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Exploratory Study of Comparison between Ultimate Strength and Performance Based Design for Reinforced Concrete Structures Subjected to Seismic Loading

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Abstract. Structural design of RC buildings has been going through a transition from allowable stress design approach (ASD) to the ultimate strength design approach (USD) leading to the present performance based design philosophy. PBD ensures that the members and structure as a whole reach a desired demand level and includes service, strength and capacity requirements. In the presented study a nonlinear analysis and design has been conducted on an intermediate rise RC building considering all seismic zones as specified by Unified Building Code in order to compare the USD and PBD approaches. Furthermore a comparison of different shear wall modeling techniques is presented along with modification to design methodology to make the structural design more economical. For the USD, UBC-97 and for PBD, ATC-40, FEMA-273 and FEMA-356 guidelines are used. The presented results reveal that the USD approach is unable to predict the soft story mechanism and shear failure governed the final failure in all seismic zones, whereas the flexural design was found to be adequate as inelastic deformations remained within the acceptable limits. However, capacity analyses revealed a non uniform damage throughout the building in contrast to PBD approach.

Keywords: Intermediate rise, Reinforced concrete structures, Performance based design, Non-linear analysis, Ultimate strength design.

1. Introduction

The kingdom of Saudi Arabia has been blessed with vast natural resources which have resulted in an immense construction boom in the past few decades. Owing to the construction boom a large number of reinforced concrete structures have been constructed, preferring them over the steel construction, as they are cheaper to construct, require less maintenance and can be constructed using locally available resources and unskilled manpower. Many of these reinforced concrete buildings are categorized as intermediate rise buildings. However in light of the rapid boom in construction industry many buildings in the kingdom have been constructed using the old design approach of the USD which lacks in providing real feedback to engineers and may suffer large damage in the event of dynamic loading such as earthquake, wind and blast loading. Furthermore the old design techniques are unrealistic as they lack to take into account the full potential of the material performance and lead to an uneconomical design. For the last several decades research has been going on in the field of structural design for reinforced concrete buildings. RC structures are particularly vulnerable to earthquakes. Earthquakes are a common phenomenon throughout the world and many of the world's most populated regions have been situated in areas of high seismic activities. Hence in order to better protect life and infrastructure governments all over the world have been encouraging research in the field of structural dynamics, structural retrofitting, strengthening and rehabilitation. Many new methods have been invented along with the refinements of old methods [1-6] which can be applied to the structure to make them safe against earthquake. Since earthquakes are unpredictable and random in nature, the engineering tools need to be developed for analyzing structural response under the action of such forces.

Performance based design has been gaining a new dimension in the seismic evaluation philosophy wherein the ground acceleration is considered. The approaches towards designing of structural members and in-particular the cross-sections have been changing over time in the past few decades. There has been an ongoing shift from allowable stress design approach (ASD) to ultimate strength design approach (USD) leading to the present strength and performance based design (PBD) in light of achieving economy by more realistic material and structural performance. The numerical and analytical methodologies used in each of these design approaches are independent of each other. This is necessary in certain aspects, but while considering the flexural design of reinforced concrete members, an integrated approach is needed that satisfies the requirements of serviceability, strength and performance. For lateral loads such as earthquake and high speed wind, performance is generally of main concern. Serviceability design ensures that deflections and vibrations for service loads are within limits. Although it checks the maximum stresses in the materials at design load levels but it is oblivious to the strength requirements and capacity ratio. The USD on the other hand ensures that a certain load factor against overload is available within a member but is silent on the topic of structural performance once the load exceeds the design level or in case the load is less than the assumed overload value. The PBD however, ensures that the member and structure as a whole reaches a desired demand level and included service, strength and capacity based design requirements.

Hence with the advent of new design approach such a PBD it is imperative that the performance evaluation of the old design method be conducted to evaluate its short coming and suggest modifications for the practicing engineers. In this respect the paper presents an exploratory study to evaluate to the performance of ultimate strength based designed structure using Performance Based Design, the moment-rotation relationship for plastic hinges adopted for the presented work is shown in Fig. (1). Rozman et. al [7] presented the research in which a three stores RC frame structure SPEAR is modeled as different variants to compare the seismic response of these variants. The first variant was considering the older building construction, only designed for vertical loading without considering the seismic loading while the 2nd and 3rd building models are designed according to Eurocode-8. They concluded that the structures modeled as per new standard EC8 code are safer than the old designed structures. Proper detailing of reinforcement which ensures suitable plastic mechanism results in greater global and local ductility of structures. Fahjan et. al [8] and Ioannis et. al [9] worked for modeling of shear walls in buildings for non-linear analysis and presented mid-pier approach and smeared approach for shear wall modeling. They showed that these shear wall modeling techniques are suitable in simulating the nonlinear behavior of shear walls. Chung and Kadid et. al [10,11] presented a technique for the modeling of the parameters of plastic hinge properties (PHP) for structure containing RC wall in the pushover analysis introduced by Anil et. al [12,13].

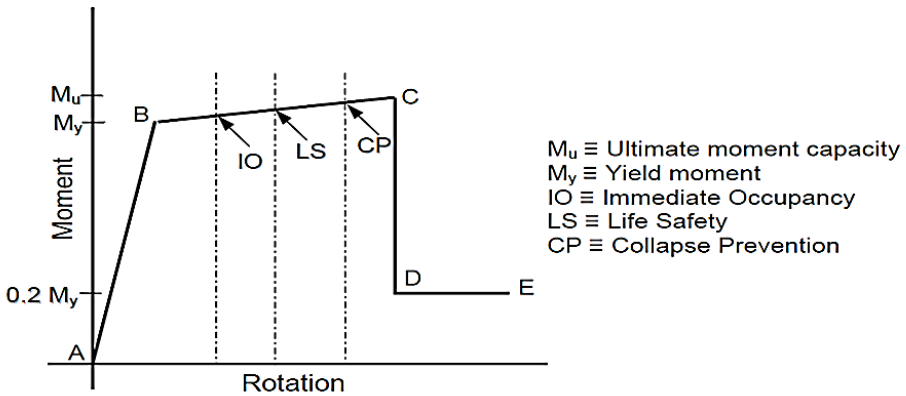


Fig. (1). Moment-rotation relation for plastic hinges

Response-2000 and Membrane-2000 codes were used to calculate the nonlinear relationship between the lateral shear force and lateral deformation of RC wall. The PHP values of each parameter were a product of two parameters α and β

for the pushover analysis in SAP2000. Experimentation was conducted to confirm the accuracy of this newly proposed method. It revealed that the method could be used professionally to help engineers to conduct the performance based design of structures containing RC shear wall using the SAP2000. Previous research work [14-18] has shown that shear failure might be the governing cause of failure for older designed structures which might prohibit them from achieving their serviceability objectives. In SAP2000 hinges can be assigned to frame elements at any location along its length. The idea of the presented work is to widen the application of this method to the RC structures containing shear wall. The advantage of modeling RC wall as a wide and flat column, frame elements, is that SAP2000 [20] can not only considers steel reinforcements exactly, but could also assign the hinges based on FEMA-356, hence the new modification could help professional engineers in conducting a more realistic and economic analysis.

2. Objectives

The presented research work aims to focus on comparing the design evaluation of USD approach and the PBD approach for intermediate rise reinforced concrete building subjected to seismic loading under all zoning categories as suggested in Unified Building Code, objectives of this research study are as explained below:

1.To compare the design framework of both the USD and PBD and to evaluate and compare the performance level of the structure after earthquake.

2.To modify the design methodology for seismic design based on the numerical results.

3.To highlight shortcomings in both of the design approaches.

3. Numerical Modeling Description

Fig. (2) presents the isometric view of the RC building selected for the proposed study. The building is a ten story structure which consists of moment resisting frame, shear walls at the four corners of the building and two lift wells. Fig. (3) depicts the typical plan of the building with the details of the moment resisting frame are mentioned in Table (1). The work was carried out in two stages. In stage-I, the analysis and design was carried out in accordance with UBC-97 [19] for all the zones. In stage-II, the performance of UBC-97 designed building was investigated in detail. The selected building has a combination of RC moment resisting frames and shear walls. Gravity loads were carried by moment resisting frame, whereas lateral loads were carried by the concrete shear wall and lift wells. Beams and columns were modeled as frame elements with the centerlines joined at the nodes. The load of slab was transferred to the beams depending upon the slab width support approach. Roof slab was 150 mm thick whereas the typical floor was 200 mm thick. For shear wall modeling, mid-pier approach was used. In mid-pier approach, shear wall were modeled as a frame element having the same cross-sectional stiffness properties as that of shear wall and the beams framing into shear wall were connected to mid-pier with the help of rigid beam offset having rigid zone factor equals to 1. It was a factor used to define the percentage of the zone specified through end offsets to be taken as fully rigid. Lateral load resistance was provided by concrete shear walls and lift-well walls in two perpendicular directions. According to their relative stiffness and rigidity, total base shear due to seismic load was resisted by shear walls and lift-well walls. L-shaped shear walls were present at the four corners of the building. Lift-wells were eccentrically situated with respect to the building geometric center. The ground floor columns were assumed to be fixed at the base. Building was analyzed and designed for all seismic zones of Pakistan as defined using UBC-97 i.e., zone-1, zone-2A, zone-2B, zone-3 and zone-4. The stiffness for cracking of the members was taken as per ACI 318-08 and was 0.7EIg for columns and shear walls (mid pier), 0.35EIg for beams and 0.25EIg for slab.

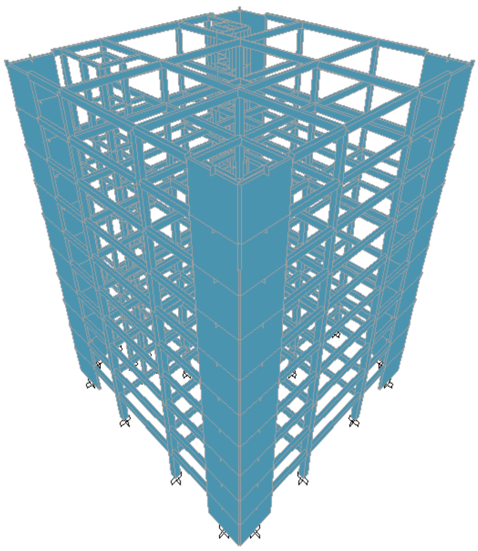


Fig. (2). Isometric view of the structure

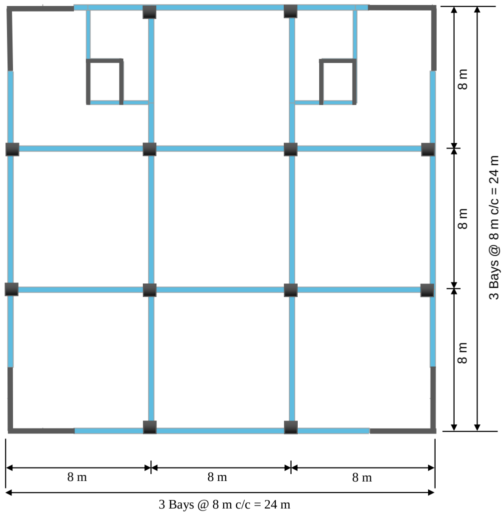


Fig. (3). Typical story plan of the structure

Table (1). Description of the moment-resisting frame

Sr. No.	Description	Values
1	Number of stories	10
2	Number of bays along X-direction	3
3	Number of bays along Y-direction	3
4	Story height	3m
5	Bay width along X-direction	8m
6	Bay width along Y-direction	8m

4. Load and Material Modeling Details

Table (2) provides the description of various types of loading and material inputs used for modeling the RC building. All the vertical loads due to structural and non-structural components of building e.g. self-weight, masonry walls, partitions, floors and roof finishes along with all other permanent construction were grouped as dead load. Loads due to occupants, moveable machineries and equipment, vehicular load and impact loadings were treated under the category of live loads. Zone factor, occupancy category, type of moment resisting frame, respective R-values and soil profile types were considered as seismic parameters. Reinforcement yield strength was considered as 414 MPa and concrete strength for different elements is provided as below;

Table (2). Loading and material input details

Sr. No.	**Description**	**Values**				
DEAD LOADS						
1	Finishes	1.44 kN/m^2				
2	Partitions	0.961 kN/m^2				
3	Roofing	0.96 kN/m^2				
4	Plaster	0.48 kN/m^2				
LIVE LOADS						
5	Typical Floor	4.8 kN/m^2				
6	Roof Floor	1.44 kN/m^2				
SEISMIC INPUT						
	Description	**zone-1**	**zone-2A**	**zone-2B**	**zone-3**	**zone-4**
7	Units	mm-KN	mm-KN	mm-KN	mm-KN	mm-KN
8	Direction	X & Y	X & Y	X & Y	X & Y	X & Y
9	Occupancy "*I*"	1	1	1	1	1
10	Moment Resisting Frame	OMRF	IMRF	IMRF	SMRF	SMRF
11	Peak Ground Acceleration "*g*"	0.05-0.08	0.08-0.16	0.16-0.24	0.24-0.32	> 0.32
12	R value	5.5	6.5	6.5	8.5	8.5
13	Soil Profile Type	S_D	S_D	S_D	S_D	S_D
14	Z	0.075	0.15	0.2	0.3	0.4
15	C_a	0.12	0.22	0.28	0.36	0.44
16	C_v	0.18	0.32	0.4	0.54	0.64
17	h_n	30 m	30 m	30 m	30 m	30 m
18	C_t	0.03	0.03	0.03	0.03	0.03
19	Eccentricity	5%	5%	5%	5%	5%
20	Eccentricity Overrides	NO	NO	NO	NO	NO
21	Period Calculation	Auto	Auto	Auto	Auto	Auto
22	Top Story	Roof	Roof	Roof	Roof	Roof
23	Bottom Story	Ground	Ground	Ground	Ground	Ground
MATERIAL INPUT						
24	Concrete Strength	*Columns*	*Shear Wall*	*Beams*		
		28 MPa	28 MPa	21 MPa		

Ultimate strength approach was based on UBC-97 for seismic design, whereas, for performance based approach, pushover analysis was used. To check the performance objective, Capacity Spectrum Method (CSM) as per ATC-40 was applied. For pushover analysis, concentrated plastic hinges as defined in FEMA-356 were used for beams, flexure (M_3) and shear (V_2) hinges, for columns and mid piers (shear walls), bi-axial bending (P-M_2-M_3) and shear (V_2 and V_3) hinges are employed. For performance objective, the zone-wise demand spectrum as per UBC-97 for S_D soil profile type was used in the analyses. Different computer models in software were generated for the study, while the seismic values were calculated as shown below in Table (3).

$$Ta = Ct\ hn^{3/4}$$

$$V = {}^{Cv\ I\ W}/_{R\ T}$$

If $T \leq 0.7$ sec, then $F_t = 0$

If $T > 0.7$ sec, then $F_t = 0.07\ T\ V$

Where,

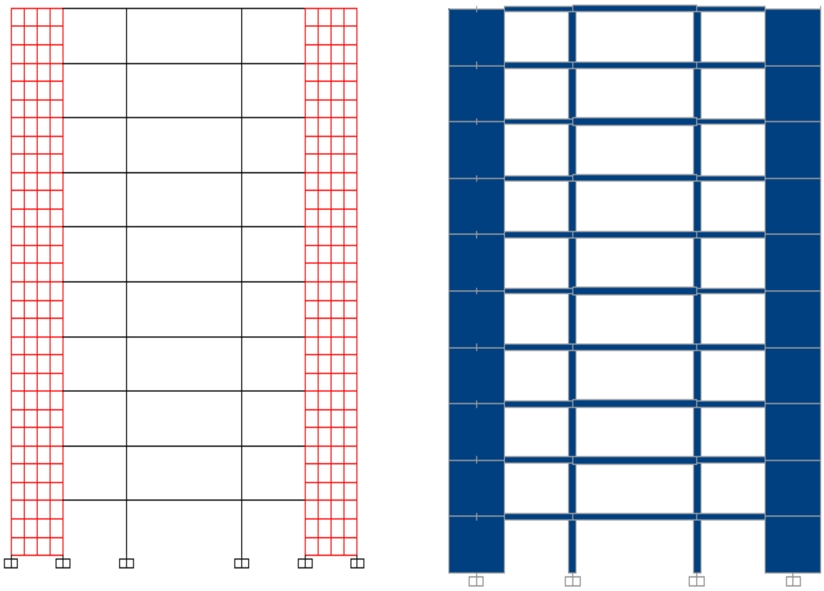
$F_t \leq 0.25\ V$

Table (3). Seismic Values for Model Input

Description	Zone-1	Zone -2A	Zone -2B	Zone -3	Zone -4
T_a	0.9375	0.9375	0.9375	0.9375	0.9375
T (sec)	1.3124	1.3124	1.3124	1.3124	1.2187
W (kN)	77562.439	79054.435	78405.268	78396.927	79488.159
V (kN)	1934.115	2965.404	3676.316	3794.852	4910.991
F_t (kN)	177.688	272.434	337.746	348.636	418.949

5. Shear wall Modeling Techniques

Fig. (4) shows the two techniques used for modeling of shear walls namely the multilayered shell element approach and the mid-pier approach. In multilayered shell element approach, shell elements were used in which non-linear smeared layers of concrete and steel were defined. Whereas, in mid pier modeling approach, frame elements were used to model shear walls having same stiffness as that of shear wall. Rigid beam offsets having rigidity equals to 1 were defined to connect mid pier with the beams framing into wall. From the plastic hinge distribution results presented in Fig. (5); it is evident that in mid pier modeling approach, the hinge formed at the base at 243 mm top displacement. Similarly, from the concrete and steel stress distribution in as shown in Fig. (6), in multilayered shell element modeling, steel stresses at the bottom of shear wall crossed the yield value at 273 mm top displacement which indicated that the hinge is formed at the base of the shear wall. Hence, both the approaches were in close agreement and were reasonable for simulating non-linear behavior of shear walls with mid-pier approach resulting in a more conservative approach. However in the presented work the mid pier modeling technique was applied in modeling for its conservative approach and since FEMA-356 guidelines were used for predicting non-linear behavior of frame elements.



Multilayered Shell Element Approach **Mid-Pier Approach**

Fig. (4). Shear wall modeling techniques

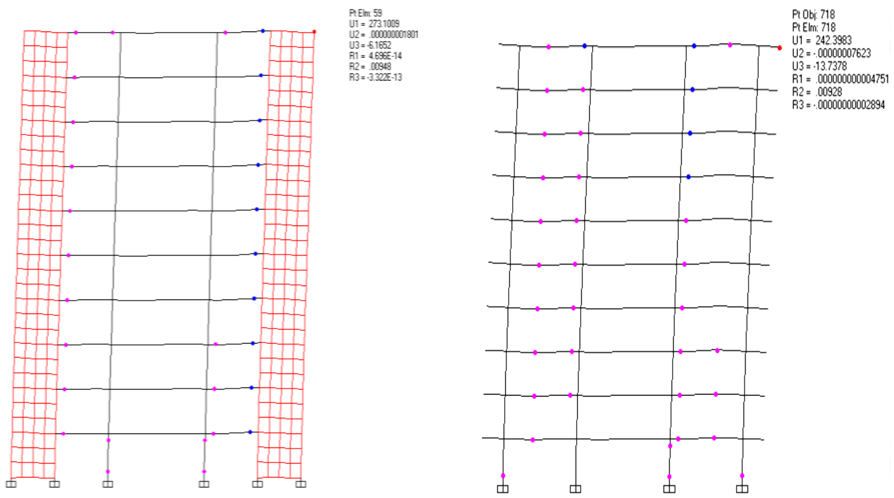


Fig. (5). Plastic hinge distribution

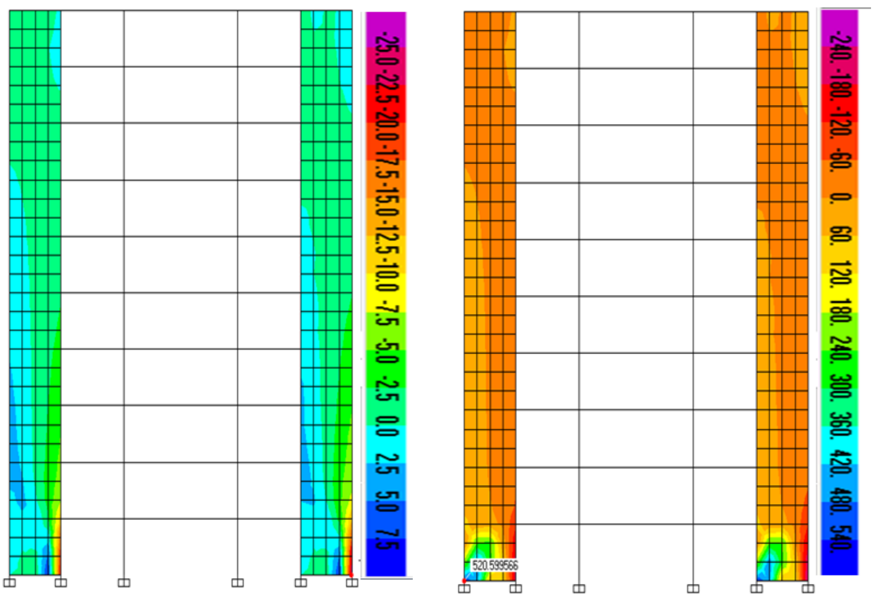


Fig. (6). Concrete and steel stress distribution

6. Performance Evaluation and Discussion

6.1 Zone-1

The building was designed as per UBC-97 in zone-1 and then pushed by applying mode-1 based on lateral load distribution; displacements were monitored accordingly in global X-direction. For nonlinearity, M_3 hinges were assigned to beams, P-M_2-M_3, V_2 and V_3 hinges were assigned to columns and shear walls. Capacity curves for building designed for zone-1 is shown in Fig. (7). Firstly the columns and shear walls of the building were modeled with bi-axial flexure and shear hinges, in this case shear failure starts first. It can be seen from Fig. (7); that the hinges could withstand 5000 KN of base shear at the top displacement of 545mm. Then, the columns and shear walls of the building were modeled with only bi-axial flexure hinges. After the structure was analyzed it was seen that it could withstand more than 6000 KN of base shear at the top displacement of 725mm. The nominal base shear calculated based on UBC-97 equivalent lateral load method was 1935 KN. The capacity curve indicated that the ultimate design capacity was over designed and the building could resist more than 6000 KN of base shear without any excessive damage. It was evident from the analysis that due to bi-axial flexure hinges only, no excessive damage occurred in the shear walls and columns while some flexure hinges in the beams reached their collapse stage. Damage degree due to the presence of shear hinges in the model showed that excessive damage had occurred in shear walls and columns at the 5th story level. Flexure hinges in beams remained within the acceptable limit of immediate occupancy (IO). Failure was concentrated in columns and shear-walls at 5th story and showed the extent of soft story mechanism. Furthermore, it was evaluated that at the performance point no excessive damage and yielding occurred in the building.

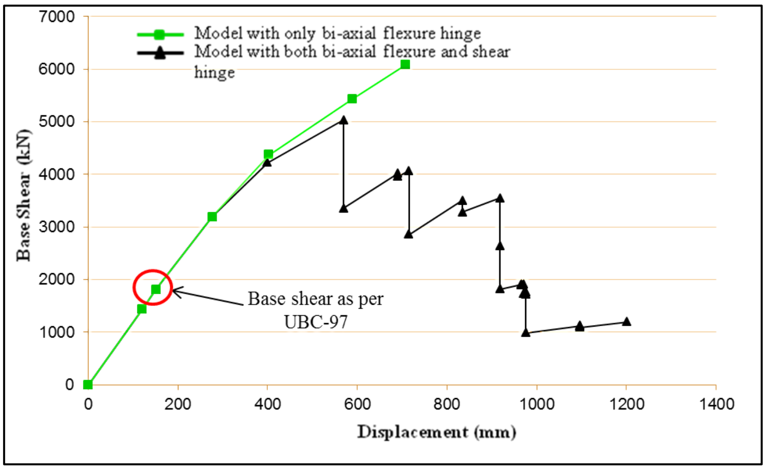


Fig. (7). Capacity curve for zone-1

6.2 Zone-2A

For analysis in zone-2A the building was designed as per UBC-97 requirements and then pushed by applying mode-1 lateral load. For non-linearity, M_3 hinges were assigned to beams, P-M_2-M_3 and V_2 or V_3 hinges were assigned to columns and shear walls. Capacity curve for the building is shown in Fig. (8). Firstly the columns and shear walls of the building were modeled with bi-axial flexure and shear hinges, this resulted in shear hinge failure. It was seen that it could sustain 5900 KN of base shear at the top displacement of 580 mm. Then, the columns and shear walls of the building were modeled with only bi-axial flexure hinges. It was seen that the building could sustain more than 7500 KN of base shear at the top displacement of 800 mm. The nominal base shear calculated by UBC-97 equivalent lateral load distribution method was 2965 KN which indicated that the lateral load capacity of the structure was much greater than the required value and the building could resist more than 7500 KN of base shear without any excessive damage. It was evident from the result evaluation that no excessive damage occurred in shear walls and columns in case of only bi-axial flexure hinge. Some flexure hinges in beams reached the collapse stage. However damage degree due to both bi-axial flexure and shear hinges showed that excessive damage had occurred in mid piers and columns at the 4th story level. Failure was concentrated in columns and shear-walls at 4th story and showed the extent of soft story mechanism. Furthermore at the performance point, no excessive damage and yielding has appeared in the structure and the building was well within the serviceable range.

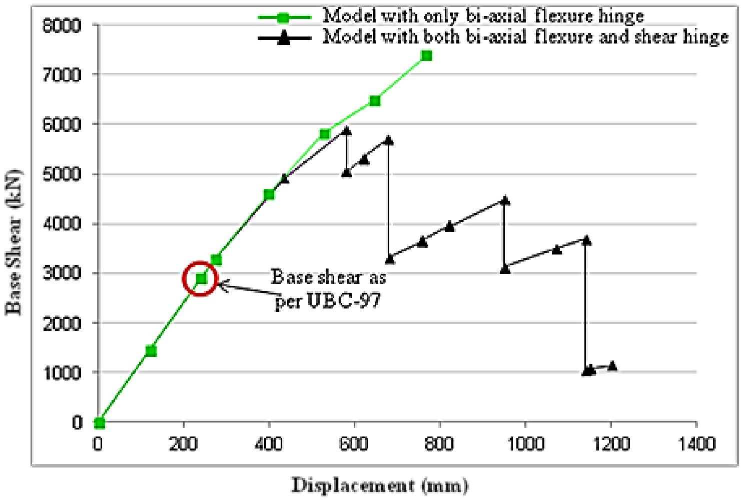


Fig. (8). Capacity curve for zone-2A

6.3 Zone-2B

The building was designed and analyzed using the input of zone-2B as described in Table (2). The non-linearity of the model was considered. Firstly the columns and shear walls of the building were modeled with bi-axial flexure and shear hinges in which case the shear hinge triggers failure first and the capacity curve is shown in Fig. (9). It can be seen that the structure was able to withstand 7400 KN of base shear at the top displacement of 600mm. Then, the columns and shear wall of the building were modeled with only bi-axial flexure hinges and it was seen that the capacity increased up to 9000 KN of base shear at the top displacement of 950 mm. The nominal base shear calculated by UBC-97 equivalent lateral load distribution was 3675 KN and which indicated that the ultimate design capacity was large. It was evident that due to only bi-axial flexure hinge, no excessive damage occurred. Damage degree due to the presence of shear hinges in the model showed that excessive damage occurred in the shear walls and columns of the 6th story level. Flexure hinges in beams remained within the acceptable limit. Failure was concentrated in columns and mid piers at 6th story and at the performance point no excessive damage or yielding appeared in the building.

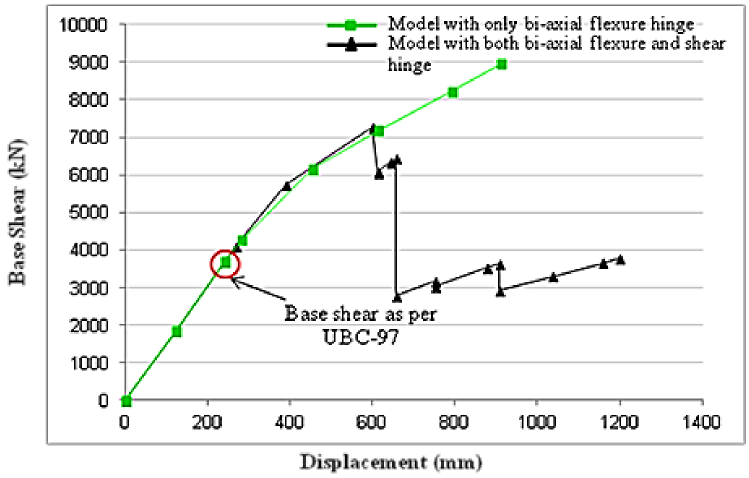


Fig. (9). Capacity curve for zone-2B

6.4 Zone-3

Capacity curves for building designed for zone-3 is shown in Fig. (10). As the columns and shear walls of the building were modeled with bi-axial flexure and shear hinges as before the shear hinge triggered failure first at the base shear level of 6900 KN and the top displacement of 500 mm. Afterwards, the columns and shear-walls of the building were modeled with only bi-axial flexure hinges and It could withstand base shear upto7775 KN at the top displacement of 650mm. The nominal base shear calculated by UBC-97 equivalent lateral load distribution was 3795 KN which was much lower than the performance level achieved by the building. Some flexure hinges in beams reached the life safety stage and the damage degree due to the presence of shear hinges in the model showed that excessive damage occurred in shear walls and columns at the 6th story level. Flexure hinges in beams remains within the acceptable limit of life safety (LS) while the failure was concentrated in columns and shear walls at 6th story and lead to a soft story mechanism, see Fig. (11). However, Fig. (12) showed that at the performance point, no excessive damage or yielding appeared in the building.

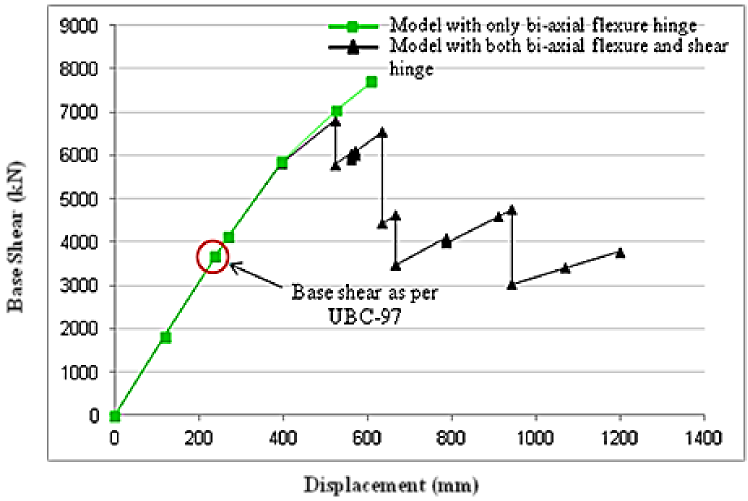


Fig. (10). Capacity curve for zone-3

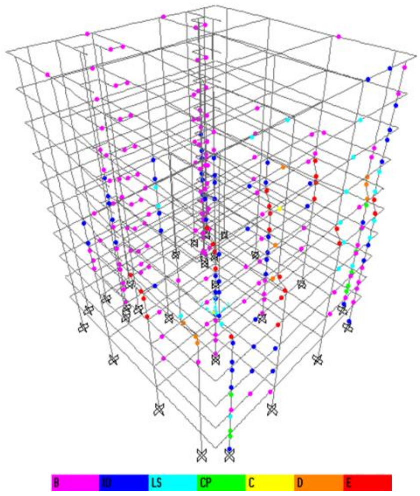


Fig. (11). Damage degree for flexure and shear hinges (P-M_2-M_3, M_2, M_3, V_2 and V_3)

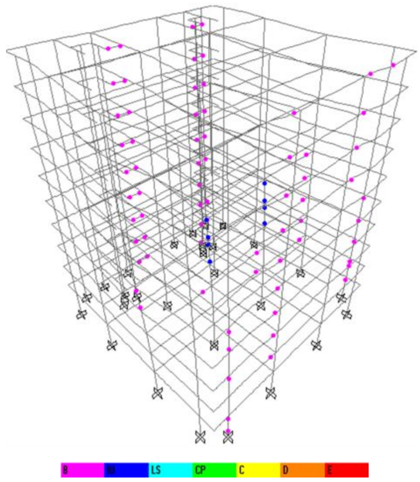


Fig. (12). Damage degree at performance point zone-3

6.5 Zone-4

Zone-4 was the most sever zone with regards to the performance requirements. Hence in order to ascertain accuracy the building was modeled with and without slab. For non-linearity in beams M_3 hinges were assigned, P-M_2-M_3, V_2 and V_3 hinges were assigned to columns and shear walls. Firstly the columns and shear walls of the building were modeled with bi-axial flexure and shear hinges, the capacity curve is as shown in Fig. (13). It can be concluded that the structure could withstand 5900 KN of base shear at the top displacement of 490 mm. Then, the columns and shear walls of the building were modeled with only bi-axial flexure hinges. It resulted in more than 12000 KN of base shear at the top displacement of 1700 mm. The nominal base shear calculated by UBC-97 using the equivalent lateral load distribution method was 4915 KN which indicates that the lateral load capacity of the building is additional. It was evident from the Fig. (13); that slab played its role as a rigid diaphragm, when the building was modeled with slab as shell elements it adds stiffness to the building and better capacity curve was achieved with and without shear hinges in columns and shear walls. It was evident from the Fig. (14); that due to only bi-axial flexure hinge, no excessive damage occurred. However, most of the beams of the exterior frame may have reached the collapse stage and showed that excessive damage occurred in shear walls and columns at the 5th story level. Flexure hinges in beams remained within the acceptable limit and failure was concentrated in columns and shear walls at 5th story. Fig. (15) showed that excessive damage or yielding due to shear failure had appeared in the building even at the performance point.

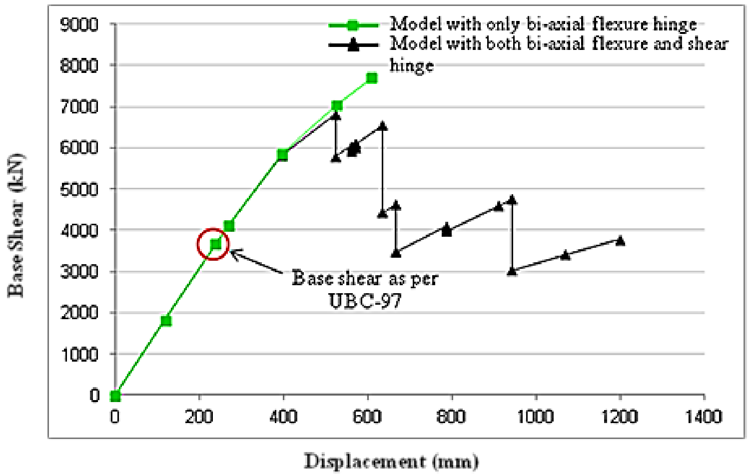


Fig. (13). Capacity curve for zone-3

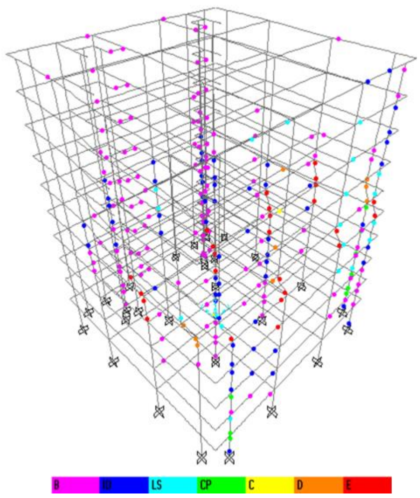


Fig. (14). Damage degree for flexure and shear hinges (P-M_2-M_3, M_2, M_3, V_2 and V_3)

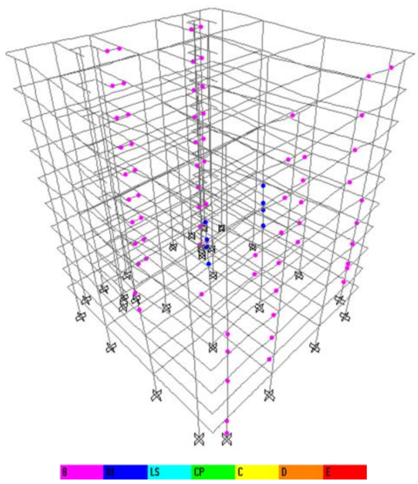


Fig. (15). Damage degree at performance point zone-4

7. Design Methodology Modification

The building designed in zone-4 showed excessive damage degree owing to shear failure. Hence it was decided to carry out a detailed performance based design to ascertain the performance of building against realistic demand. Therefore, the building was re-analyzed and re-designed by selecting the performance objective of Life Safety (LS). Performance based design facilitates the engineers in deciding the performance objective and demand for the building. The members which cross the life safety performance limit were revised by increasing the stiffness. This process was continued until the desired performance objective was achieved. From the capacity curve shown in Fig. (16), it is evident that by revising the cross-sectional properties and corresponding design of the members crossing the Life Safety performance objective, better performance of the building was achieved for the same demand. Before deciding the higher performance objective, the building could resist 5900 KN of base shear with top displacement of 490 mm. While, after revising the performance objective of LS, the building could resist 7400 KN of base shear with top displacement of 800 mm. By comparing the ATC-40 capacity spectrum before and after deciding the performance objective as shown in Fig. (17) and Fig. (18), it was observed that the performance point after revising the stiffness of the members improved from base shear of 3197.845 KN and displacement of 466.806 mm to base shear of 6654.527 KN and displacement of 598.238 mm. Revised analysis and design of the building in zone-4 revealed that performance based design is the more suitable design methodology in high seismic regions as ultimate strength design remained ambiguous about the performance of discrete elements of the building when the base shear calculated as per UBC-97 equivalent lateral load distribution was exceeded.

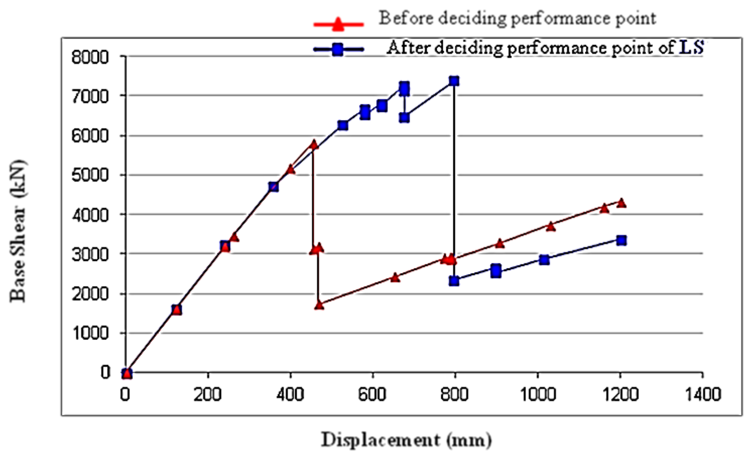


Fig. (16). Comparison of capacity curve in zone-4

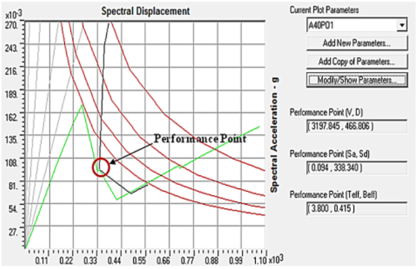


Fig. (17). ATC-40 Capacity spectrum before deciding the performance objective

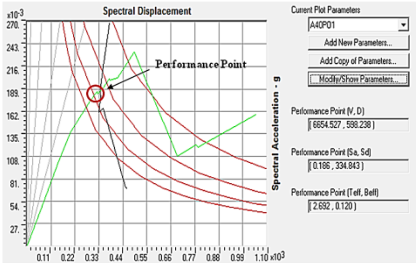


Fig. (18). ATC-40 Capacity spectrum after deciding the performance objective of LS

8. Conclusions

An exploratory study regarding the design approaches of USD and PBD is presented for a ten story reinforced concrete building analyzed under various seismic zones as defined by UBC-97. From the detailed analysis, design and assessment of the building the following conclusions can be drawn;

1. It is evident from the presented results that for intermediate rise buildings designed using USD; shear failure governs the final failure mode in all seismic zones.

2. Lowest performance point occurred in zone-4 owing to the higher performance requirements. Hence the structures designed should be made more ductile and inelastic deformations should be allowed to occur at critical sections to dissipate energy.

3. USD based design was found to be adequate as inelastic deformations remained within the acceptable limits, however the approach lead to an uneconomical design since the realistic material capacity were not considered.

4. It was evident from the damage degree observation that the building designed using USD showed soft story failure while the building designed using PBD showed uniform damage throughout the building. Hence the soft story failure mechanism can be avoided by adopting PBD approach.

5. Comparison of mid pier and multilayered shear wall approach reveals that in mid pier approach, plastic hinge was formed at the base of mid pier at the displacement of 242 mm. Similarly in multilayered shell modeling, steel crosses yield stress at the base at the displacement of 273 mm. Hence it can be stated that these techniques are in reasonable agreement, while mid pier approach being more conservative, these approaches can be used for simulating nonlinear response of shear walls.

6. In mid pier approach, rigid zone factor should be selected with much care as it effects the moment redistribution in frame elements. The rigid beam offset with rigidity equals to 1 effect the moment distribution within a member up to 3 to 5%.

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تصميم دراسة استكشافية للمقارنة بين القوة القصوى والأداء على اساس التصميم لتعزيز هياكل الخرسانة التي تتعرض لأحمال الزلزال

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ملخص البحث التصميم الإنشائي للمباني مازال مستمر من خلال الانتقال من أسلوب تصميم الإجهاد المسموح به (ASD) إلى اسلوب تصميم القوة القصوى في نهاية المطاف (USD) مما يؤدي إلى فلسفة التصميم القائمة على الأداء الحالي .الأداء على اساس التصميم (PBD) يضمن ان القوام والهياكل الإنشائية تصل إلى المستوى المطلوب وتشمل متطلبات الخدمة، والقوة والقدرة. في الدراسة التي قدمت تم إجراء التصميم و التحليل الغير خطي على ارتفاع متوسط للمبنى الإنشائي مع الاعتبار في جميع المناطق الزلزالية على النحو الذي يحدده قانون البناء الموحد من أجل مقارنة تصميم القوى القصوى (USD) والأداء على اساس التصميم (PBD) وعلاوة على ذلك يتم تقديم مقارنة في جدار القص بين مختلف التقنيات جنبا إلى جنب مع بعض التعديلات لتصميم المنهجية لجعل التصميم الإنشائي أكثر اقتصادا. USD، UBC-97 و PBD، ATC-40، FEMA-273 و FEMA-356 تستخدم كمبادئ توجيهية. تظهر النتائج المقدمة أن منهجية تصميم القوى القصوى (USD) غير قادر على التنبؤ على آلية الليونة وانهيار القص الذي يحكم الانهيار النهائي في جميع المناطق الزلزالية، في حين تم العثور على تصميم الانحناءات لتكون كافية حيث ظلت التشوهات الغير مرنة ضمن الحدود المقبولة. ومع ذلك، كشفت التحليلات قدرة الضرر الغير موحدة في جميع أنحاء المبنى على النقيض من نهج الأداء على اساس التصميم(PBD)