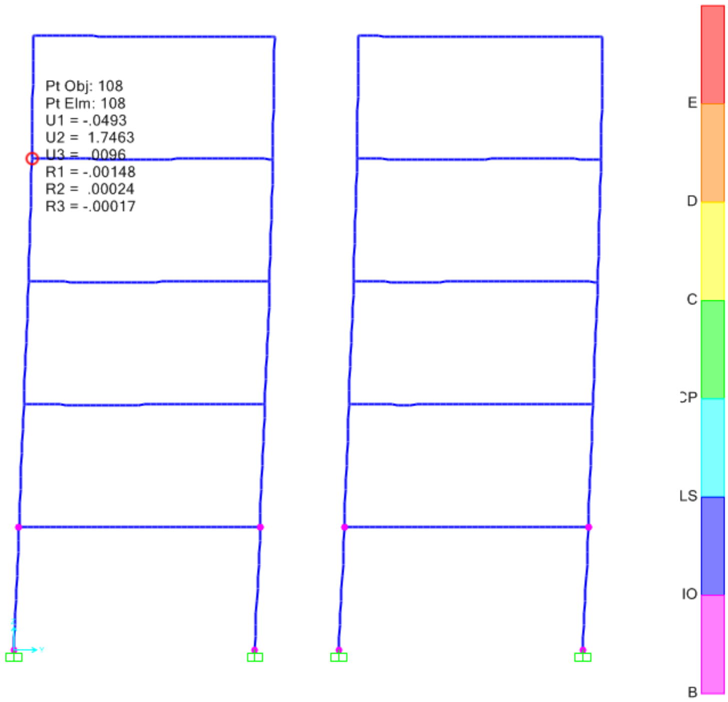

Figure 5.11. Hinge state of frame in Y7 grid line during pushover step-3 pushover

Table 5.10. Inter-storey drift of frame in Y7 grid line for pushover step-2

Storey	Displacement (in)	Drift (%)
5	1.65	0.12
4	1.49	0.23
3	1.19	0.32
2	0.77	0.30
1	0.37	0.28

Table 5.11. Inter-storey drift of frame in Y7 grid line for pushover step-3

Storey	Displacement (in)	Drift (%)
5	1.92	0.13
4	1.75	0.25
3	1.42	0.34
2	0.97	0.35
1	0.51	0.39

Hinge states corresponding to the maximum deflection in the two orthogonal directions has also been investigate and is presented in Fig.5.12 and Fig. 5.13. In case of X-direction, the maximum inter-storey drift ratio at maxim deflection is found to be 1.57 (Table 5.12) which indicates that the structure can be pushed to the damage control limit state but will not be able to sustain deformation corresponding to the life safety limit state as significant strength and stiffness degradation occurs after maximum deflection is reached. Similar behaviour is observed for maximum deflection in Y-direction as evident from Table 5.13 and Figure 5.13.

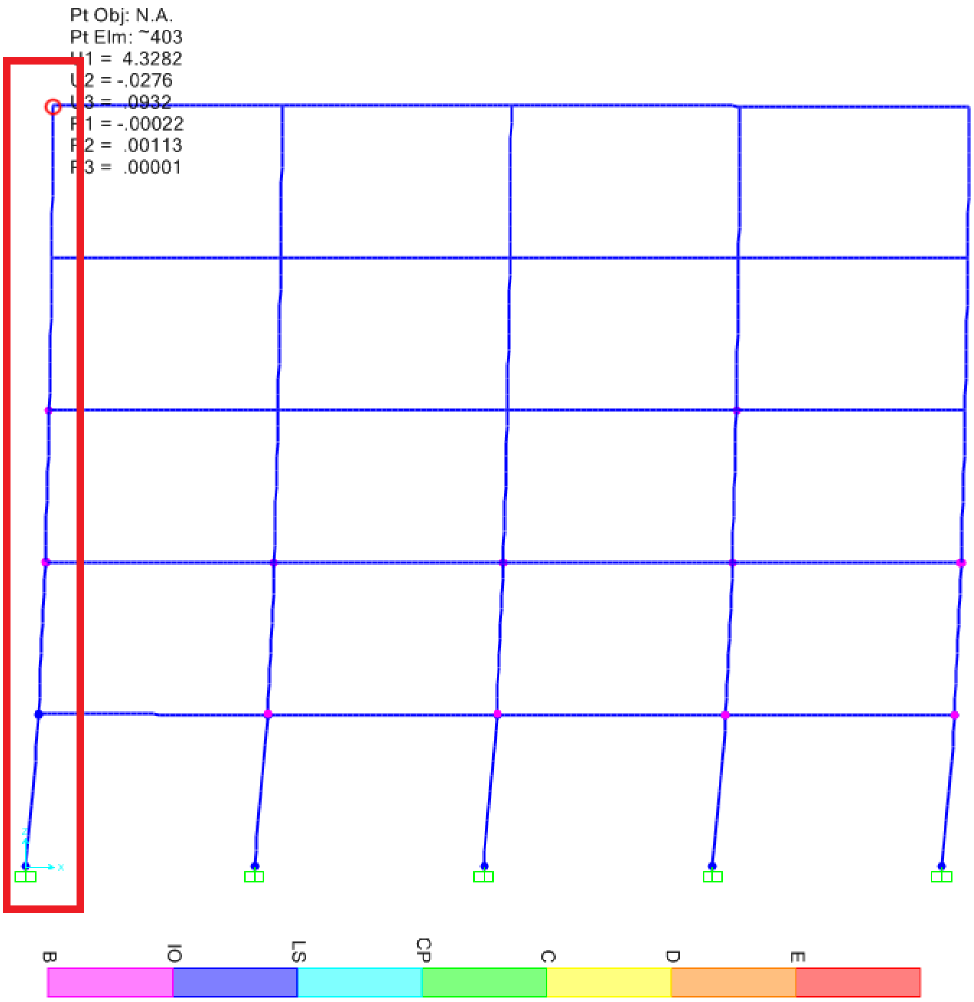

Figure 5.12. Hinge state of frame in X4 grid line corresponding to maximum displacement in X-direction

Table 5.12. Inter-storey drift of frame in X4 grid line corresponding to maximum deflection in X-direction

Storey	Displacement (in)	Drift (%)
5	4.33	0.17
4	4.11	0.31
3	3.7	0.45
2	3.1	0.78
1	2.07	1.57

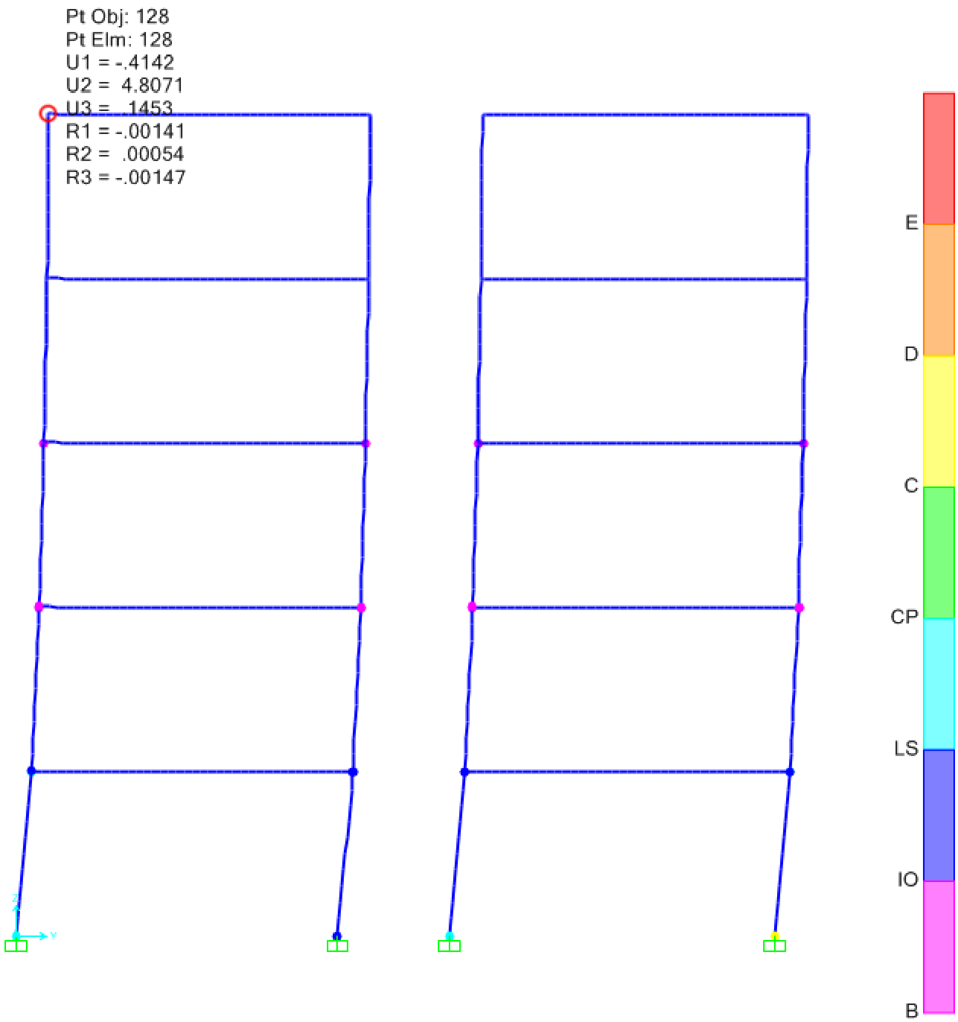

Figure 5.13. Hinge state of frame in Y7 grid line corresponding to maximum displacement in Y-direction

Table 5.13. Inter-storey drift of frame in Y7 grid line corresponding to maximum deflection in Y-direction

Storey	Displacement (in)	Drift (%)
5	4.8	0.18
4	4.56	0.33
3	4.12	0.52
2	3.43	0.88
1	2.27	1.72

6 CONCLUSION

Earthquake was considered unavoidable for a long time. It was accepted that building would be damaged as a result of earthquake ground motion. Preventive measures were therefore mostly limited to disaster management preparedness. The structural damages observed during several earthquakes in the past which are very educative in selecting suitable design earthquake performance level and also in identifying the suitable structural system. The traditional design approach to seismic design of a building is a force-base design without any measure of the deformation capability of a member or of a building. As a relatively new development, Pushover-based seismic design and evaluation methods offer a great opportunity to overcome the weakness of the traditional design approach.

The aim of this study is to increase the knowledge of the structural behaviour of the mid-rise apartment building in Dhaka city (Seismic Zone 2). The main topics of the interest are to determine deficiencies along with present situation focusing seismic conceptual design requirement of buildings as per BNBC 1993, ACI 2005, ATC 40, 1996 and FEMA 356, 2002. The selected structure is then evaluated in the light of seismic requirement as per codes to determine seismic deficiencies as presented in Chapter 5. The capacity curve and capacity spectrum curves obtained from the analysis are studied for Serviceability Earthquake (SE) at E=0.50 as well as for Design Earthquake (DE) at E=1.00. Then the structure, which is non-conforming (NC) to performance level, is again modelled with necessary retrofit treatment to attain the desired performance level for earthquake (DE) at E = 1.00 as presented.

With the limitation of structure of this study the following conclusion can be drawn.

1. The performance evaluation of the case study building in two orthogonal direction indicates that capacity curves do not intersect the 5% damped elastic spectra in their elastic range, which means some inelastic deformation will occur if the structure is subjected to the design level earthquake.

2. In two orthogonal directions the maximum inter-storey drift (0.38% and 0.74%) fall well below the 1 % drift ratio limit corresponding to immediate occupancy performance level as per ATC-40 guideline.

3. Capacity of a structure in any direction basically depends on the structural capacity of the structural members in that direction not on the number of bays present in that direction. As for example in X-direction we have 4 bays and in Y-direction we have 3 bays, but it has been found that the capacity in Y-direction (900 kip) is 30% greater than the X-direction (710 kip).

4. Global Performance of the Secretariat Clinic Building designed as per provision of BNBC, 1993 meet the Immediate Occupancy (OI) performance level.

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