A PYTHON-BASED PLATFORM TO AUOTMATE SEISMIC DESIGN AND NONLINEAR ANALYSIS OF STEEL MOMENT FRAMES

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Abstract

This paper introduces the development of an end-to-end computational platform that could automate the process of conducting seismic design, constructing nonlinear structural models, and performing response simulation (static and dynamic) of steel moment resisting frames. The seismic design module is able to iteratively generate code-conforming section sizes and detailing for beams, columns, and beam-column connections based on the relevant input design variables such as the building configuration (e.g., the numer of stories, the number of lateral-force resisting systems, and the building dimensions), loads (e.g., dead and live loads on each floor), and site conditions (mapped MCE_R spectral acceleration parameters). The nonlinear model generation module takes the design results as input, automates the construction of nonlinear structural models that capture the flexural strength and stiffness deterioration and performs the pushover and response time history analyses. Three illustrative examples are presented to demonstrate the reliability, accuracy, and efficiency of the developed platform. The significance of this platform is a drastic reduction in time and effort involved in conducting iterative structural designs and nonlinear analyses, both of which are necessary for performance-based seismic design. Additionally, the platform could be used to develop an extensive database of archetype steel moment frame buildings towards the development of analytics-driven design methods.

Keywords: computational platform; seismic design automation; nonlinear structural analysis; archetype buildings

1 Introduction

The archetype concept has been extensively used in recent years to investigate the seismic performance of building structures. Some of the earliest work in this area utilized the archetype approach to assess the collapse risk of both modern (ductile) and older (non-ductile) reinforced concrete (RC) special moment frame buildings [1]. The Applied Technology Council (ATC) developed a systematic approach to assess seismic design provisions for later force resisting systems (LFRS's)based on archetype buildings/models [2]. Soon after, the archetype concept was adopted as part of the FEMA P695 [3] guidelines, which provides a comprehensive methodology for quantifying the collapse vulnerability of different LFRS's. Since then, several other studies utilizing archetypal buildings have been published (e.g., [4, 5]). Given the growing significance and popularity of the archetype concept in performance-based earthquake engineering, there is a need to develop a set of computational tools and processes to automate seismic design, nonlinear structural model construction and seismic response analyses (static and dynamic).

This study introduces a Python-based platform that is able to automate the seismic design, nonlinear structural model generation and response analyses for steel moment resisting frames (SMRF's). The Automated Seismic Analysis and Design (AutoSAD) platform is developed using the object-oriented programing paradigm and has the two modules shown in Fig. 1. This first AutoSAD module takes in building geometry and load information as input and generates code-conforming designs (e.g., SMRF member sizes and beam-column joint details). The second module takes in the design information generated by the first module as input and constructs two dimensional (2D) nonlinear structural models. It further automates the process of conducting nonlinear static and dynamic analyses and post-processing the results. By utilizing the AutoSAD platform, less time and resources are needed to design the archetype buildings, construct and analyze the associated nonlinear structural models and post-process the results, thus reducing some of the challenges associated with utilizing the archetype concept. Moreover, while the current version of the AutoSAD platform is specific to SMRF's, it is developed and presented in a generalized manner such that it can be extended to other LFRS's. The remainder of the paper begins by introducing the algorithms and programming structures used for developing AutoSAD. Then, three illustrative examples are provided to demonstrate the reliability and efficiency of the platform. Finally, the features of the AutoSAD platform are compared with these of commercial design software to illustrate its advantages and limitations.

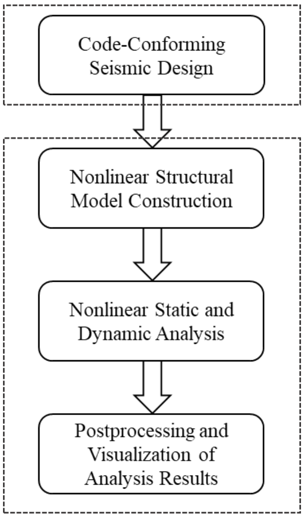


Fig. 1 – Overview of the main AutoSAD platform modules

2. Seismic Design Module

2.1 Overview

The seismic design module is developed based on the design criteria specified in the relevant building code and design standards [6–9] and currently only equivalent lateral force (ELF) analysis method is adopted to design SMRFs. An overview of the workflow for the seismic design module is presented in Fig. 2. A one-time preprocessing of the electronic steel section database is first performed. Then, the relevant seismic design parameters are received as input, beam and column sizes are initialized, and the member-sizes are adjusted to satisfy the drift requirement. Subsequently, the beams, columns, and connections are checked to ensure that they satisfy the relevant strength and detailing requirements and the member sizes are revised as needed. Lastly, the member sizes are adjusted to account for ease of construction and the final design is generated. The details of each of the main steps in the procedure are presented in the following sections.

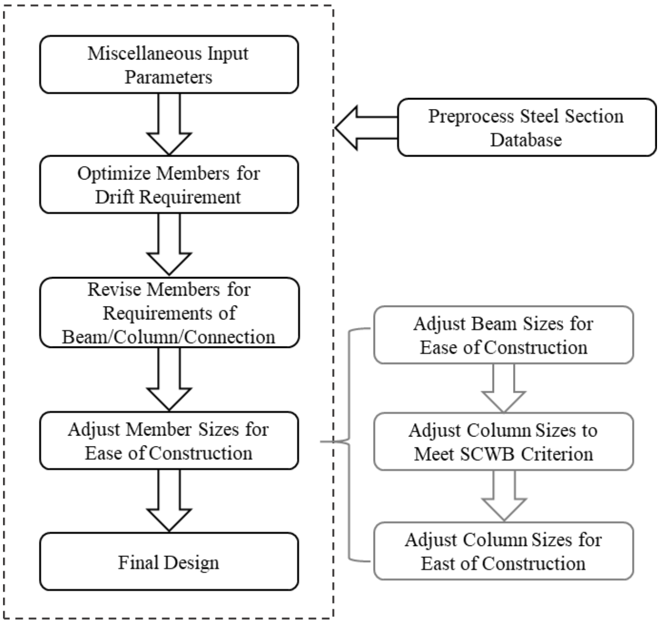


Fig. 2 – Overview of the seismic design module

2.2 Preprocessing the Electronic Database of Wide Flange Sections

To facilitate the iterative adoption of SMRF beam and column sizes, the electronic database of wide flange sections provided by the American Institute of Steel Construction (AISC) is pre-processed. The entire database includes all wide flange shapes that are currently manufactured in the industry. However, only those satisfying the high ductility requirement can be used as a SMRF column or beam section. Moreover, the adoption of RBS connections introduces more stringent requirements on the beam and column sections. More specifically, the section depth, weight, and flange thickness of beams must be less than W36, 300 lb/ft, and 1.75 inches, respectively. Also, column section depths must be less than W36. Based on these requirements, the original database is filtered to create two new sub-databases of all possible beam and column sections. The section sizes are listed in descending order of the moment of inertia and the plastic section modulus in the column and beam sub-database, respectively. An index beginning from zero and incremented at values of one is attached to each sub-database to denote the order of section sizes (zero represents the strongest section).

2.3 Design Automation Algorithm

The sub-algorithm used to optimize the member sizes to meet the relevant drift requirement is presented in Fig. 3. Two important coefficients are predefined by the user: the moment of inertia ratio between the exterior and interior columns ($I_{col,ext}/I_{\mathrm{col,int}}$) and the plastic section modulus ratio between the beam and interior column ($Z_{\mathrm{bm}}/Z_{\mathrm{col,int}}$). These two coefficients impose additional constraints on the beam and column section sizes, which will be enforced throughout the entire design process. Based on a review of industry-generated SMRF designs [10], ($I_{col,ext}/I_{\mathrm{col,int}}$) typically ranges from 0.6 to 0.8. The typical range for ($Z_{\mathrm{bm}}/Z_{\mathrm{col,int}}$) is 0.7 to 0.8 for buildings with less than 10 stories and 0.45 to 0.7 for taller buildings.

As shown in Fig. 3, after taking in the relevant input parameters, the algorithm begins by initializing all the beams and columns in the SMRF with the maximum allowable sizes specified in their respective section sub-database. Then a linear elastic model of the SMRF is constructed in the Open System for Earthquake Engineering Simulation (OpenSees) [11] platform and subjected to equivalent lateral story forces in accordance with ASCE 7 Section 12.8. Subsequently, the story drifts obtained from the elastic analysis are compared with the allowable limit: 2% at the design basis earthquake (DBE) hazard level. The design module also allows the user to specify a different drift limit based on a desired performance objective. Fig. 3 summarizes the "brute force" approach that is used to target the desired design drift level. First, the story that has the minimum drift is identified and labeled as the "target story". The size of the interior columns in the "target story" is decreased such that the new section is just one index higher than the previous one in the column section sub-database. Then, the design of the beams and exterior columns is revisited to ensure that ($I_{col,ext}/I_{\mathrm{col,int}}$) and ($Z_{\mathrm{bm}}/Z_{\mathrm{col,int}}$) is within some acceptable tolerance of the value defined by the user. At this stage, new section sizes are assigned to the members in the "target story" and the elastic analysis is repeated to obtain updated drift demands and member forces. This process is repeated until the maximum story drift exceeds 2%, which is followed by a comparison between the sections in the current and initial designs (which is based on the maximum allowable sections). If these two designs are the same, the implication is that no valid design exists within the member sub-databases. Otherwise, the current member sizes are reduced by one index at a time until the design drift is less than (but closest to) the allowable limit. At this stage, the member sizes have been optimized to meet the drift requirement. Since P-Δ effects are explicitly included in the elastic analysis, a frame stability check is not implemented.

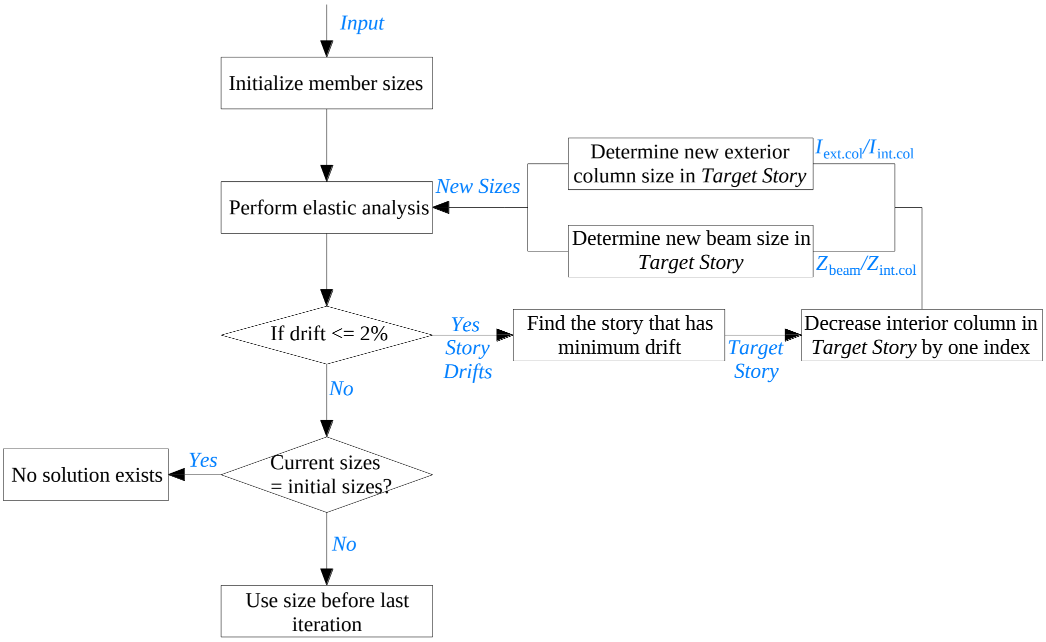


Fig. 3 – Overview of sub-algorithm used to achieve the desired target drift demand

As shown in Fig. 4, after the member sizes are proportioned to meet the drift requirements, component-by-component checks are performed. While SMRF beam and column sizes are typically governed by the drift requirements, strength evaluations are still performed to ensure the robustness of the algorithm. Each column is individually evaluated to ensure that the axial, shear, flexural, and axial-flexural strength requirements specified in ANSI/AISC 360 [8] Chapters E3, G2, F2, and H1, respectively, are satisfied. If a column does not satisfy at least a single strength requirement, its section size is increased such that the new size is one index lower in the column section sub-database. Based on the newly determined column size, the elastic analysis is repeated to ensure that the drift demands are still within the required limit. Meanwhile, the force demands in each member are updated based on the results of this analysis. This procedure is repeated until all columns in the SMRF satisfy the relevant strength requirements. A similar process is implemented for the beams to ensure that they all satisfy the relevant requirements.

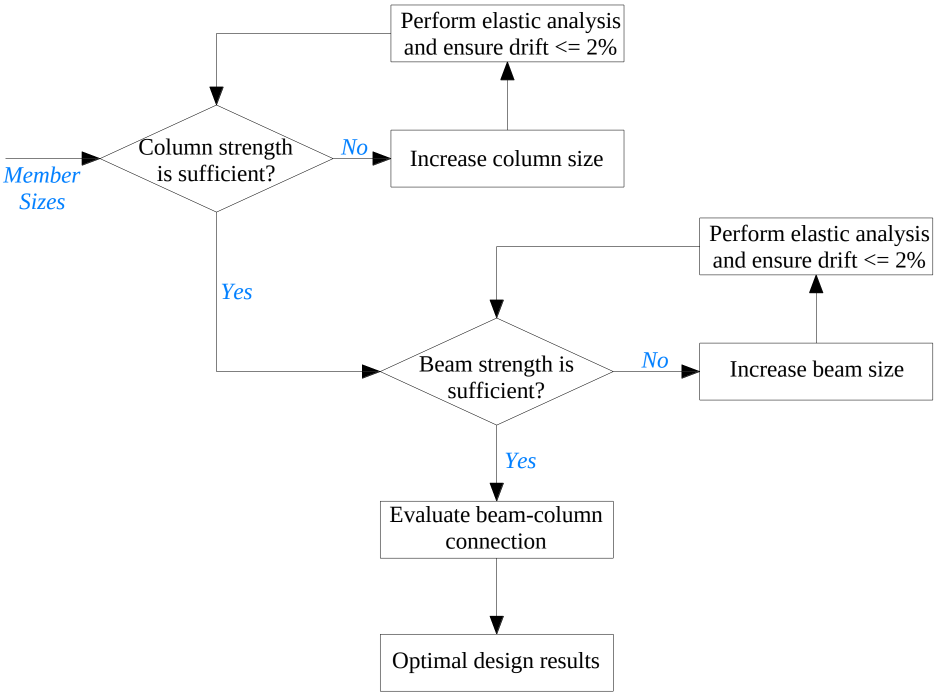
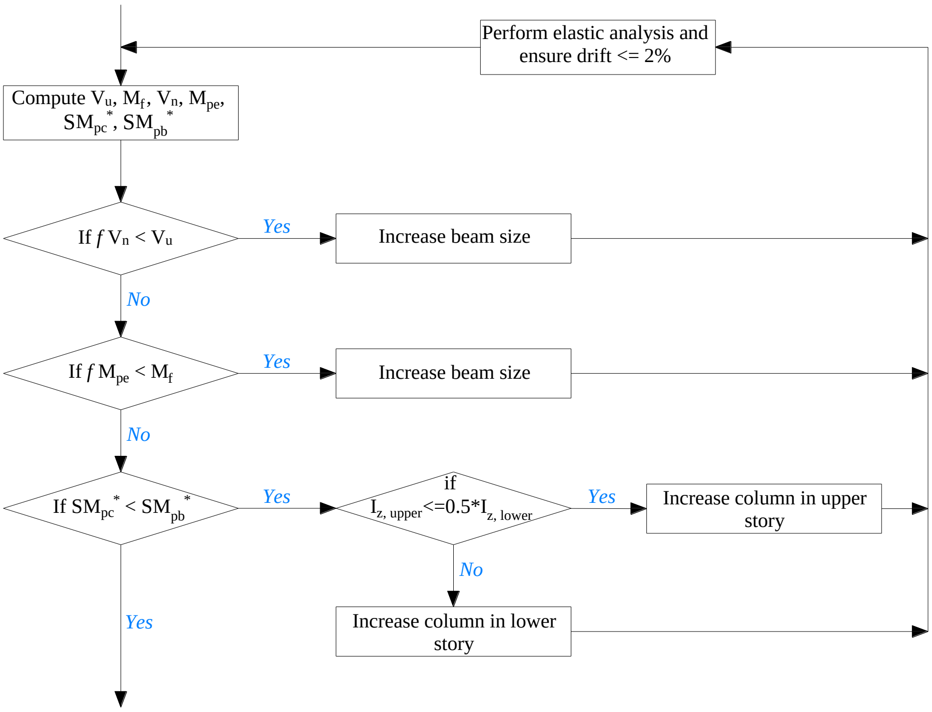


Fig. 4 – Overview of sub-algorithm used to check the feasibility of beams, columns, and connections

The algorithm subsequently evaluates each beam-column connection based on the requirements specified in AISC 358 [12] Chapter 5. More specifically, the beams are checked to ensure that they can resist the demands based on the expected flexural strength at the center of the RBS. The beam-column connections are further checked to ensure that they comply with the SCWB criterion. As shown in Fig. 5, the following parameters are first computed: (1) the required shear strength of the beam (V_u), which is based on the probable moment and shear caused by gravity, (2) the probable maximum moment at the column face (M_f), (3) the shear capacity of the beam (V_n), (4) the plastic moment of the beam based on expected yield stress (M_{pe}), (5) the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines (ΣM_{pc}^*), and (6) the sum of moments in the beams at the intersection of the beam and column centerlines (ΣM_{pb}^*). A typical "switch-case" programing structure is then adopted to evaluate which design requirement is violated and an appropriate ameliorative measure is taken. If the shear or bending moment capacity is found to be less than their corresponding expected demands, the algorithm revises the beam design such that the new section size is one index lower than the old one in the section sub-database. If the SCWB criterion is not met, the column section is increased. Since a typical connection (with the exception of the roof) consists of two columns (one in the upper story and the other in the lower story), the one that is adjusted is determined based on the relationship between the plastic section modulus of those two columns. The column in the upper story is selected if its plastic section modulus is found to be 50% less than the one in the lower story. Otherwise, the column in the lower story is selected. Note that the assumed 50% threshold could be adjusted by the user. The entire process of evaluating the strength and SCWB ratio for the connections is repeated until all members and connections satisfy the design requirements.

At this stage, the algorithm has generated SMRF member sizes that satisfy all design requirements. Subsequently, the member sizes are further adjusted to account for ease of construction. The beam sections are first adjusted such that identical section sizes are used over a specified number of adjacent stories. The detailed algorithm for adjusting beam sizes are presented in Fig. 6. Starting from the roof and moving downward, the beams are grouped by the number of adjacent floors specified by the user. Then, the algorithm determines whether the beams at the current floor level i and the next lower level (i-1) belong to the same group (i.e., whether beams at levels *i* and *i*-1 are supposed to have identical section sizes). If they belong to the same group, the appropriate adjustments are made such that they have the same section size. Otherwise, the algorithm takes another set of different actions to ensure that the beam at level *i*-1 has a larger depth and moment of inertia than the one at level *i*. This process is coded using a compound "if-else" structure and, as shown in Fig. 6, there are ten possible cases in total. It should be noted that the algorithm shown in Fig. 6 is generic and can be used for other LFRS's (e.g., RC moment frames) to ensure ease of construction. After adjusting the beam sizes, the size of those columns that do not meet the SCWB criterion is increased. The previously described process is repeated for columns to ensure that the same section size is used over a user-specified number of adjacent stories.

The algorithm eventually generates a design that complies with the relevant code and standards while accounting for ease of construction. An additional elastic analysis is performed to obtain the updated design information, including the column-to-beam flexural strength ratio, demand-to-capacity ratio, design drifts, etc.



Nomenclature:

V_u: required shear strength based on probable moment and shear caused by gravity

M_f: probable maximum moment at column face based on expected yield strength

V_n: shear capacity of beam

M_{pe}: plastic moment of the beam based on expected yield stress

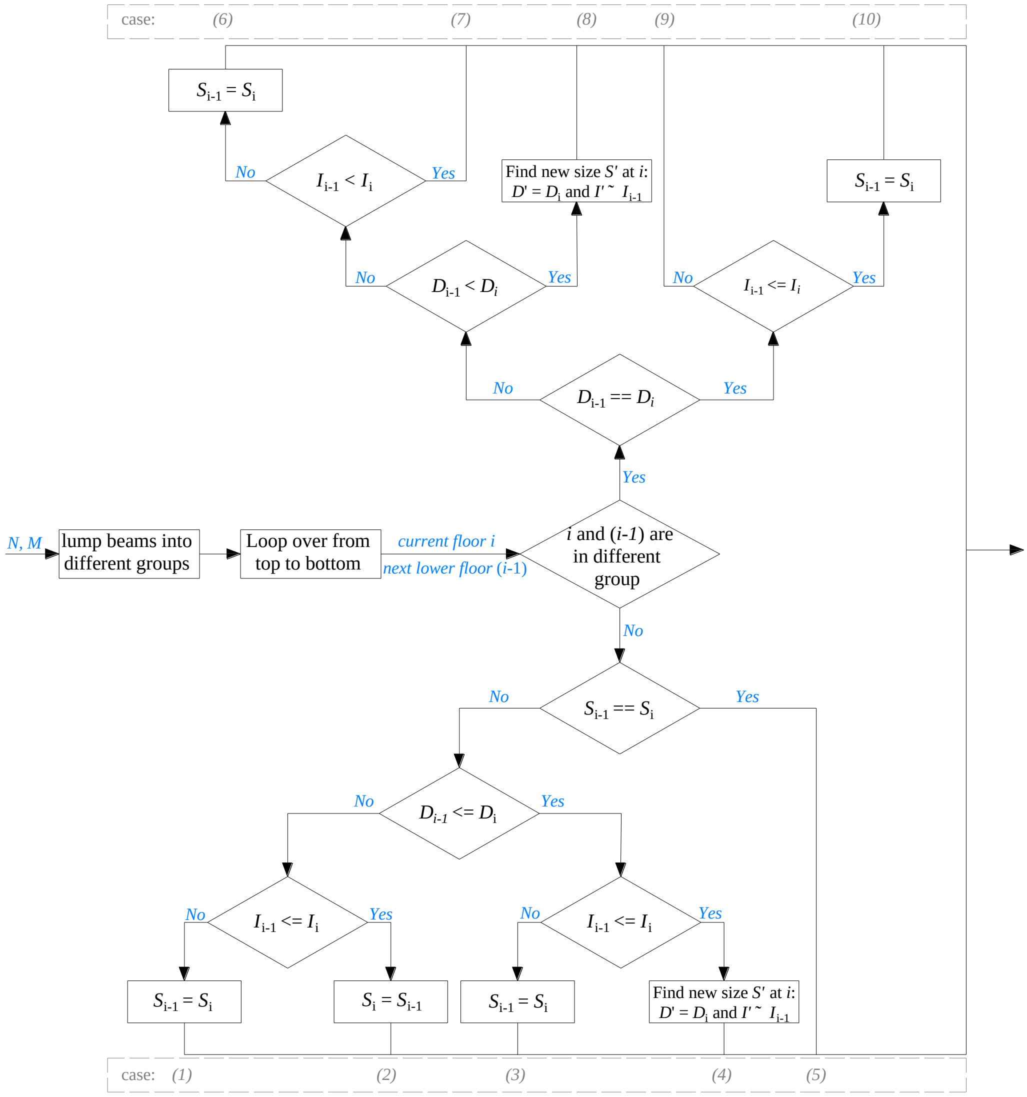
ΣM_{pc}^*: the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines

ΣM_{pb}^*: the sum of the moments in the beams at the intersection of the beam and column centerlines

$I_{z,\,upper}$: moment of inertia of column in upper story

$I_{z,\,lower}$: moment of intertia of column in lower story

Fig. 5 – Overview of the sub-algorithm used to ensure that the feasibility for all connections



Nomenclature:

N: number of stories for the building
M: number of stories that should be adjusted to have identical size
S_i: section size at floor level i
D_i: section depth at floor level i
I_i: moment of inertia of section at floor level i

$S_i == S_{i-1}$: condition if current section size is the same as the lower section size
$S_i = S_{i-1}$: change the section size at floor level i to be the same as that at level (i-1)

Fig. 6 – Overview of the sub-algorithm used to revise the beam sizes for ease of construction

3. Nonlinear Model Construction and Analysis

The nonlinear model construction and analysis (NMCA) module takes the design results as input, constructs nonlinear structural models, and performs static and/or dynamic structural response analyses. The current version of the AutoSAD platform allows the user to building a 2D structural model using the concentrated plasticity model for the beam-column elements. The plastic hinges at both beam and column ends are represented as zero-length rotational springs assigned with modified Ibarra-Medina-Krawinkler material model [13]. The panel zone is modeled using a combination of elastic elements and zero-length rotational springs. More specifically, eight elastic elements with very high axial and flexural rigidity are used as boundary elements, which form a parallelogram with a width corresponding to the column depth and a height that is the same as beam depth. The remaining three corners are modeled as pinned connections. The thickness of the panel zone is taken as the sum of the column web and doubler plate thicknesses. A leaning column is included to account for P-Δ effects. The hinge for the leaning column is modeled as a zero-length rotational spring with very small rotational stiffness so that it does not add lateral stiffness to the structure. The gravity load on the SMRF is uniformly applied to the beam elements, whereas the load on the part of the gravity system that is not explicitly modeled is applied to the leaning column. The future versions of the AutoSAD platform will be adapted to incorporate 3D models and other types of modeling techniques (e.g., fiber elements or finite length hinge). The overall programing structure and associated class definition for the NMCA module is shown in Fig. 7. There is only one class: *NonlinearModelGeneration*. It provides a set of static methods to write the text files, which are further used to generate the nonlinear structural models in OpenSees and perform eigenvalue, static pushover over, and dynamic analyses (including incremental dynamic analysis to collapse). Once all the necessary text files have been generated, the OpenSees execution file is called directly from the Python environment. Additionally, the NMCA module provides a set of independent functions to visualize the analysis results. The building weight, nodal forces and nodal displacements from the pushover analysis is used to plot the normalized base shear vs. roof drift. Peak story drifts and floor accelerations and residual story drifts can also be extracted from the dynamic analysis results. Three statistical methods, maximum likelihood estimation, minimizing sum of squared errors and *Probit* regression, are available to fit the extracted engineering demand parameters at different intensities to a lognormal distribution [14], which can then be visualized (e.g., collapse fragility curve).

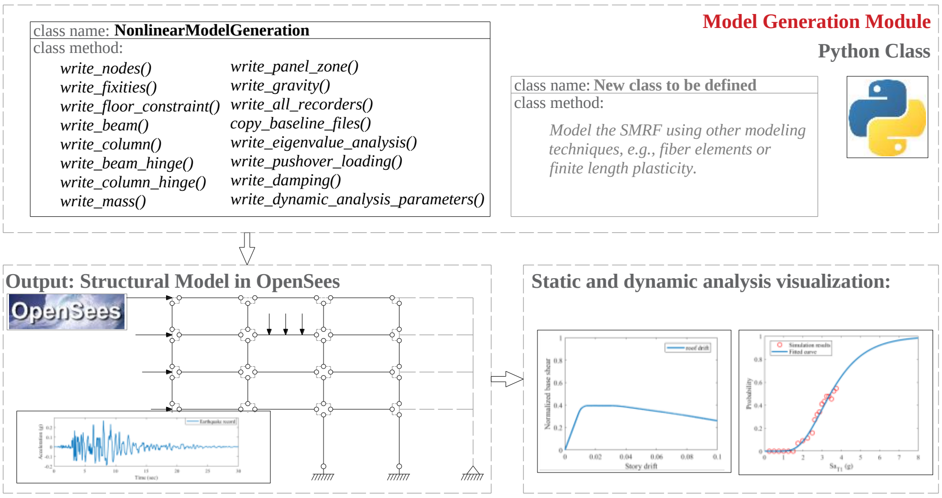


Fig. 7 – Programming structure of the NMCA module

4. Illustrative Examples

Despite the AutoSAD platform's efficiency and the ability to keep track of the entire process, the reliability of the generated designs is a major concern. To address this issue, three buildings designed and reviewed by industry structural engineers and researchers are selected to assess the reliability of the module.

4.1 Comparing with designs produced by *Englekirk Structural Engineers*

As part of the ATC-123 project [10], three- and nine-story SMRF office buildings located in Los Angeles were designed by Englekirk Structural Engineers (*https://www.englekirk.com*). Both buildings have two SRMFs in the two orthogonal directions. The SMRF in the North-South direction is designed using AutoSAD and compared with the *Englekirk* designs. The dead loads on a typical floor and roof for both buildings are 106 psf (5.08 kN/m^2) and 83 psf (kN/m^2), respectively. The site class D and the associated response accelerations are S_s = 2.25 g and S_1 = 0.6 g. More building details can be found in reference [10].

The comparison between the Englekirk designs and the ones generated by the AutoSAD platform are summarized in Tables 1 and 2. With the realization that two engineers start with the same design information would generate different design results, the designs generated from *Englekirk* and the module are comparable.

Table 1 – Comparing member sizes between designs produced by *Englekirk* and the AutoSAD platform for the three-story building

Story	Englekirk			Seismic Design Module		
	Exterior column	Interior column	Beam	Exterior column	Interior column	Beam
3	W14X211	W14X311	W27X94	W14X132	W14X159	W27X94
2	W14X311	W14X370	W33X130	W14X311	W14X455	W30X132
1	W14X311	W14X370	W33X130	W14X311	W14X455	W30X132

Table 2 – Comparing member sizes between designs produced by *Englekirk* and the AutoSAD platform for the nine-story building

Story	*Englekirk*			Seismic Design Module		
	Exterior column	Interior column	Beam	Exterior column	Interior column	Beam
9	W14X233	W14X311	W27X94	W14X132	W14X145	W33X130
8	W14X233	W14X311	W30X116	W14X311	W14X455	W33X130
7	W14X370	W14X398	W36X150	W14X311	W14X455	W33X130
6	W14X370	W14X398	W36X150	W14X398	W14X550	W33X152
5	W14X398	W14X426	W36X182	W14X398	W14X550	W33X152
4	W14X398	W14X426	W36X194	W14X426	W14X605	W36X160
3	W14X455	W14X500	W36X232	W14X426	W14X605	W36X160
2	W14X455	W14X500	W36X232	W14X500	W14X665	W36X170
1	W14X455	W14X550	W36X232	W14X500	W14X665	W36X170

4.2 Comparing of designs produced by researchers

A four-story office building located in Los Angeles was developed by Lignos [15] and its seismic performance was evaluated. The building has two SMRFs in each orthogonal direction and the one in the North-South direction was designed using the AutoSAD platform. The floor and roof weights are 1050 kips (4670 kN) and 1200 kips (5338 kN), respectively. The site class is D and the associated spectral acceleration parameters are S_s = 1.5 g and S_1 = 0.6 g. The allowable drift limit is taken as 2.5% because the building was designed using the 2003 International Building Code [9]. Additional details of the building can be found in Chapter 5 of reference [15]. The comparison between the designs reported by Lignos [11] and the one generated by the AutoSAD platform is summarized in Table 3 It is observed that the section sizes for the two designs are quite comparable, which again demonstrates the reliability of the AutoSAD platform.

Table 3 – Comparing member sizes between designs produced by the AutoSAD platform and Lignos [15]

Story	Lignos [15]		Seismic Design Module	
	Exterior/Interior column	Beam	Exterior/Interior column	Beam
4	W24X76	W21X93	W24X76	W21X68
3	W24X76	W21X93	W24X76	W21X68
2	W24X117	W27X102	W24X131	W27X102
1	W24X117	W27X102	W24X131	W27X102

4.3 Comparing features of AutoSAD with commercial software: *RAM* and *SAP 2000*

The seismic design module is compared with commercial software to illustrate its advantages and limitations. *RAM Steel* and *SAP 2000*, which are commonly used structural design software, are selected for this purpose. The comparison is summarized in Table 4. In both *RAM Steel* and *SAP 2000*, the user manually constructs the elastic SMRF model using a graphical user interface (GUI). In contrast, this process is automated in the AutoSAD platform without the user's intervention. As for the design process, *RAM Steel* can only evaluate the feasibility of a specified design. In other words, the member sizes must be assigned and adjusted by the user. While *SAP 2000* provides an "auto-list" function that implies seismic design automation, it simply determines the member sizes based on the demands from the previous elastic analysis without updating the demands. The user needs to perform a manual iteration to finalize the design. These issues are not present in the seismic design module of AutoSAD. As noted earlier, another advantage of the AutoSAD platform is that it automatically tracks each change in member size during the entire design process. However, both *RAM Steel* and *SAP 2000* do not have this capability. Moreover, the time spent to perform a design using *RAM Steel* and *SAP 2000* depends on the user and is on the order of several hours to days. In contrast, the longest running time for a single design using the AutoSAD platform is less than one hour. A limitation of the AutoSAD platform is that its current version is limited to 2D modeling and it cannot deal with torsional irregularity or other three-dimensional effects. However, given the adopted object-oriented programming structure and modular framework, the AutoSAD platform can be easily extended to consider three-dimensional issues. Also, the current version of the AutoSAD platform can only produce designs using equivalent lateral force analyses, whereas both *SAP2000* and *RAM Steel* can accommodate produce response spectra analysis (RSA) and response history analysis (RHA) based designs.

Table 4 – Comparing features of *RAM Steel*, *SAP 2000*, and the AutoSAD platform

Features	*RAM Steel*	*SAP 2000*	Seismic design module
Auto-modeling	×	×	√
Auto-iterative design	×	×	√
Auto-tracking of the design process	×	×	√
3D modeling (torsional irregularity)	√	√	×
Design duration	depends on the user	depends on the user	< 1 hour
RSA and RHA-based design	√	√	×

4.4 Nonlinear static and dynamic analysis of SMRF buildings

The three-story building used in the ATC 123 project is used to illustrate the features of the NMCA module. Both nonlinear static and dynamic analyses, including incremental dynamic analyses, are performed. The pushover loading pattern is determined using the equivalent lateral force procedure prescribed in Chapter 12 of ASCE 7-16 [6] and assuming that the response is governed by the first mode of vibration. The pushover response for the North-South SMRF is shown in Fig. 8. The frame base shear is normalized with respect to its tributary seismic weight. The maximum base shear observed in Fig. 8 is 0.39, whereas the design base shear is 0.1. The overstrength factor, which is defined as the ratio of the maximum base shear to the code-design base shear [3], is computed as 3.86, which is greater than the code-specified minimum of 3.0. The period-based ductility ratio, which is defined as the ratio of the roof drift corresponding to a 20% drop in the maximum base shear to the yield roof drift [3], is also computed. The drift corresponding to a 20% drop in the maximum base shear is 0.075 and the yield drift is 0.011. Therefore, the ductility ratio is 6.97. The resulting values for the overstrength factor and ductility ratio serve as further indication of the reliability of the AutoSAD-generated designs.

The truncated results are used to generate a collapse fragility, where a lognormal distribution function is used to fit the simulation data via the maximum likelihood method. The collapse fragility curve for the three-story building is shown in Fig. 9. The collapse margin ratio (CMR), which is defined as the ratio of the median collapse spectral acceleration to S_{aMCE}, is computed. The median collapse capacity observed from Fig. 9 is 3.44 g and the S_{aMCE} is 1.22 g, which corresponds to a CMR of 2.81. This result is further adjusted by applying a spectral shape factor of 1.36 (FEMA P695 Table 7-1). Thus, the adjusted collapse margin ratio (AMR) is 3.82. According to Table 7-3 of FEMA P695, the minimum permissible ACMR, which is based on an MCE level collapse probability of 10%, is 1.83. The ACMR for the three-story building is twice the permissible value, indicating acceptable collapse performance. This observation serves as further evidence of the reliability of the AutoSAD-based designs.

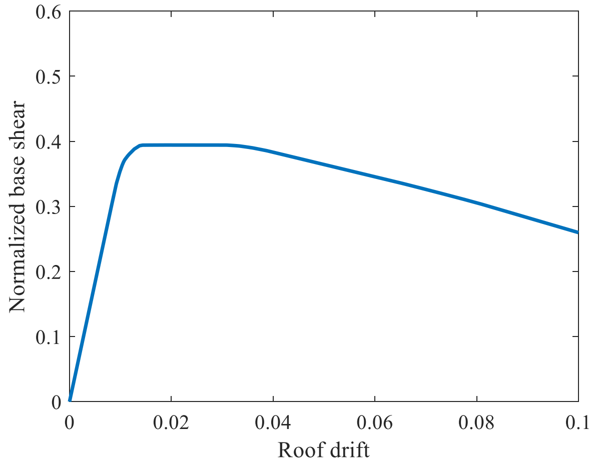


Fig. 8 – Pushover curve for the three-story building

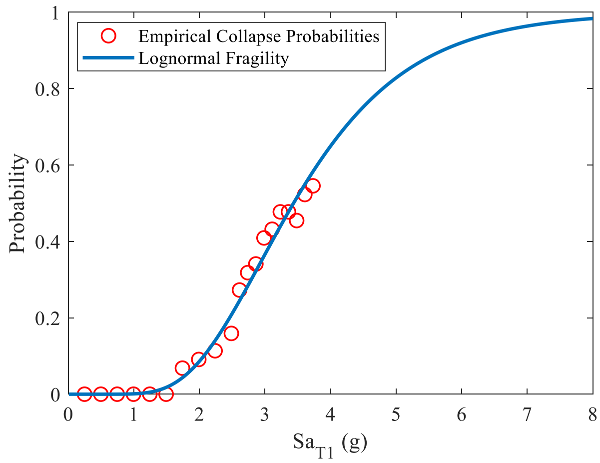


Fig. 9 – Collapse fragility for the three-story building

5. Conclusions

This paper presents a Python-based platform that automates the seismic design, nonlinear structural model generation, and response simulation of steel special moment resisting frames (SMRFs). The first module of the automatic seismic analysis and design (AutoSAD) platform takes building configuration, loads, and site parameters as input and outputs SMRF designs that comply with the latest building code provisions while accounting for ease of construction. A second module constructs two-dimensional nonlinear structural models in OpenSees based on the generated designs and performs nonlinear static and dynamic analyses towards a comprehensive evaluation of seismic performance. The efficiency, reliability, and accuracy of the AutoSAD platform are demonstrated using three illustrative examples. The modular framework object orientated programming structure makes the platform easily adaptable. Potential future enhancements include the use of alternative strategies to account for beam-column material nonlinearity, 3D modeling and economic loss assessment. The broad implication of the AutoSAD platform is a drastic reduction in the time and effort involved in performance-based seismic design. Moreover, it can be used to develop a database of archetype steel moment frame buildings towards the development of analytics-driven design methodologies. It is worth noting that the development details (e.g., platform structure and algorithm) documented in this paper can be used to create similar platforms for other types of structural systems. A key limitation of the current version of the AutoSAD platform is that it only allows the design of SMRFs using the ELF method. This limitation can be addressed by adding a feature that generates designs using the results from response spectrum and/or response history analyses.

6. Acknowledgements

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7. Reference

[1] Liel AB, Deierlein GG (2008): Assessing the collapse risk of California's existing reinforced concrete frame structures: metrics for seismic safety decisions. *Doctoral Dissertation*, Stanford University.

[2] Deierlein GG, Liel AB, Haselton CB, Kircher CA (2008): ATC 63 methodology for evaluating seismic collapse safety of archetype buildings. *Structures Congress: Crossing Borders*, Vancouver, Canada.

[3] FEMA P695 (2009): Quantification of building seismic performance factors. *Applied Technology Council*. Redwood City, CA, USA.

[4] Elkady A, Lignos DG (2015): Effect of gravity framing on the overstrength and collapse capacity of steel frame buildings with perimeter special moment frames. *Earthquake Engineering & Structural Dynamics*. 44, 1289–1307.

[5] Zareian F, Lignos D, Krawinkler H (2010) Evaluation of seismic collapse performance of steel special moment resisting frames using FEMA P695 (ATC-63) methodology. *Structures Congress*, Orlando, FL, USA.

[6] ASCE 7-16 (2016): Minimum design loads for buildings and other structures, Reston, VA, USA.

[7] American Institute of Steel Construction (AISC) (2016): Seismic provisions for structural steel buildings. ANSI/AISC 341-16., American Institute of Steel Construction: Chicago, IL, USA.

[8] American Institute of Steel Construction (AISC) (2016): Specification for structural steel buildings. ANSI/AISC 360-16, American Institute of Steel Construction: Chicago, IL, USA.

[9] IBC (2018): International building code, International Code Council, Inc.(Formerly BOCA, ICBO and SBCCI). 4051 (2018) 60478–5795.

[10] FEMA P-2012 (2018): Assessing seismic performance of buildings with configuration irregularities, *Applied Technology Council*. Redwood City, CA, USA.

[11] Mazzoni S, McKenna F, Scott MH, Fenves GL (2006): The open system for earthquake engineering simulation (OpenSEES) user command-language manual.

[12] American Institute of Steel Construction (AISC) (2016): Prequalified connections for special and intermediate steel moment frames for seismic applications. ANSI/AISC 358-16, American Institute of Steel Construction: Chicago, IL, USA.

[13] Ibarra LF, Medina RA, Krawinkler H (2005): Hysteretic models that incorporate strength and stiffness deterioration. *Earthquake Engineering & Structural Dynamics*. 34, 1489–1511.

[14] Baker JW (2015): Efficient analytical fragility function fitting using dynamic structural analysis. *Earthquake Spectra*, 31, 579–599.

[15] Lignos D (2008): Sidesway collapse of deteriorating structural systems under seismic excitations, *Doctoral Dissertation*. Stanford University.

