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Pushover Analysis of Retrofitted Reinforced
Concrete Buildings

BY

Hamidreza Nahavandi

A research project report submitted in partial fulfillment
Of the requirement for the degree of

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Project Advisor:
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ABSTRACT

For the past decade or more, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis. Modeling for such analysis requires the determination of nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis is carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 such as SAP2000. While such documents provide the hinge properties for several ranges of detailing, programs may implement averaged values. In the realm of retrofitting of existing structures, few tested procedures have been identified such as steel jacketing and carbon fiber reinforced polymer (CFRP) composite jacketing system techniques in order to increase lateral stiffness of existing structures. Several methodologies have been developed using these processes in recent years. In some cases, the codes require nonlinear analysis for the verification of design proposed for a retrofitting a structure.

In this study, the seismic behavior of a typical residential building in South of Tehran, Iran, was investigated by performing static pushover analysis before and after retrofitting the columns. In the selected reinforced concrete (RC) structure, seismic analysis was performed for the structure retrofitted by two methods: 1) Steel jacketing and 2) CFRP jacketing technique assuming full composite action between jacketing and the existing concrete columns. By using nonlinear static (pushover) analysis, the performance level of structural members were evaluated for all structures, before and after retrofitting. The selected model building is a good representation of all typical residential buildings constructed in 1970's in Iran. These structures are almost always without proper seismic detailing. In this study, to investigate the effectiveness of the retrofitting systems, a comparative study was performed.

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1. Introduction

Throughout the world many existing reinforced concrete (RC) structures, constructed prior to earthquake-resistant design procedures, suffer from the inability to supply adequate ductility during earthquakes. Particularly, columns without adequate transverse reinforcement may fail due to lack of sufficient deformation capacity, and cause collapse of the building. Retrofit of this type of column by means of forming an additional steel jacket layer may supply the required transverse reinforcement and enhance the seismic performance by providing additional ductility, and reducing the seismic force demand. In this study, the seismic behavior of a typical existing building in Tehran, Iran is investigated by using pushover analysis before and after retrofitting its columns by fully steel jackets and CFRP jackets.

❖ Steel Jacketing

Confining reinforced concrete column in steel jackets is one of the effective methods to improve the earthquake resistant capacity, reported by Kenji Sakino and Yuping Sun [1]. Steel jacking has remarkable advantages in comparison with hoops and spirals rebar's warped around columns. Two major reasons of implanting steel jacketing can be addressed as first, having a vast amount of transverse steel which provides more confinement to the compressed concrete. Second, preventing concrete crumbling out of plane which could be considered as critical reason for deterioration of rebars and buckling of longitudinal bars in a column. Furthermore, steel jacketing is not only less interruptive, less time consuming and less expensive, but also results in minimum loss of floor area. Practically, steel jacketing (or caging) consists of steel angles at corners of RC columns and steel straps at few places along the height which provide composite action at the interface of steel and concrete element [2].

❖ CFRP Retrofit Technique

Another innovative retrofit technique, according to Richard D. Iacobucci and Shamim A Sheikh [3], is wrapping the concrete member with carbon fiber-reinforced polymer (CFRP). This material has been used as an attractive and constructive procedure for retrofitting of existing columns. Specifically the columns constructed prior to 1971 [3] which have shown vulnerability

in regards to inelastic conditions and may fail instantly without showing adequate warning during an intensive seismic event. Assembled fabric sheets are consisting of synthetic fibers along with resinous matrix which can be applied to any concrete sections. There are many advantages to this method. CFRP lightweight enables installation duration to be accomplished quickly with less labor expenses and service disruption of the building. This material also shows resistance to corrosion in chloride environments which can possibly leads to reduction in maintenance cost. The retrofit of vulnerable reinforcement concrete columns with CFRP will provide more ductility and increase energy dissipation capacities substantially along with vast improvement in terms-of total seismic resistance. The confinement provided by CFRP jacketing technique can cover the disadvantages of insufficient steel rebars and also ramp up shear and moment capacities as the jacketing transforms column response from brittle and non-ductile action to a more ductile response. CFRP retrofit technique substantially illustrates ductility enhancement and improved seismic behavior in comparison to previously damaged columns while the level of improvements are completely correlated to the intensity of damages. As the level of defectiveness grows [4], more CFRP layers are required in order to achieve a performance similar to undamaged retrofitted columns. CFRP jackets will be mostly provided for columns because of maintaining the discontinuity between columns and footings or beam-column joints [4] which strengthen the column sections and shift plastic hinge zones away from the interface to sections with lower stiffness capacities such as beams.

2. Purpose of Pushover Analysis

The purpose of the pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands in designing earthquake resistant buildings by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element and connection forces (for elements and connections that cannot sustain inelastic deformation). The inelastic static pushover analysis can be viewed as a method for

predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behavior. The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are examples of such response characteristics (Helmut Krawinkler) [5]:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam-to-column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformation demands for elements that have to deform in-elastically in order to dissipate the energy imparted to the structure by ground motions. Consequences of the strength deterioration of individual elements on the behavior of the structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have become the focus of thorough detailing.
- Identification of the strength discontinuities in plan or elevation that will lead to changes in the dynamic characteristics in the inelastic range.
- Estimates of the inter-story drifts that account for strength or stiffness discontinuities and that may be used to control damage and to evaluate P-delta effects. Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength, and the foundation systems.

❖ Pushover Analysis-Background Information

The static pushover analysis has no rigorous theoretical foundation. It is based on the assumption that the response of the structure can be related to the response of an equivalent single degree-of-freedom (SDOF) system. This implies that the response is controlled by a single mode, and that the shape of this mode remains constant throughout the time history response. Clearly, both assumptions are incorrect, but pilot studies carried out by several investigators (Lawson [6], Fajfar [7], Saiidi [8]) have indicated that these assumptions lead to rather good predictions of the maximum seismic response of multi degree-of-freedom (MDOF) structures, provided their response is dominated by a single mode. The formulation of the equivalent SDOF system is not unique, but the basic underlying assumption common to all approaches is that the deflected shape of the MDOF system can be represented by a shape vector $\{\Phi\}$ that remains constant throughout the time history, regardless of the level of deformation. Accepting this assumption and defining the relative displacement vector X of an MDOF system as $X=\Phi X_t$, (X_t = roof displacement), the governing differential equation of an MDOF system can be written as:

$$(1) \quad M\{\Phi\}\ddot{X}_t + C\{\Phi\}\dot{X}_t + Q = -M\{1\}\ddot{X}_g$$

Where M and C are the mass and damping matrices, Q denotes the story force vector, and $\ddot{X}_g$ is the ground acceleration. Then the definition of the reference SDOF displacement x^* will be as follows:

$$(2) \quad X^* = \frac{\{\Phi\}^T M \{\Phi\}}{\{\Phi\}^T M \{1\}} X_t$$

and pre-multiplying Eq. (1) by $\{\Phi\}^T$, and substituting for X_t using Eq. (2), we obtain the following differential equation for the response of the equivalent SDOF system:

$$(3) \quad M^*\{\Phi\}\ddot{X}_t^* + C^*\{\Phi\}\dot{X}_t^* + Q^* = -M^*\ddot{X}_g$$

$$(4) \quad M^* = \{\Phi\}^T M \{1\}$$

$$(5) \quad Q^* = \{\Phi\}^T Q$$

$$(6) \quad C^* = \{\Phi\}^T C \{\Phi\} \frac{\{\Phi\}^T M \{1\}}{\{\Phi\}^T M \{\Phi\}}$$

Presuming that the shape vector $\{\Phi\}$ is known, the force-deformation characteristics of the equivalent SDOF system (Q^* - X^* relationship, see *Figure 1*) can be determined from the results of a nonlinear incremental static analysis of the MDOF structure, which usually produces a base shear (V) - roof displacement (X_t or δ_t) diagram of the type shown with solid lines in *Figure 2*.

Figure 1 Force-displacement characteristics of MDOF structure and equivalent SDOF system.

Figure 2 Static base shear vs. roof displacement of MDOF.

In order to identify nominal global strength and displacement quantities, the multi-linear $V-\delta_t$ diagram shown in **Figure 2** needs to be represented by bilinear relationship that defines a yield strength, V_y , an effective elastic stiffness, $K_e=V_y/\delta_{t,y}$, and a hardening (or softening) stiffness, $K_s=\alpha K_e$ for the structure. Some judgment may be needed to define these properties. The simplified bilinear base shear-roof displacement response curve is needed to define the properties of the equivalent SDOF system.

The yield value of the base shear V_y and the corresponding roof displacement, $X_{t,y}$ ($\delta_{t,y}$ in **Figure 2**), are used together with Eqs. 2 and 5 to compute the force – displacement relationship for the equivalent SDOF system as follows:

$$(7) \quad X_y^* = \frac{\{\Phi\}^T M \{\Phi\}}{\{\Phi\}^T M \{1\}} X_{t,y}$$

$$(8) \quad Q_y^* = \{\Phi\}^T Q_y$$

where Q_y is the story force vector at yield; $V_y = \{1\}^T Q_y$

The initial period of the equivalent SDOF system, T_{eq} , can be computed as

$$(9) \quad T_{eq} = 2\pi \sqrt{\left\{ \frac{X_y^* M^*}{Q_y^*} \right\}}$$

The strain hardening ratio α of the $V-\delta_t$ relationship of the MDOF structure defines the strain hardening ratio of the equivalent SDOF system.

The basic properties of the equivalent SDOF system are now known. The roof displacement of the structure, X_t , is related to the equivalent SDOF displacement, X^* , by means of Eq. 2. Thus, the target displacement can be found if the displacement demand of the equivalent SDOF system can be estimated for the design earthquake. The utilization of inelastic spectral demand information requires the estimation of the ratio of elastic strength demand to yield strength of the equivalent SDOF system, it is convenient to divide Eq. 3 by M^* to obtain the differential equation of the unit mass equivalent SDOF system:

$$\ddot{X}^* + \frac{C^*}{M^*} \dot{X}^* + \frac{Q^*}{M^*} = -\ddot{X}_g$$

This Equation indicates the response of a unit mass SDOF system with period T_{eq} and yield strength $F_{y,eq}$ illustrated as

$$F_{y,eq} = \frac{Q_y^*}{M^*}$$

The use of T_{eq} and shape vector for estimating the properties of the equivalent SDOF system and the target displacement requires elaborate computations and time consuming iterations. Recognizing all the assumptions and approximations inherent in the pushover procedure, there is no good justification to be extremely precise in computations leading to the estimate of the target displacement.

So many studies have shown that the difference between first mode structure period and T_{eq} is usually small and its effect on the target on the target displacement can be neglected unless the design spectrum is very sensitive to small variations in period. Simplifications in the shape vector $\{\phi\}$ should also be acceptable. The use of a shape vector corresponding to the deflected shape at the target displacement is only a recommendation and has no theoretical foundation.

❖ Lateral Load Patterns

For a realistic performance evaluation the load pattern selection is likely more critical than the accurate determination of the target displacement. The load patterns are intended to represent and bound the distribution of inertia forces in design earthquake. It is clear that the distribution of inertia forces will vary with the severity of the earthquake and with time within an earthquake. The basic assumptions are that the distribution of inertia forces will be reasonably constant throughout the earthquake within the structure's stories. These assumption likely are reasonable if the structure response is not severely dependent on higher mode effects. It is attractive to utilize adaptive load patterns that follow more closely the time variant distribution of inertia forces. There are lot of different procedures have been implemented in this regards. The process that utilized in this project and reflected in SAP2000 program is Pseudo Lateral load which is based on modal analysis and

spectral response acceleration of structure. The pseudo lateral load in a given horizontal direction of a building shall be determined using Equation below: (EQ 3-10 in FEMA 356)

$$V = C_1 C_2 C_3 C_m S_a W$$

Where:

- C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.
- C_2 = this coefficient adjusts design values based on component hysteresis characteristics, stiffness degradation, and strength deterioration.
- C_3 = Modification factor to represent increased displacements due to dynamic P- Δ effects specified in Section 3.2.5.2 in FEMA 356.
- C_m = Effective mass factor to account for higher mode mass participation effects obtained from Table 3-1 in FEMA 356

After acquiring the lateral load, the vertical distribution shall be defined as below: (FEMA 356 section 3.3.1.3.2)

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{W_x h_x^K}{\sum_{i=1}^n W_i h_i^K}$$

Where:

- C_{vx} = Vertical distribution factor
- $K = 2$ for $T \geq 2.5$ s and 1 for $T \leq 0.5$ s
- W_i = Portion of the total building weight W located on or assigned to floor level i
- W_x = Portion of the total building weight W located on or assigned to floor level x
- h_i = Height (in ft) from the base to floor level i
- h_x = Height (in ft) from the base to floor level x

❖ Target Displacement

In pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as displacement demand for the corresponding equivalent SDOF system transformed to the MDOF domain through the use of a shape vector. Inherent in this approach is the assumption that maximum MDOF displacement is controlled by a single shape vector without regards to higher mode effects. Parametric studies [9] have shown that for frame and wall structure with a first mode period of less than 2 seconds this assumption is rather accurate for elastic system and conservative for inelastic systems.

All the important structural response characteristics in the prediction of the SDOF displacement demand implies the ability to represent the global load-deformation response of the structure by an equivalent SDOF system with appropriate hysteretic characteristics. For this purpose the simplified bilinear base shear-roof displacement diagram shown in **Figure 2** may serve as a skeleton, defining a yield level and an effective elastic and post elastic stiffness.

According to FEMA-356 (section 3.3.3.3.2), the target displacement is calculated by:

$$\delta_t = C_0 C_1 C_2 C_3 S_a (T_e^2 / 4\pi^2) g$$

Where:

- C_0 = Modification factor for SDOF $\rightarrow$ MDOF
- C_1 = Modification Factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
- C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response
- C_3 = Modification Factor to represent increased displacements due to dynamic P- Δ effects.
- S_a = Response spectrum acceleration
- T_e = Characteristic period of the response spectrum.

❖ Modeling Criteria of Structure

In general, a building needs to be modeled and analyzed as a three-dimensional assembly of elements and components. All elements that are part of the lateral or gravity load system and have significant rigidity or limited deformation capacity need to be represented in the analytical model. The elastic and inelastic strength and stiffness characteristic of each element need to be modeled to the extent that their important effects on the response of the building are reasonably represented. If elements have limited inelastic deformation capacity but their weakening in strength does not necessarily lead to unacceptable performance, then their force-deformation model should incorporate the post-deterioration range, as is shown in **Figure 3**. In a typical building, nearly all elements, including many nonstructural components, will contribute to the building's overall stiffness, mass, and damping, and consequently its response to earthquake ground motion. However, not all of these elements are critical to the ability of the structure to resist collapse when subjected to strong ground shaking. Elements and components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of lateral deformation of the structure, shall be classified as primary or secondary, even if they are not part of the intended lateral-force-resisting system. Elements and components that provide the capacity of the structure to resist collapse under seismic forces induced by ground motion in any direction shall be classified as *primary*. Other elements and components shall be classified as *secondary*.

The generalized force versus deformation curves used to specify component modeling and acceptance criteria for deformation-controlled actions in any of the four basic material types as is illustrated in **Figure 3**.

Figure 3 Element Deformation

Linear response is depicted between point A (unloaded component) and an effective yield point B. The slope from B to C is typically a small percentage (0-10%) of the elastic slope, and is included to represent phenomena such as strain hardening. C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD). Beyond point D, the component responds with substantially reduced strength to point E. At deformations greater than point E, the component strength is essentially zero. The sharp transition as shown on idealized curves in **Figure 3** between points C and D can result in computational difficulty and an inability to converge when used as modeling input in nonlinear computerized analysis software. In order to avoid this computational instability, a small slope may be provided to the segment of these curves between points C and D. Acceptance criteria for deformation or deformation ratios for *primary* members and *secondary* members [10] corresponding to the target Building Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) are illustrated in **Figure 4**.

Figure 4 Component or Element Deformation Acceptance Criteria and Performance Level

❖ Analysis System

The analysis consists of the application of gravity loads and represented lateral load pattern, and an incremental event-by-event analysis in which the load pattern is applied in increments corresponding to stiffness changes in each structural component. The first load step consists of an elastic analysis of the structure and scaling of loads to a level that corresponds to achievement of the first discontinuity in the force-deformation response which implies the first or first series of

hinges created in the elements (also known as first event). For the next load increment, the stiffness of that particular component is modified and another elastic analysis is performed, with the incremental loads again scaled to a level that corresponds to achievement of the next discontinuity in the force-deformation response in any of elements (second event). This process will continue until the summation for roof displacement of the structure reaches to aimed displacement. Therefore, the performance level at that point will be compared with the demand which is the target displacement calculated initially.

Generally, the pushover analysis provides information on force and deformation demands at target displacements that are associated with specified levels of performance. Performance evaluation consists of a capacity/demand evaluation of relevant parameter, such as the roof displacement, interstory drifts, inelastic deformations in elements and connections. The most significant and controlling factor in pushover analysis is the realization that life safety hazards are caused primarily by brittle failure modes in components and connections that are important parts of the gravity and lateral load paths. Consequently, the emphasis in performance prediction needs to be on:

- Verification that an adequate load path exists,
- Verification that load path remains sound at the deformation associated with target displacement level,
- Verification that critical connections remain capable of transferring loads between the elements that form part of the load path,
- Verification that individual elements that may fail in a brittle mode and that are important parts of the load path are not overloaded,
- Verification that localized failure do not pose a collapse or life safety hazard. Also the loads tributary to the failed elements can be transferred safely to other elements and that the failed element itself does not pose a falling hazard.

In the implementation of pushover analysis, modeling is one of the important steps. The model must consider nonlinear behavior of structure/elements. Such a model requires the determination of the component in the structure that are quantified by strength and deformation

capacities. The ultimate deformation capacity of a component depends on the ultimate curvature and plastic hinge length. The use of different criteria for the ultimate curvature and different plastic hinge length may result in different deformation capacities. In practical use, most often the default properties provided in the FEMA-356 documents are preferred, due to convenience and simplicity. These default properties can be implemented in well-known linear and nonlinear static and dynamic analysis programs such as SAP2000 [11].

In SAP2000, nonlinear behavior is assumed to occur within frame elements at concentrated plastic hinges. The default types include an uncoupled moment hinges, an uncoupled axial hinges, an uncoupled shear hinges and a coupled axial force and biaxial bending moment hinges. Below are the key elements of analysis process in SAP2000:

- Definition of the control node: control node is the node used to monitor displacements of the structure. Its displacement versus the base-shear forms the capacity (pushover) curve of the structure.
- Developing the pushover curve which includes the evaluation of the force distributions. To have a displacement similar or close to the actual displacement due to earthquake, it is important to consider a force displacement equivalent to the expected distribution of the inertial forces. Different forces distributions can be used to represent the earthquake load intensity.
- Estimation of the displacement demand: this is a crucial step when using pushover analysis. The control is pushed to reach the demand displacement which represents the maximum expected displacement resulting from the earthquake intensity under consideration.
- Evaluation of the performance level: performance evaluation is the main objective of a performance based design. A component or action is considered satisfactory if it meets a prescribed performance. As it is mentioned previously in section 2.3, FEMA-356 document has developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force deformation criteria for hinges used in pushover analysis. As illustrated in **Figure 4**, five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge and three points labeled IO, LS and CP

are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The SAP2000 static pushover analysis capabilities, which are fully integrated into the program, allow quick and easy implementation of the pushover procedures prescribed in the FEMA-356 documents for both two and three-dimensional buildings.

3. Step by Step Analysis Process

A typical residential concrete frame building which was constructed in 1970s and located in south of Tehran city in Iran, is subjected to retrofitting procedures using SAP2000 program. For this specific building, the soil class D has been assumed. The mapped MCER spectral response acceleration parameter at short periods and at a period of 1s have been identified as: $S_s = 2.320g$ and $S_1 = 0.883g$. A typical floor plan of this 6-story building which is used for housing purposes as well as complete 3D perspective view are given in **Figure 5a** and **5b**. As shown in the figure, it can be seen that all columns are rectangular and the structural system is symmetric in the orthogonal directions. Characteristic compressive strength of concrete is assumed as 3000 psi, which is a commonly accepted mean value for relatively older existing structures in Iran. Both longitudinal and transverse reinforcement are plain bars with a characteristic yield strength of 60000 psi. The column schedules of the original building are shown in **Table 1**. The longitudinal reinforcement of the original structure, consisting #3 bars at 8-inch spacing as well as all beams with the constant dimensions of 16"x16" having different size longitudinal rebars are listed in **Table 1**. Almost all of the columns are found to be inadequate in terms of flexure which will be addressed in following section. Since lateral stiffness of the structure is quite low due to small cross-sectional areas of its columns, natural frequency of the first mode is 1.04 Hz. It should also be noted that the high level of axial stresses on columns also reduces the ductility.

Figure 5a 3D Model View of the Structure

Figure 5b Plan View and Elevation View

Schedules of Reinforcement	Longitudinal Rebars		Transvers Bars Tie/Stirrup	
	Columns	Beams	Columns	Beams
Story 1&2	16"x16" 20#6	16"x16" 12#6/ Longer Spans 16#6	#3 every 8"	#3 every 8"
Story 3&4	16"x16" 16#6	16"x16" 12#6/ Longer Spans 16#6	#3 every 8"	#3 every 8"
Story 5&6	16"x16" 12#6	16"x16" 12#6/ Longer Spans 16#6	#3 every 8"	#3 every 8"
Graphical View	Columns		Beams	
Story 1&2				
Story 3&4				
Story 5&6				

Table 1 Schedules of Reinforcement

❖ Evaluation of Existing Building

As it is mentioned previously, push over analysis is more of an evaluation process rather than an analytical process for a design. In order to understand the deficiencies in terms of member sizing and number of rebars for this particular building, a **Response Spectrum Analysis** has been performed. The location of building is in Tehran, Iran, with almost stiff Soil. Furthermore, the spectral response acceleration parameters have been defined by “user specified” option integrated in SAP2000 as it shown in **Figure 6a**. Once again the mapped MCER spectral response acceleration parameter at short periods and at a period of 1s have been identified as: $S_s = 2.320g$ and $S_1 = 0.883g$. First step in response spectrum analysis is to define response spectrum function which in our case has been nominated from IBC 2012 as it is illustrated in **Figure 6a**. The next step would be the defining of the load case. It is shown in **Figure 6b** that load applied as an

acceleration in direction of consideration using characteristic of modal analysis allowing for CQC procedure for combination of results. In **Figure 6c**, the graphical results of response spectrum analysis have been illustrated. It is clear that all columns are found inadequate in terms of shear and shear-flexure (P-M interaction).

It should be mentioned that the live load that has been taken into account in this project is 40 psf which implies the maximum Live load for residential buildings. The value of base shear Calculated by response spectrum analysis is **2132 kips**.

Figure 6a Response Spectrum Function

Load Case Data - Response Spectrum

Load Case Name RS U1		Notes Modify/Show...		Load Case Type Response Spectrum	
Modal Combination		Directional Combination		Mass Source	
☒ CQC ☐ SRSS ☐ Absolute ☐ GMC ☐ NRC 10 Percent ☐ Double Sum		GMC f1: 1. GMC f2: 0. Periodic + Rigid Type: SRSS		☒ SRSS ☐ CQC3 ☐ Absolute Previous (MSSSRC1)	
Modal Load Case					
Use Modes from this Modal Load Case: MODAL					
☒ Standard - Acceleration Loading ☐ Advanced - Displacement Inertia Loading					
Loads Applied					
Load Type	Load Name	Function	Scale Factor		
Accel	U1	RS	386.4		
Accel	U1	RS	386.4		
				Add	
				Modify	
				Delete	
☐ Show Advanced Load Parameters					
Other Parameters					
Modal Damping		Constant at 0.05		Modify/Show...	
				OK	
				Cancel	

Figure 6b Response Spectrum Load Case

Figure 6c Graphical Result of RSA

Since the main purpose of this project is to determine the performance level of the building before and after the retrofiting, push over analysis is performed for an exciting building. Although it is clear that all columns are inadequate in terms of design loads, push over analysis determines

the performance level of the structure and demonstrates the trend of the failure at the event of earthquake.

The details of applying the push over analysis in SAP2000 are noted here explicitly. Therefore, in following sections mostly results of push over will be discussed.

After the modeling the structure, the load case for push over analysis should be created as shown in **Figure 7a**. In defining the load case for push over analysis, there are few steps that need attention. First, the type of analysis should be specified as nonlinear. Second, in order to achieve more accurate data, the P-Delta effect should be considered. P-Delta effect typically involves large external forces upon relatively small displacements. If deformations become sufficiently large as to break from linear compatibility relationships, then Large-Displacement and Large-Deformation analyses become necessary. Third, for capturing the nonlinearity of elements, the defined dead load (it is assumed that a dead load case was generated initially) should be set as nonlinear which allows push over load case to have nonlinear dead load case as a starting point. Accordingly, the initial condition will start from the nonlinear dead load case. Forth, controlling the displacement requires a control joint which can be specified in Load Application tab as demonstrated in **Figure 7a**.

It should be noted that the lateral load patterns need to be defined. In this project, as mentioned before, the Pseudo lateral load system along with vertical distribution of seismic forces according to FEMA 356 have been utilized. **Figure 7b** demonstrates the defining of push over lateral load case using response spectrum function in order to achieve S_a based on dominant mode of structure. Additionally, it is shown that the modification factors corresponding to Pseudo lateral load can be defined in this stage.

Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. The user defined flexural hinge properties capability of SAP2000 accurately represents FEMA 356 definition of hinges in columns and beams. The beams and columns were modeled with concentrated plastic hinges at the column and beam faces, respectively. Beams have only moment (M3) hinges, whereas columns have axial load and biaxial moment (PMM) hinges. The moment–rotation relations and the acceptance criteria for the performance levels of the hinges were obtained from FEMA 356 guideline (see **Figure 7c**). Next step is to determine a target displacement for the structure. The target displacement is an estimate of the seismic top displacement of the building

(roof level), when it is exposed to the design level earthquake excitation. The target displacement is calculated in respect to section 3.3.3.3.2 of FEMA 356. Then, a pushover analysis is carried out on the structure until the displacement at the top of the building reaches the target displacement. The extent of damage experienced by the building at the target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking.

After creating accurate model including proper hinges by the process discussed previously, the model analysis has been performed. The graphical results are shown in **Figure 7d** which clearly illustrate performance level in every hinges. The overall result demonstrate that at first floor, the building experience *soft story* (buildings are classified as having a "soft story" if that level is less than 70% as stiff as the floor immediately above it, or less than 80% as stiff as the average stiffness of the three floors above it) phenomenon which requires the immediate attention in terms of seismic retrofiting. The normalized base shear-top displacement relationships can be obtained by SAP2000 through Display tab and Show Static Pushover Curve. For the sake clarity, the values of push over curve (FEMA 356 Coefficient Method) have been extracted from the program and represented in **Figure 7f**.

As it is shown in **Figure 7f**, the maximum base shear that the exciting building can withstand is **1390 kips** which clearly less than our demand **2132 kips** that calculated by response spectrum analysis. The reason that target displacement is not calculated by the program is, the structure will experiences collapse mechanism before reaching the target displacement.

Load Case Data - Nonlinear Static

Load Case Name

Push

Set Def Name

Notes

Modify/Show...

Load Case Type

Static

Design...

Initial Conditions

☐ Zero Initial Conditions - Start from Unstressed State
 ☒ Continue from State at End of Nonlinear Case

DEAD

Important Note: Loads from this previous case are included in the current case

Modal Load Case

All Modal Loads Applied Use Modes from Case

MODAL

Loads Applied

Load Type	Load Name	Scale Factor
Accel	UX	1.
Accel	UX	1.

Add

Modify

Delete

Other Parameters

Load Application

Displ Control

Modify/Show...

Results Saved

Multiple States

Modify/Show...

Nonlinear Parameters

Default

Modify/Show...

Analysis Type

☐ Linear
 ☒ Nonlinear
 ☐ Nonlinear Staged Construction

Geometric Nonlinearity Parameters

☐ None
 ☒ P-Delta
 ☐ P-Delta plus Large Displacements

Mass Source

Previous

OK

Cancel

Load Application Control for Nonlinear Static Analysis

Load Application Control

☐ Full Load
 ☒ Displacement Control

Control Displacement

☐ Use Conjugate Displacement
 ☒ Use Monitored Displacement

Load to a Monitored Displacement Magnitude of

12.

Monitored Displacement

☒ DOF

U1

at Joint

35

☐ Generalized Displacement

OK

Cancel

Figure 7a Push over Load Case

Parameters For FEMA 356 Coefficient Method

Pushover Parameters Name

Push

Units

Kip, in, F

Demand Spectrum Definition

Effective Viscous Damping (0 < Damp < 1)

0.05

☒ Defined Function

RS

Scale Factor

386.

Characteristic Period of Resp Spec, Ts

1.

☐ FEMA 356 General Response Spectrum

Mapped Spectral Accel at Short Period, Ss

Mapped Spectral Accel at 1 Sec Period, S1

Site Class

Selected Coefficients

☒ User Value for C2

1.

☒ User Value for C3

1.

☒ User Value for Cm

0.9

Items Visible On Plot

☒ Show Capacity Curve

Color

☒ Show Idealized Bilinear Force-Displ Curve

Color

Reset Default Colors

Update Plot

Set Axis Labels and Range...

OK

Cancel

Figure 7b Pushover Lateral Load Case

Figure 7c Hinge Properties in Accordance with FEMA 356

Figure 7d Graphical result of Original Building Push over Analysis

Figure 7f Normalized Base Shear-Top Displacement for Original Structure

❖ Retrofitting The Structure by Steel Jacketing Method

As it is stated previously, confining reinforced concrete column in steel jackets in one of the effective methods to improve the earthquake resistant capacity. In this project, the concrete columns are strengthen by 9.5 mm (3/8") steel jackets on all four sides of the column under consideration (see **Figure 8a**). The jacketing arrangement is fully anchored to concert columns by 25mm (1") anchor bolts. These strong anchor bolts provide the fully composite action between concrete column and steel jacket system which fulfills the design assumptions in terms of modeling in SAP2000.

In this project, a residential building is under investigation, and the target performance level desired to meet has been selected as CP (collapse prevention) for sever earthquake. In SAP2000, the light blue color represents CP level of performance which is our goal. A process needs to be established in order to come up with the number of columns that requires retrofitting in terms of design efficiency. For that reason, a trial and error method has been performed. As presented in **Figure 8b**, three steps have been taken to identify the actual quantity of columns that

have need of retrofitting procedure. In first try, three rows of columns on each corner received full steel jacketing for all stories. Second trial, 8 more columns in two first floor added to the process. As it is shown in **Figure 8b**, even after second trial, the building suffers from low performance in first floor at the base level. Once the third trial took place which made 6 more columns in two first floor under retrofitting process, the building performance reached to CP level which is acceptable for a residential building.

The graphical results of the modeling has been displayed in **Figure 8c**. It is clear that all the hinges throughout the columns meet CP level of performance.

The normalized base shear-top displacement relationships obtained by pushover analysis for retrofitted structure by steel jacketing method of rehabilitation are presented in **Figure 8d**. As it is shown (red color line) the demand base shear is 2639 kips with target displacement of 6 inch. On the other hand, the capacity of the structure within performance level of CP which was the target level performance is determined around 3400 kips (green color line).

Figure 8a Typical Steel Jacket Detail for Reinforced Concrete Columns

Figure 8b Trial and Error Process

Figure 8c Graphical Result of Retrofitted Building by Steel Jacketing

Figure 8d Normalized Base Shear-Top Displacement for Retrofitted Structure by Steel Jacketing

❖ Retrofitting The Structure by CFRP Jacketing Method

CFRP confining of RC columns has received considerable attention for use in feeble structures due to its unique properties, such as high strength-to-weight ratio, stiffness-to-weight ratio, as well as corrosion and fatigue resistance. CFRP increases both ultimate strength and ductility of RC members. Besides, retrofitting techniques using CFRP-confined columns are sometimes directed at increasing flexural strength, when necessary. Thus, the retrofitting method should be used carefully because increasing flexural capacity increases the forces transferred to the foundation and the connections between the structure/column.

In this project, a specific CFRP has been chosen. Tyfo CFRP composite which the properties is shown in in **Figure 9a**, is one of the known brands in composite products for structural strengthening and retrofitting. Implementing the CFRP is generally simpler than other retrofitting

methods. For any member that requires CFRP wrapping, CFRP can be applied on a thin epoxy layer which has been already coated over the surface of the member under consideration. The surface must be clean, dry and free of protrusions or cavities, which may cause voids behind the CFRP composite.

The process of the modeling in CFRP in SAP2000 is similar to steel jacketing method. The main difference would be the specific material that needs to be defined in program in order to mimic the CFRP Tensile strength, Tensile Modulus, Density and Poisson's Ratio. In this study, the directional material type has been selected as isotropic which implies the material is uniformity in all orientations. A trial and error procedure has been taken into account to identify minimum quantity of columns that requires CFRP wrapping. The results came out similar to steel jacketing method with only one difference. For all inner columns, instead of two first stories, the wrapping had to go all the way up to third floor in order to meet CP level of performance. **Figure 9b** represents the three major steps of the trails.

The graphical results of the modeling has been presented in **Figure 9c**. As the results illustrate, all the hinges throughout the columns meet CP level of performance.

The normalized base shear-top displacement relationships obtained by pushover analysis for retrofitted structure by CFRP wrapping method of rehabilitation are presented in **Figure 9d**. As it is shown the demand base shear is 2294 kips (red color line) with target displacement of 4.52 inch. On the other hand, the capacity of the structure within performance level of CP which was the target level performance is determined around 2800 kips (green color line).

TYPICAL DRY FIBER PROPERTIES	
PROPERTY	TYPICAL TEST VALUE
Tensile Strength	550,000 psi (3.79 GPa)
Tensile Modulus	33.4 x 10 ⁶ psi (230 GPa)
Ultimate Elongation	1.7%
Density	0.063 lbs./in. ³ (1.74 g/cm ³)
Minimum weight per sq. yd.	19 oz. (644 g/m ²)

Figure 9a Tyfo® SCH-41 Reinforcing Fabric Properties

Figure 9b Trial and Error Process

Figure 9c Graphical Result of Retrofitted Building by Steel Jacketing

Figure 9d Normalized Base Shear-Top Displacement for Retrofitted Structure by CFRP wrapping

❖ Cost Estimation

The estimated cost for both steel jacketing and CFRP jacketing are provided as a comparison tool for evaluating a proper option in retrofitting.

Steel jacketing cost can be calculated as the price of 3/8" steel sheets and 40-1" fully threaded retrofitting bolts including installation and bolt fastening labor. Additionally, the welding cost of steel sheets around columns has been taken into account as it is shown in **Table 2**. CFRP jacketing cost mostly will sum up in material cost and labor cost for installation, surface preparation and applying epoxy. In general, the surface must be clean, dry and free of protrusions or cavities before applying the fiber carbon epoxy. The cost of CFRP implementation have been calculated in **Table 3**. As it is illustrated between two tables, the cost of steel jacketing is 49% higher than CFRP jacketing within the same targeted performance level. Although in steel

jacketing method, the structure will gain more capacity in terms of base shear, implementing the CFRP wrapping will deliver significant reduction in overall cost of the retrofiting process. It should be emphasized that these cost data are only estimates developed based on the experience of the author and conversations with colleagues. In practice, there may be a significant variation in costs compared to the data presented here.

	Steel Jacketing Cost \$
3/8 " Steel sheet per column 16 3/4" x 118"	1200
40 x 1" all threaded retrofitting bolt per column	100
Fastening Bolt Labor Cost per Column	3200
Steel Sheet Welding Cost per Column	2000
Number of Columns	84
Total Cost	546000

Table 2 Steel Jacketing Cost

	Steel Jacketing Cost \$
Cost of Tyfo CFRP and Epoxy per Column	900
Cost of Surface Preparation per Column	700
Cost of Applying Epoxy and CFRP per Column	1500
Number of Columns	91
Total Cost	282100

Table 3 CFRP Jacketing Cost

❖ Comparison of the Results

The deformed shapes and plastic hinges of the original structure and retrofitted ones are presented in **Figure 10a**. It can be interpreted that the original building experiences very low strength and ductility in columns. This is a clear demonstration of soft story phenomenon at first floor will lead to complete collapse of the structure the event of earthquake.

As it is illustrated in Figure 10a, the ductility of columns were increased noticeably due to CFRP wrapping and steel jacketing methods. Consequently, the retrofitted structures exhibit larger displacements without collapsing and also develops plastic hinges within CP level of performance. The normalized base shear-top displacement relationships obtained by pushover analysis for original and retrofitted structures are presented in **Figure 10b**. In comparison with the original structure, both retrofit techniques enhanced the strength and ductility characteristics of the building. The occupant friendly CFRP wrapping retrofit technique supplied good displacement capacity but less lateral strength than the other jacketing technique. On the other hand, the structure retrofitted by steel jacketing exhibited a more rigid behavior so that structural and non-structural elements could suffer less damage. Because of increased lateral stiffness, the natural frequencies of the retrofitted structures were increased as shown in **Table 4**. In comparison between demand and capacity base shear, it is clearly shown that the capacity demand ratio has been improved significantly. Additionally, the target displacements of the structures calculated in respect to FEMA 356 (section 3.3.3.3.2) have been demonstrated in the table.

The total cost between two methods of rehabilitations are assessed with 49% difference. The cost reduction in CFRP method in comparison with steel jacketing method shows that CFRP technique may be considered a cost effective method for retrofitting concrete structures.

Figure 10a Deformation and distribution of plastic hinges for original and retrofitted structures

	Natural Frequency (Hz)	Demand Base Shear (kips)	Capacity Base Shear (kips)	Target Displacement (in)	Material + Labor Cost \$
Original Structure	1.04	2132	1390	-	-
Steel Jacketing	1.17	2639	3400	6	546 k
CFRP Wrapping	1.08	2294	2800	4.52	282 k

Table 4 Analysis Results

Figure 10b Normalized base shear-top displacement for original and retrofitted structures

4. Conclusions

The following conclusions are obtained after making an attempt in analyzing the inelastic behavior of a typical existing reinforced concrete structure with various deficiencies before and after retrofitting it. Both retrofitting techniques improved the ductility characteristics of the structure. In the case of CFRP jacketing in addition to significant enhancement in ductility, flexural strength also increased slightly due to the contribution of CFRP jacketing. The columns retrofitted with full steel jackets developed the overall structural performance in terms of ductility and lateral strength, strength being more pronounced due to larger cross-sections and additional longitudinal reinforcement. Consequently, steel jacketing may be more preferable when lateral drifts must be limited, which in turn limits the damage as well. However, when fewer tenant disturbances are required and a relatively higher level of damage is acceptable against severe earthquakes, CFRP

jacketing may be more preferable. In this technique, since the dimensions of the columns are not changed, the seismic demand is not increased either.

In general, a carefully performed pushover analysis will provide insight into structural aspects that control performance during severe earthquakes. For structures that oscillate primarily in the fundamental mode, the pushover analysis will likely provide good estimates of global as well as local inelastic deformation demands. The analysis will also expose design weaknesses that may remain hidden in an elastic analysis.

In the author's opinion, pushover analysis can be implemented for structures whose higher modes are judged not to be significantly important, and it can provide an effective tool to evaluate performance level of structures.

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