Performance Based Seismic Design

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***Abstract*— Presented in this paper is Performance based Seismic design method. In which plastic design methodology is performed for the building under altered earthquake ground motions. The Performance based design (PBD) method is a recent evolving perfect method for the future of seismic design. PBD is a direct design method in this method nonlinear analysis is more concerned in determining the damage that may cause to the structure, the performance objects are pre- computed and the members of the frame is detailed to achieve the intended yield mechanism and the structure that have to behave in the required limit during earthquake seismic events.**

***Keywords*— Performance Based Seismic Design, Nonlinear Analysis, Performance Objectives, Design basis earthquake, Maximum Considered earthquake.**

I. Introduction

It is observed that building structures during severe seismic ground motions experience huge cyclic deformations. To produce sound design, it is significant to accurately apply the design forces to get the required member sizes, at this point this would limit the post yield deformation within acceptable limit, and this procedure would be time consuming and also highly complex for the multi-story structures because the multi-story structures in the post-yield state have complex seismic response properties. The type of yielding mechanism and higher modes of the structure play important role in the post-yield response of the building. Especially in the inelastic stage different structures have different yield mechanisms and modes of vibration, without understanding this factors, the design will be un-conservative which would lead to unacceptable damage level.

The performance-based seismic design has two primary goals: appropriately calculating the uncertainties associated with the performance assessment process and satisfactorily distinguishing the associated structural damage that may occur, to evaluate in the design. The purpose of a performance-based seismic design methodology is to counter to a variety of demands of the clients or the stake holders, where various levels of performance objectives and design target are clearly defined and termed. In addition, in the practical design procedure, design parameters have to be clearly identified and their criteria to realize each performance level or objective have to be worked out and should be fulfilled to meet the seismic demands. The PBSD will act as a useful tool to determine design performance in each specific project, a more reliable design methodology can be recognized to realize included performance requirements in buildings taking the concept of risk management into account.

The PBSD is the methodology to respond to a variety of seismic demands, where various levels of performance objectives and design target are clearly defined and described, and their criteria to realize each performance level or objective have to be worked out and satisfied to meet the seismic demands. Simply stating, that the goal of performance based design might be, “that when the building is subjected to varying levels of hazards (earthquake or wind loads), the building performs in such a way that, agreed acceptance criteria is satisfied”. When dealing with earthquake we might have, Service level earthquake (SLE) or DBE-Design basis earthquake and MCE-Maximum considered earthquake.

SLE/DBE might have 50% probability of occurrence in 30 years, MCE may have 2% of occurrence in 50 years which is rare to occur. For low level earthquake like SLE/DBE we might assume the building remains essentially elastic, i.e. no significant yielding of structural elements will be allowed, and this analysis may typically be done using linear elastic response spectrum. However for larger earthquake (MCE’s) there will be significant nonlinear behavior, certain structural elements will yield, to maintain adequate ductility to such an extent, that the building will not lose gravity load carrying capacity for experiencing excessive drift or instability issues.

II. Objective And Methodology

A. Objective

Our objective is, to define performance objectives, corresponding design criteria and methodology and work out for the targeted performance of structure to meet the seismic demands by comparing to past earthquake ground motions recorded, i.e. for Design basis earthquake the building should remain essentially elastic, i.e. no significant yielding of structural elements will be allowed and for Maximum considered earthquake certain structural elements can yield, but adequate ductility should be maintained to such an extent, that the building will not lose gravity load carrying capacity for experiencing excessive drift or instability issues.

B. Methodology

1) Perform initial analysis design to size the appropriate members.
2) Identify Deformation controlled components that should be detailed for nonlinear behaviour.

3) Perform an analysis using a lower level earthquake like SLE/DBE to make sure that the building remains primarily elastic.
4) Perform MCE Non-linear Time-History analysis using expected strengths, to determine post yield behaviour of components.
5) Verify results meet the appropriate acceptance criteria.

C. Performance Objective

In the following graphical representation, the relationship between performance objectives, earthquake probability occurrence, and the facility type is developed and then the response parameters are related to each performance objective as shown in fig 1. These parameters are then recognized and then the evaluations for the required performance level for the type of structure are computed as shown in the following Table I.

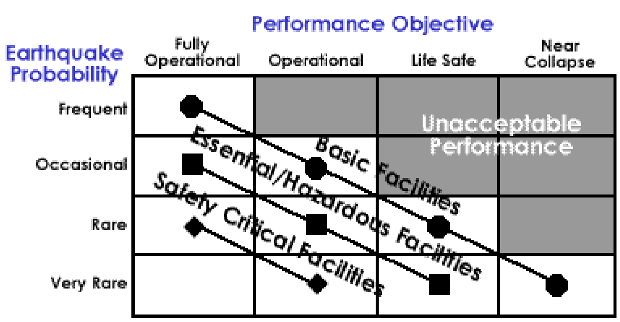

Fig1 Relation between Performance objective and probability of earthquake

Table I Performance objective for the probability of earthquake.

	EQ Level		
Performance Level	**DBE**	**MCE**	**Applied use example**
No Damage (CP limit)	Fully operational	Fully operational	Atomic power station, etc.
Slight Damage (CP-IO limit)	Fully operational	Operational	Disaster prevention centre, central hospital
Small-Scale Damage (CP-IO limit)	Fully operational	Life Safe	Ordinary hospital, refuge facility, head office, etc.
Serious Damage (IO-LS limit)	Operational	Near Collapse	Ordinary building

Table II Performance levels of the structure

Performance Objective	**Definition**
Fully operational	Continuous service. Negligible structural and non-structural damage.
Operational	Most operations and functions can resume immediately. Structure safe for occupancy. Essential operations protected, non-essential operations disrupted. Repair required to restore some non-essential services. Damage is light.
Life Safe	Damage is moderate, but structure remains stable. Selected building systems, features, or contents may be protected from damage. Life safety is generally protected. Building may be evacuated following earthquake. Repair may be economically impractical.
Near Collapse	Damage severe, but structural collapse prevented. Non-structural elements may fall. Repair generally not possible.

D. Acceptance Criteria

1) Drift: According to Clause 7.11.1 of IS 1893(Part 1), The Storey Drift is limited to 0.004 times the storey height. Chapter 16 ASCE/SEI 7-16 limits drift to twice the standard design limits of ASCE 7-16 of Table 12.12-1. For this structure (Risk Category II non-masonry building taller than four stories), this implies a mean story drift limit of 4%.

Table III Drift Acceptance Criteria

	DBE	**MCE**
Story Drift	1.5%	Mean value shall not exceed 3%. Maximum drift shall not exceed 4%.

2) Beam Hinge Status: The beam elements for any structure irrespective if type of importance of the building, during DBE should remain primarily elastic and fully operational (IO-intermediate occupancy).

The behavior of the structure during a MCE depends on the owners or clients requirements i.e. at what limit a structure can respond for a maximum considered earthquake. In case of important structures some of the beam elements for MCE can be allowed to yield and should be near collapse limit (CP-collapse prevention) or it can also be fully operational depending on the owner's choice or the importance of structure.

III. EXPERIMENTAL ANALYSIS DETAIL

A. Analysis and Modeling Approach

The experimental model has a 7 story of 30.8m (101.049 ft.) height, with 8-meter bays in both X-Y axis, X-axis has 7 bays and Y-axis has 5 bays, first story is 6.8m high and all the above six stories are 4m high. The analysis was performed in three dimensions using ETABS 2015. The Elastic Lateral Force (ELF) method is a necessary and primary part of the analysis and design process, it is to be carried out initially and appropriate member sizes has to be decided. Further the Non-Linear Response History (NLRH) analysis is carried out. The results from the analysis procedures are then computed to satisfy the objectives and the acceptance criteria. The Building Seismic Design Information is shown in the below table IV.

Table IV Building Seismic Design Information

Code:	ASCE/SEI 7-10
Occupancy Category:	II
Earthquake Design Data of location Downtown Stockton, California,	
Seismic Importance Factor: I	1.0
Mapped Spectral Response:	S_s : 1.041 g S_1 : 0.373 g
Site Class:	C
Site Class Coefficients:	F_a : 1.0 g F_v : 1.427 g
Spectral Response Coefficients:	S_{ds} : 1x (1.041) x (2/3) = 0.694 g S_{d1} : 1.427x (0.373) x (2/3)= 0.3548 g
Modification Coefficient, *R*:	8.0
Over strength Factor, Ω_0:	3.0
Deflection Amplification Factor, C_d:	5.5
Seismic Response Coefficient, C_s:	0.0305
Design Base Shear, *V*:	420588.5 kips

B. Primary Design by Elf Method

With the ELF method using the forces shown in Table V, initial member sizes confirming to strength requirements were designed as shown in Figs 2

Table V Summary of Equivalent Lateral Seismic Design Forces

Story Level	**h_x(ft.)**	**w_x(kips)**	**$h_x w_x^k$**	**C_{vx}**	**F_x(kips)**	**V_x(kips)**
7	101.0	1892736	$4.4E^{+11}$	0.2260	95061	95061
6	87.92	1925862	$4.0E^{+11}$	0.2019	84951	180013
5	74.80	1957976	$3.4E^{+11}$	0.1762	74132	254145
4	61.67	1929254	$2.8E^{+11}$	0.1420	59754	313899
3	48.55	1951407	$2.2E^{+11}$	0.1138	47873	361772
2	35.43	1966675	$1.6E^{+11}$	0.0840	35355	397127
1	22.30	2035027	$1.1E^{+11}$	0.0557	23460	420588
SUM=	-	13789788	$1.9E^{+12}$	-	-	-

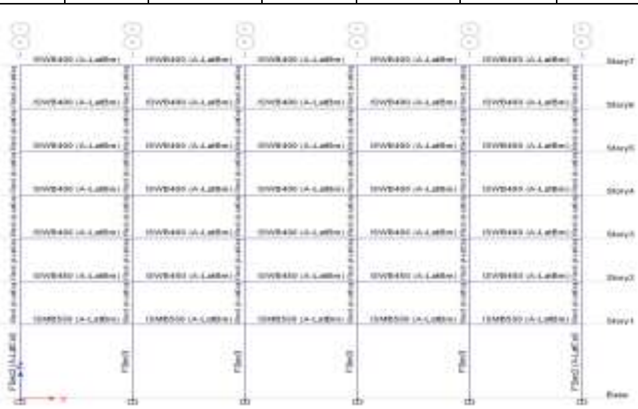

Fig 2 Member size of the frame A in Y-axis

C. Nonlinear Analysis of Building

The nonlinear analysis is modeled using ETABS, Nonlinear Time History Modal (FNA). Computers and Structures a program that is used for nonlinear analysis in earthquake engineering practice.

For measuring post-earthquake functionality of the structure the demand parameters used are story drifts and story drift ratios which provides more trustworthy measure for accessing the performance and behavior of the buildings components. Also the local demand parameters is used in accessing the performance of structural components by classifying them as force controlled and deformation-controlled components. In this model, Plastic hinge rotations in beams and columns leading to significant strength/stiffness degradation are modeled as deformation based on the consequence of failure of the components (from Section C 16.4.2.2 of ASCE 7-16) and Axial compression forces in columns due to combined gravity and overturning forces are modeled as Critical force-controlled components (based on section C 16.4.2.1 of ASCE/SEI 7-16).

1) Ground Motions: For Design Based Earthquakes (DBE's) Ground motions are developed as per ASCE/SEI 7-10 for the site, downtown Stockton, California, from ground motion parameters- S_s and S_D mentioned in the above table 5. The design response spectrum is shown in the fig 3, below.

For Maximum Considered Earthquake (MCE's) Ground motions are developed based on the risk-targeted Maximum Considered Earthquake (MCE). The target response spectrum was developed as per ASCE/SEI 7-10. Horizontal ground motion LACC NOR-1 and LACC NOR-2 shown in fig 4 were selected from ETABS and applied in the orthogonal directions x-y respectively.

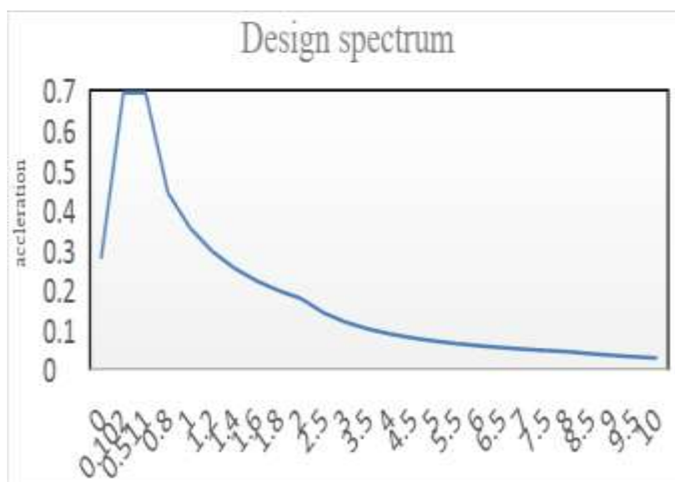

Fig 3 Design response spectrum

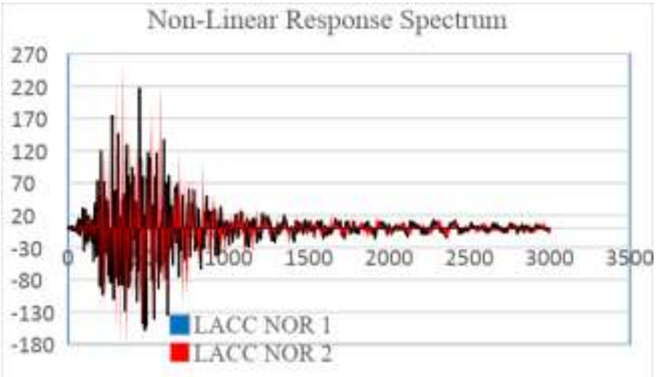

Fig 4 Non-Linear response spectrum

IV. RESULTS

A. *Story Drift*

Story drifts are a common fundamental acceptance criterion used in many standards and guidelines. The most common story drift measurement is a simple calculation of the relative displacement between two stories, normalized by the story height (also called story drift ratio)

I. *1) Elemental Level Storey Drift:*

Table VI Elemental Level Storey Drift

Story	Load Case/Combo	Label	Max Drift X	Label	Max Drift Y	limit (0.004xH)
Story7	Nonln-TH-Resp Max	1	0.00158	129	0.000404	0.016
Story6	Nonln-TH-Resp Max	43	0.000529	130	0.000107	0.016
Story5	Nonln-TH-Resp Max	132	0.000432	132	0.000112	0.016
Story4	Nonln-TH-Resp Max	131	0.000367	129	9.70E-05	0.016
Story3	Nonln-TH-Resp Max	129	0.000323	132	0.000103	0.016
Story2	Nonln-TH-Resp Max	1	0.000712	129	0.000216	0.016
Story1	Nonln-TH-Resp Max	43	0.000855	42	0.000223	0.000272

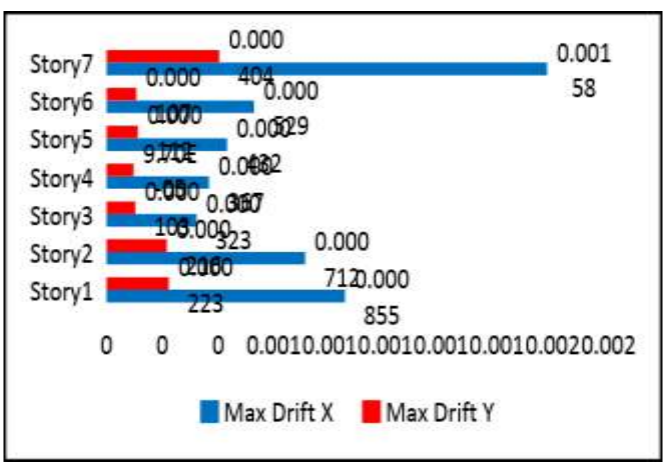

Fig 5 Graphical representation of Elemental Level Story Drift

II. *2) Diaphragm Storey Drift:*

Table VII Diaphragm Storey Drift

	Elevation (m)	Location	X-Dir	Y-Dir	limit (0.004xH)
Story7	30.8	Top	0.00158	0.00040	0.016
Story6	26.8	Top	0.00053	0.00007	0.016
Story5	22.8	Top	0.00043	0.00011	0.016
Story4	18.8	Top	0.00037	0.00010	0.016
Story3	14.8	Top	0.00007	0.00007	0.016
Story2	10.8	Top	0.00071	0.00022	0.016
Story1	6.8	Top	0.00040	0.00009	0.0272

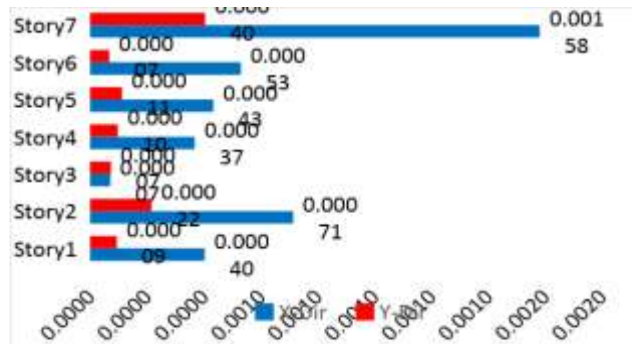

Fig6 Graphical representation of Diaphragm Story Drift

B. *Beam Hinge Results*

1) *Beam Hinge Results on Application of DBE*

It can be seen that for lower level earthquake Ground motions like DBE the building remains primarily elastic, i.e. the hinges in the all beams are in the criteria of Intermediate occupancy level as reported in the Table VIII below of Beam hinge results for DBE (only some hinge results of story7 are shown in table below).

Table VIII DBE Beam Hinge Results

Story	Frame/ Wall	Load Case	Hin-ge	Generated Hinge	M3 kN-m	Hinge Status
7	B1	Nonln-THX DBE Max	M3	B1H1	49.1	A to IO
7	B7	Nonln-THX DBE Max	M3	B7H1	36.9	A to IO
7	B8	Nonln-THX DBE Max	M3	B8H1	48.9	A to IO
7	B9	Nonln-THX DBE Max	M3	B9H1	99.7	A to IO
7	B10	Nonln-THX DBE Max	M3	B10H1	102	A to IO
7	B11	Nonln-THX DBE Max	M3	B11H1	107	A to IO
7	B12	Nonln-THX DBE Max	M3	B12H1	99.9	A to IO
7	B13	Nonln-THX DBE Max	M3	B13H1	90.0	A to IO
7	B14	Nonln-THX DBE Max	M3	B14H1	34.5	A to IO

2) *Beam Hinge Results On Application Of MCE*

It can be seen that for Maximum Considered Earthquake (MCE's) Ground motions like LACC NOR-1 and LACC NOR-2 some of the elements of the building remains elastic and the other elements starts yielding, i.e. some of the hinges status of the beams are in the criteria of Intermediate occupancy level and other hinge status of the beams have reached to the Collapse Prevention criteria as reported in the Table IX below for Beam hinge results for MCE (only some hinge results of story7 are shown in table below).

Table IX MCE Beam Hinge Results

Sto-ry	Fra me/ Wal l	Load Case	Hinge		M3	Hinge Status
			Assi gned	Gener ated	kN-m	
7	B1	Nonln-THX MCE MAX	M3	B1H1	0	LS- CP
7	B7	Nonln-THX MCE MAX	M3	B7H1	1059	A-IO
7	B8	Nonln-THX MCE MAX	M3	B8H1	0	LS- CP
7	B9	Nonln-THX MCE MAX	M3	B9H1	0	LS- CP
7	B10	Nonln-THX MCE MAX	M3	B10H1	0	LS- CP
7	B11	Nonln-THX MCE MAX	M3	B11H1	0	LS- CP
7	B12	Nonln-THX MCE MAX	M3	B12H1	108	LS- CP
7	B13	Nonln-THX MCE MAX	M3	B13H1	367	LS- CP
7	B14	Nonln-THX MCE MAX	M3	B14H1	727	LS- CP

V. CONCLUSIONS

In the present study, based on the experience in performing the advanced type of seismic analysis (PBSD), various conclusions can be arrived. The conclusions have been divided into subdivisions according to the challenges faced in studying and performing the project.

A. Modeling and Analysis Conclusions

1) Many structural analysis programs are already available and are presently in use by practicing professionals but only some of the subset programs are suitable for nonlinear response history analysis design tasks.

2)Performing nonlinear time history analysis by practicing professional by Recognizing challenges is a hypothesis as many of the natural parameters (Expected reoccurrence MCE's, subsurface conditions, Features of ground motion, nearby source effects, regional tectonic setting, geology and seismicity) regional tectonic setting, geology and seismicity) are to be considered which cannot be accounted much accurately

3)For a project complexity for a nonlinear model is high and an experienced professional is needed to understand and classify the parameters to be included to get maximum accurate response of a structure during an earthquake for a particular location of ground motions.

4) For the immediately conceivable future of performance based seismic design still more reasonable, less complex and simplified technique for nonlinear analysis is required.

5) At present there are no proper procedures and standard are available for selection of ground records, still research groups are working to produce suitable ground motion accelerations records.

B. Analysis Techniques Conclusions

The analysis results confirmed that the building performs within expectations and meets the ASCE/SEI 7-10 requirements for response under DBE and MCEs, and the following are some specific observations regarding the analyses

1) The objective to achieve i.e. The building should remain primarily elastic in DBE and some elements of the frame can yield during MCE keeping into account the life safety is satisfied.

2) Also the fundamental acceptance criteria i.e. story drift was obtained to be in expectations and less than the 4% limit specified by ASCE/SEI 7-10.

3) The local demands on deformation controlled components (RBS hinge rotations, column base rotations, and panel zone shear deformations) are not satisfied and are to be analyzed in different software with further research.

Finally, it should be underscored that these observations are based on the one of the example of a simple frame building and that the observations and conclusions are not meant to generally apply to other buildings. Indeed, the intent of these project is to implement the use of nonlinear analysis as much as possible to a highest accuracy with the available research papers and guidelines.

C. Assumptions and Limitations

1) In our study of PBSD which is advanced evolving analysis, it was tried to get the various parameters into the prescribed limit, but some of the important parameters like panel zone shear deformations, beam column base connections, has to be considered and analyzed for practical models.

2) This study has presented analysis results (drifts, beam hinge level status) which are limited conclusion parameters presented from these analysis.

3) Also force controlled and deformation controlled components should be properly classified in the model and the analysis has to be performed for the accuracy.

4) In case of practical purpose the chosen method (PBSD)

for assessing the accuracy of the procedure studied in this project should not be decided only on the explored results, but comparison of explored results should be performed. For ex, in ASCE 41-06 the modification factors used for linear analysis often infer altered allowable ductility demands than do the nonlinear component deformation limits for the same performance level. This should be taken into consideration in research in future.

VI. FUTURE SCOPE

The Performance based design method as the name itself describe the type of method, in this type of method the various performance objective to be performed by the structure, during seismic event has be defined earlier by the requirements of the owner for the structure for which a performance menu is prepared for each specific building, based on the menu the performance level is to be satisfied, and various parameters as studied in this paper and also other associated parameters should be studied with accuracy for each different type of building which may vary for the particular type of building, using the further updated codes like ASCE 7-16 and also the specified limits according to the code should be satisfied.

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