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Seismic Performance Assessment of a School Building before and after Retrofit: A Case Study

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Abstract
2015 Gorkha earthquake showed the vulnerability and risk associated with the non-engineered and deficit framed structure. Many school buildings collapsed or performed poorly due to insufficient ductility and lack of proper supervision during the construction. Furthermore, the new seismic code NBC-105 2020 has been enforced and these buildings may not comply to the new building code. Many school buildings are still at risk of damage during another earthquake. Strengthening by suitable retrofit technique is significant to comply with the new building code. Building is modelled in ETABS capable of capturing Nonlinear behaviour of structural elements. Performance assessment of the building is done by performing Non-Linear Static Procedure (NSP). Performance of the building in association with their seismic demands are compared before and after retrofit

Keywords
Retrofit, Nonlinear static procedure, concrete jacketing

1. Introduction

For the seismic performance evaluation of existing building [1] recommends the capacity spectrum method. In this method capacity curve which is the plot of base shear and roof displacement is plotted in Acceleration Displacement Response Spectra (ADRS) format, demand curve is also plotted in the same spectrum and the point of intersection of the capacity curve and the demand curve is called performance point which is useful for the assessment of the seismic performance of the existing building. Performance level of Immediate Occupancy(IO), Life safety(LS) and Collapse Prevention(CP) is defined in the ATC document. Typical capacity spectrum from ATC 40 is shown in the figure 1.

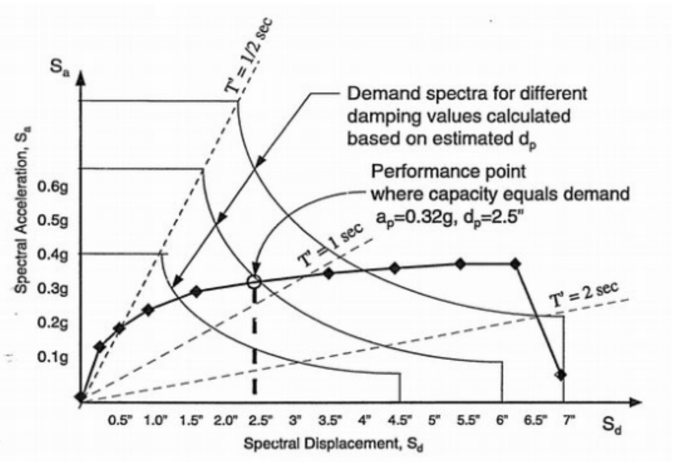

Figure 1: Performance point from ATC 40

Furthermore [2] provides an improved method over the one which is described in the ATC 40. In this method capacity curve is generated as per ATC 40 and Demand spectrum is adjusted for damping and both the curve are in ADRS format. Bi-linear representation of the capacity curve is done which defines the Initial period, Yield displacement and the yield period. Then Post elastic stiffness and ductility can be calculated as per the equation formulated in the FEMA 440 document. From these values effective damping and the effective time period is found. And for the solution of the performance point three methods are outlined in the document, one of which is Modified acceleration displacement spectra (MADRS) Locus of possible performance point. In this method locus of possible performance point is generated by assumed solution of performance points with their corresponding ductility. Modification factor is found out from effective period as outlined in FEMA 440 and Initial ADRS is adjusted to effective damping and acceleration component is multiplied by modification factor for MADRS. Possible performance point is intersection of Secant period with MADRS. Assumed performance point is increased or decreased to generate a locus and thus actual performance point is the intersection of the locus and the capacity spectrum. Typical locus of possible performance point is shown in figure 2.

For the purpose of this study FEMA 440 equivalent linearization method is used and to determine the performance point of the retrofitted structure locus of the possible performance point is used utilizing the incorporation of the method in the finite element modelling software ETABS.

Retrofitting is the process of strengthening structural components on a global or a component level so as to improve the seismic behavior of the structure. [3] categorized the strategies into 3 groups A,B and C based on how they improve the performance of the existing building. First group improves the deformation capacity of the building such as by applying Fiber reinforced polymer (FRP), Second group improves the strength and stiffness of the existing building such as by addition of new elements to the structural system.

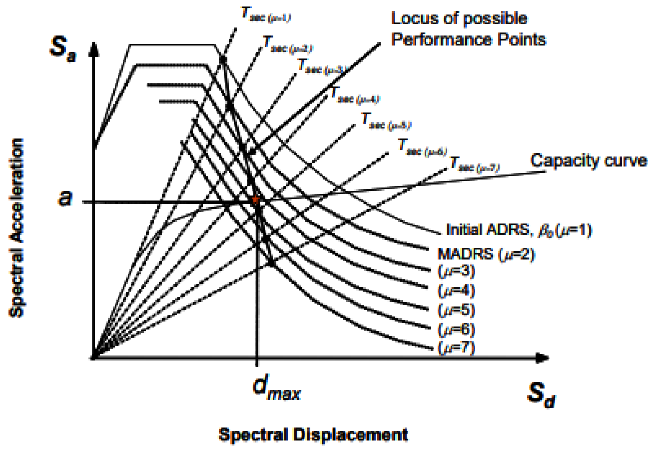

Figure 2: Locus of possible performance point using MADRS from FEMA 440

Third group improves both the stiffness and the deformation capacity of the building such as concrete jacketing.

This study aims for the evaluation of the seismic performance of the existing school building. Furthermore if the deficiencies are found in the building it will be retrofitted by the suitable technique and the seismic performance after the retrofitting will also be evaluated.To asses the performance of the retrofitted building nonlinear static pushover analysis will be carried out. Seismic performance will be compared in terms of base shear, time period, maximum displacement.

2. Existing Building Model

2.1 Geometric Characteristics

The building has the following geometric characteristics with identical first and second floor plan as shown in figure 3 with the staircase cover as a roof.

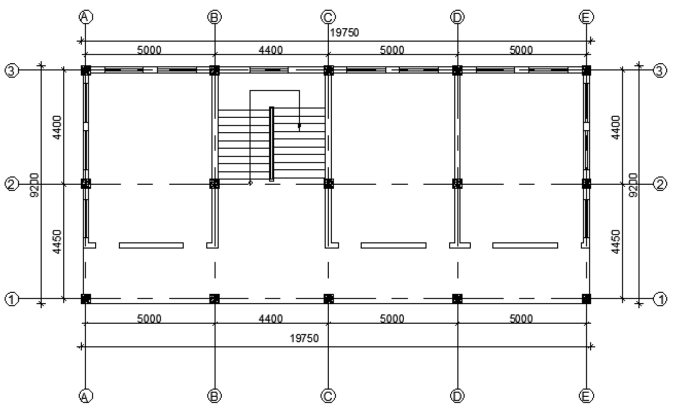

Figure 3: Geometric Characteristics of Existing Building

2.2 Structural Characteristics

The buildings consist of Moment resisting frame system with all columns having size of 350 × 350 in all floors. From reinforcement details found from existing structural drawings of the building, the column consists of 4 corner bar of 20mm and 4 edge bar of 16mm. Fe 415 and M15 is the grade of Steel and concrete respectively. Size of beam is 230 × 425 in first floor level and 230×355 in second and roof level. Typical beam consists of 2 top and bottom bars of 20 mm and 2 extra bar of 12mm at the support and center. 3-D Finite element modeling of the structure is done in ETABS as shown in figure 4.

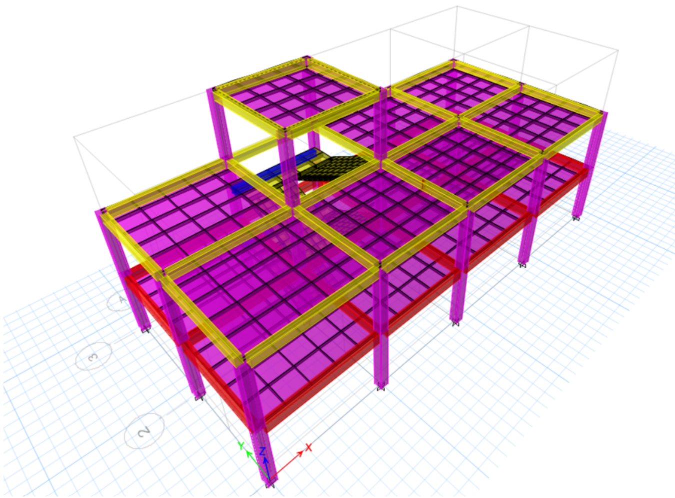

Figure 4: 3-D Finite Element Modelling of Existing building

Upon the Linear Static method of analysis performed on ETABS software with Equivalent static earthquake force applied with Base shear coefficient according to [4] and results shows certain deficiencies are present on the building. Torsion irregularities and some column failing under column beam capacity are major deficiencies on the building.

2.3 Column Beam Capacity Ratio

According to [4] Ratio of sum of the moment capacities of column end to the sum of beam end moment capacities at a joint should be greater than 1.2 i.e.,

$$\frac{\sum Mc}{\sum Mb} > 1.2$$

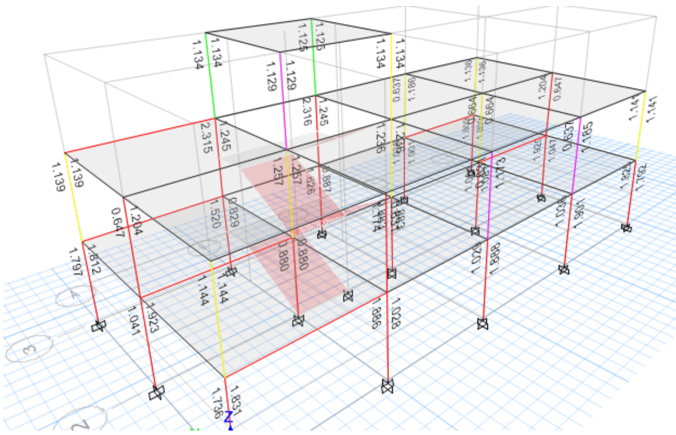

Figure 5: Existing Column Beam Capacity of Existing Building

Since the criteria set in [4] is not satisfied by several columns more critically by B1, C1, D1, B2, C2, B3, C3, D3 in the ground floor and D3, D2, D1 in the first floor. So there is chance of

Table 1: Torsional Irregularity Check on Existing Building

story	Load case	Max/Min	check
First	EQx	1.78	Not OK
First	EQy	1.5	Not OK
Second	EQx	1.73	Not OK
Second	EQy	1.57	Not OK

forming of global mechanism as column are weaker and there is a need to strengthen the column do that the moment of resistance can be increased. Existing column beam capacity is shown in the figure 5.

2.4 Torsion Irregularity

According to [4] the torsion irregularity exists when the maximum horizontal displacement at one end of the story is more than 1.5 times the minimum horizontal displacement at the other end when lateral forces are applied in the center of the mass.

It is evident from the table 1 that the torsional irregularities exists in the building mainly in the first and second floor when the earthquake forces are applied in both X and Y directions.

3. Retrofit of the Building

Due to the deficiencies as seen in the existing building mainly torsional irregularities and the failure at column beam capacity ratios, the building is retrofitted. Strengthening and stiffening retrofit strategies is employed with jacketing system employed at column. 3 column jacketing section is designed as shown in figure 6,7 and 8 and are placed at strategic location as shown in figure 9 so as to balance the torsion irregularities present in the existing structure. Sectional size and the reinforcement detail were calculated based on [5].vNominal sizes were obtained but the guidelines recommends minimum of 100mm thick jacket. Furthermore 150mm and 200mm jacket sections were also designed and placed so that the center of stiffness could be shifted towards the mass center in order to counteract the inherent torsion. Column beam capacity ratio and the torsion as well as the performance of the retrofit technique is checked to verify the effectiveness of the retrofit system.

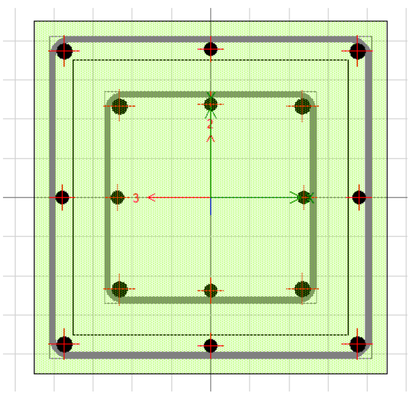

Figure 6: Jacket Section 450×450

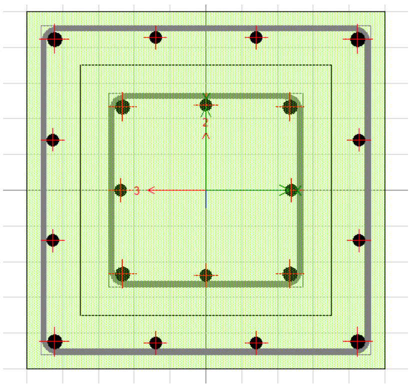

Figure 7: Jacket Section 500×500

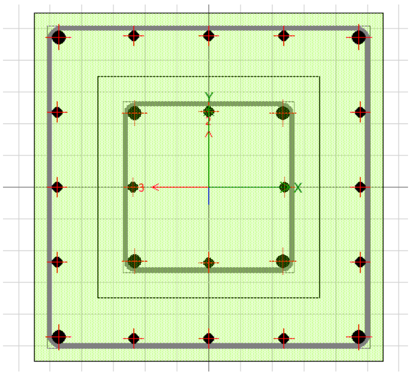

Figure 8: Jacket Section 550×550

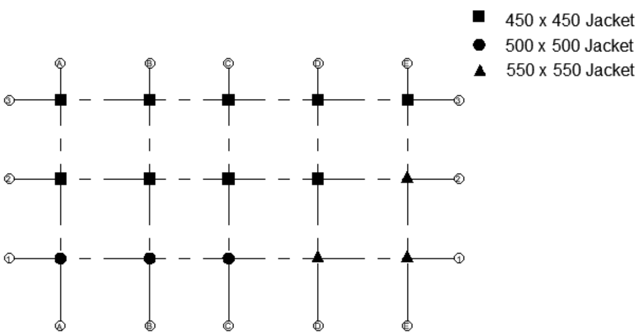

Figure 9: Location of the jacket sections

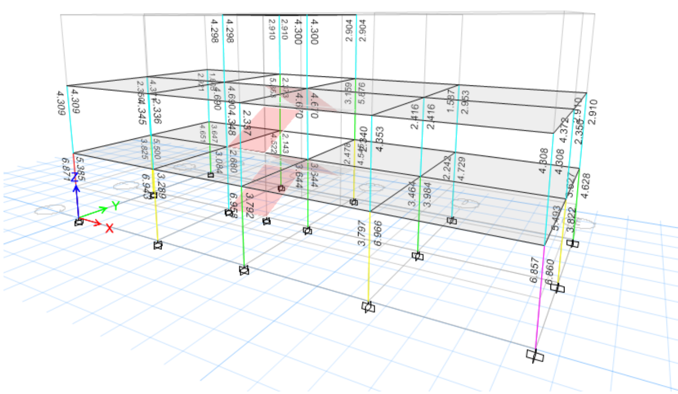

Figure 10: Column Beam Capacity of Retrofitted Building

After the application of the retrofit scheme the torsion irregularities is checked on the floors where torsional irregularities was seen in the existing building. From table 2 it is clear that the torsional irregularities no longer exist in the retrofitted building as the ratio of maximum horizontal displacement to the minimum horizontal displacement in the direction of applied earthquake loads in mass centers are within the permissible limit.

story	Load case	Max/Min	check
First	EQx	1.25	OK
First	EQy	1.32	OK
Second	EQx	1.40	OK
Second	EQy	1.27	OK

Table 2: Torsional Irregularity Check on Existing Building

After the application of the retrofit scheme the Column beam capacity is checked as the existing building failed to satisfy the criteria of column beam capacity design. It can be seen that from figure 10 column beam capacity is satisfied in all the columns of the retrofitted building as:

$$\frac{\sum Mc}{\sum Mb} > 1.2$$

3.1 Non linear Modelling for retrofitted structure

To determine the accurate response of structures during seismic excitation Non linear dynamic analysis is preferred but the computational effort required is of great magnitude. However non linear static analysis also can represent the response with accurate enough accuracy. Nevertheless model should be capable of capturing the non linear effects. For this hinges were assigned to the frames. P-M-M fiber hinges were assigned to the both ends of the columns which will account the non linearity from the material level. Non linear material properties are used from the software itself i.e. mander model is selected for confined concrete and for reinforcement bars default model. For beams auto generated plastic hinge were assigned. Moment curvature relation and the acceptance criteria for material were imported from [6].

4. Results and Discussion

Through the eigenvalue analysis of existing school building model it was found that the fundamental time period of first mode of vibration was 0.47seconds. Fundamental mode of vibration was observed in rotational degree of freedom. [7] recommends for the structure with torsion as a fundamental mode of vibration pushover analysis does not yield the correct results. So response spectrum analysis was performed which resulted in the base shear of 648.9 KN in both the direction. Furthermore maximum displacement was found to be 13mm in X direction and 18mm in Y direction.

Figure 11 shows the pushover curve of the building in the X-Direction of the retrofitted building. Two curves can be visualised in the figure which are demand and capacity curves. Their intersection is known as performance point. Performance point is essentially the measure of the response of structures with associated seismic demands. Displacement of 16mm was found with base shear of 2789.06 KN. All the assigned hinges in non linear model are under the performance level of life safety in given performance point. Hinges start to cross the performance level of life safety along the direction of loading (-X). First column to cross the performance limit of collapse prevention is C2 and C3 in ground floor.

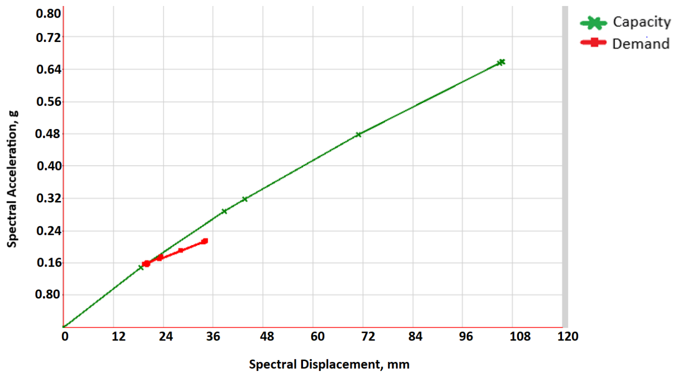

Figure 11: Pushover curve of Retrofitted Building in X-Direction

Similarly figure 12 shows the pushover curve in the Y direction in which displacement of 26.74mm was found and base shear of 6463.64 KN was found. Also all the assigned hinges are under the performance limit of life safety.Hinges of the ground floor column B2 and C3 are the first to cross the performance limit of life safety along loading direction (-Y)

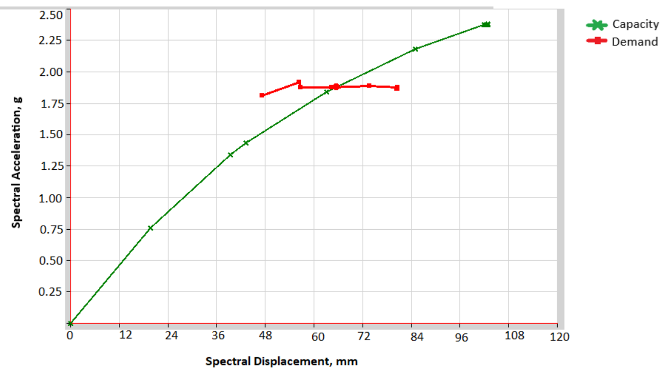

Figure 12: Pushover curve of Retrofitted Building in X-Direction

From eigen value analysis the time period of retrofitted building was found to be 0.3 seconds and the fundamental first mode of vibration of the building was changed from rotational first mode to translation 1st mode in X direction.

5. Conclusion

From result section the effect of retrofit is evident enhancing the performance of the building. Strength of the building has increased. Time period of the building was decreased considerably and there is slight increase in the deformation capability of the building. Furthermore detrimental rotational first mode of vibration was changed to translation mode after retrofitting.

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