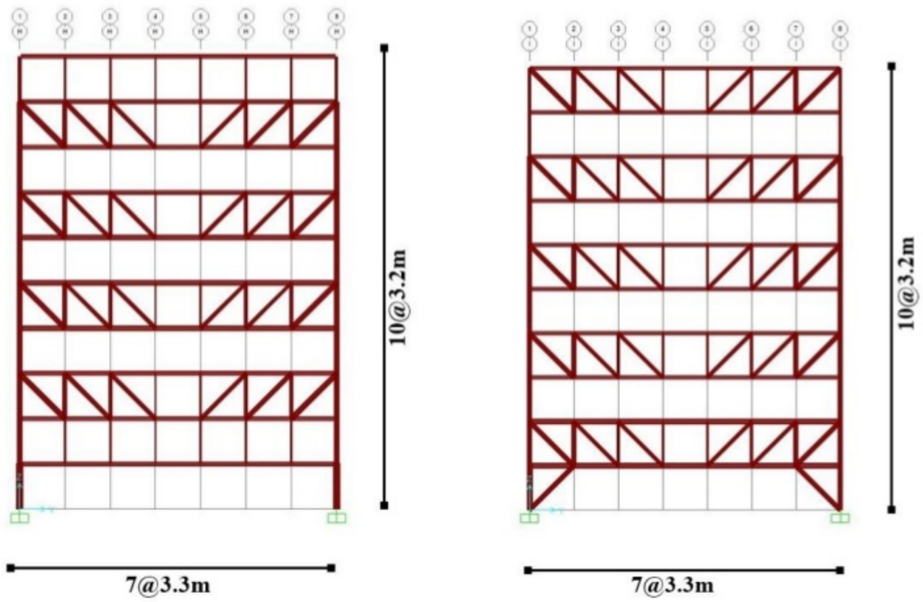


Figure 4-12: Staggered Truss Fames (STS) in row (a) and row (b)

Chapter 3 of this study described this model completely. Actually, the AISC LRFD [1] regulations and load combinations for gravity and lateral loadings were applied to design the structure.

The plastic hinge definitions are the most important aspects of a nonlinear analysis. Chapter 3 mentions all plastic hinge calculations for members of columns, truss, and braces. In SAP2000, one of the characteristics of the behavior of plastic hinge is a piece-wise linear relationship between plastic rotation and moment [24]. Diagrams of a beam and a truss member for plastic hinge are shown in Figures 4.2-(a) and 4.2- (b). As can be seen, in a beam, the actual path follows path of skeleton.

Unlike the beams of the columns, the actual path usually does not follow the skeleton path. Deduction of this could be due to application of the normal force on the relationship between the moment and plastic rotation.

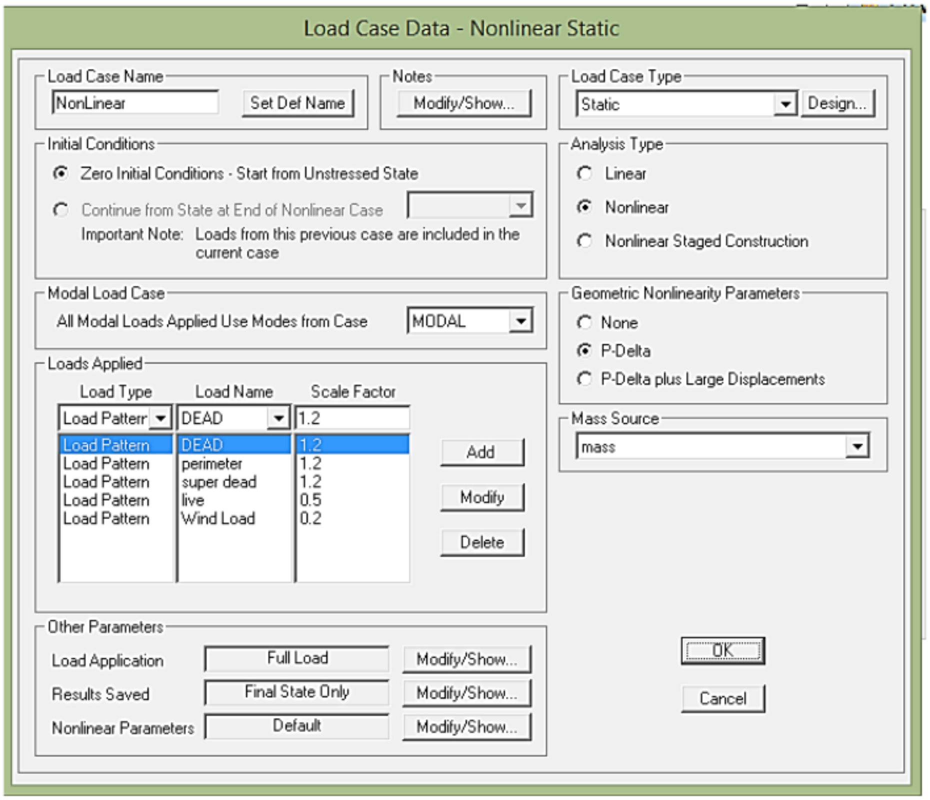


Figure 4-13: Using Parameters of Equation 4.2 for Nonlinear Static Load Case.

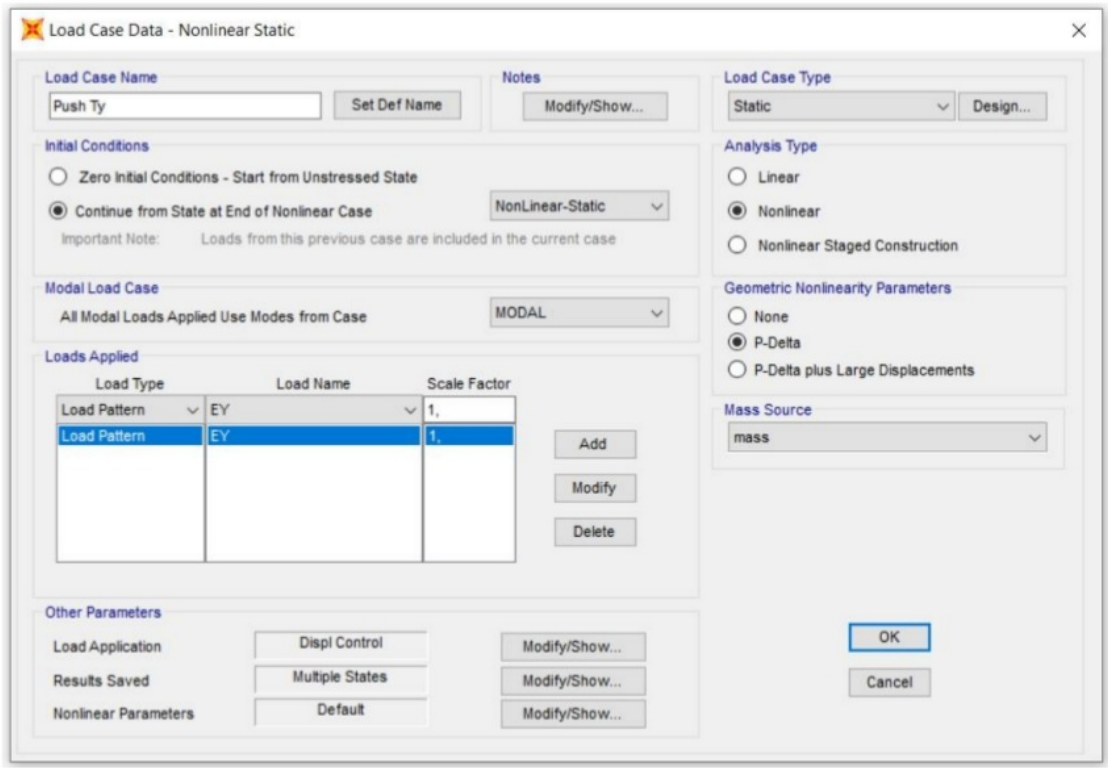


Figure 4-5: Pushover Analysis parameters in SAP 2000.

4.1 Criteria for Acceptance of Structural Steel

The nonlinear acceptance criteria of Life Safety condition are used from ASCE 41 as shown in tables of Chapter 5 for the secondary and initial components. These components are considered as the components controlled by deformation. On the contrary, in case of larger $\mathbf{P/P_{CL}}$ ratio than 0.5 for the columns, they should be regarded as the components controlled by force in SAP2000. In this case, high axial load influences the column. If the $\mathbf{P/P_{CL} \leq 0.5}$, there should be equation of interaction in which the moment is regarded as the moment controlled by deformation and the behavior controlled by force is axial load. Table 4.1 show these nonlinear criteria , from Tables 5-5 and 5-6 and 5-7 of ASCE 41 [2].

Connection Type	Parameters of Nonlinear Modeling (1)			Nonlinear Criteria for Acceptance	
	Angle of Plastic Rotation, radians		Residual Strength Ratio	Angle of Plastic Rotation, radians	
	a	b	c	Initial(2)	Secondary(2)
Fully Restrain Moment					
Improved WUF with Bolted Web	0.021 - 0.0003d	0.050 - 0.0006d	0.2	0.021 - 0.0003d	0.050 - 0.0006d
Reduced Beam Section (RBS)	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.050 - 0.0003d	0.070 - 0.0003d
WUF	0.0284 - 0.0004d	0.043 - 0.0006d	0.2	0.0284 - 0.0004d	0.043 - 0.0006d
Side Plate®	0.089 - 0.0005d(3)	0.169 - 0.0001d	0.6	0.089 - 0.0005d	0.169 - 0.0001d
Moment Connections Partially Restrained (Relatively Stiff)					
Double Split Tee					
a. Bolt Shear	0.036	0.048	0.2	0.03	0.040

b. Bolt Tension	0.016	0.024	0.8	0.013	0.020
c. Tee Tension	0.012	0.018	0.8	0.010	0.015
d. Tee Flexure	0.042	0.084	0.2	0.035	0.070
Simple Connections Partially Restrained (Flexible)					
Double Angles					
a. Bolt Shear	0.0502 - 0.0015d_{bg}[4]	0.072 - 0.0022d_{bg}	0.2	0.0502 - 0.0015d_{bg}	0.0503 - 0.0011d_{bg}
b. Bolt Tension	0.0502 - 0.0015d_{bg}	0.072 - 0.0022d_{bg}	0.2	0.0502 - 0.0015d_{bg}	0.0503 - 0.0011d_{bg}
c. Angles Flexure	0.1125 - 0.0027d_{bg}	0.150 - 0.0036d_{bg}	0.4	0.1125 - 0.0027d_{bg}	0.150 - 0.0036d_{bg}
Simple Shear Tab	0.0502 - 0.0015d_{bg}	0.1125 -	0.2	0.0502 - 0.0015d_{bg}	0.1125 - 0.0027d_{bg}

Table 4-1: Parameters of Modeling and Criteria for Acceptance of Nonlinear Modeling of Steel Frame Connections [12].

4.2 NON-LINEAR STATIC ANALYSIS

Non-linear static analysis also called analysis of pushover has been expanded in the past years and has been used as an effective analysis method for design and evaluation of performance. Due to the relatively simple procedure, there are simplifications and approximations for which variation is always needed for evaluation of the seismic demand. The reliability and accuracy for prediction of the seismic and global demands for all structures have been discussed though analysis of pushover has been found to get main characteristics of structural response. In order to remove the specified limitations. The pushover methods which have been improved have been suggested. However, such procedures are impractically used in engineering codes and profession due to the computationally demanding and conceptually complex procedures which have been improved. The pushover analysis aims to assess the required performance of a structural system through estimation of its strength and demands for deformation in design earthquakes based on the static inelastic analysis, and to comparing these demands with the existing capacities. To predict the deformation demands and seismic force approximate representing internal forces redistribution in case of the structure being subjected to inertia forces not resistible within the structural behavior elastic range, Pushover analysis is used.

The seismic demands are computed in the latest NEHRP guidelines [52] using the non-linear static analysis of the structure which is subjected to lateral forces which are monotonically increasing with a non-varying height-wise distribution until reaching a target displacement. Both the target displacement and the force distribution are based on the assumption that the fundamental mode controls the response and that the mode shape will not change. After the structure yields, both assumptions are approximate.

The non-linear structural analysis method has gone one step further once the FEMA-356 [57], FEMA-440 [58] and FEMA-273 [52] documents have been recently published, with many suggestions for the acceptable values of deformation and force parameters for evaluation of performance.

4.2.1 Non-linear Static Analysis Process

The structure two- or three -dimensional mathematical model with relationship between deformation and load and for all members is first made and gravity loads are first applied. Then, a pattern of lateral load distributed with the building height is applied. The pattern of lateral load is selected in this specific study as the first mode shape of the structure. There will be step by step increase of the lateral forces until yield of member(occurrence of plastic hinge). Then, the model is changed for modification of yielded member stiffness and there is an increase in the lateral forces until the yielding of more members.

The process will continue until the control displacement gets specified level or the structure is regarded as an unstable mechanism.

In this specific research, the full capacity of the system is investigated based on the final state of the analysis. On the contrary, the target displacement keeps at most 3% of the total height which is regarded as the final performance point sometimes to prevent further abnormal results from occurring.

The displacement plot versus the base shear yields the structure's global capacity curve. This study monitors displacement as the mean value of the roof nodes displacement.

4.2.2 Relationships of Force Deformation

Proper modeling of the force deformation relations for all members is one of the main steps of the non-linear analysis. Concentric plastic hinges which are assigned to the selected sites along members of the frame represent this basic relation. As the highest probability is that there will be yielding at both ends of the members under lateral loads, the plastic hinges will be assigned to those sites. Yielding behavior and behavior after yielding can be modeled as a curve of moment-41 rotation for flexural yielding which is applied for the members of beam, as an axial deformation -axial force curve for members of brace and also as a 3D axial interaction between force and bending moment for members of column members.

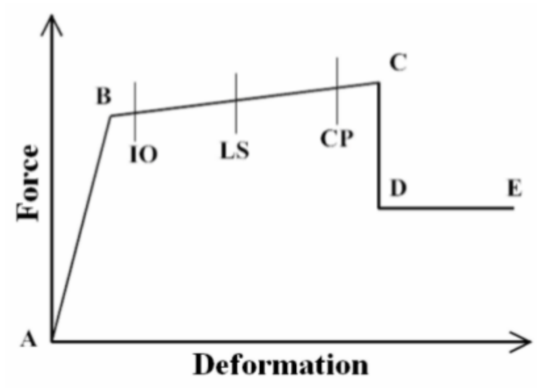


Figure 3.5: Component Force-Deformation Curve

Figure 3.5 shows a behavior curve of generic component. The software vendor [69] is used to express the points which are marked on the curve as follows:

• The origin is point A.

• yielding is represented as point B. there is no deformation in the hinge to point B not considering the value of deformation determined for point B. The rotation or deformation at point B will be deducted from the deformations at points D, C, and E. The plastic deformation will be exhibited from point B hinge.

• The final capacity of the analysis of pushover is represented as point C but a positive slope from C to D can be determined for other purposes.

• A residual strength of the analysis of pushover is represented as point D but a positive slope from D to E or C to D can be determined for other purposes.

• Total failure is represented as point E. The hinge will drop load down to point F (not shown) out of point E which is below point E directly along the horizontal axis.

If it is not desirable for the hinge to fail in this way, there will be a deformation large value at point E to be specified.

The additional measures of deformation can be specified at points LS i.e. life safety, IO i.e. immediate occupancy, and CP i.e. prevention of collapse. These are regarded as the informational measures found in results of the analysis and applied for design based on performance. They are effective in the structure behavior.

The whole deformation is linear before reaching point B , occuring in the element of Frame and does not occur in the hinge. There is plastic deformation out of point B of the hinge plus any elastic deformation occurring inside the element. When there is elastic unlading of the hinge, there will be no plastic deformation meaning that it is similar to slope A-B.

Figure 3.6 shows the force - deformation curve of member in FEMA-356 [57] for the structural modeling and it is suggested for the parameters such as a, b and c as shown in in the appendix of Table A.4.1. The rows which are highlighted in Tables determine the parameters selected for modeling of the curve of behavior in this specific study.

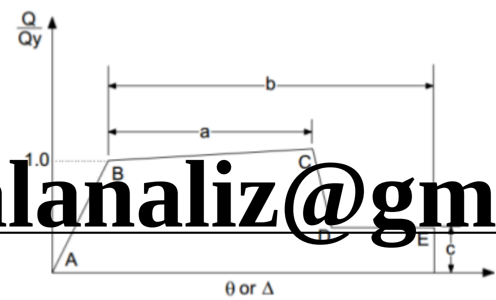


Figure 3.6: Component Force-Deformation Curve as given in FEMA-356 [57]

The FEMA-356 [57] contain the following equations for the yield rotation and yield moment of steel beams and columns calculation.

$$\text{Beams:} \quad \theta_y = \frac{Z \cdot F_{ye} \cdot l_b}{6EI_b} \qquad M_y = Z \cdot F_{ye} \tag{3.6}$$

$$\text{Columns:} \quad \theta_y = \frac{Z \cdot F_{ye} \cdot l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \qquad M_y = 1.18 \cdot Z \cdot F_{ye}\left(1 - \frac{P}{P_{ye}}\right) \le Z \cdot F_{ye} \tag{3.7}$$

where, θ_y : Yield Rotation

M_y : Yield Moment

F_{ye} : Yield strength of steel

Z : Plastic section modulus

l_b : Beam length

l_c : Column length

E : Modulus of elasticity

I_b : Moment of inertia of beam with respect to the bending axis

I_c : Moment of inertia of Column with respect to the bending axis

P : Axial force in the member

P_{ye}: Expected axial yield force of the member ($A_g F_{ye}$)

The curves of force deformation which are idealized are taken from the "backbone curves" also fundamentally taken from experimental data on a member hysteretic behavior. Figures 3.7 and 3.8 schematically represent how to acquire data on a plastic hinge model. Figure 3.8 shows the classification of the behavior as semi-ductile, ductile, and brittle where the performance after yielding appears to determine the characteristics.

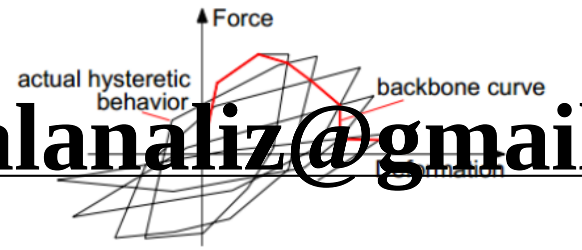


Figure 3.7: Actual Hysteretic Behavior and its Backbone

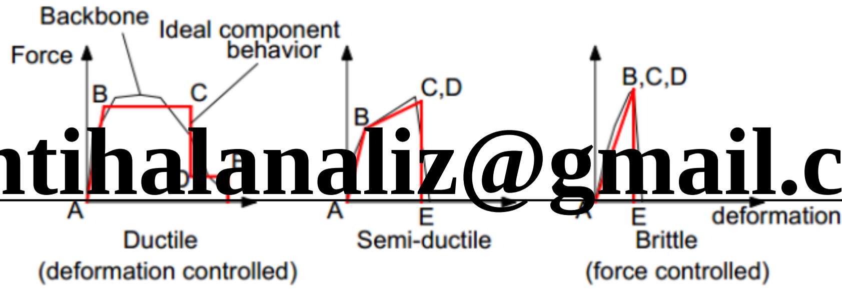


Figure 3.8: Backbone Curves Further Idealized as Component Behavior Curves

As shown in in Figure 3.8, a typicalductile behavior is related to the first curve. Its characteristic is an elastic range i.e. point A to point B on the curve, before plastic range i.e. points B to E including strain softening or hardening at points B to C , and a strength-degraded range from points C to D, with residual force resisting significantly below than the final strength, but it is still considerable.

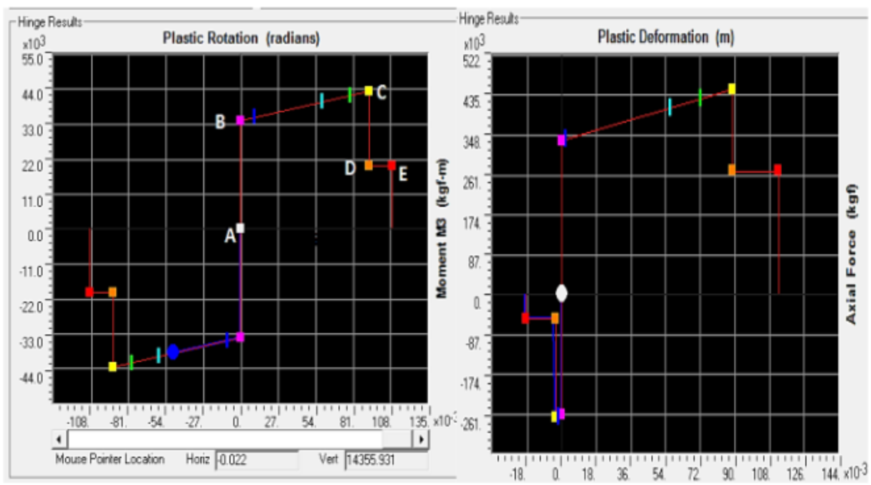
Actions of the component which show this behavior are controlled by deformation. The second curve represents another ductile behavior type. Its characteristics are a plastic range and an elasticrange before a complete and quick loss of strength. The sufficiently large plastic range i.e. 2 times as high as the elastic deformation range will lead to categorization of this behavior as the behavior controlled by deformation. In another case, it will be categorized as the behavior controlled by force. As shown in Figure 3.8, the third curve shows non-ductile or brittle behavior. Its characteristic is just anelastic range before a complete strength loss. Components which show this behavior are always categorized as the behavior controlled by force.

Members of Columns and Beams

Seismic resistance of the steel moment frames is developed through steel columns and beams bending and also the connections between beam and column resisting against moment. Connections of this frame are designed for increase of the moment resistance in the column and beam joint. For this purpose, the behavior of the frames resisting against steel moment is generally dependent on configuration and detailing of the connection. Behavior of the column and beam members as DESCRIBED shown in Chapter 3 was classified by the FEMA-356 [57] along the flange and web slenderness limits. As shown in this specific research, the W sections which have been selected for columns and beam are in the flange and web slenderness limits as Tables 3.4 and 3.5 show and discuss.

To see the perpendicular loads, the model will be first tested and based on the related codes, the model against earthquake loads will be designed. In this study, as has been described in Chapter 3, earthquake designing was done using the UBC 97. The model in this case will be similar to a true conventional building. Sudden removal scenarios for the component can be performed either with a nonlinear or a linear procedure after the model was designed. There is an essential difference between these two methods in consideration of the geometrical nonlinearities and inelastic behavior in nonlinear dynamic method . Thus, the concrete and steel nonlinear specifications such as yield stress and ultimate stress of steel section should be defined for modeling of the prototype with SAP2000. Tensile stress $F_u = 448.15$ N/mm and yield stress of steel sections $F_y = 344.7$ N/mm were used in this study. Chapter 3 describes all data of concrete and steel stress strain.

The plastic hinge definitions are the most important aspect of a nonlinear analysis. Chapter 3 shows all calculations of the plastic hinge for truss members, columns, and braces. In SAP2000, a piece-wise linear relationship between plastic rotation and moment defines the plastic hinge behavior[24]. Figures 4.2-(a) and 4.2- (b) show plastic hinge diagrams of a truss member and a beam . As can be seen, the skeleton path is followed by the actual path in a beam. on the contrary to the beams of the columns, the skeleton path is not usually followed by the actual path. The reason for such deduction could be the normal force which is applied on the relationship between moment and plastic rotation.



(a) Diagram of Beam plastic hinge (b) Diagram of plastic hinge of truss member

Figure 4-14: Definition of Plastic hinge in a truss and a beam member.

Point B in Figure 4.2 (a) is showing the moment of yield and point C shows the final moment point and the related plastic rotation. This relationship between moment and plastic rotation for columns is also dependent on the normal force and there could be activation of this interaction in SAP2000 [24].

4.3 P- Δ Effects

To analyze the structures, both geometric and material nonlinearity should be considered. A geometric nonlinearity was the P- Δ Effect.

P- Δ analysis is included in SAP2000 by selecting the P- Δ from *Geometric Nonlinearity Parameters* section on page of Time History load definition .

4.3.1 Finite Element Computer Model Building

Chapter 3 of this study completely described this model. Actually the design of structure is done in accordance with the load combinations for lateral loadings and gravity and regulations of AISC LRFD [1]. There was optimal design of all primary structural sections which included columns, beams, truss members and braces with P-M ratios near 0.95. The model used in this approach is a 3, 6, and 10 story STS building of which 2 Moment Frames were located in the structure left and right sides (X direction) and 8 Staggered Truss Systems (STS) in transverse (Y) direction. Figure 4.3, 4.4, 4.5 show details of the architectural geometry of the model; Chapter 3 of this context gives additional information.

Idealization of Capacity Curve

The initial information for assessing the structures response modification factor is represented by the capacity curve but it should be first idealized for extraction of the related information from the plot.

The aim is to get the over strength factor and factor of reduction of the ductility through investigation of the pushover curve. For this purpose, a bi-linear curve is incorporated into the curve of capacity in which the first segment begins with the origin, passes across, intersecting with the second segment at the significant yielding point and then the second segment which has started with the intersected point is finished at the final point of displacement. Individual changes in the plot increments slopes are traced to find the first segment slope. All increments mean slope is determined in each step and then compared with the second one by finding the dramatic change. The first segment which is related to the elastic part is determined with the successive parts mean slope in the curve until occurrence of the considerable change. the significant yield point is acquired to plot the second segment which is the post-elastic part through the concept of equal energy where there is equality between the area under the bi-linear curve and the area under the curve of capacity. the abovementioned methodology is utilized to develop an AutoLISP program in order to plot and read the pushover data and then fit the bi-linear curve. An optimized version of this one as suggested by FEMA 273 [52] is this technique, giving the visual process of trial and error and suggesting intersection of the first segment with the original curve in 60% of the meaningful strength of yielding. On the contrary, intersection of the cure in the investigated plots in 60% of the meaningful strength of yielding does not create the boundary condition because the condition is satisfied by all curves based on their almost direct elastic parts. Improvement of this method will lead to no necessity of the visual trial and error, however, the obtained bi-linear curves are investigated to see the false interpretation. Figure 3.26 shows general description of the bi-linear approximation.

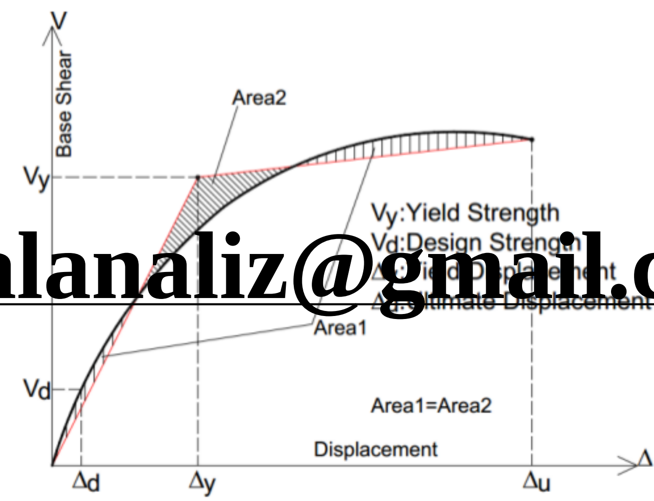


Figure 3.26: Bi-linear Idealization of a Generic Capacity Curve

the essential components will be provided by the bi-linear idealization. These main componnets are the displacement of significant yield, significant strength of yield and also the strength of design and the ultimate displacement which have been detremined before. The overstrength factor viewed generally in Section 2.2 can be detremined can be calculated easily using the these available data, as the yield strength to the design strength ratio.

In addition, ratio of ductility can be detremined as final displacement to yield displacement ratio as the main element incalculation of the factor of ductility reduction based on Eq. 2.3.32 as suggested by Miranda [16].

	3 Story		6 Story		10 Story	
R	3,5249	**R**	3,2554	**R**	**2,9671**	
λ	9,375	**λ**	5,3571	**λ**	**6,0267**	
Rλ	1,8799	**Rλ**	2,025	**Rλ**	**1,8544**	
Rs	1,875	**Rs**	1,6071	**Rs**	**1,6**	
Story	R	λ	Rλ	Rs	TX	TY
3	3,5249	9,375	1,8799	1,875	0,78979	0,48693
6	3,2554	5,3571	2,025	1,6071	1,74238	0,8154
10	2,9671	6,0267	1,8544	1,6	3,14	1,35197

5. Discussion and Conclusion

In this chapter, we reviewed the staggered truss system and related work.

Initially, staggered truss systems were generally described, and a brief history of the system was presented. Subsequently, different parts of the system, such as floors, columns, and trusses, were introduced separately.

This chapter following sections reviewed the evaluation of codes related to the relevant regulations. Then seismic designs and works in this field are discussed. The AISC 14 and ASCE 41 standards are then described, and their essential properties are reviewed, respectively.

We then discussed the design considerations and experiments performed on this system. In the next section, we discussed STS numerical models, including Pushover and

Incremental Dynamic Analysis. In general, this chapter introduces concepts related to staggered trusses and reviewed the related works.

In this Section, results of the evaluations and analysis in this study are summarized. There will be increasing trend of the factor of ductility reduction (Rμ) when it increases from 3 stories to 6 stories and to 10 stories before the slight increase or decrease or constant plateau. For all systems of framing, the overstrength factor (Rs) will have significant reducing trend with elongated periods to be traced. On the contrary, there is no significant decrease from 6 to 9 stories as there will be no significant decrease from 3 to 6 stories. The modification factor of overall response resulting from the sub-factors mentioned above tends to be reduced with elongated period. This is reliable for all systems with no exception.

In case of comparing the performance of connection, we can say that modeling of panel zone reduces the lateral load which carries the frames capacity by increasing the structural period and this results in R factors values which are slightly lower.

The related R values will get the same condition. This study shows that "R" factors general results which are acquired for most systems are significant higher than those of the code values. This result may be found for the following reasons:

- The frames which are designed with high ductility should satisfy the condition of the columns which are more powerful than the beams. This will result in high overdesign ratios because the gravity loads govern the design of beam. Thus, there is significantly unused capacity of the lateral loads in the column with high plastic modules than the beams. This is found in the design phase as sections of column which are hardly used for 50% of the permissible stress level.
- The loads of gravity not the lateral loads particularly in 3-story frames will control the design, leading to excessive lateral strength which increases over strength factor.
- The target displacement value which is used for analysis of pushover corresponds to the structure collapse state. Therefore, any specified capacity with its full capacity will be assessed. On the contrary, if the target values are selected based on the design state, there will be remarkably lower ductility ratios. Based on the top drift by 1% which is

defined as the level of life safety, the result will lead to mean value of 50% values of factor $R\mu$ and then 50% lower values of R.

- one of the causes of high R values may be the use of permissible stress design rather than the updated technique. Reducing the Effective Ground Acceleration Coefficient by 4 times led to significant reduction of the base shear of the design and it seemed that the lateral actions in the phase of design disappeared. Significant change of the seismic zoning led to reduction of sections in lighter systems under pressured of gravity loads. The non-linear evaluation was expected to be affected by change of the earthquake zone. very high over strength factors resulted from the lower base shears of the design, leading to slight reduction of the factor of ductility reduction.

Conclusions

In this study, methodology which is used for determination of “R”, is based on rule of equal displacement. The application is simplified by this idea while ignoring the structure post-elastic behavior. The effects of stiffness and strength degradation and inelastic behavior negative or positive slopes are neglected completely. Another idea which is called rule of equal area is equal to the total absorbed energy and then the inelastic behavior is somewhat included. On the contrary, the displacement demands even cannot be estimated approximately. Ambiguous results of “R” will be due to two unrealistic approaches. The present seismic code can adjust the factor “R” based on the structure stiffness. Very short vibration periods of the structure will lead to streamlining of the “R” to lower values. On the other hand, the strength has not been issued for determination of R. level of the structural strength should be also controlled because the underdesign and overdesign will lead to the undesirable and unexpected behaviors. Some structures which were designed in this study appear not to yield even in a less intense earthquake.

The desired performance will not be produced by the application of the response change in the design earthquake. “R” single value cannot be determined for a specified type of framing without correlating the basic structural properties including the geometry of plan, layout of framing, type of connection and height. Due to unique structure and boundary conditions of it, a seismic behavior which is controlled well will not be provided by doing the parametrical studies to create a detailed table. However, many variables of design are connected to the R single value. There is a belief that more reliable and better seismic performance will result from incorporation of the different parameters into the selection of R factor. The factor "R" mainly aims to use the structure inelastic capacity. Inelasticity will be due to design of the building for a base shear which is significantly lower than the expected rate only uncontrollably. The inelastic behavior main components including total displacement, plastic rotations and story drift ratios will not be known. In 5% of the modal damping, there is fixed damping of structures in the present seismic design.

Future Work

This work only focused on calculation of R factor for 3, 6, and 10 stories STS structures where the structure in Y direction is moment frame and in transverse direction is staggered truss system. It is suggested to carry more analysis on high rise buildings as well. It is recommended to do more analysis on calculation of R factor in STS systems when the structure in X direction is braces system. It is also suggested to do further analysis for the same structures when different types of braces such concentric brace and eccentric braces are used in Y direction of the building.

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