A Simplified Framework for Performance-Based Design of Reinforced Concrete Frame Elements

FINAL YEAR PROJECT – UG BATCH 2018

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(2022)

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Final Year Project Titled

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for the undergraduate degree **in**

CIVIL ENGINEERING

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ACKNOWLEDGEMENTS

The completion of our project is solely due to the vision and guidance of our Supervisor A/P Dr. Fawad Ahmed Najam. His utmost efforts and guidance enabled us to develop a deep understanding of not only our project but also its associated topics too. His friendly guidance and continuous encouragement enabled us to envision beyond the horizon and strive harder than ever to develop an understanding of our topic.

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Abstract

This report offers a new and a simplified solution to the complex and time taking procedure of Performance-based Design (PBD). This is achieved by assessing element level performance instead of the structure level performance. This new approach makes Performance-based Design (PBD) more adaptable in the field of structural engineering, especially in developing countries like Pakistan. Specifically, the simplified framework is provided through development of a computer software coded in Excel VBA. The solution is validated against the complicated and time taking Non-Linear Time History Analysis (NLTHA) procedure and its results. The report further discusses the importance of the adaptability of Performance-based design (PBD) as the need of high-rise construction emerges in Pakistan.

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Being a structural engineer, one is bound by certain prescriptions and limitation. We follow a specific design code and it most of the times limit our ability to use the structure to its full ability. Often, we are constrained by these codes to follow a specific path that does provide an optimal solution. We're somewhat becoming experts at navigating through the code provisions quickly rather than developing solutions that are both innovative and creative.

The solution to this issue is catered in the newer design philosophy known as the Performance Based Design (PBD). In this approach the end goals or performance criteria are set prior to the design process. This performance-based design approach involves the design criteria to be expressed as certain performance objectives when the structure is subjected to certain level of hazard. These performance objectives may be expressed in terms of a stress level not to be surpassed, a damage state, a load, or a limit damage state. In general, the seismic performance is evaluated upon the three levels IO (Immediate occupancy), LS (Life safety) and CP (Collapse prevention). IO means the structure is at the serviceable limit state. LS means the structure is at the safety limit state while CP means that the structure has achieved significant damage and is close to the collapse limit state.

The process of performance-based design includes the designer to have a discussion with the client to set the actual performance levels. After that a specific performance criterion is set upon the agreement. After that the designer develops a preliminary design. In the next step we evaluate whether the design meets the specific set of performance levels already been. If yes, we move on to the construction if not the whole process is repeated iteratively until the design meets the specific set of performance levels.

Performance based requires knowledge of detail non-linear modeling and analysis. Currently, performance-based design is being used in the world for the important structures such as high-rise building, hospitals, schools, and buildings of strategic significance. In Pakistan however we lack the capability and skill level require for the performance-based design. The designers are often looking for a quick alternative rather than lengthy procedure such as the Performance based design but considering the seismic risk based on the geographic location of Pakistan there is a dire need to make Performance based design a common practice.

Although there is still a long way to go but performance-based design presents a good alternative to the traditional code-based design. There can be a possibility of the use of PBD in the common structures other than the structures of high importance only if we can speed up its process and equip people with the desired level of skill.

1.2 Problem Statement

Performance based design proves to be better alternative to the traditional prescriptive methods of design. The problem associated with this process is that it requires the knowledge of detailed non-linear modelling to evaluate the performance levels. Other than that, the method itself is quite lengthy and time consuming. Due to these reasons, it is not a very common practice especially in Pakistan where designers are not well equipped with the desired skill level. Our project tries to solve this problem at hand by an excel based software that predicts the performance of a specific cross section using the fiber modelling approach. A person who is not knowledgeable in the detailed non-linear modeling and analysis can now perform PBD using this software.

1.3 Objectives

The objectives of this study include,

1) The detailed non-linear modelling and analysis of three real life case study building on CSI ETABS 2018 and their seismic performance assessment.
2) The development of an application on EXCEL for the simplified performance assessment of cross sections.
3) A comparison of the performance results acquired by the developed application with those determined using the detailed non-linear modelling using CSI ETABS.

1.4 Methodology

For the development of a newer simplified approach there was a need to develop a new computer application and validate its results against a given set of results obtained through an established software which in this case was Etabs 2020.

Firstly, a case study structure was selected, and its linear model was created using the structural drawings. Non-linearity was introduced into the model using Fiber modeling and plastic hinges after which the model was run against selected ground motions. A total of 5 ground motions were selected from the Pacific Earthquake Engineering Research Center (PEER). Specific selection criteria was used to obtain the ground motions which represented the site conditions of our selected structure. Using the non-liner model and the selected ground motions Non-linear Time History Analysis was performed (NLTHA) and the first set of performance results was extracted. Moving on, a computer application was programmed which was able to perform non-linear cross-sectional analysis and demonstrate the new simplified framework, named Fibrica. Specific column and beam cross sections from the case study structures were analyzed using the new software and the second set of performance results was extracted. These two sets of performance results were compared against each other to see if the results from the simplified approach and the full PBD procedure are consistent with each other or not. Finally, the study looks into how the simplified approach and the new application can be incorporated into the daily design practice to make it more efficient and effective.

1.5 Scope and Limitations

Fibrica is 2-d modelling software, it allows for the modelling of cross-sections only. When it comes to material properties, there are only a few limited options of material; stress-strain models to choose from and there is no option of providing a user-defined stress-strain curve. Furthermore, Fibrica depends on Etabs for the overall output of its results, as the demand point that is to be plotted on the generated capacity curve will come from the linear time-history analysis of Etabs.

CHAPTER 2 LITERATURE REVIEW

2.1. Performance-Based Design

The current design codes do address the life safety and damage control in case of some minor seismic activity while prevent collapse in case of some major earthquakes. But how much reliable the design is in achieving these objectives is still unknown. This causes uncertainty in the prescriptive design procedures regarding the seismic design and seismic capacity of the structure to be designed.

To cater for this issue, we are introduced with a new design philosophy termed as the Performance Based Design. Performance based design is a more general design philosophy in which the design criteria are expressed in terms of achieving stated performance objectives when the structure is subjected to stated levels of seismic hazard (Performance-based design in earthquake engineering: state of development by Ahmed Ghobarah).

Often, Performance based design and displacement-based design are used interchangeably. This is explained by the fact that the performance objectives and structural damage can be correlated, which in turn can be in a relationship with the storey drift and displacement but, this assumption is not entirely accurate one can however term displacement-based design a subset of performance-based design. The level of damage is also related to many other factors such as failure modes of different elements, the duration period, and cycles of an earthquake and in case of some secondary elements their acceleration levels. For effective design criteria, the actual performance of structure in case of an earthquake should be calibrated against drift and damage.

There are many approaches for the performance-based design. One of such approach is that in the first step the traditional force-based analysis is done and after the design is done the deformation and damage are estimated and checked against the established limits. Other approach is such that firstly the displacement and drift associated with certain performance level are established then propotion the structure and then carrying out actual respons analysis (Performance-based design in earthquake engineering: state of development by Ahmed Ghobarah).

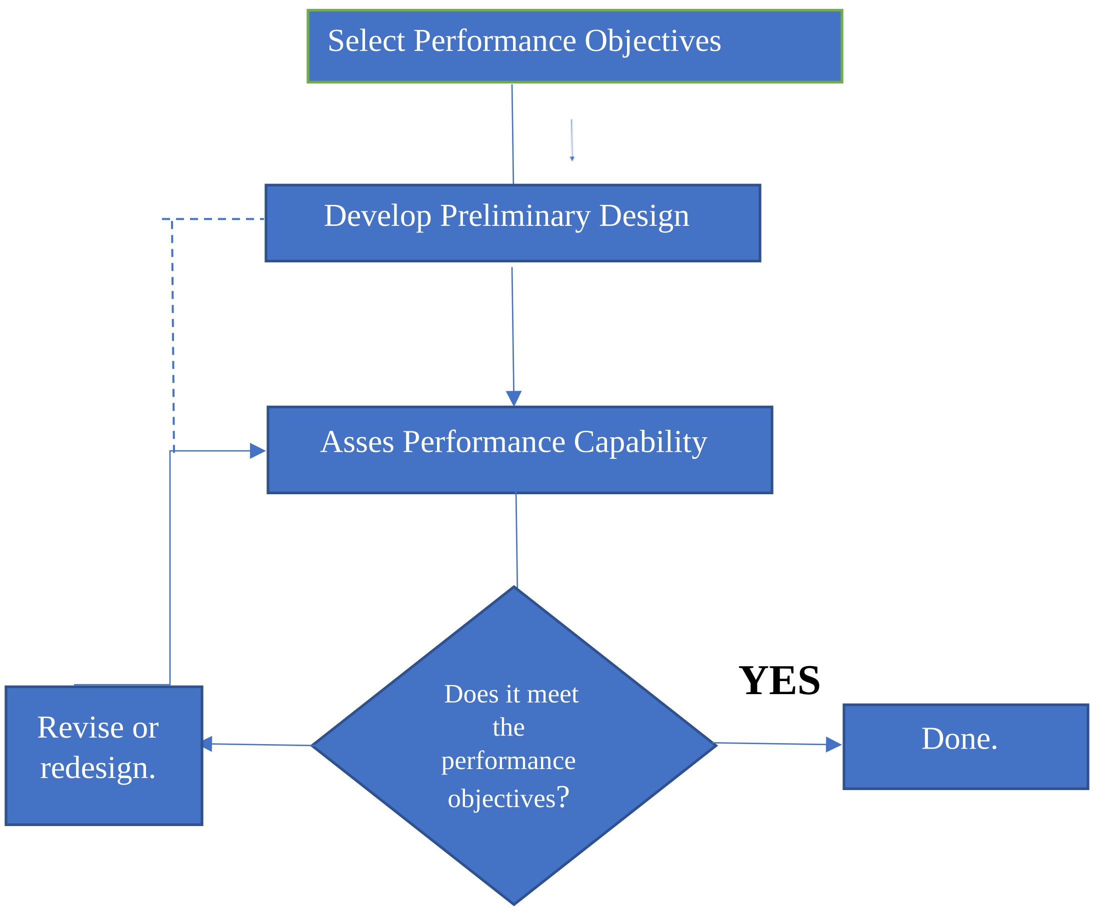
The process of performance-based design starts with establishing a performance criterion articulated through one or more performance objectives. (Performance-Based Design: John L. Galinski, PE). Performance objectives are statements of acceptable performance of the structure. The performance target can be specified limits on any response parameter such as stresses, strains, displacements, accelerations, etc. It is appealing to express the performance objective in terms of a specific damage state or the probability of failure against a prescribed probability demand level. (Performance-based design in earthquake engineering: state of development by Ahmed Ghobarah).

Each performance objective is a statement of the acceptable risk of incurring different levels of damage and the consequential losses that occur because of this damage. Losses can be associated with structural or nonstructural damage, and can be expressed in the form of casualties, direct economic costs, and loss of service costs. (Performance-Based Design: John L. Galinski, PE).

Performance Level	Damage State	Drift
Fully Operational, Immediate Occupancy	No Damage	< 0.2%
Operational Damage, Moderate	Repairable Damage	<0.5%
Life Safety	Irreparable Damage	<1.5%
Near Collapse, Limited Safety	Severe	<2.5%
Collapse		>2.5%

Table 1: Performance levels and damage states

The given table describes performance level in terms of storey drifts. When specific performance levels are set the designer moves to the preliminary design.



Flow Diagram of performance-based design: Source Hamburger 2003

The above figure illustrates the steps used in the performance-based design.

In the next step we evaluate whether the design meets the specific set of performance levels already been. If yes, we move on to the construction if not the whole process is repeated iteratively until the design meets the specific set of performance levels.

One of main step in performance-based design is the design evaluation. The methods that we use in the design evaluation are based upon non-linear modelling and analysis. These methods include non-linear Pushover and nonlinear time history analysis. For this one need to have the knowledge of nonlinear modelling. Nonlinear modelling can be done in the sophisticated software like ETABS and PERFORM 3D. The nonlinear model of a structure is capable of clearly identifying the structural damage and performance in terms of deformation demand-to-capacity ratios. (Nonlinear Modelling and Analysis of RC Buildings using ETABS (v 2016 and onwards)

The non-linear modelling can be further categorized into Fiber Modelling approach and Plastic hinge approach. In Fiber Modeling approach each structural member is divided into number of fibers and each fiber is assigned a material stress strain curve. In case of Plastic hinge modelling, we tend to assume that the inelastic deformation is concentrated at specific points of zero length hypothetical elements termed as plastic hinge elements.

The linear structural model is converted into a non-linear model using either of these approaches and then the detailed nonlinear static pushover analysis or the nonlinear time history analysis can be performed on these computer models to assess the performance of the designed structure whether it fulfills the design criteria that was established in the first step.

Currently this approach of Performance based design is used in the designing of very important structure such as hospitals, High rise buildings and structures of strategic importance. This is the situation in the developed nation. In case of developing countries there is little to no use of this approach partially due limited skill set of the designers and partially due to the time-consuming nature of performance-based design.

Here the issue arises about the time-consuming aspect of the nonlinear modelling. Nonlinear modelling requires a certain level of skill which is not very common and other than that the analysis itself is very time consuming that makes the whole process of performance-based design quite tedious. First modelling the preliminary design on a computer software and then performing the nonlinear analysis and if your design fails to fulfill the performance levels set then the whole process is repeated all over again. These aspects question the use of Performance based design in the todays fast paced world.

Concluding that the performance-based design is a new approach of design which offers a lot of benefits to the traditional prescriptive methods of design but the downside to this approach is the lack of required skill set and the time-consuming nature. There is potential to increase the speed of this procedure and research should be done to explore new option.

CHAPTER 3: METHODOLOGY

3.1: Selection of the Case Study Buildings:

For our case study we used a real-life building. Its and B+G+8 storey building located in the residential area of NUST H-12.

This is how our building looks it's a moment resisting frame plus shear wall structure. It has one core wall and 4 planer walls.

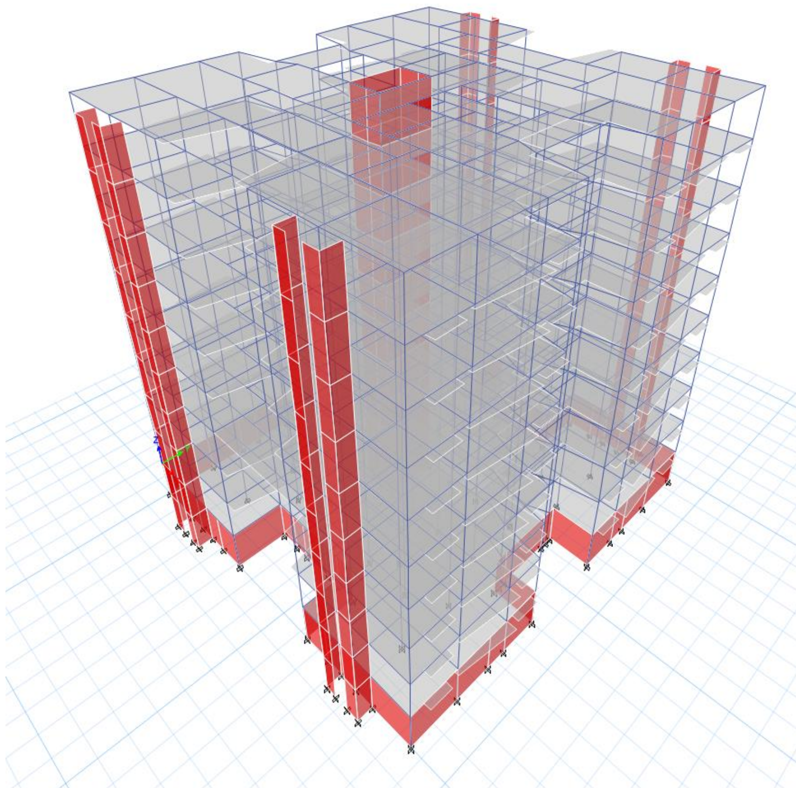


Figure 1-A: 3D elevation of the building model

This building was chosen because it has all the necessary elements required to make our results more credible and increases the scope of the of our research.

It has a basement and nine floors including the ground floor. Below are the floor plans of the building.

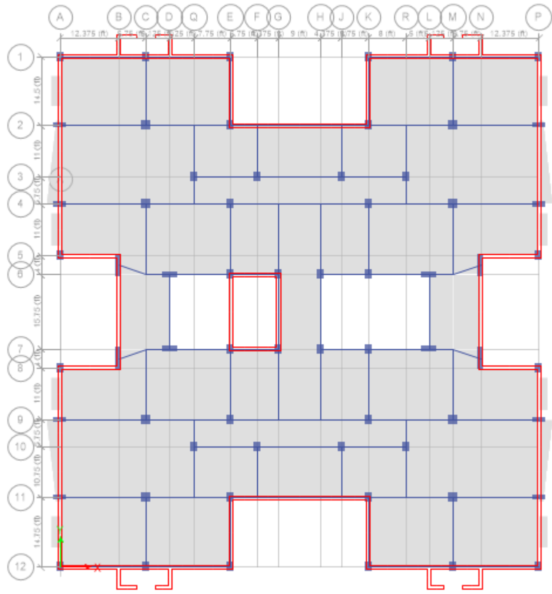


Figure 1-B: Basement plan for the B+G+7 structure

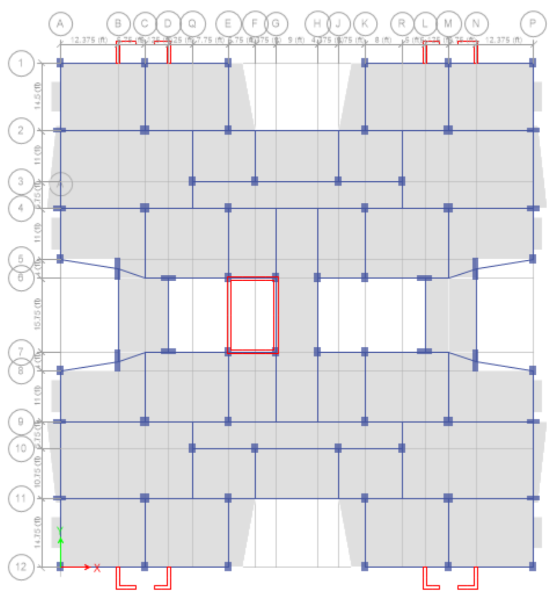


Figure 1-C: Ground floor to 8th floor plan

3.2: Linear Modelling of Structures:

While modelling the structure in ETABS the first foremost important step is to input the material properties of the structure. Linear elastic elements obey Hooke's Law i.e a linear relationship exists between stress and strain which can be represented by the equation $\sigma=E\epsilon$.

In ETABS when we do linear modelling we keep E, L, I A, G and k all constants where

E= Young's Modulus

L= Member Length

I= Moment of Inertia

A= Cross sectional Area

G= Shear modulus of elasticity

K being the function of all these variables listed above so it is also constant.

ETABS uses the finite element models to model a structure. Through meshing a discrete model of the structure is then created which has finite degrees of freedom. For analysis ETABS uses the equation F=KU where F is load matrix and U is displacement matrix to produce responses. This analysis procedure is termed as finite element analysis.

The following steps were used in ETABs to model our structure.

- Grids were defined according to the structural drawings
- Material properties were defined for both concrete and steel to be used
- The cross-sectional properties of beams, columns and shear walls were defined. The beams and columns were defined as frame elements while slabs were modelled as thin shell elements and shear walls were modelled as thick shell elements. These cross sections were defined as per the structural drawings.
- The stiffness modifiers for beam were 0.35 for moment of inertia along 2 and 3 axes while for columns the value was 0.7 for both the moment of inertia in 2 and 3 axes.
- Frame elements were drawn on the grids to create overall layout of the structure referring to the structural drawings.
- The Shell elements were also drawn using the grids as per the structural drawings of the building.

The following figures illustrate the how these steps were performed.

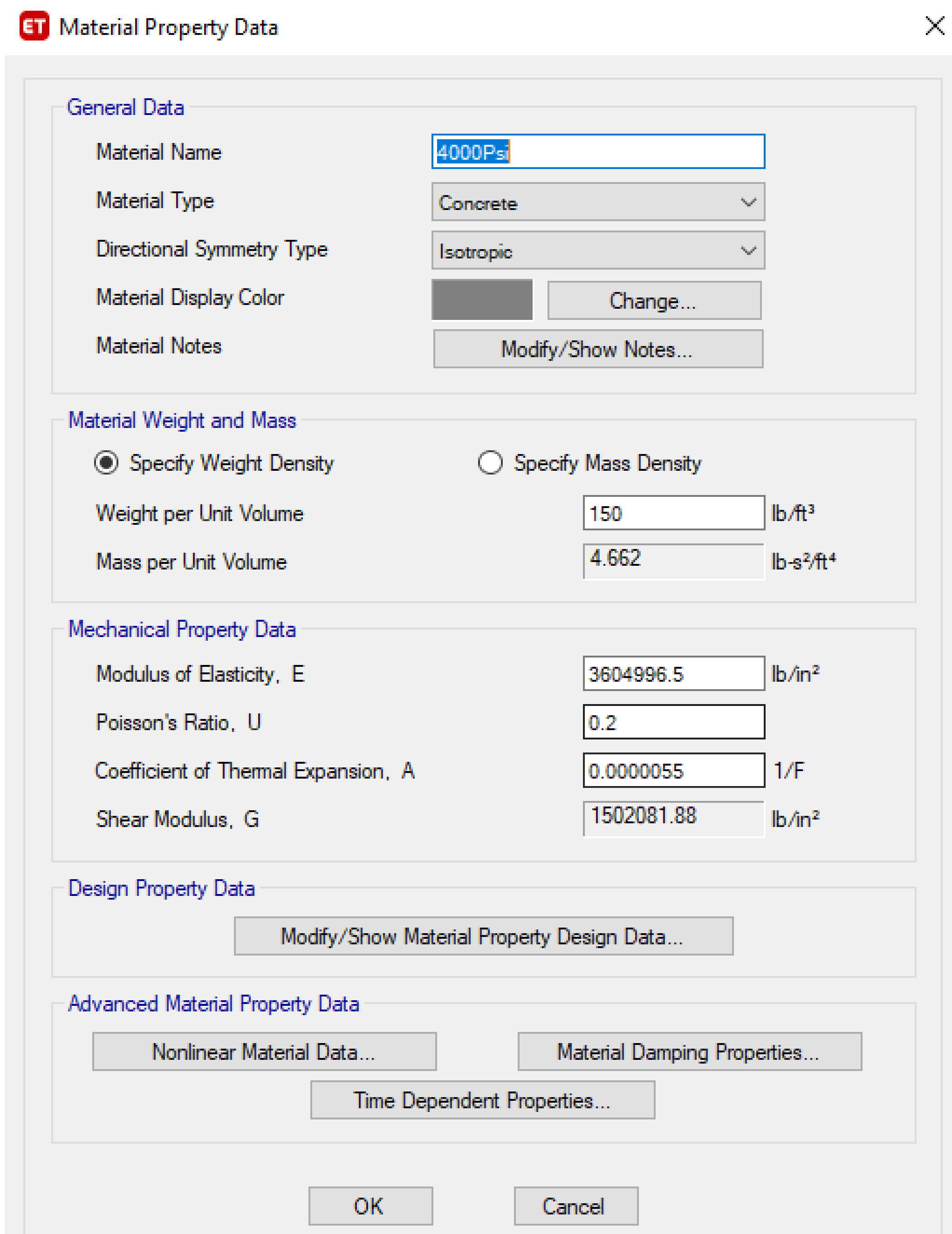


Figure 2: Definition of material properties

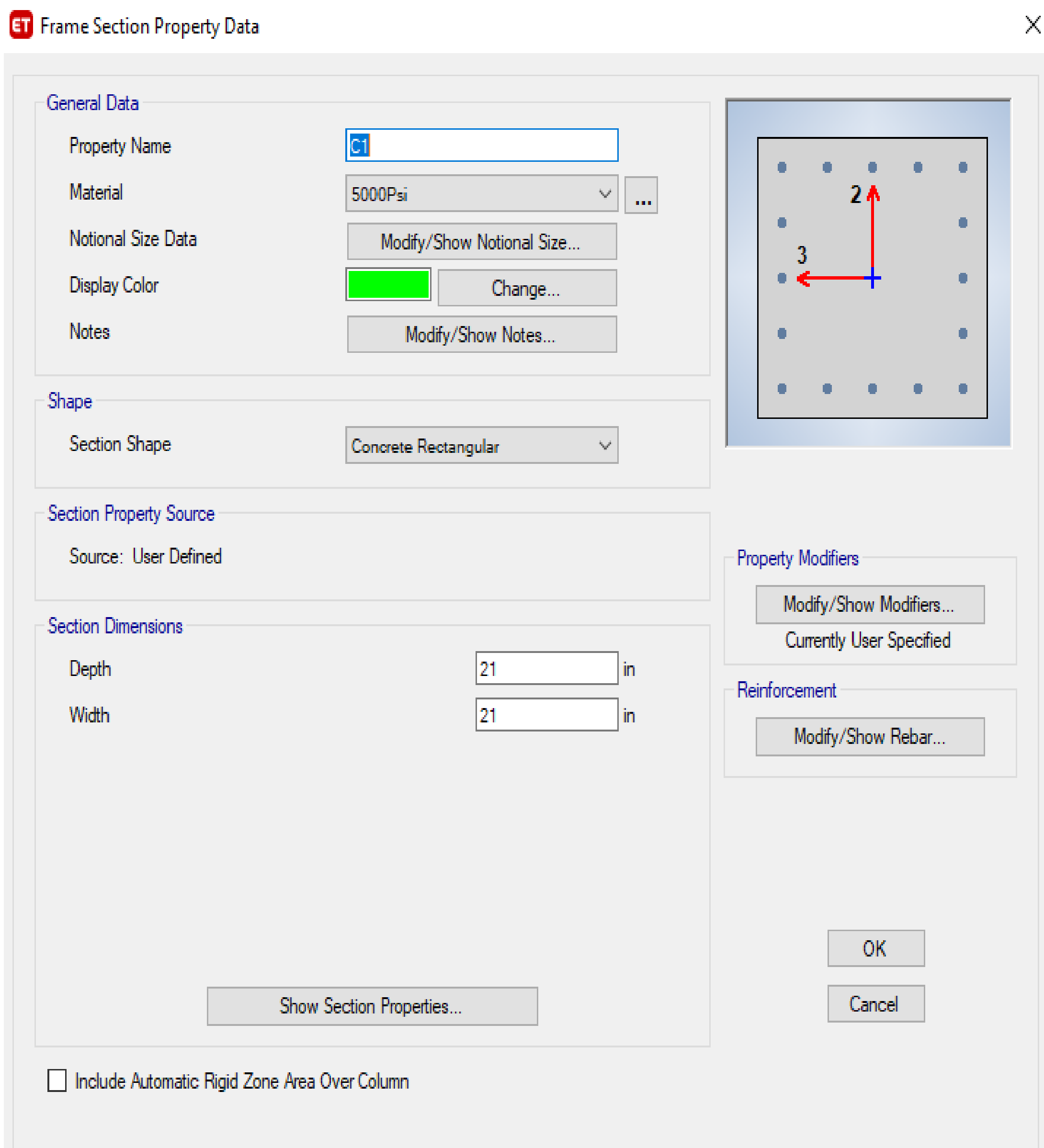


Figure 3: Definition of a column cross-section

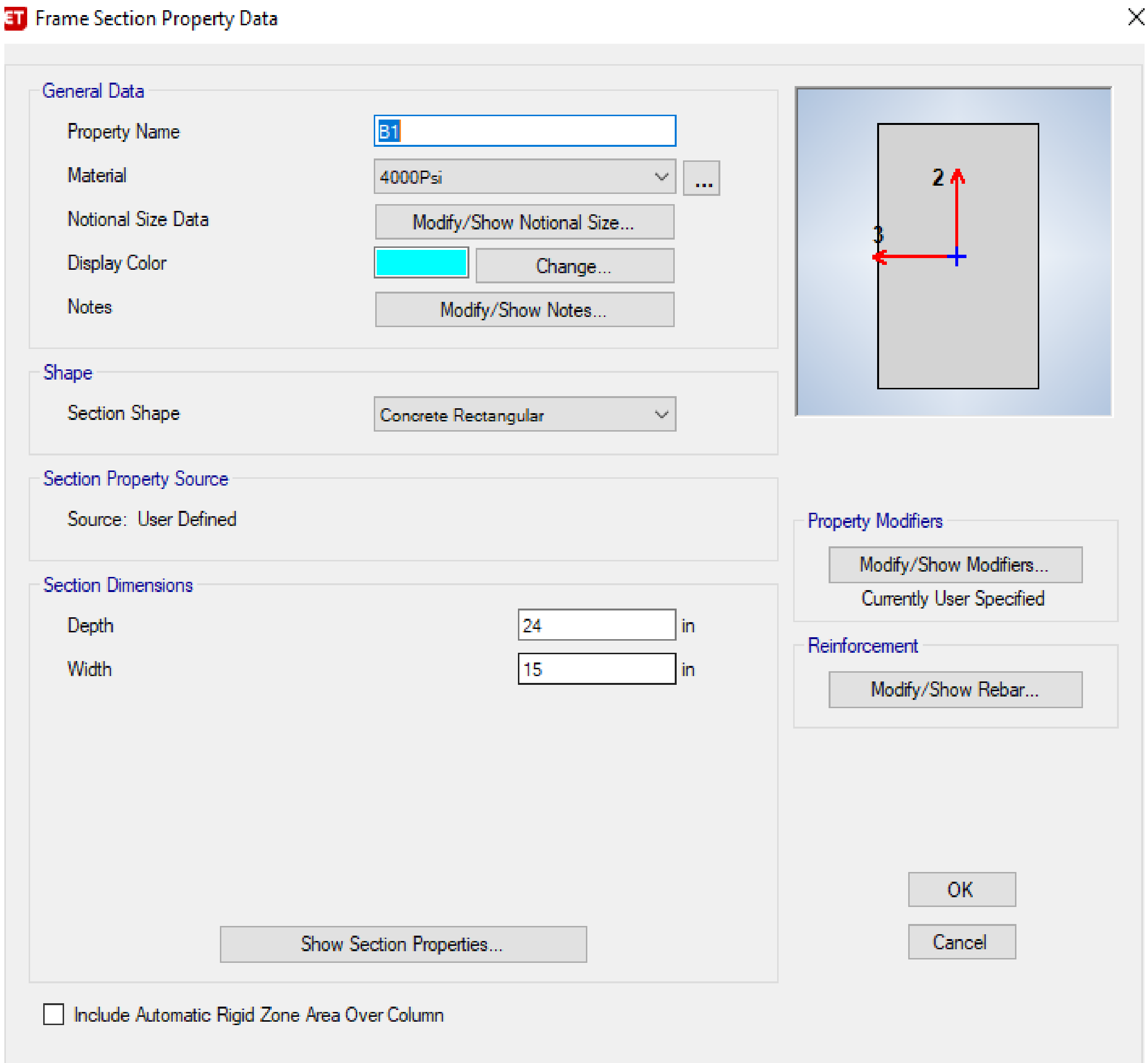


Figure 4: Definition of a beam cross-section

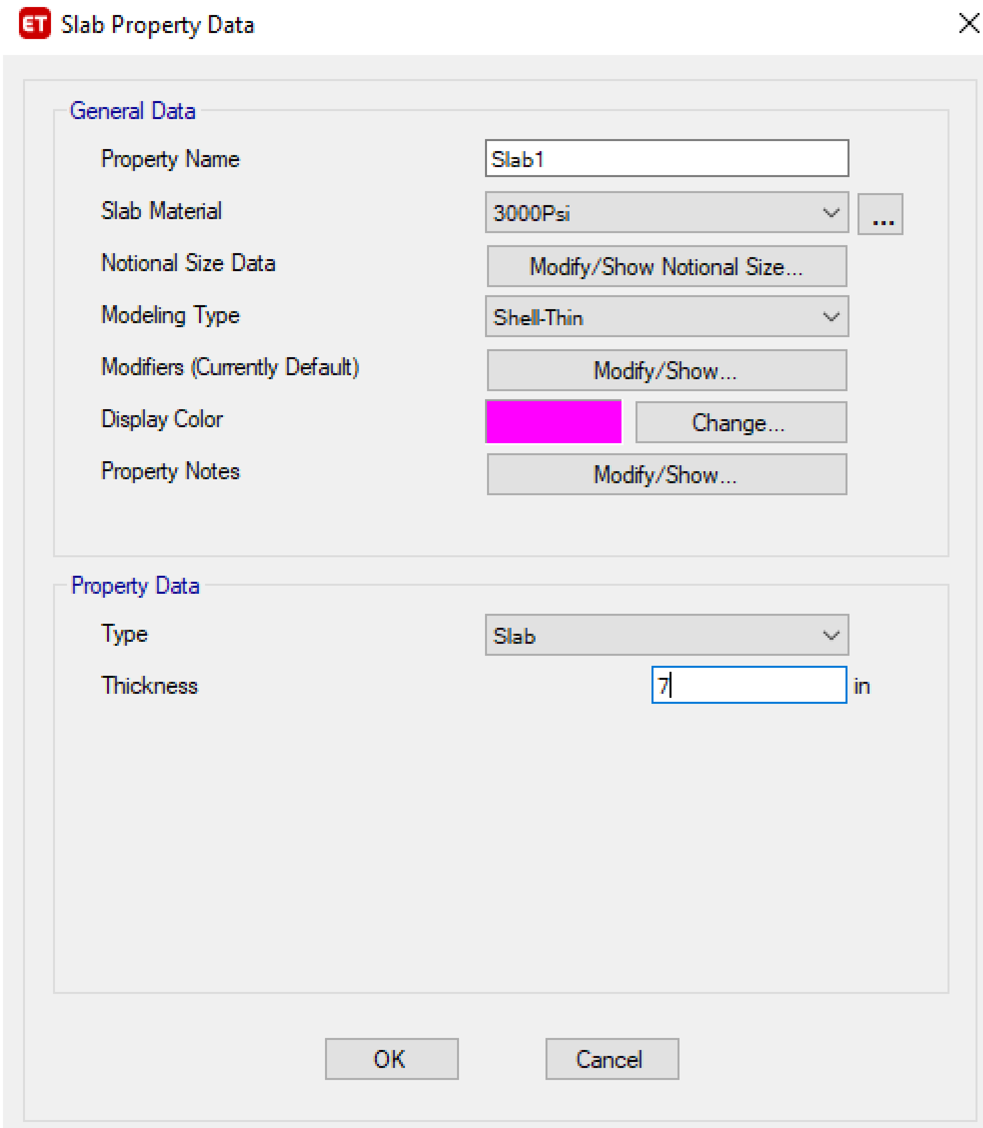


Figure 5: Definition of a slab cross-section

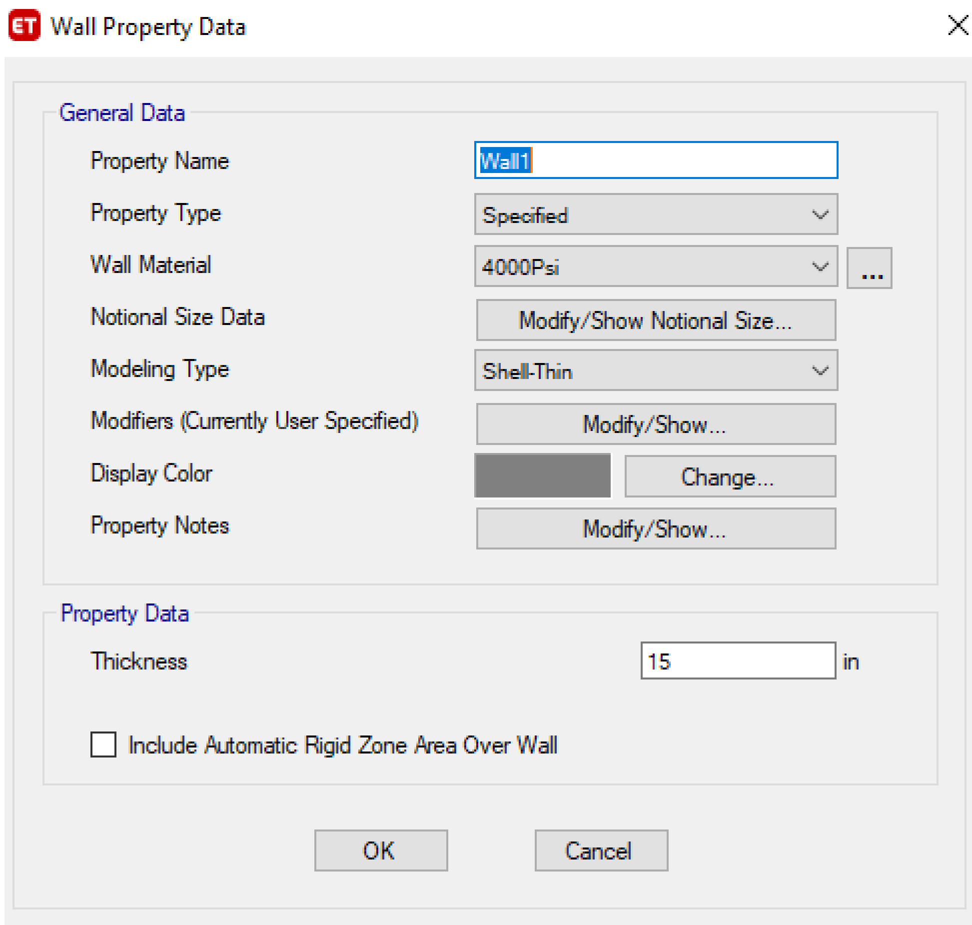


Figure 6: Definition of a shear wall cross-section

3.2.1 Modelling Considerations:

All the elements were modelled as linear elastic as the properties E, L, G, A and K were assumed to be constant. Due to the complex nature of modelling stairs were not modelled.

3.3: Nonlinear Modelling of structures:

There are two approaches to model nonlinear behavior of a structure in ETABS i.e Fiber Modelling and Plastic hinge modelling.

Fiber Modelling approach: This is the more detailed approach in this approach, the cross section of a structural member is divided into number of uniaxial fibers running along the length of the member and each of the fiber is assigned a stress-strain curve.

The stress strain curve captures the various aspects of non-linearity in the fiber. These fibers can be throughout the length of the member or at a percentage of total length of the member.

This approach can account for the axial-flexural interaction also the axial deformation caused by bending in the shear walls and columns and shear walls but not the shear behavior. The Shear behavior is to be modelled separately. This approach uses the nonlinearity at the material levels.

Columns and shear walls were modelled as per this approach.

Plastic Hinge approach: This approach defines the non-linearity at cross sectional or member levels. In this approach, hypothetical elements known as plastic hinges are defined at the start and the end of the member these plastic hinges are points where it is assumed that all the inelastic deformation lies.

Using these plastic hinges, the inelastic relationship is defined at the cross sectional or the member level of the structure to include the effects of non-linearity instead of defining it the to the material level as we do in the fiber modelling approach.

Beams were modelled using this approach.

3.3.1 Non-linear Modelling of Columns:

- Before assigning fiber hinges to the columns first the nonlinear material curve for the fibers must be defined. For this step, click on the define option and select material and select the material property you want to modify and in the nonlinear material data you can use the Models available in ETABs or define your own stress strain curve. Mander’s model was used in this case.

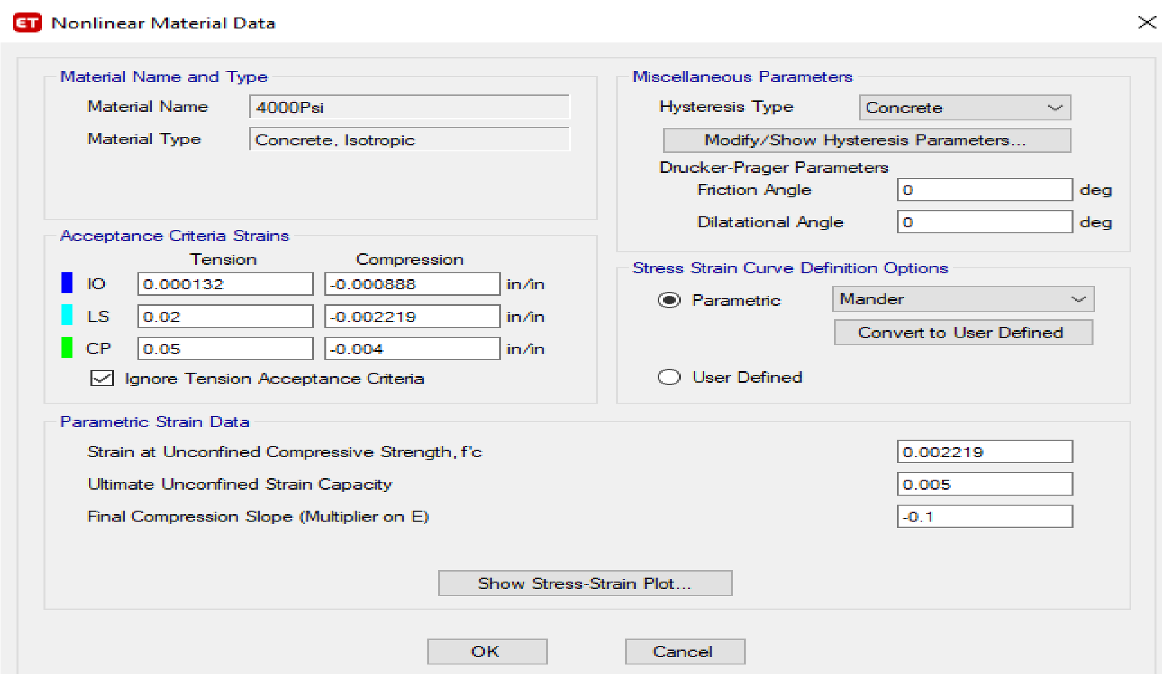


Figure 7: Definition of non-linear material property for concrete

Here the performance levels must also be defined for Etabs to evaluate your structure against.

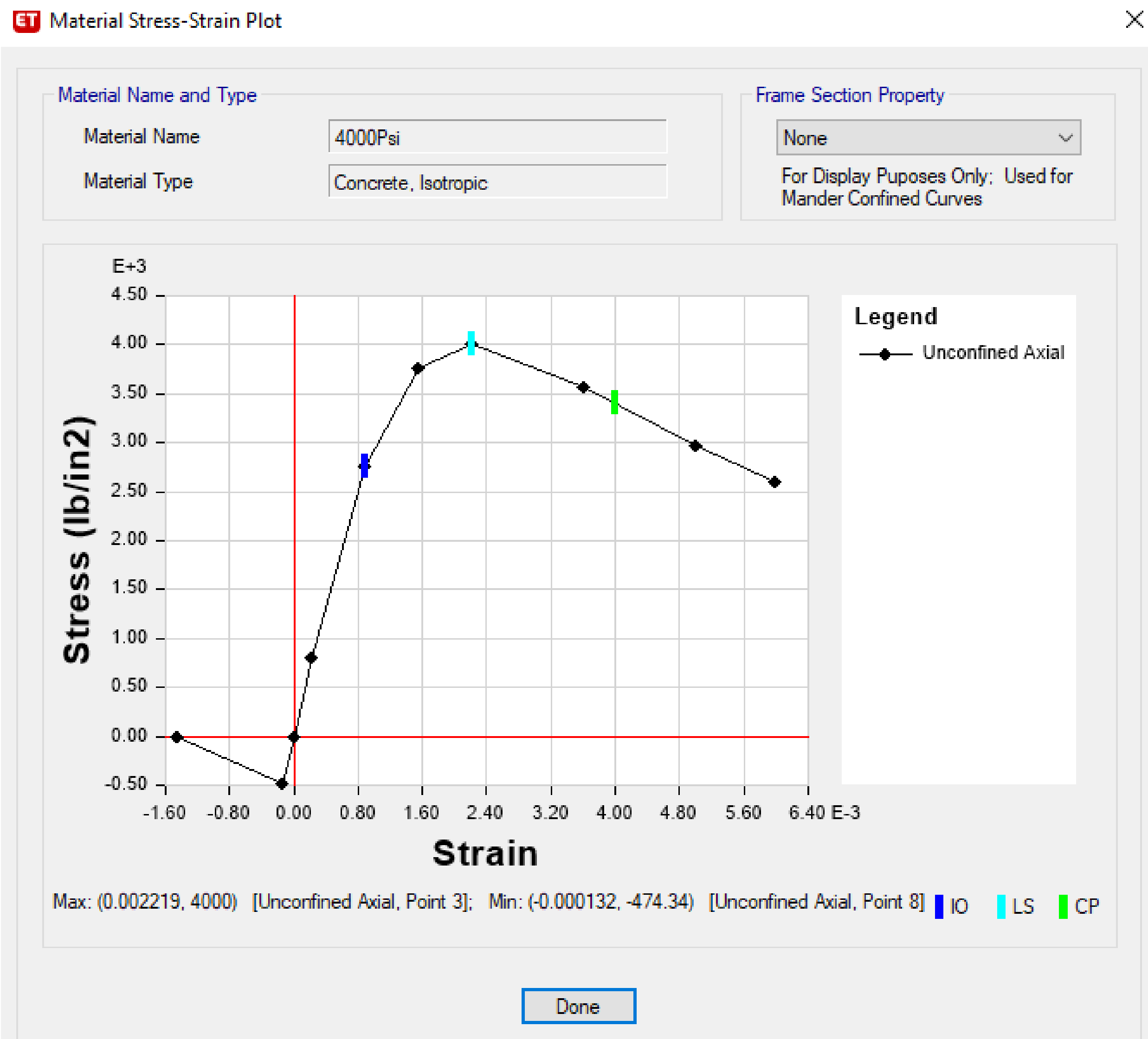


Figure 8: Stress-strain curve for concrete

Similarly the stress starin curve for steel was also defined.

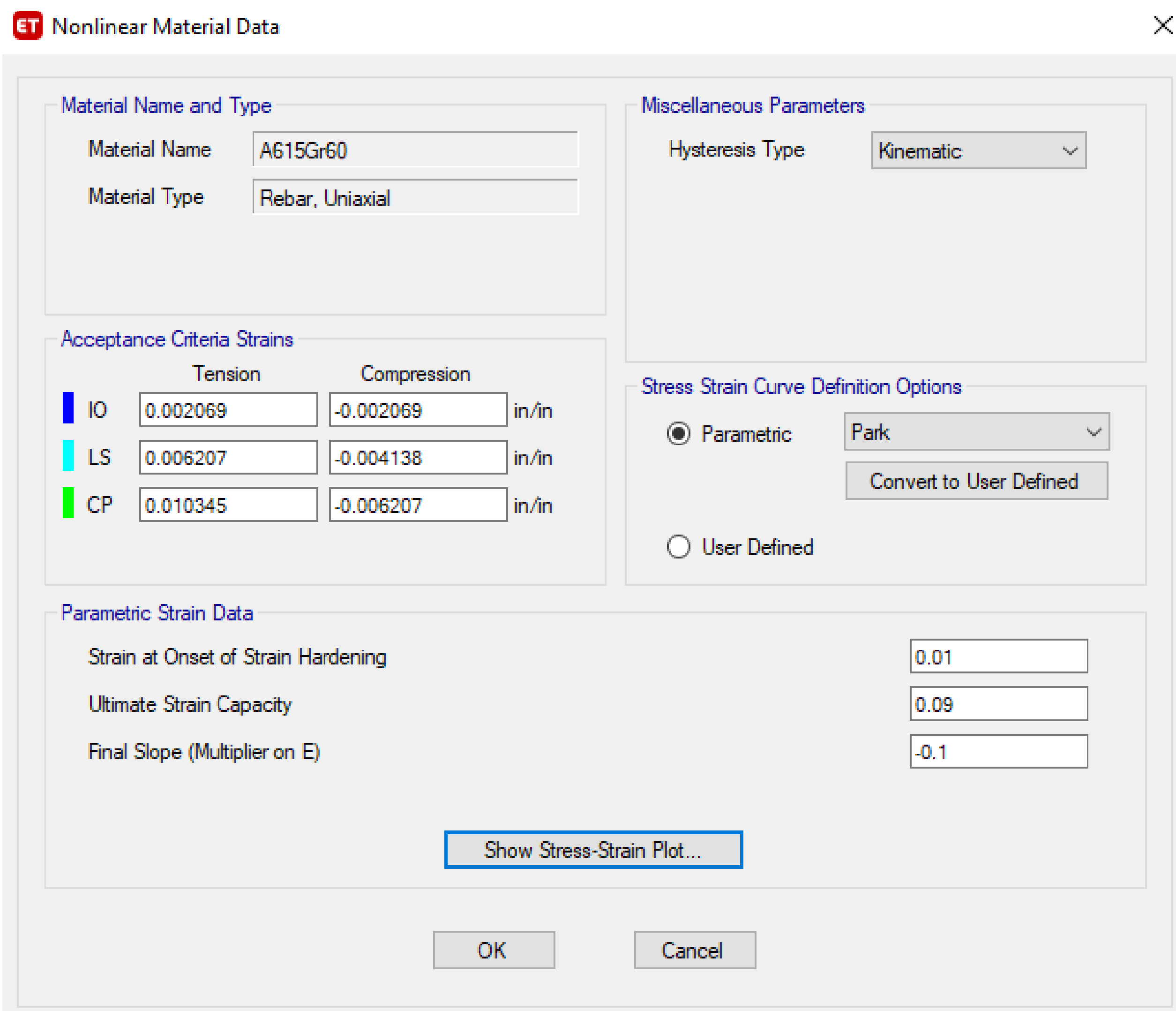


Figure 9: Definition of non-linear material property for steel

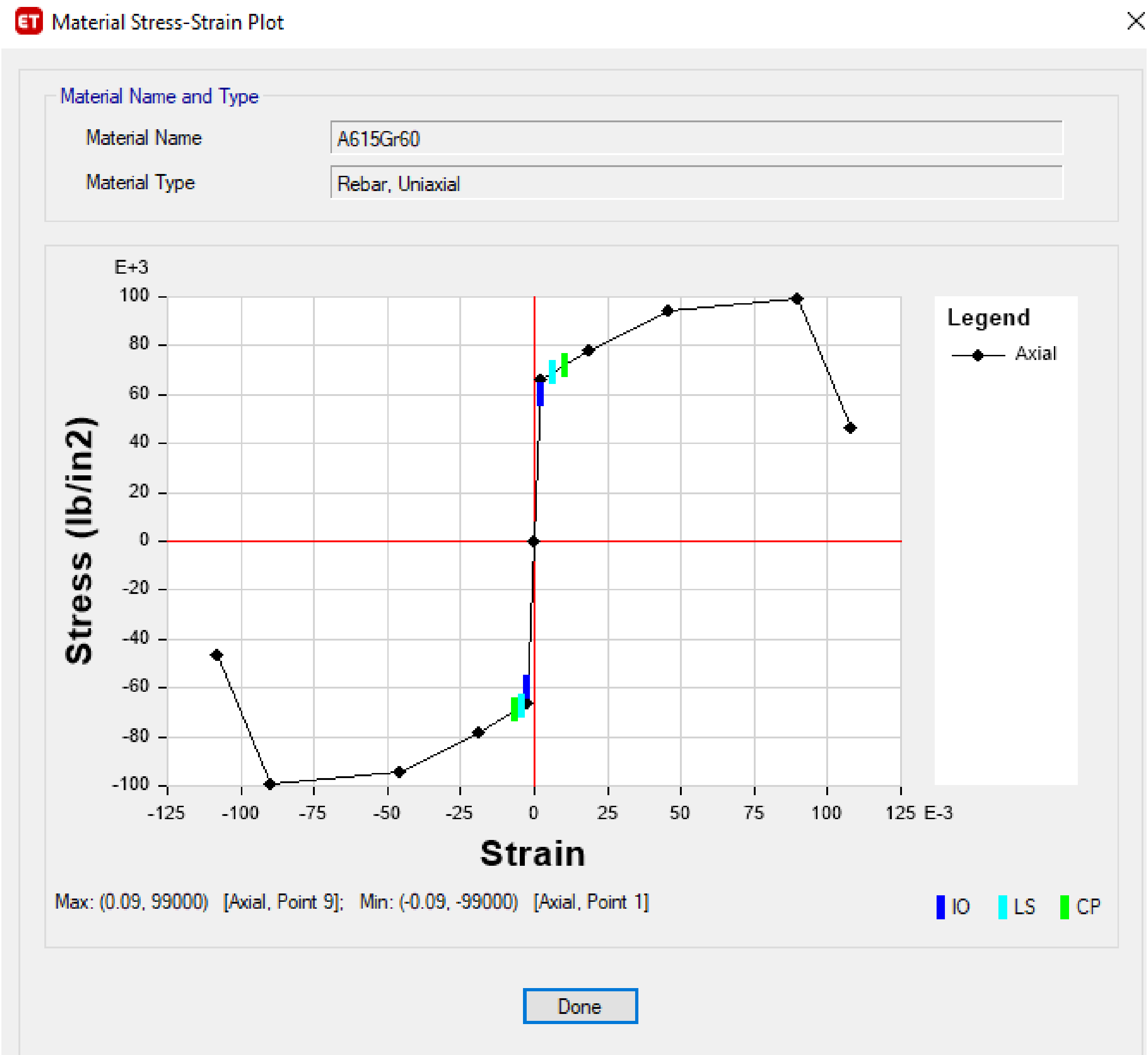


Figure 10: Stress strain curve for steel

- After the cross-sections were defined, next the reinforcement was assigned to each cross-section as per the structural drawings. Before assigning any hinge to an structural elements it is necessary to provide reinforcements to the cross-sections to achieve accurate non-linear behaviour.

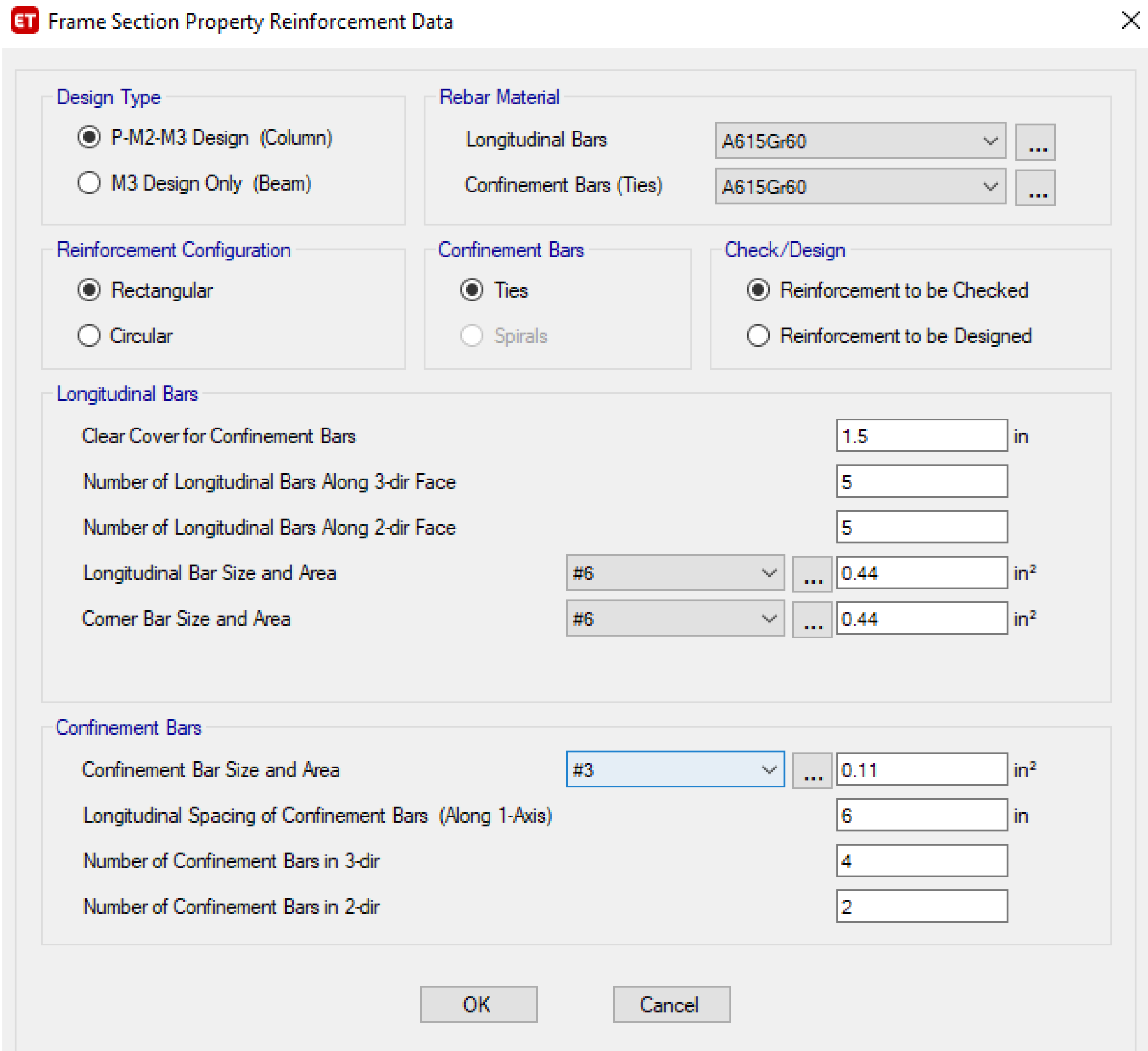


Figure 11: Assignment of reinforcements for the corresponding column cross-section

- Next a fiber P-M2-M3 hinge was defined in ETABS. This was a deformation-controlled hinge defined for concrete material. The hinge was placed at 0.1 of the total length of the member.

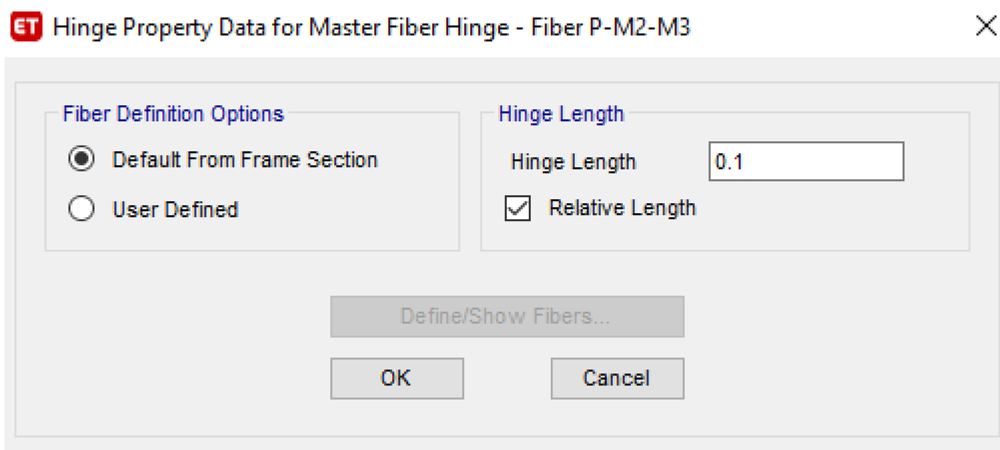


Figure 12: Generation of Master Fiber P-M2-M3 Hinge

- To assign fiber hinges, all the columns were selected and assigned the hinge that was generated above. This way Etabs assigns the master hinge property to each of the columns while giving a unique name to each of the hinges.

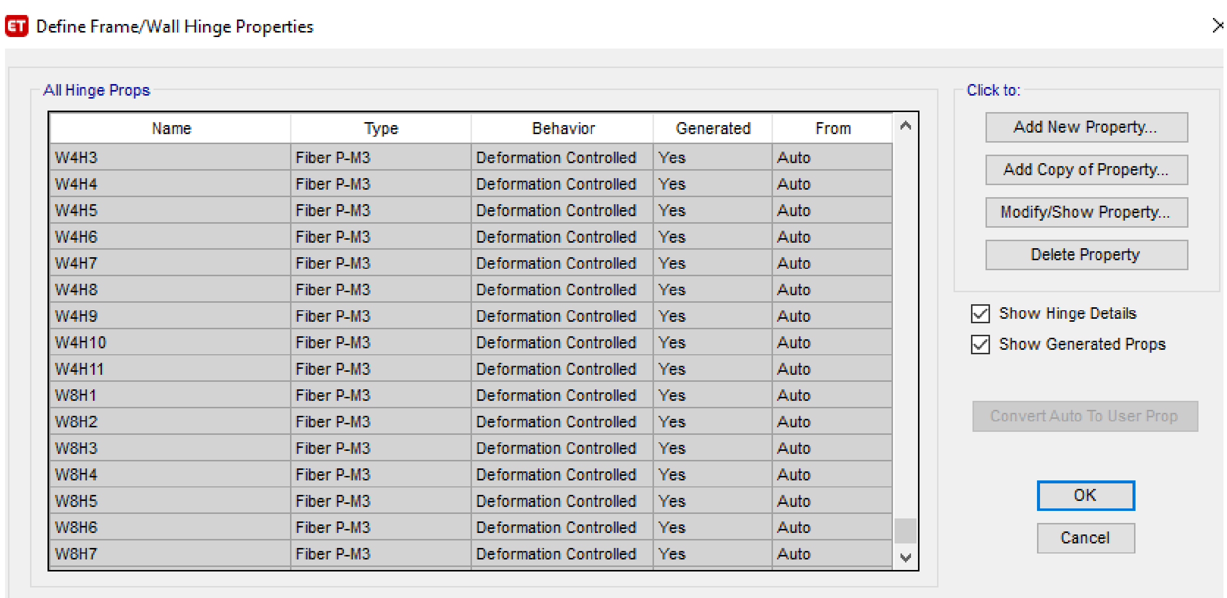


Figure 13: Automatically generated P-M2-M3 hinges

3.3.2 Non-Linear Modelling of beams:

For this case study, as mentioned earlier, beams were modelled using the plastic hinge approach.

- The first step was to assign reinforcements for the beam cross-sections as per the structural drawings.

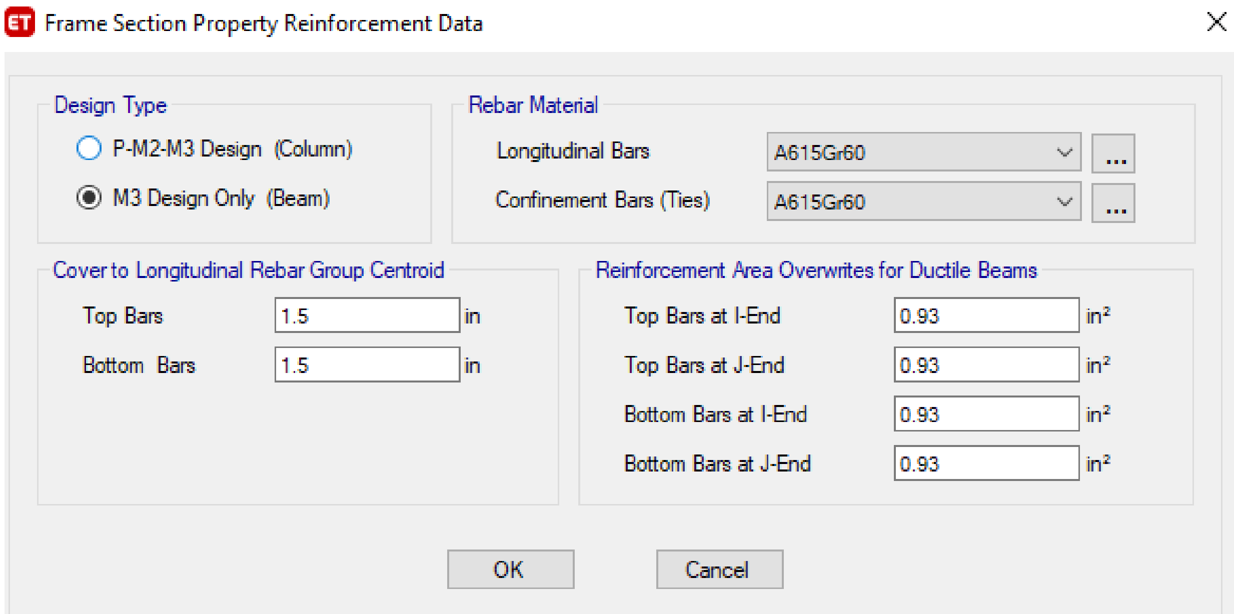


Figure 14: Assignment of reinforcements for the corresponding beam cross-sections

- Next plastic hinges were assigned to beams, for that purpose all the beams were selected and auto hinges using ASCE 41 were generated and assigned at relative distances of 0 and 1 to the length of the member.

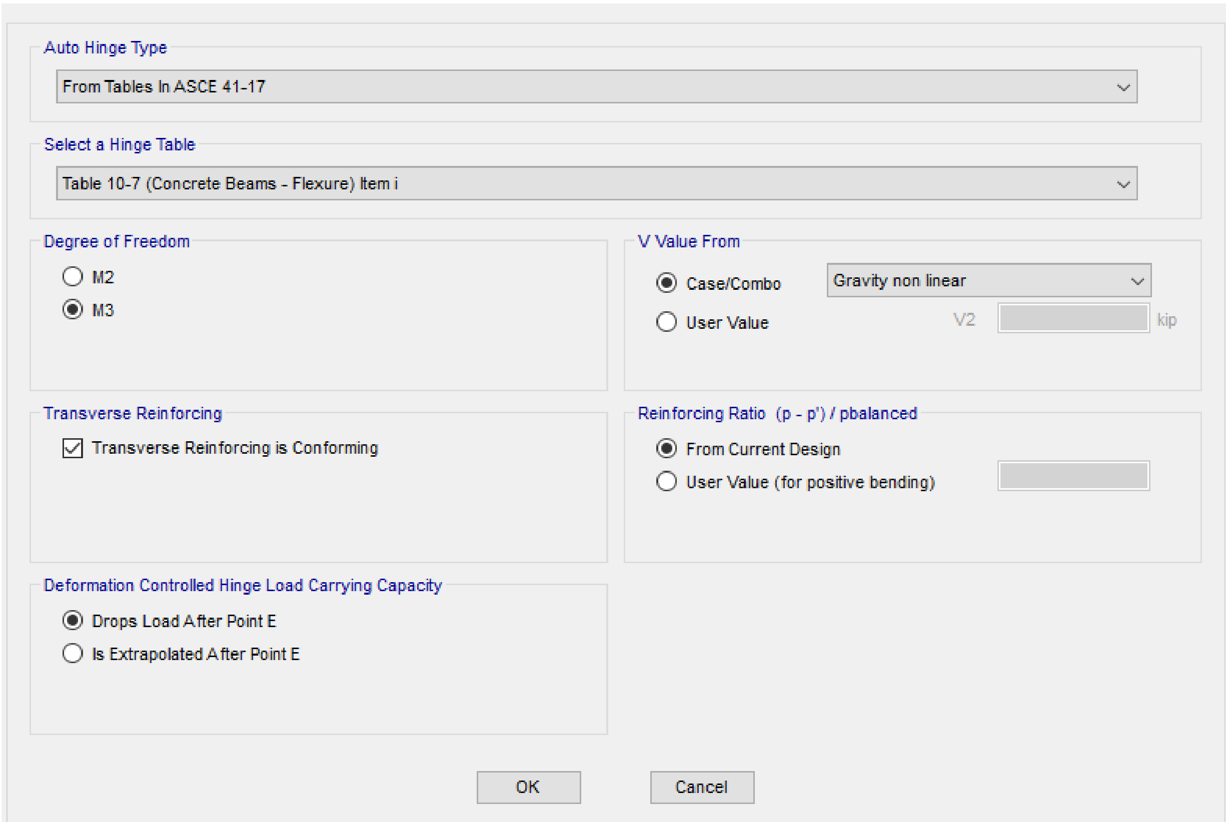


Figure 15: Auto hinge generation using ASCE 41

- The shear value (V) can automatically be taken either from the analysis results of a particular load case/combination or a user-defined value can be provided based on manual calculations.
- Similarly, the factor $\rho - \rho / \rho bal$ can either be automatically obtained by the program using the "From Current Design" option or it can be manually entered for positive bending. For the transverse reinforcement, the check box can be checked if it is conforming. This automatically assigns the moment curvature curve taken from ASCE-41 to the beam.

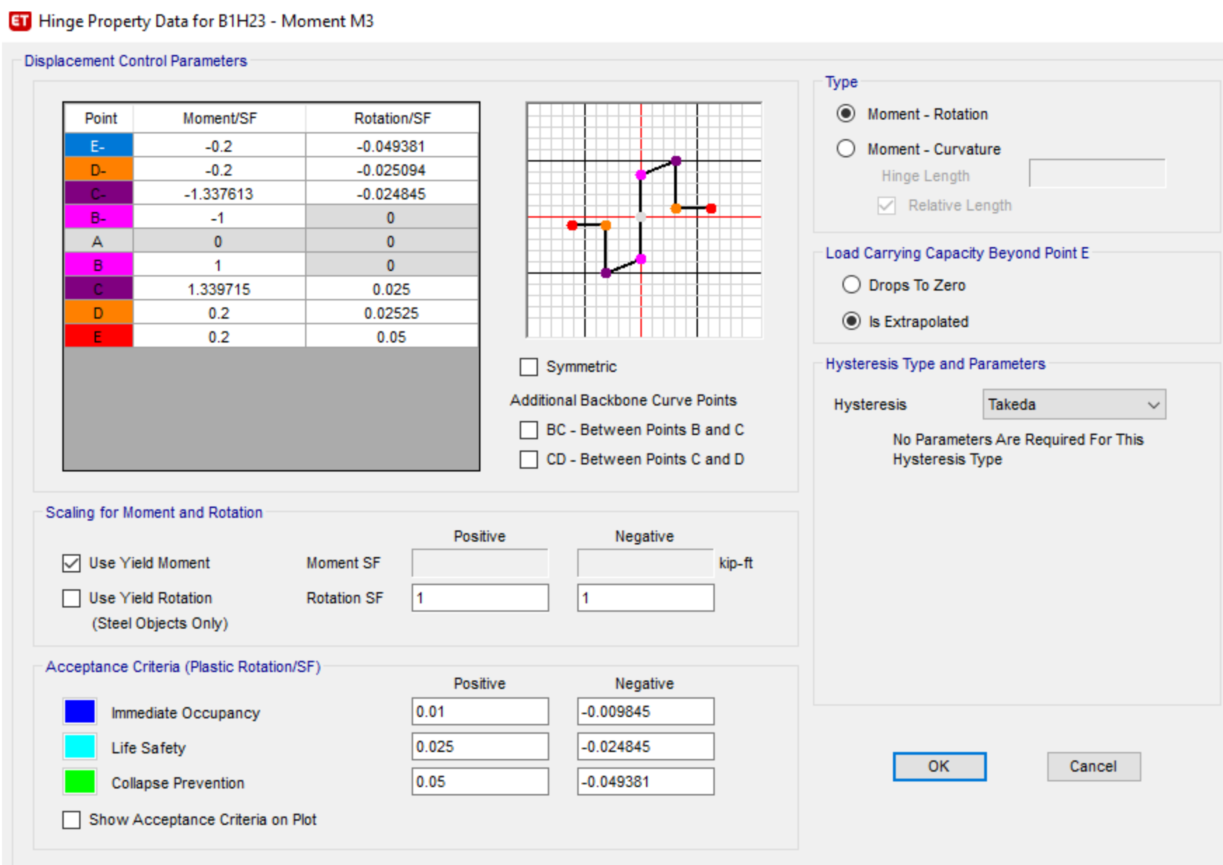


Figure 16: Plastic hinge properties tab

3.3.3 Non-Linear Modelling of Shear walls:

Shear walls were modelled using the fiber modelling approach.

- Before assigning automated fiber hinges to the shear walls the first step is to input the reinforcements in the defined shear wall sections as per the structural drawings.

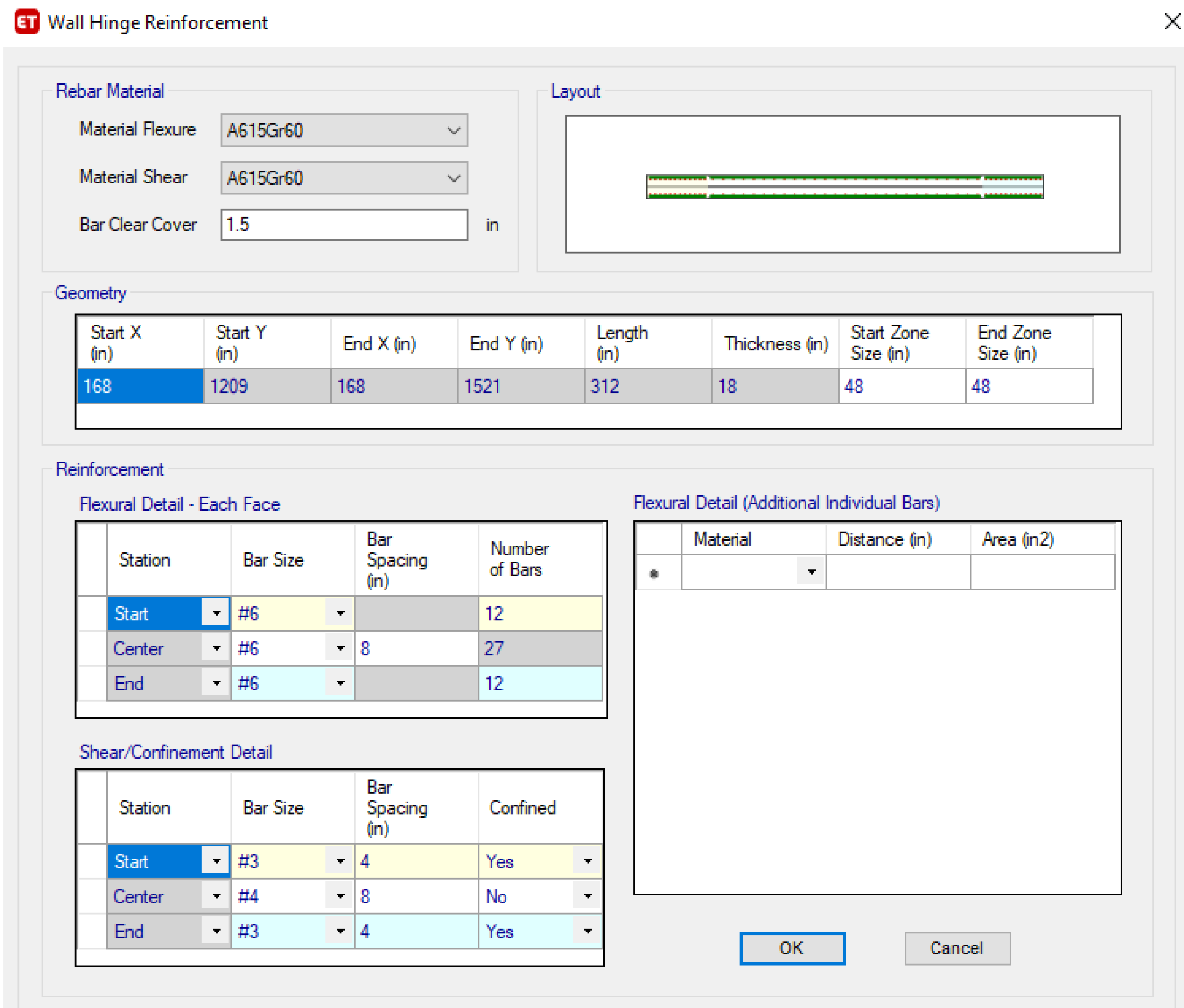
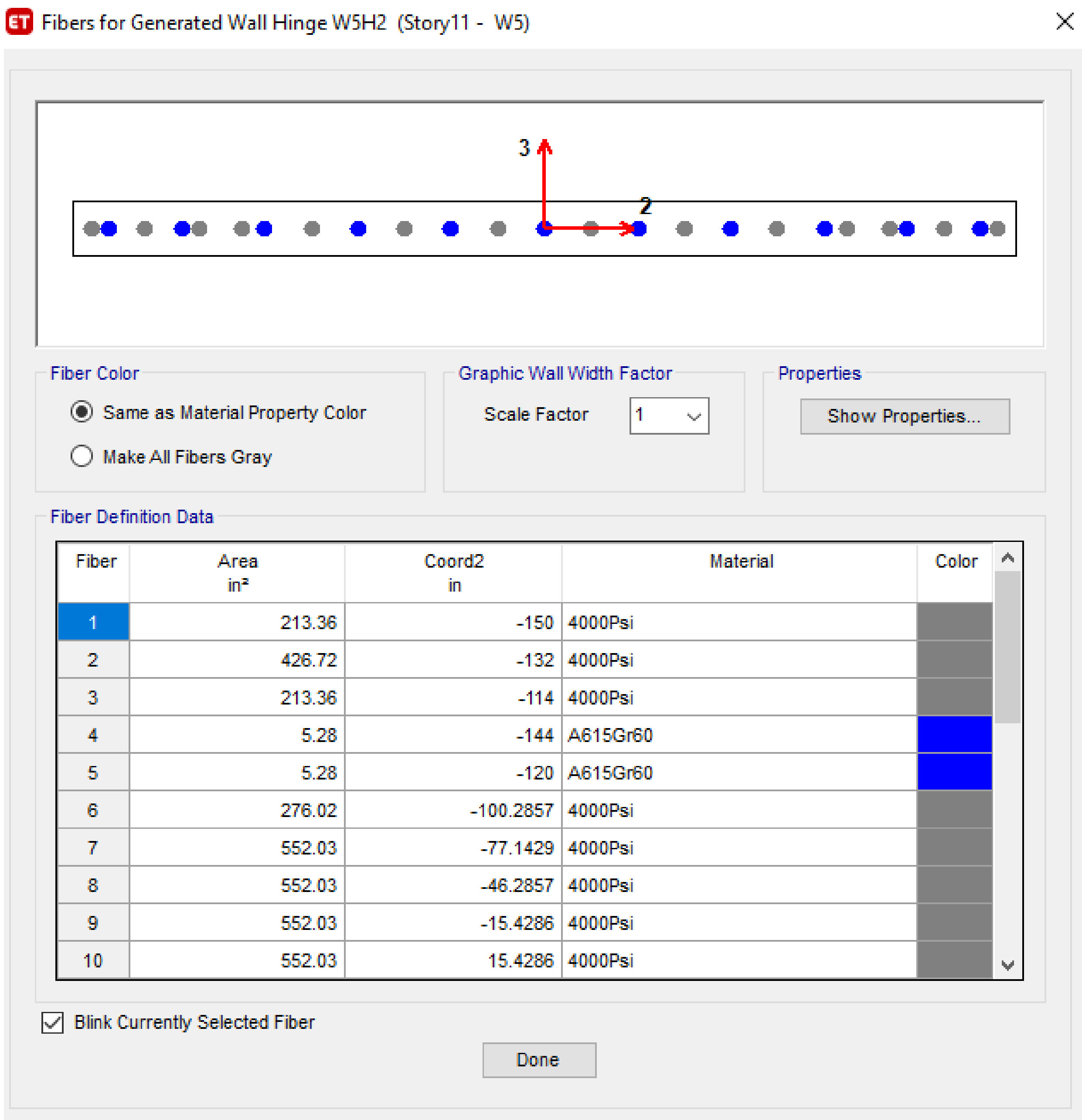


Figure 17: Shear wall reinforcement

- Next, all the shear walls were selected and assigned the master hinge defined in the previous sections. The concrete and steel fibers are defined automatically in these generated fiber hinges.
- Automatically generated can be checked through the hinges option by clicking on define then on section properties.



Fiber	Area in²	Coord2 in	Material	Color
1	213.36	-150	4000Psi	
2	426.72	-132	4000Psi	
3	213.36	-114	4000Psi	
4	5.28	-144	A615Gr60	
5	5.28	-120	A615Gr60	
6	276.02	-100.2857	4000Psi	
7	552.03	-77.1429	4000Psi	
8	552.03	-46.2857	4000Psi	
9	552.03	-15.4286	4000Psi	
10	552.03	15.4286	4000Psi	

Figure 18: Fiber hinge for Shear Wall

3.4: Analysis Procedure:

Two different types of analysis were performed on the selected case study structure namely equivalent lateral force procedure (ELF), which is a linear static analysis procedure, and the non-linear time history analysis, which is a non-liner dynamic analysis.

3.4.1 Equivalent Lateral Force Procedure (ELF)

ELF is the most common and most used method for seismic analysis that because of its simplistic approach.

In this method, equivalent lateral forces are applied as one concentrated force at the mass center of each storey of the building. The base shear is then calculated and distributed along each storey. The method is then repeated along all the directions. It is an approximate method is often used for small regular buildings.

The code that was used for seismic analysis of the structures is **ASCE 7-16.** The main inputs that were require are the spectral acceleration parameters Ss and S1, the values of which were obtained from UBC-97 supplements for Islamabad the city where our case study building is located.

The other inputs that are required for ELF are Cd, R, omega and Ie. Where Cd is deflection amplification factor. Omega being system overstrength. R is response modification while Ie is importance factor. The corresponding for our building which a RC frame plus shear wall structure were used.

The other important parameters are site class co-efficient which are Fa and Fv which were taken as 0.8 and 0.9 as per our site conditions.

Parameters	**Values**
Ss	1.302
S1	0.381
Cd	5.5
Omega	3
Ie	1
R	5
Fa	0.9
Fv	0.8

Table 2: Seismic input parameters and their corresponding values

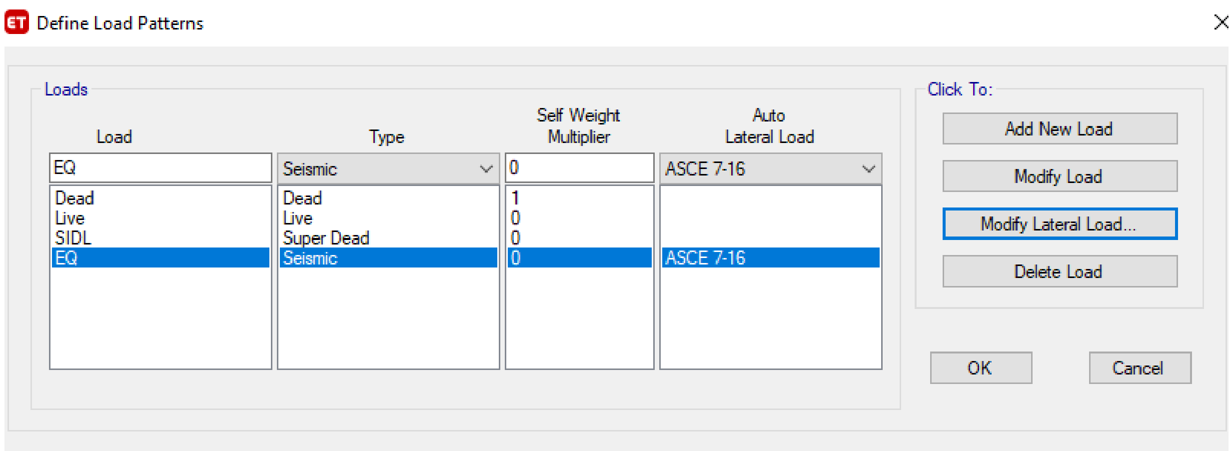


Figure 19: Definition of load patterns

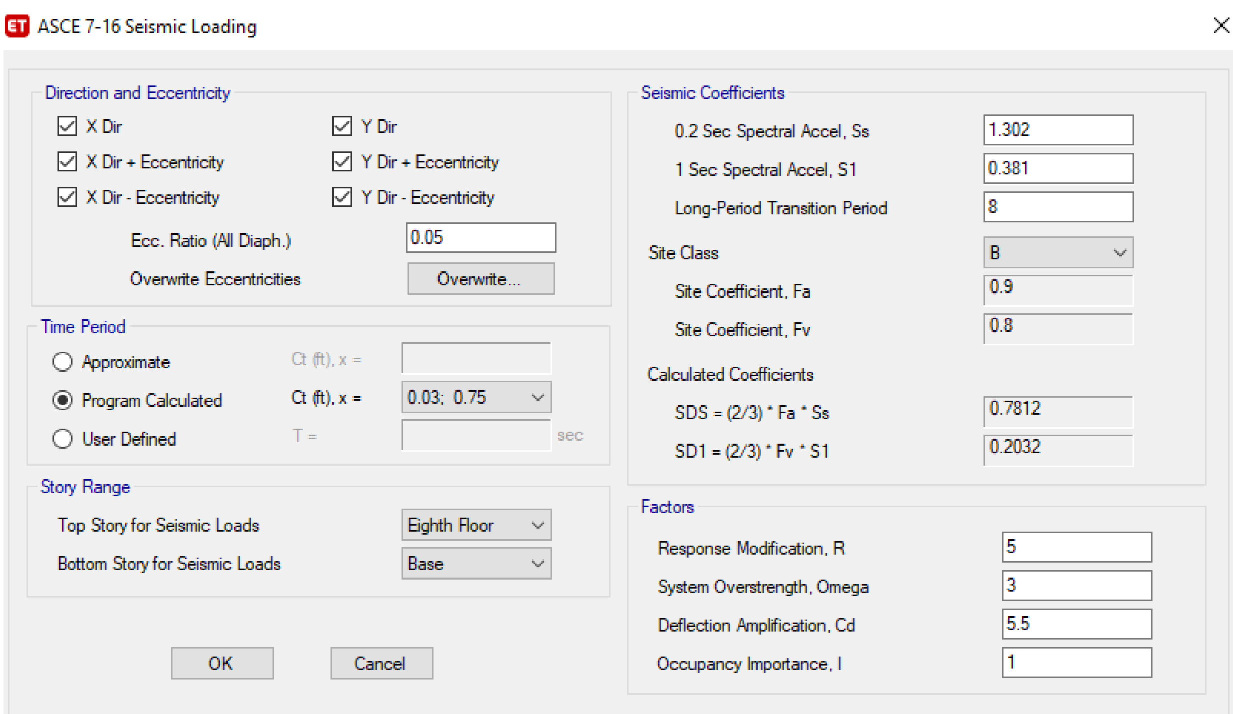


Figure 20: Definition of the seismic load pattern

After defining load patterns, the mass source was defined as the effective seismic weight which includes dead loads plus 25% of the live load as per the ASCE 7-16. All the elements were meshed, and the analysis was run.

3.4.2 Nonlinear Time history Analysis:

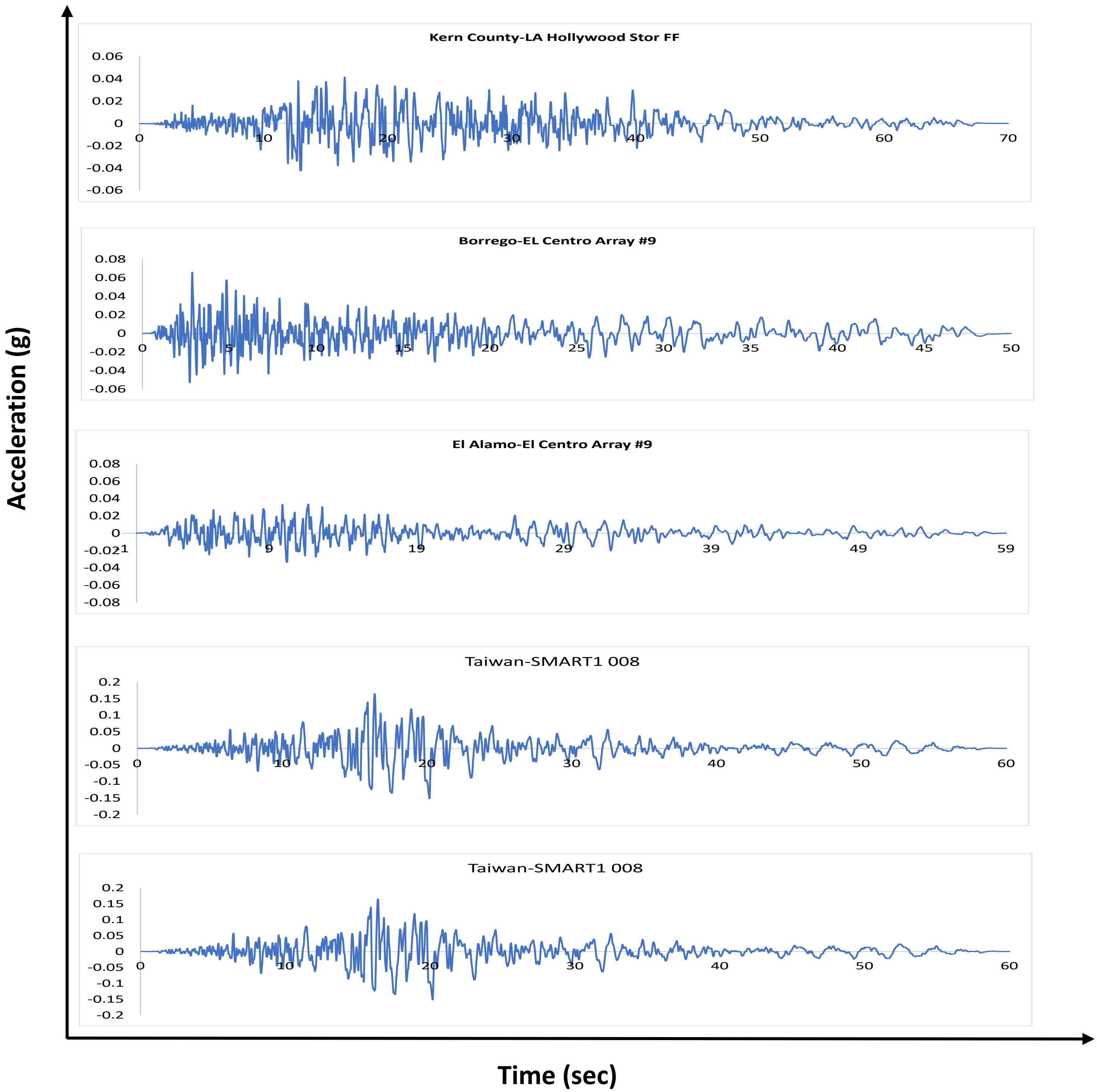
It is known as Dynamic analysis. It is an important technique for structural seismic analysis especially when the evaluated structural response is nonlinear. To perform such an analysis, a representative earthquake time history is required for a structure being evaluated. Time history analysis is a step-by step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading of representative earthquake (Wilkinson and Hiley, 2006).

Selecting the seismic loading for design and/or assessment purposes is not an easy task due to the uncertainties involved in the very nature of seismic excitations. One possible approach for the treatment of the seismic loading is to assume that the structure is subjected to a set of records that are more likely to occur in the region where the structure is located. For that purpose, 5 different ground motions were selected according to the selection criteria shown in table 3. These earthquakes/ground motions represented our site conditions, and they were obtained from Pacific Earthquake Engineering Research Centre (PEER).

Sr. No.	Selection Criteria	Values	Reason
1	Fault Type	Reverse/Oblique	The closest fault and most contributing to Karachi and hence site is Nagar-Parkar Fault which is a reverse fault
2	Magnitude	6.4 to 8.2	On average Nagar-Parkar fault causes earthquakes of this magnitude range
3	R_{JB} (Km)	40 to 150 Km	Distance of Nagar Parkar fault to Karachi city
4	R_{RUP} (Km)	40 to 150 Km	Distance of Nagar Parkar fault to Karachi city
5	V_{S30} (m/s)	180 to 360 m/s	Corresponding to Class D
+6	D5-95 (sec)	30 to 50 sec	To ensure number of cycles for peak response
7	Pulse	No Pulse-like Records	Not Applicable

Table 3: Selection criteria for the ground motions

The original time histories of selected earthquakes are shown below.


Acceleration (g)
Kern County-LA Hollywood Stor FF
Borrego-EL Centro Array #9
El Alamo-El Centro Array #9
Taiwan-SMART1 008
Taiwan-SMART1 008
Time (sec)

As the original time histories were unable to meet site target so for that a site target spectrum was obtained from the building code of Pakistan and spectral matching the target spectrum was done.

Below are matched spectra of the earthquakes.

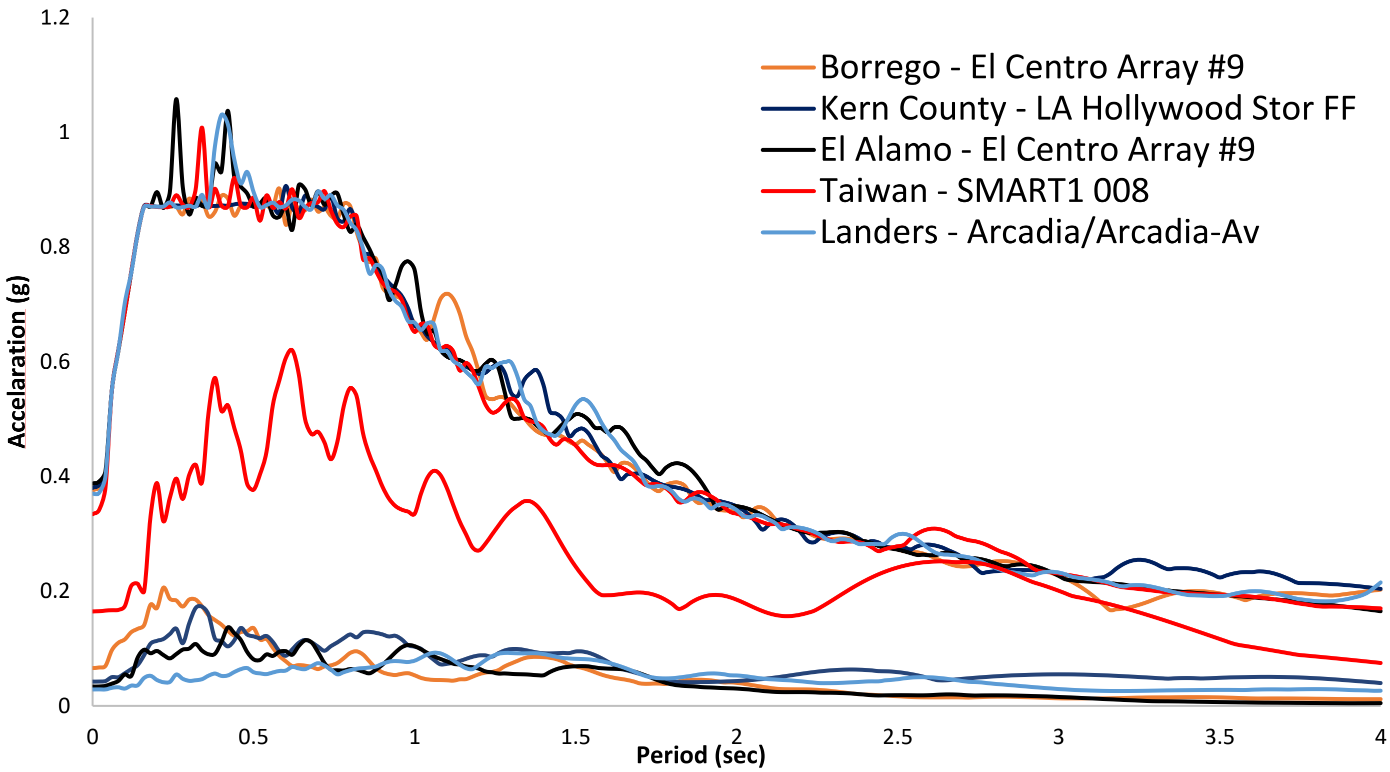


Figure 21: Original and matched spectra of selected time histories

Finally, in ETABS the nonlinear load cases were defined. Nonlinear dynamic analysis for was run for all the earthquakes to get the first set of performance results. It took us a while to complete this since it is time consuming process even the analysis run can take up to 12 hours. This marked completion of our case study part of our project.

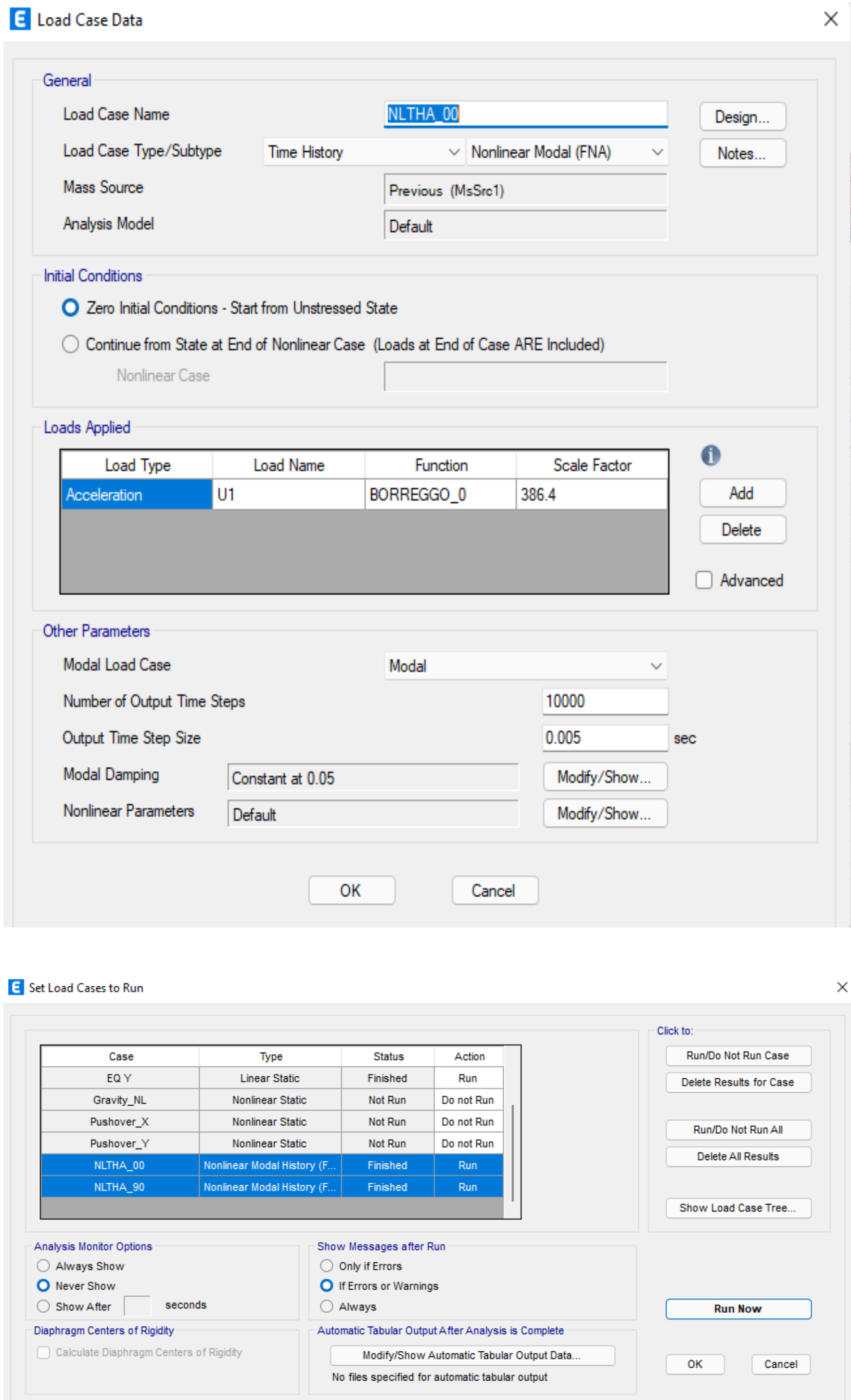


Figure 22: Non-linear load cases definition

CHAPTER 4: Development of Fibrica

4.1: Front-End of Fibrica

The software was developed in Excel VBA and each of the module discussed below can be seen in the corresponding sheets. This approach intended for our software to not act as a black box rather the processing results be available to users wanting to access. The software is named **"FIBRICA"** which at present is a Non-Linear cross section analysis software.

The development included the front and back-end of the software. Excel provides with an Object-Oriented Programming (OOP) based approach to design the user-form which is the Graphical User Interface (GUI) of the software. The GUI consists of 4 tabs

- Cross-Section Properties
- Material Properties
- Analysis
- Results

4.1.1: Cross-Section Properties Tab

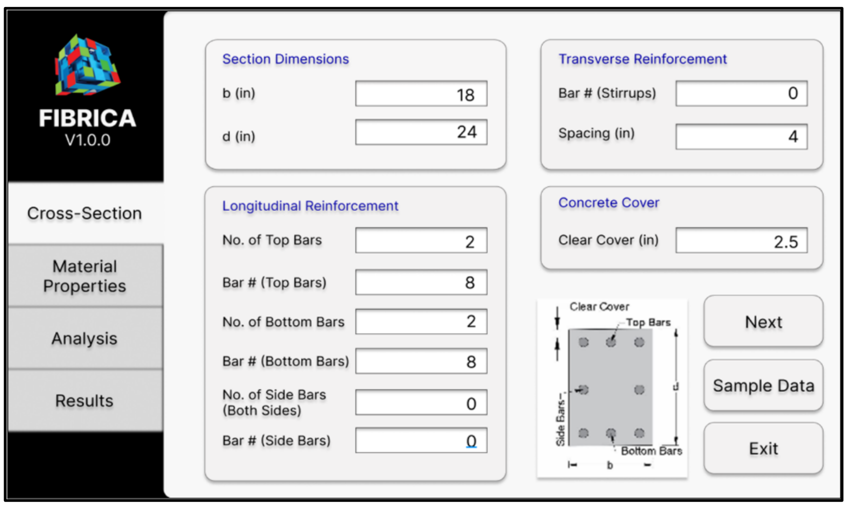


Figure 23-A: Cross-Section properties tab of Fibrica

This tab takes input from the user which then defines the cross-section accordingly. The input parameters include the section dimensions, the reinforcement details, and the concrete cover. This is where the software gets an overall picture of the cross-section and divides it into smaller individual fibers.

4.1.2: Material Properties Tab

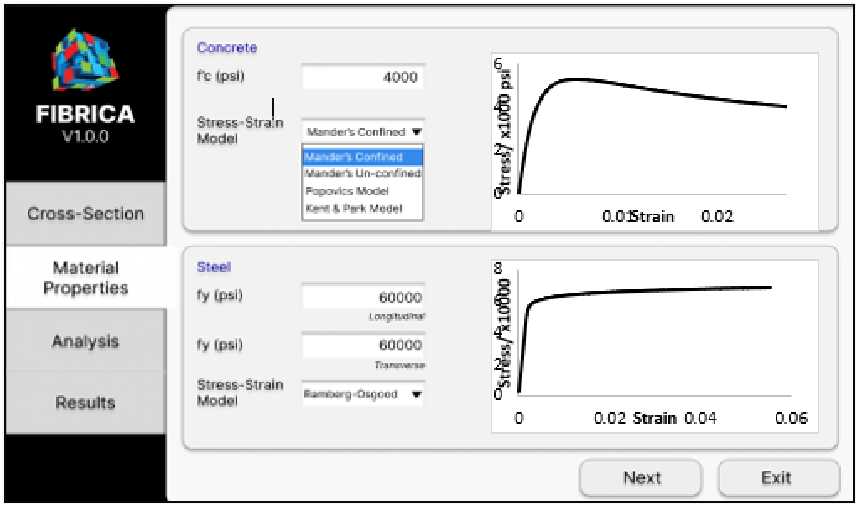


Figure 23-B: Material properties tab of Fibrica

This tab takes the required stress-strain model for both the concrete and the steel and their corresponding strengths. Plots of the model being used can be seen on the screen aswell.

The different concrete models that Fibrica allows the user to choose from are:

- Mander’s Model (Confined and Unconfined)
- Popovics Model
- Kent & Park Model

And the options for steel models include:

- Ramberg-Osgood Model
- Elastic Perfectly Plastic Model (EPP)

After the material stress-strain models have been assigned by the user the software assigns the respective models to the fiber that were created in the previous section.

4.1.3: Analysis Tab

This tab is where cross-sectional analysis parameters are defined. The parameters include the type of frame element under study, the acceptance levels and the load actions obtained through linear analysis of the structure.

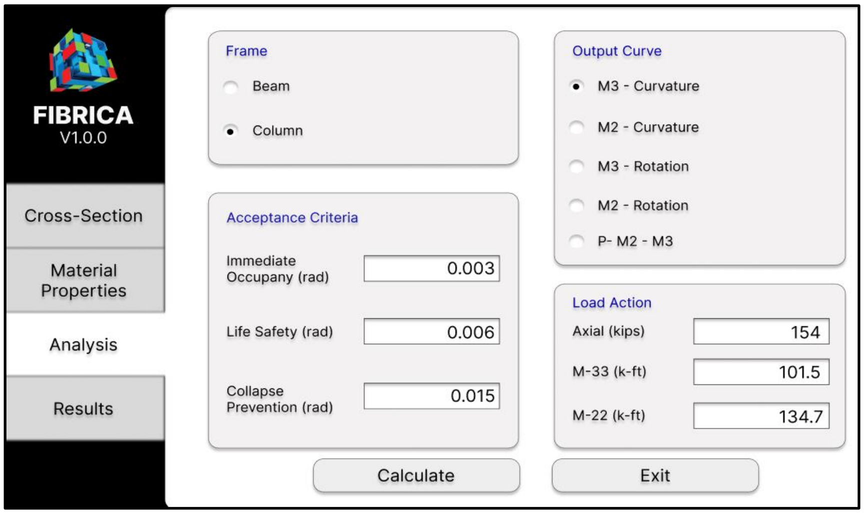


Figure 23-C: Analysis tab of Fibrica

4.1.4: Results Tab

After the cross-sectional analysis have been completed, the performance results can be seen in this tab. It shows capacity curve of the frame element selected along with the demand point. The performance of the frame element can be directly predicted using this output.

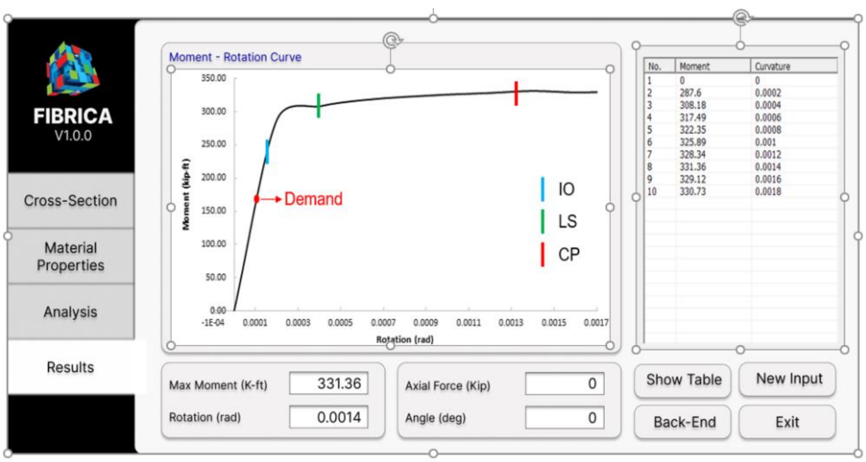


Figure 23-D: Results tab of Fibrica

4.2: Back-end of Fibrica

The program was coded in form of modules, each performing a specific functionality. The modules were linked in a complex branched system to perform the cross-sectional analysis.

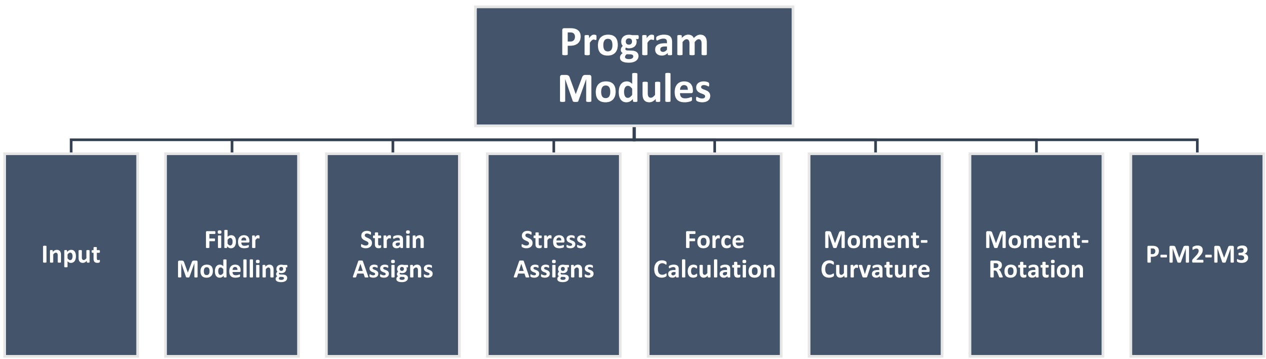


Figure 24: Program modules

The GUI takes the input from the user and populates the corresponding excel sheet with the input parameters. the program uses the approach of fiber modelling as discussed in the earlier chapters to discretize the cross section under study. It divides the section into a mesh of 3600 fibers and distinguishes on the basis of their coordinates. Fibers representing different materials have different designations. This is the second module of the program.

The strains are calculated against an assumed value of curvature “phi” and is plotted through trial & error method to locate the neutral axis. Once each fiber has been assigned the strain value, the program shifts towards the module of stress assigns. The stress is assigned based on the stress-strain models fed in the input module. The program picks up the stress corresponding to the strain that was assigned to each of the fiber earlier. The program populates the stress sheet of the excel and moves on to the next module. This module calculates the force contribution of each fiber by multiplying the stress to the area of each of the fiber. The forces above the neutral axis contribute in compression and the ones below contribute in tension. This is because the beam is sagging under normal loading. Both the forces form a couple about the neutral axis which give us the moment of the beam against the initially assumed curvature. This process is repeated with curvature increments to plot the Moment-Curvature curve for the beam.

The capacity curves for columns are in form of axial-moment interaction and therefore gives P-M curves which can be converted into a PMM surface. This is because in case of columns, axial force and moment about both the axes interact to give the strength or capacity of the columns. The program follows another modular route to calculate the capacity curves and surfaces for the columns. In this case, we calculate the curvature based upon already determined strain profiles.

The profiles are as follows

1. Pure Compression
2. Bar Stress Near Tension Face of Member Equal to Zero, ($\varepsilon s = fs = 0$)
3. Bar Stress Near Tension Face of Member Equal to 0.5 fy, ($fs = -0.5\ fy$)
4. Bar Stress Near Tension Face of Member Equal to fy, ($fs = -fy$)
5. Bar Strain Near Tension Face of Member Equal to 0.005 in./in., ($\varepsilon s = -0.005$ in./in.)
6. Pure Tension

The curvature against each profile is used to calculate the strain of each fiber and the process repeats as in the case of beams. To plot the surface, the plot rotates the angle of neutral axis in 15-degree increments and repeats the whole process for each of the orientation. This calculates the curves for each of the angle. The curves can then be used to plot a 3-D capacity surface for the columns.

CHAPTER 5: Analysis and Results

The results and discussion part has been divided into 2 parts namely comparison of sectional analysis results and the comparison of performance assessment results. The aim is to first validate the credibility of the new software and then to see how it can be used in the real-life design practice and in field. For the validation part first, the cross-sectional analysis will be used, and then performance-based assessment, and once that has been successfully achieved, the report looks into the implementation part.

For the first comparison Fibrica's section analysis results were compared to those obtained from Etabs section designer. Section designer is a tool within Etabs that can perform cross-sectional analysis. An example cross-section shown in figure 25-A was used to generate moment-curvature curves using both softwares. The curves generated are shown in figure 25-B. The curve in red was generated using Etabs section designer whereas that in black was generated using Fibrica. As it can been seen both the curves are highly consistent which validates the credibility of the results obtained using Fibrica.

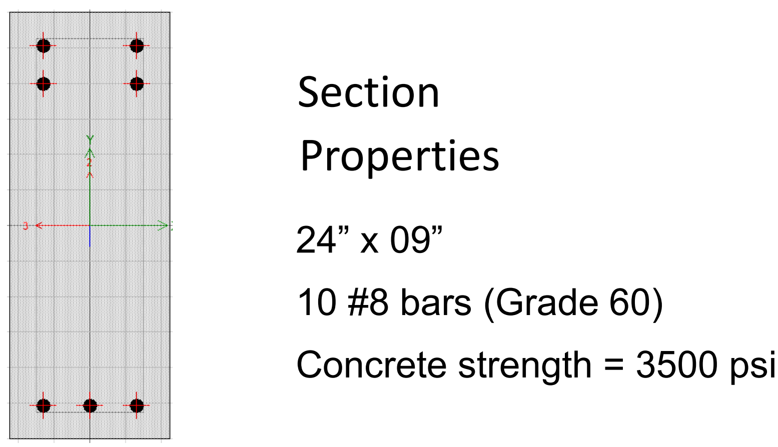


Figure 25-A: Example cross section under consideration

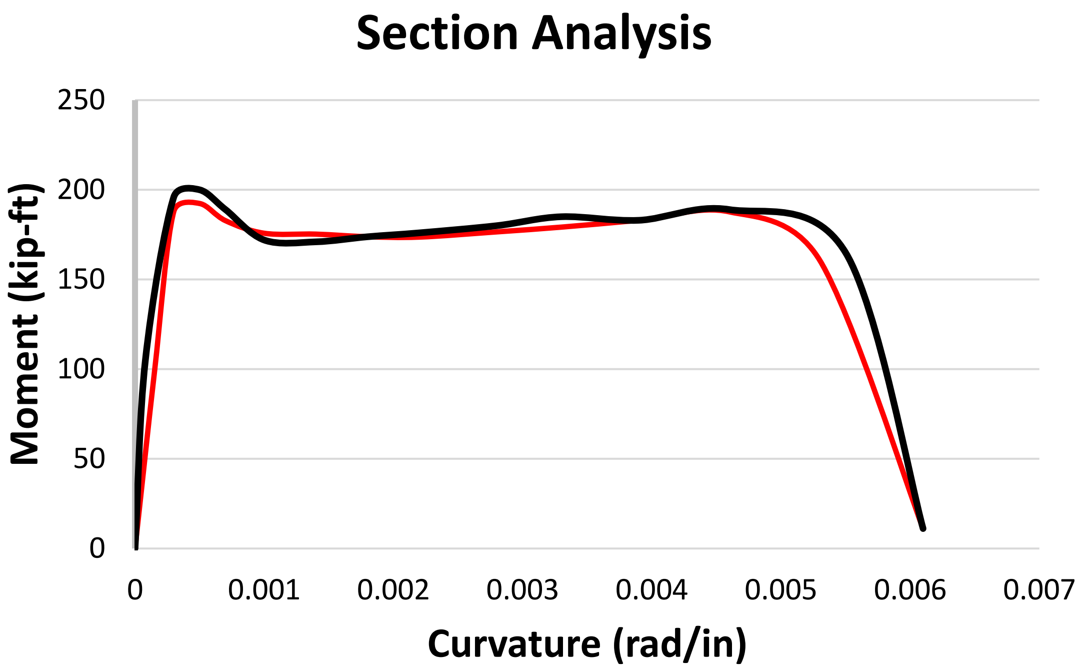


Figure 25-B: Comparison of results from Fibrica and Etabs

Moving on, detailed seismic assessment of the case study structures was carried out using the full performance-based design procedure on Etabs. The structures were evaluated against ground motions that were selected as mentioned in section 3.4.3, and performance results were extracted for both structure and member level. For the sake of this comparison, the 3rd case study structure, and its results against the Borrego-EL Centro Array #9 earthquake, obtained after rigorous non-linear time history analysis, are being considered. The structural responses are shown in figure 26-A. Theses results give an idea about the maximum storey drifts, displacements, and overturning moments. Figure 26-B shows that the the structure under consideration is under the 'Immediate Occupancy' level and a further look into the results shows that the structure is not even utilizing 50% of its 'Immediate Occupancy'. However, since the whole idea was to carry out the detailed performance assessment on a member level, we are more interested in the member-level results.

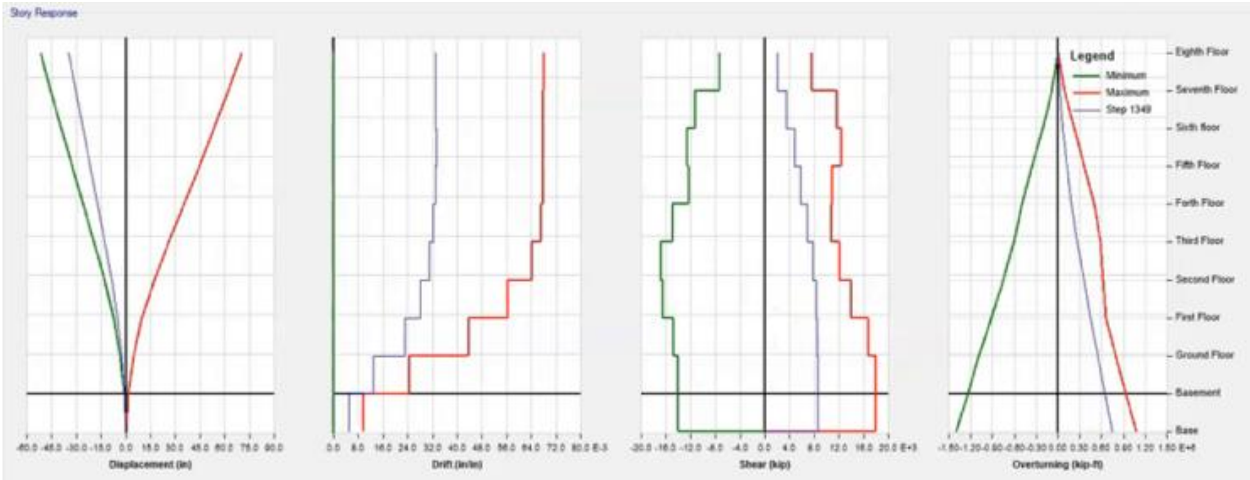


Figure 26-A: Combined storey response plots

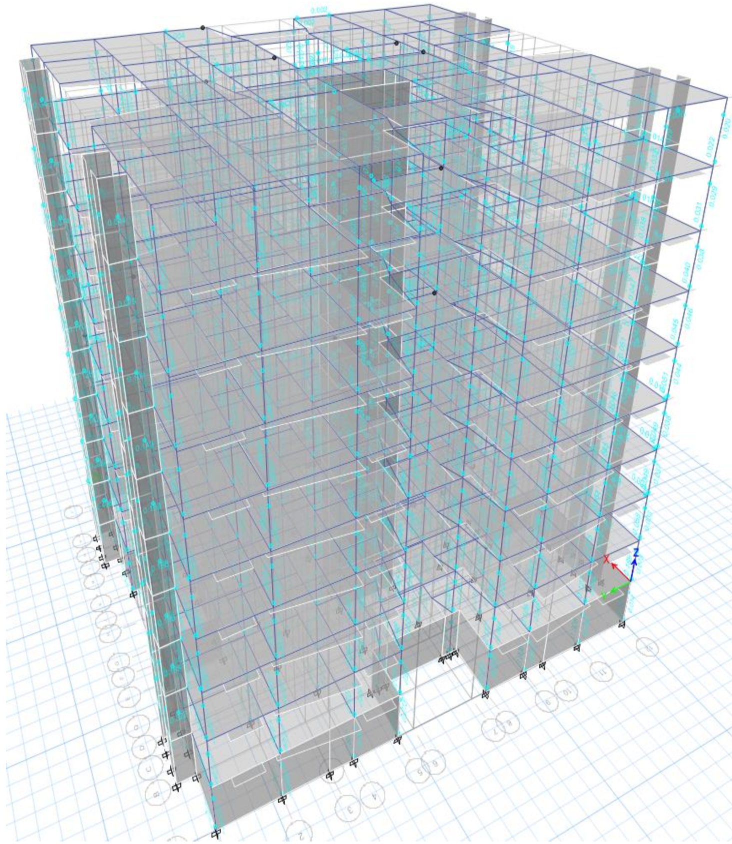


Figure 26-B: Performance level of the formed hinges

For this part of the study, the performance results of an example beam from the above-mentioned structure will be compared to those obtained from Fibrica for the same member. As expected, figure 27 shows that the beam is over-designed, so much so that it only utilizes only a minute fraction of its actual capacity. The beam lies below the IO level. Lets now take a look at what Fibrica's predicted performance for the beam is. Firstly, to again reinforce the credibility of the new software, figure 28-A shows the comparison between the capacity curves generated using Fibrica and section designer. Secondly, figure 28-B shows the capacity curve, obtained from Fibrica, along with the performance point which came through the linear time history analysis of Etabs (figure 28-C). The performance predicted by Fibrica for the same cross-section is similar to the one obtained through the full PBD procedure. This part validates the credibility of the simplified framework proposed and establishes that this simplified framework can indeed give us similar results to the ones obtained through the full detailed PBD procedure.

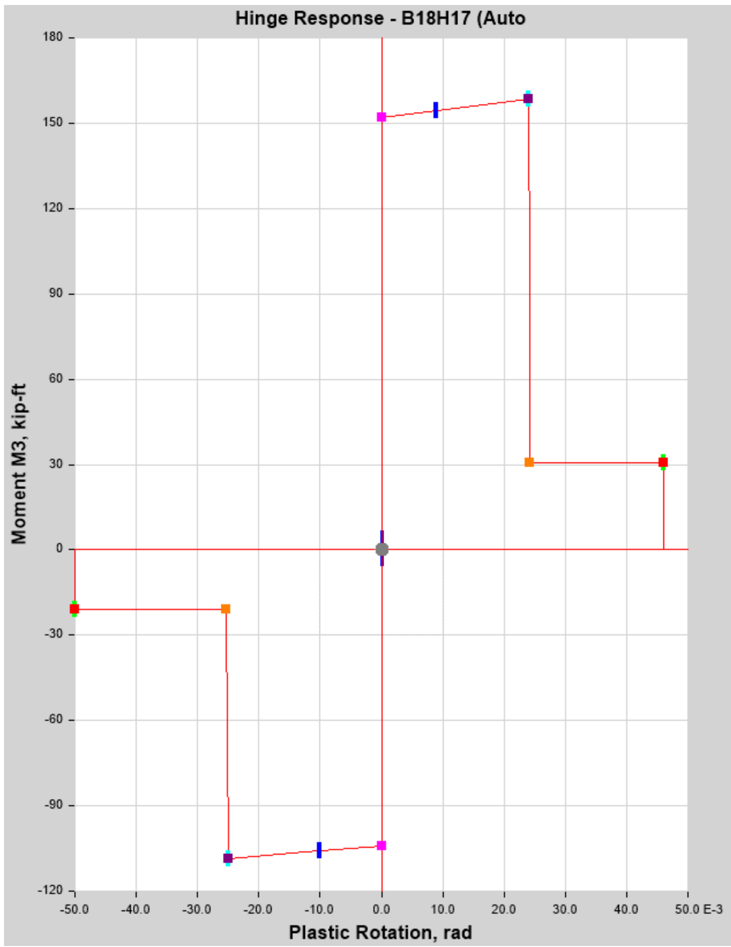


Figure 27: Example beam's capacity curve and corresponding demand

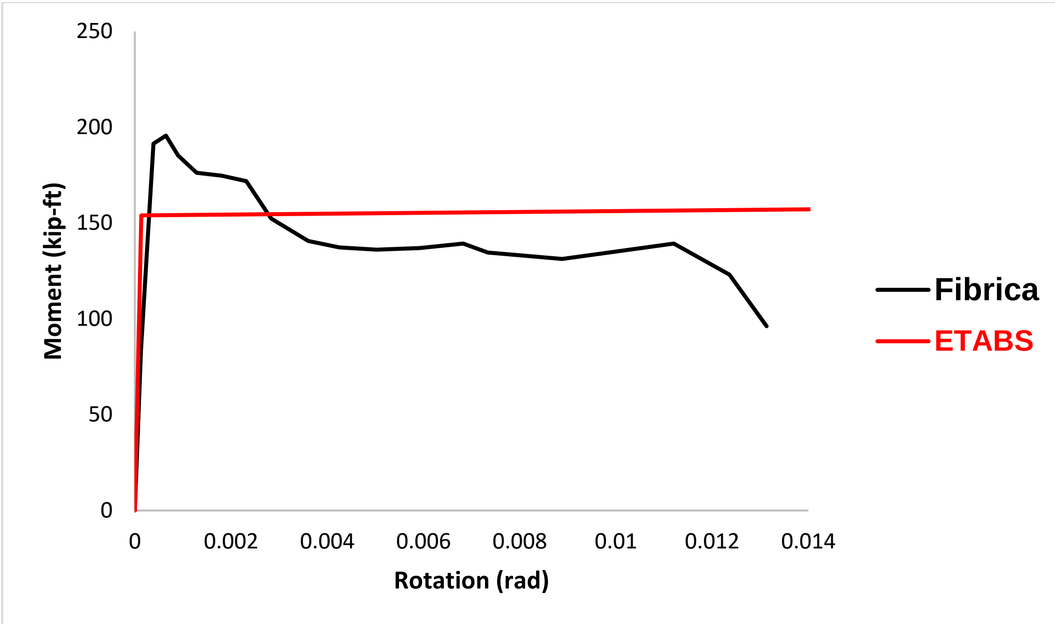


Figure 28-A: Capacity curves obtained from Etabs (ASCE 41) and Fibrica

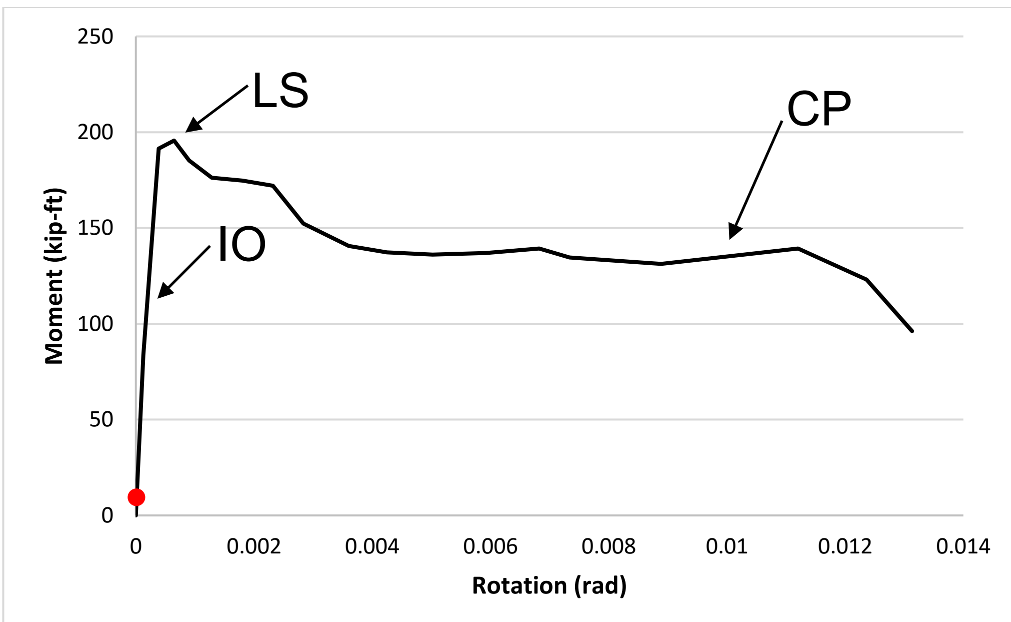


Figure 28-B: The capacity curve from Etabs and the demand point from Linear time history analysis of Etabs as per figure 45

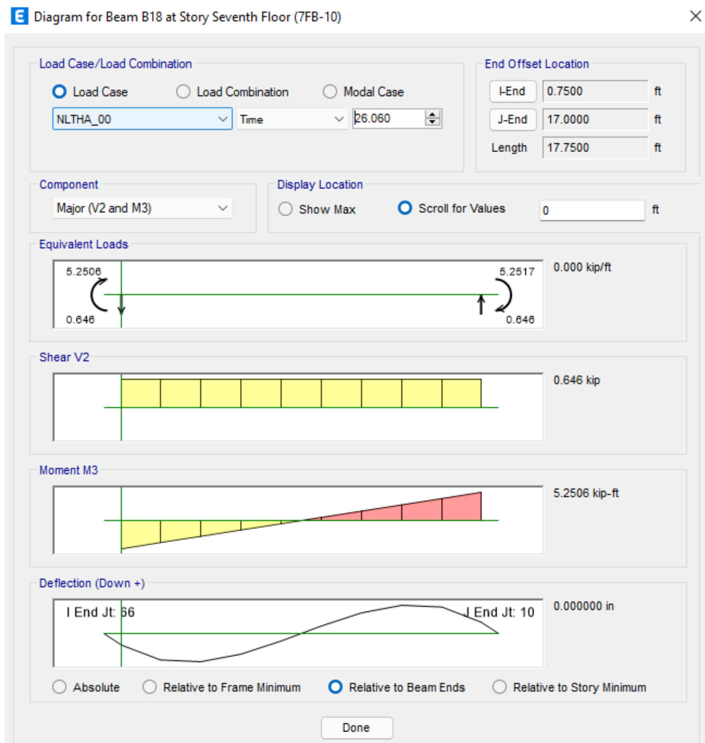


Figure 28-C: The linear time history demand from Etabs

Finally, when it comes to the designing of structures, the design is supposed to be of a sufficient capacity but at the same time it should be economical. After a design iteration, if the designer finds out that the design is under-designed or over-designed, the design must be revised. With the use of the detailed procedure of PBD, as mentioned earlier, it is going to take months before we have finalized a design, that is appropriate. Fibrica helps to bring that time down to just days. Figure 47 A to C shows how easy and fast it is to optimize a member cross-section with Fibrica. An example cross section as shown in figure 46 was considered, again from the above-mentioned structure. With the help of just 3 iterations, which took less than 30 minutes, a cross-section size that is economical and is utilizing just less than its IO-level capacity is achieved.

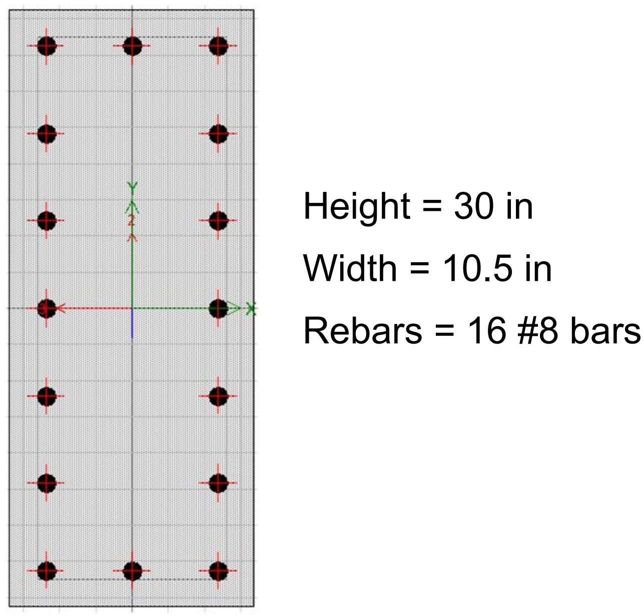


Figure 29-A: Example column cross-section

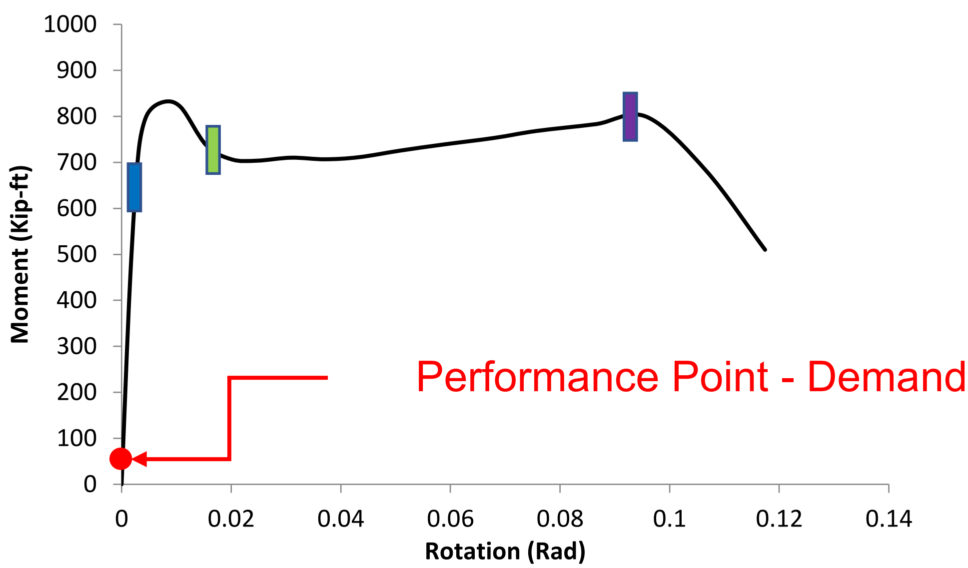


Figure 29-B: 1st iteration

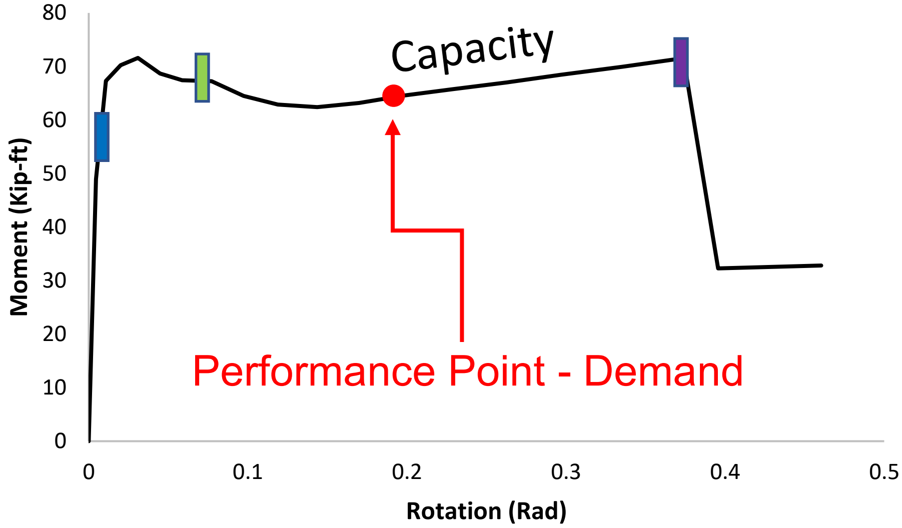


Figure 29-C: 2nd iteration

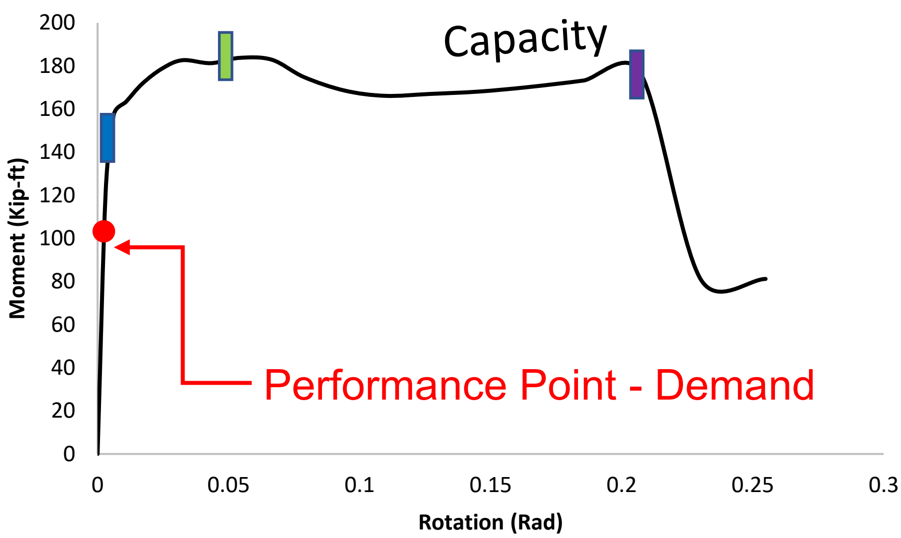


Figure 29-D: 3rd iteration

Figure 29: Design iterations using Fibrica

CHAPTER 6: Conclusion

With the development of the simplified framework, the following have been achieved by this study:

1) The full Performance-based Design procedure requires a great level of expertise. A high level of knowledge and understanding is required to understand the complex non-linear bahaviour of the materials that are present in the structure and hance the overall analysis as well. Apart from the knowledge and understanding, the designer needs to have a polished skill set to create a non-linear model and perform non-linear analyses. As mentioned in this study, it is a very lengthy and complicated process to create a non-linear model of the structures. A lot of decisions are being made when a non-linear model is being created, and hence without the proper knowledge of the different parameters and modeling considerations a comprehensive non-linear model cannot be created. Basically, the overall process is very complex, and an average structural designer is not skilled enough to opt for performance based design. With Fibrica, the complexity involved in the entire process is greatly reduced. With just the definition of the cross-section and the assignment of material properties one can generate performance results of the a given cross-section. Furthermore, the demand point that is used as an input in Fibrica comes from the linear time history analysis and not the non-linear time history analysis. Linear time history analysis does not require a non-linear model, making it way simpler than the non-linear time history analysis and saves a lot of time as well.
2) As mentioned earlier the entire process of a full performance-based design is highly complex, time consuming and tedious. However, apart from that it even requires a high amount of computational effort and extensive data processing. Once non-linear analyses are complete you are presented with a lot of data, and without proper understanding the designer will not know what to do with it. Just the interpretation of the obtained data and extracting something useful out of it is a very demanding job. With the simplified framework all these problems have been solved. The entire analysis procedure takes minutes now instead of weeks or months. There is no need of a highly amount of computational effort and the data presented to the user is very simple that in the form of a capacity curve, while also giving the entire picture of the performance of the cross-section.
3) In the field of structural designing the decision of initial cross-section sizing is very important. Initial sizing usually decides how many iterations it would take to reach the final design. However, for designer the initial cross-section sizes are just a guess or at best are rough estimates. There is no proper framework for the initial sizing and preliminary design of cross-sections. This entire process gets better and more efficient as the designer gets more and more experienced. However, for fresh structural engineers who are just starting in the field, it is harder and makes their work less efficient and effective. With Fibrica, these fresh structural engineers ,or any structural engineer for that matter, can use Fibrica not just as a tool for the simplified performance-based design procedure but also the for the preliminary design and initial sizing of the cross-sections.

4) In performance-based design the entire goal for the designer to achieve is to reach a certain level of performance which is accepted by the client. The performance of the structure depends upon the material properties and the cross-section sizes. When a design iteration does not predict a performance that is above par with the desired performance, the designer reiterates the entire process using different cross-section sizes. So the entire process and the number of iterations depends upon the proper selection of cross-section sizes. As mentioned in the previous point, that Fibrica helps to make the entire process of preliminary design much more efficient, the designer can use the optimized cross-section sizes obtained from Fibrica in the full detailed performance-based design procedure to reduce the overall number of iterations it took to reach the final design.

CHAPTER 7: References

1. Ahmed, G. (2001). Performance-Based Design In Earthquake Engineering: State Of Development. *Engineering Structures*. *DOI:* 10.1016/S0141-0296(01)00036-0
2. Xuemei, Z., Yu-Fei, W., A.Yt, L., Heung, L. (2011). Plastic Hinge Length in Reinforced Concrete Flexural Members. *Procedia Engineering. DOI:* 10.1016/j.proeng.2011.07.159
3. Yusuf, S. (2019). Determination of Plastic Hinge Length for RC Beams Designed with Different Failure Modes under Static Load. *Special Issue of the 8th International Advances in Applied Physics and Materials Science Congress. DOI:* 10.12693/APhysPolA.135.955
4. Neena, P., K.S, B. (2013). Effect Of Variation Of Plastic Hinge Length On The Results Of Non-Linear Analysis. *International Journal of Research in Engineering and Technology*.
5. A. Marchis, T. Moldovan, A. Loani. (2018). Progressive Collapse Potential Of Seismically Designed Rc Framed Structures Subjected To Column Removal
6. Michael, S., Keri, R. (2013). Moment-Rotation Behavior of for ce-Based Plastic Hinge Elements. *Earthquake Spectra*. *DOI*: 10.1193/1.4000136
7. A. Eslami, H.R. Ronagh. (2014). Effect Of Elaborate Plastic Hinge Definition On The Pushover Analysis Of Reinforced Concrete Buildings. *The Structural Design of Tall and Special Buildings. DOI:* 10.1002/tal.1035
8. CSI Analysis Reference Manual (SAP 2000, ETABS and CSI Bridge), Computers and Structures Inc., USA.
9. Graham, P. (2010). Modeling for Structural Analysis, Computers and Structures Inc.