Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

OPTIMAL ESTIMATION OF LINEAR SHEAR-TYPE MODELS FOR BENCHMARK STEEL STRUCTURES IN DYNAMIC ANALYSIS

Maziar Fahimi Farzam (1), Mohammad Salehi (2)

(1) *Department of Civil Engineering, Faculty of Engineering, University of Maragheh, m.farzam@maragheh.ac.ir*
(2) *Graduated Student from the University of Maragheh, mohammadsalehicivil70@gmail.com*

Abstract

Reducing the degree of freedoms of building models significantly decreases the computational costs in time consuming structural engineering problems like dynamic analysis of structures, nonlinear analysis or any optimal design of structural systems. In this study, the finite element models of 3- and 9-story benchmark steel buildings with numerous degrees of freedoms are simplified to 3- and 9-degree-of-freedom linear shear-type buildings and reduced to just 3- and 9-degree of freedom buildings, respectively. First, initial linear shear-type models were derived by estimating the stiffness of the stories, ensuring that their fundamental frequency matches the corresponding frequencies of the finite element models. Then, an optimization problem is defined and solved using a Genetic Algorithm (GA) to achieve more significant accuracy in the higher frequencies of the initial linear shear-type models. Two objective functions were established and assessed for the optimization problem: one is the difference in frequencies between the finite element models and the initial linear shear-type models with equal weighting, and the other is the first objective function improved with the modal participation percent weighting. The stiffness of the stories in the shear models are selected as the design variables in both optimization problems. Finally, the models are subjected to four benchmark earthquake excitations and different structural responses are compared with those of finite element models to evaluate the accuracy of optimal linear shear-type models based on the time responses of buildings under natural ground motions. The Root Mean Square (RMS) and peak of the displacement, velocity, and absolute acceleration of the building's roofs have been selected and compared as the performance criteria of the proposed objective functions. The results show that the model derived from the weighted objective function outperformed the objective function with equal weighting. Moreover, by comparing the proposed optimal 3- and 9-story models with those of available in the literature, it can be shown that the proposed optimal linear shear-type models are significantly more accurate both in frequency and time-history analysis.

Keywords: Dynamic Analysis, Finite Element Models, Degrees of Freedom, Shear-Type Models, Genetic Algorithm, Benchmark Steel Structures, Benchmark Earthquake Excitations.

1. Introduction

Engineering structures are increasingly being constructed on a larger scale, leading to greater structural sophistication and more detailed design requirements. Various methods have been employed for analysis and design. This analysis involves calculating displacements, deformations and stresses within a structure over a specified period under defined dynamic loads. Therefore, reliable dynamic analysis tools are essential throughout the design process [1]. Dynamic analysis can be conducted analytically for simple structures under specific loading conditions. However, advanced computational methods are necessary for more complicated structures and intricate loading conditions. The introduction of the Finite Element (FE) [2] method marked a pivotal milestone in the history of computational dynamic analysis. FE techniques are essential in structure analysis, design, and control. The more precise these methods are, and the more accurate the results they can provide, the more complex and computationally intensive the modeling and computational effort required becomes [3].

Unfortunately, FE models often possess hundreds of thousands of degrees of freedom (DoF), making dynamic response computations extraordinarily time-consuming and costly. When multiple designs and loading conditions require evaluation, completing a dynamic assessment of the system within a reasonable time duration becomes unfeasible [4]. Engineers must balance two conflicting objectives to achieve accurate and precise modeling while minimizing time, cost, and computational effort. Model

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

Order Reduction (MOR) techniques present a valuable and practical approach for approximating the original model with a significantly more subordinate order. In other words, these techniques create a Reduced-Order Model (ROM) that simulates the behavior of large-scale dynamic systems. So this allows for reduced computation time, cost, and effort while maintaining the accuracy of the analysis results [5]. Today, FE analysis is widely used to analyze and design structural systems, making the need for MOR techniques essential in this context.

Reducing DoF in engineering structures serves various purposes, such as seismic performance assessment, seismic demand analysis, health monitoring, damage detection, damage evaluation, and control system design. Engineering structures, such as buildings, often possess many DoF and are simplified and reduced to lower-order models, including single-degree-of-freedom (SDoF) and shear-type models.

Guyan introduced the concept of reducing DoF in 1965 by reducing mass and stiffness matrices [6]. After Guyan, other researchers have also employed reducing DoF and using ROMs in their studies. Qu et al. reduced the order of the mass and stiffness matrices of a 40-story shear-type building, where ROM is updated iteratively until the desired ROM is achieved [7]. Liu et al. proposed a simplified method for the preliminary design of structural dampers, which simplified 3, 9, and 20-story benchmark steel strutures to a shear-type models [8]. Bagchi proposed an iterative method for reducing the DoF in 6 and 12-story concrete buildings, where these were reduced to SDoF models [9]. Cimellaro et al. simplified a 9-story benchmark steel structure by deriving an equivalent linear shear-type model. This purpose is achieved by minimizing the natural frequencies and mode shapes discrepancies between the FE model and ROM [10]. Larki and Hosseini introduced a method to reduce the DoF in high-rise shear-type models, precisely 40 and 60-story models, by condensing them into 10 and 15-story models [11].

Alonso-Rodríguez and Miranda introduced a method for simplifying large-scale buildings, which was applied to a 20-story building [12]. Niti et al. proposed a fundamental analytical formulation for evaluating high-rise buildings' natural frequencies and mode shapes in the early design stages for dynamic response assessment. This method reduces the structural systems' DoF to 3 per floor [13]. Ghara et al. proposed a matrix-based analytical approach for assessing the behavior of friction-damped structures. In this approach, the DoF are divided into primary and subsidiary groups, with frictional connections modeled using 4 DoF, ultimately condensing the primary structure's matrices [14]. Ahmadi Amiri et al. presented a practical method for seismic risk analysis, which reduces both the DoF in the FE model and the number of iterative nonlinear time history analyses (NLTHA) required [15]. Fiore et al. developed a heuristic algorithm for obtaining a structures' mass and stiffness matrices and determining natural frequencies, requiring minimal input data and accelerating the process compared to traditional FE methods [16].

In this paper, an optimal linear shear-type model of 3 and 9-story benchmark structures is estimated based on the study by Ohotori et al., achieving a very high level of accuracy [17]. Initially, a preliminary shear-type model is created by estimating the story stiffness values, although it shows a considerable discrepancy with the FE model. By adjusting the coefficient in the stiffness equation for each structure, the initial frequencies of the preliminary models are matched to those of the FE models.

An optimization problem is defined to enhance the accuracy of these preliminary models, which is solved using a weighted sum approach. Two objective functions are defined based on the frequency discrepancies between the preliminary shear-type and the FE models, with one function weighted equally and the other weighted by modal participation percent weighting. The design variables in the optimization problem are the story stiffness values, and the search space is set within the range [2/3k, 4/3k], where k represents the stiffness of each story in the preliminary shear-type model. Upon solving the optimization problem for both objective functions, the solutions in which the story stiffness values are identified and corresponding shear-type models are modeled and compared to FE models in terms of frequency and time-history analysis accuracy.

The floor masses in both structures are identical to those in the Ohotori et al. study, and the damping matrix is derived using Rayleigh damping with a 2% damping ratio for the first and second modes. For

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

time-history analysis comparison, the optimized and FE models are subjected to identical inputs from four benchmark ground motions: two far-field earthquakes (Elcentro and Hachinohe) and two near-field earthquakes (Kobe and Northridge). The differences in their time-history responses, including peak and RMS values for the roof floor's displacement, velocity, and absolute acceleration, are compared.

2. Structures

2.1. Description of Case Studies

This study employs the 3 and 9-story benchmark buildings designed for the SAC Phase II Steel Project. Although these structures have not been constructed, they conform to seismic design codes and represent typical low, mid and high-rise buildings in Los Angeles, California. These buildings were selected as benchmark structures for SAC studies, providing a broader basis for comparative analysis of the results [17]. Details of these structures are supplied in [18]. The benchmark structures, as studied by Ohtori et al., have been modeled by Farzam and Kalajahi [19] using FE software, achieving significantly higher accuracy compared to Ohtori's models. These models serve as the foundation for the FE models utilized in this article. Figures (1,2) show the schematic diagram, plan, and connection details of the 3 and 9-story structures, along with the specifications of the beams and columns, respectively.

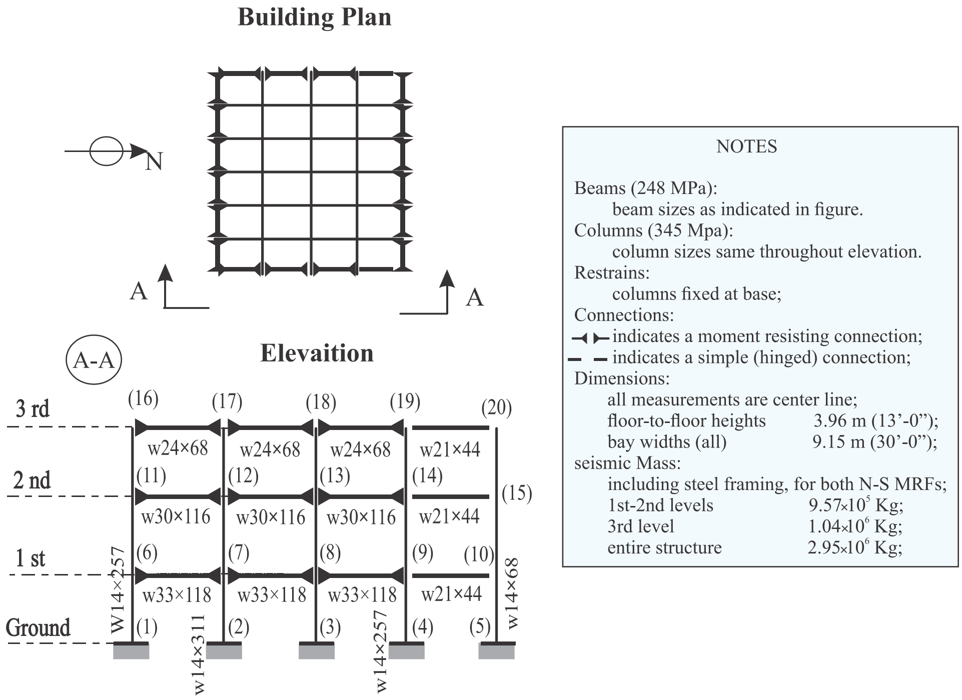

Figure 1. 3-story benchmark building north–south moment-resisting frame

2.2. Preliminary Shear-Type Models

A ROM is essential to simplify and reduce the DoF in the benchmark structures. For this purpose, a linear shear-type model is estimated, and the 3 and 9-story FE models, with 33 and 185 DoF, respectively, are reduced to 3 and 9 DoF shear-type models. In this approach, each floor acts as a rigid diaphragm, and the horizontal displacement of each floor's DoF is linked to the motion of the rigid floor, with the masses concentrated at each floor level. Consequently, the mass matrix of the structure is diagonal. The floor stiffness is calculated for each story, encompassing all cross-sections, and because the structures are modeled linearly, the principle of superposition applies; thus, the stiffness of each

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

floor is obtained by summing the stiffnesses of the beams and columns on that floor. With the mass and stiffness matrices established, the damping matrix of the structure is also determined. Figure (3) illustrates an N-story shear-type model subjected to earthquake excitation.

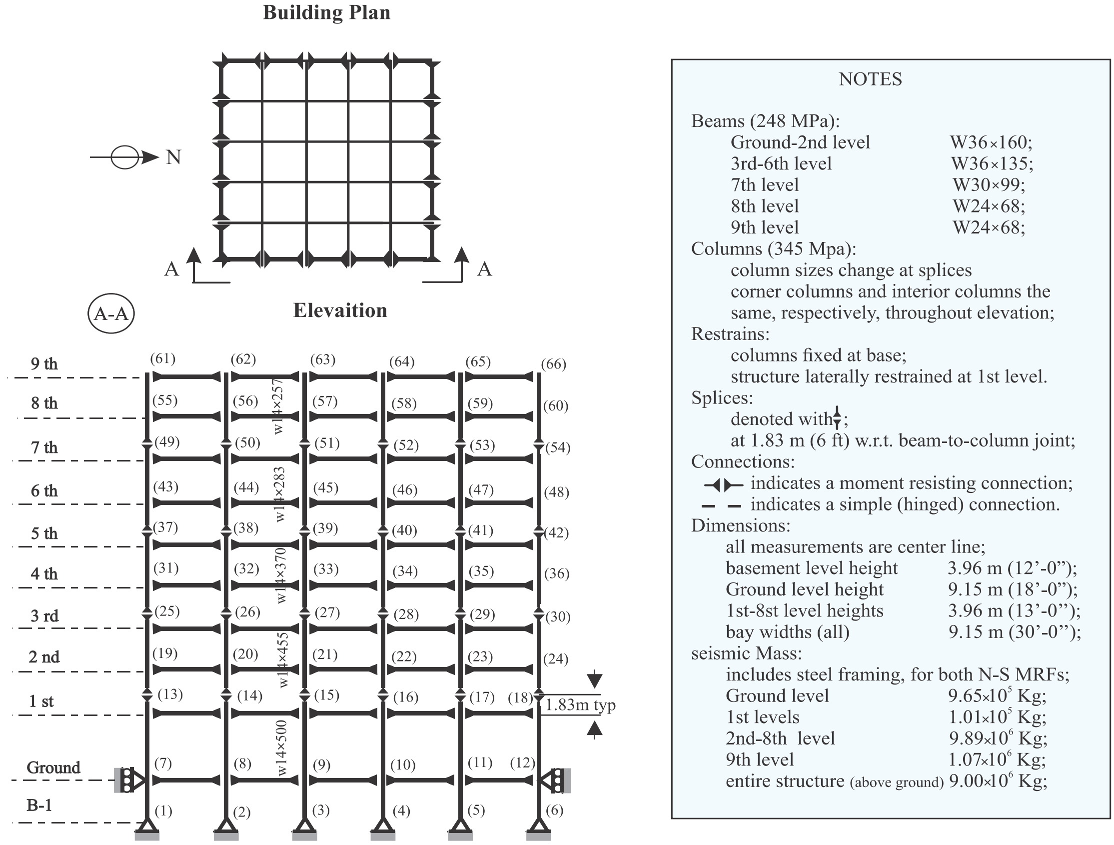

Figure 2. 9-story benchmark building north–south moment-resisting frame

To estimate the story stiffness in moment-resisting frames, a simplified method based on engineering judgment is provided by [8] to develop linear shear-type models for benchmark structures. The lateral stiffness in moment frames can be approximated from the lateral force and the resulting displacement. For frames with strong columns and weak beams, the total displacement under lateral force combines column and beam displacements, with approximately 0.4 and 0.6 contributions to the total displacement. Therefore, the lateral stiffness is estimated as follows:

$$K=\sum_{columns}\frac{4.8\times EI_c}{h^3} \tag{1}$$

E represents the modulus of elasticity of the columns, I_c denotes the moment of inertia of the column section, and h refers to the story height. The benchmark 3 and 9-story structures feature a moment-resisting frame system with a strong column and weak beam design approach. Therefore, the lateral stiffness K for each story in all structures is calculated using Equation (1). In Equation (1), a coefficient of 4.8 is utilized; however, comparing the first frequencies of the linear shear-type and the FE models reveals a significant discrepancy. Therefore, the lateral stiffness K for each story in all structures is calculated using Equation (1). In Equation (1), a coefficient of 4.8 is utilized; however, comparing the first frequencies of the linear shear-type and the FE models reveals a significant discrepancy. Since aligning the initial frequencies, mainly the first frequency, is essential for establishing an accurate preliminary shear-type model, the coefficient of 4.8 was adjusted for both the 3 and 9-story structures.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

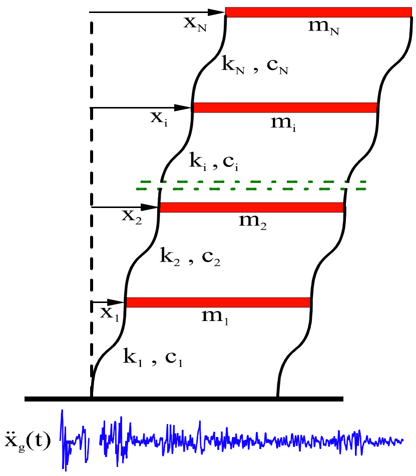

Figure 3. Sketch of an N-story shear-type model under earthquake excitation

The adjusted coefficients are 4.655 for the 3-story structure and 3.891 for the 9-story structure, which ensures that the first frequencies of the preliminary shear-type models match those of the FE models. Table 1. compares the first to third frequencies of the preliminary shear-type and the FE models. The closer the initial shear-type model represents the finite element model in terms of initial frequencies, mainly the first frequency, the less effort the Genetic Algorithm requires to solve the optimization problem. Based on the modification coefficients, as the number of stories and the structure height increase, the stiffness equation coefficient value decreases further. The preliminary model of the 9-story structure exhibits more accurate frequencies than the preliminary model of the 3-story structure.

Table 1. Comparison of natural frequencies of preliminary shear-type and FE models

Frequency (Hz)				
Structure	**Error (%)**	**Preliminary**	**FE**	**No.Mode**
3-Story	0	0.979	0.979	1
	9.302	2.769	3.053	2
	27.523	4.042	5.577	3
9-Story	0	0.442	0.442	1
	1.180	1.200	1.186	2
	0.148	2.035	2.032	3

3. Genetic Optimization Algorithm

Genetic algorithms (GA) are exploratory search approaches that can be applied to various optimization problems. This flexibility makes them particularly appealing for various practical optimization challenges. Since the early 1980s, extensive research has been conducted on applying and developing GA across different fields. Using GA to optimize various engineering problems has proven successful in structural engineering. GAs are based on the principle of evolution, a concept first introduced by Charles Darwin [20]. Holland initially proposed GA in 1975, and researchers such as Goldberg [21] have further developed and refined them.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

4. Mathematical Formulation

4.1. Governing Equation of Motion

To determine the response of a structure under seismic earthquake excitation, it is necessary to establish the mathematical relationships governing the dynamic displacements of the structure. Subsequently, by solving these relationships, the time-history responses of the structure can be obtained, where the most fundamental response in a dynamic analysis is its displacement. The equation of motion for a system with n DoF subjected to an earthquake is expressed as follows [22]:

$$M\ddot{x} + C\dot{x} + Kx = -M\Gamma\ddot{x}g \tag{2}$$

M, *C*, and *K* are the structure's n×n mass, damping, and stiffness matrices, respectively. Additionally, *x* is the n-dimensional displacement vector, and $\ddot{x}g$ represents the ground acceleration. The matrix *Γ* is of size n×1 and indicates the ground acceleration. Solving the above equation determines the structural response values.

Equation (2) can be expressed in state space form as follows:

$$\dot{Z} = AZ + E\ddot{x}g \tag{3}$$

In the above relation, *Z* is a 2n-dimensional state vector defined as follows:

$$Z = \begin{bmatrix} x \\ \dot{x} \end{bmatrix} \tag{4}$$

Additionally, *A* is the state matrix of the system with dimensions 2n×2n, and *E* is the earthquake position matrix, which has dimensions 2n×1. These matrices are defined as follows:

$$A = \begin{bmatrix} O(n,n) & I(n,n) \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \quad E = \begin{bmatrix} O(n,1) \\ -\Gamma \end{bmatrix} \tag{5}$$

4.2. Definition Optimization Problem

An optimization problem is formulated to identify a linear shear-type model of benchmark structures that closely approximates FE models. This problem minimizes the frequency discrepancies between preliminary shear-type and FE models. A GA and weighted sum method are used to solve it. In this approach, the objective functions in the problem are combined through weighted coefficients to create a final objective function. The weight of each objective function depends on its significance, with more critical objectives assigned larger weights [23]. The objective function for this problem is defined as follows:

$$f(X) = \sum_{i=1}^{k} w_i\, f_i(x) \tag{6}$$

In the defined optimization problem, the design variables are the story stiffnesses, which are identified by finding the optimal stiffness of each story according to the specified objective function in the search space. The optimization problem is unconstrained, and the objective function aims to minimize the frequency discrepancies between the preliminary shear-type and FE models. Since matching the initial frequencies of FE models with shear-type models is highly important, the weighting coefficients for the objective functions associated with initial frequency discrepancies are assigned larger values. Appropriate weights are determined based on the dynamic characteristics of the FE models, assigning higher weights to more critical objective functions. Modal participation percent and period-based weightings from FE models can be used as possible weightings. Here, modal participation percent is employed as a weight in this optimization problem. Two objective functions are defined to account for

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

frequency differences between the preliminary shear-type and the FE models: one with equal weights and the other with weights proportional to the modal participation percent. The model obtained from the equally weighted objective function is designated Model 1, while the model derived from the objective function weighted by modal participation percent is designated Model 2. The search space for stiffness is defined within the range [2/3k, 4/3k], where k is the stiffness of each story in the preliminary shear-type model. The optimal story stiffnesses are determined by solving the optimization problem for both objective functions, and corresponding shear-type structures are modeled. These optimized shear-type models are then compared with the FE models in terms of frequency and time-history responses to identify the most optimal model. The story masses for both structures are set equal to those in Ohotori et al.'s article, and the damping matrix is derived using Rayleigh damping with a 2% damping ratio for the first two modes.

5. Results Analysis and Comparison

5.1.Frequency Analysis

In the 3 and 9-story structures for Model 1, where the objective function applies equal weighting, the discrepancies of all frequencies relative to their counterparts in the FE model are reduced uniformly. In contrast, for Model 2, where modal participation percent weights are applied to the objective function, the primary frequencies have more minor discrepancies compared to the FE model frequencies. Tables (2,3) present the first three frequencies of the 3 and 9-story models, comparing them with their respective values in the FE model.

Table 2. Comparison of frequencies between optimized the 3-story and FE models

Frequency (Hz)				
3-Story	**Error (%)**	**Optimum**	**FE**	**No.Mode**
Model 1	6.843	1.046	0.979	1
	0	3.053	3.053	2
	17.464	4.603	5.577	3
Model 2	0	0.979	0.979	1
	2.915	2.964	3.053	2
	18.002	4.573	5.577	3

Table 3. Comparison of frequencies between optimized the 9-story and FE models

Frequency (Hz)				
9-Story	**Error (%)**	**Optimum**	**FE**	**No.Mode**
Model 1	4.29	0.423	0.442	1
	1.096	1.173	1.186	2
	1.033	2.011	2.032	3
Model 2	0	0.442	0.442	1
	0	1.186	1.186	2
	0.147	2.035	2.032	3

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

In each structure, Model 2 is undoubtedly the more optimal choice. For both the 3 and 9-story structures, model 2 is compared in terms of frequency with the FE and the preliminary model, as shown in Figures (4,5). In the 3-story structure, the optimized model 2 achieves a 0% discrepancy for the first frequency, 2.915% for the second, and 18.002% for the third, demonstrating greater accuracy than the preliminary model and the optimized model 1. Similarly, in the 9-story structure, model 2 shows a 0% discrepancy for the first and second frequencies and 0.147% for the third, surpassing the preliminary model and optimized model 1 in accuracy. At higher frequencies, only the fifth frequency of the preliminary model shows a smaller discrepancy than that of the optimized Model 2.

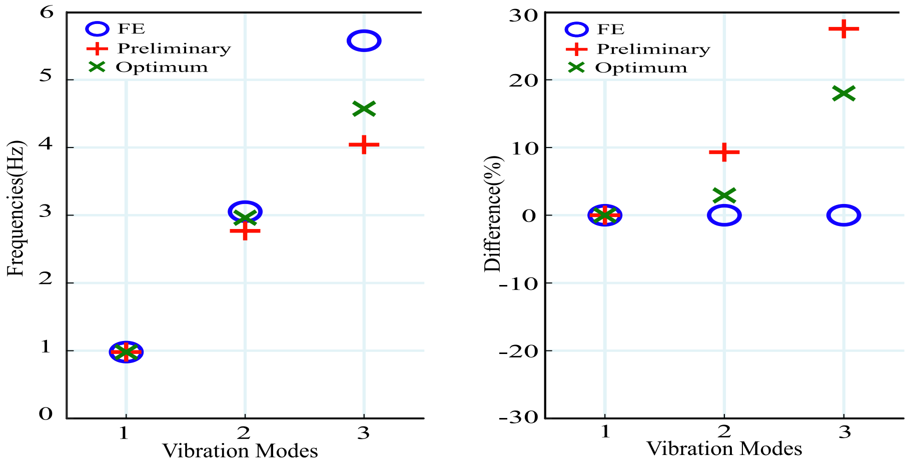

Figure 4. Comparison of Frequencies between Optimized Model 2 of the 3-Story Structure, the FE model, and the Preliminary Shear-type Model

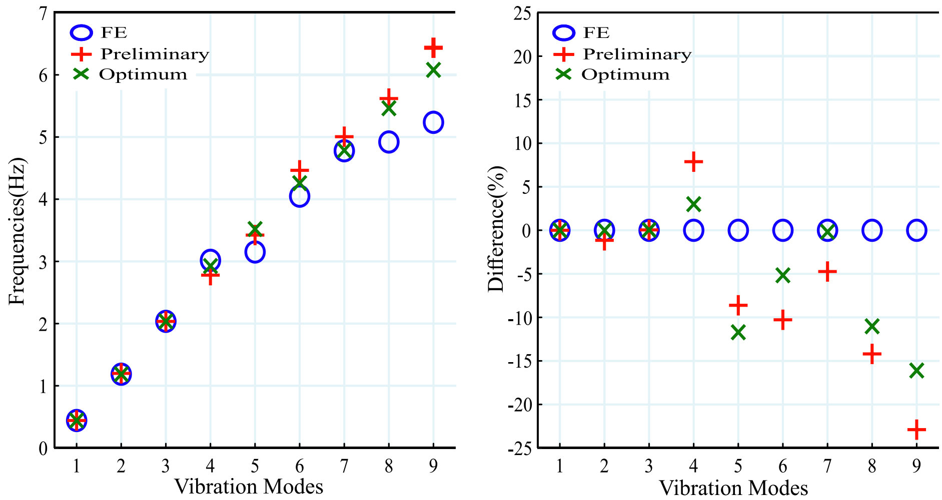

Figure 5. Comparison of Frequencies between Optimized Model 2 of the 9-Story Structure, the FE model, and the Preliminary Shear-type Model

Figures (6,7) illustrate the average absolute discrepancy across all modes during the optimization process and the convergence of preliminary shear-type model frequencies to the FE model frequencies for optimized Model 2 of the 3 and 9-story structures. In the upper figures, the vertical axis represents the percentage, while the horizontal axis denotes the number of analyses. The green curve indicates a reduction in the average absolute discrepancy across all modes throughout the optimization process, with a more pronounced decrease observed in the 9-story structure. The lower figures show the comparison of frequencies between the FE model and optimized Model 2 throughout the optimization process. Across various analyses, the frequencies of the preliminary shear-type model rapidly converge to the FE model frequencies, indicating both a highly accurate initial model and an improvement in its accuracy as the optimization progresses.

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

5.2. Time-History Analysis

This section analyzes the time-history responses of optimized Models 1 and 2 for the 3 and 9-story structures. The FE and optimized models are subjected to four benchmark earthquake excitations, and their time-history response discrepancies are evaluated and compared. For this purpose, a time-history response has been obtained by subtracting the time-history response of the finite element model from that of the optimized model. Subsequently, the resulting time-history response Peak and RMS were compared. The model with lower values is considered more optimal and accurate.

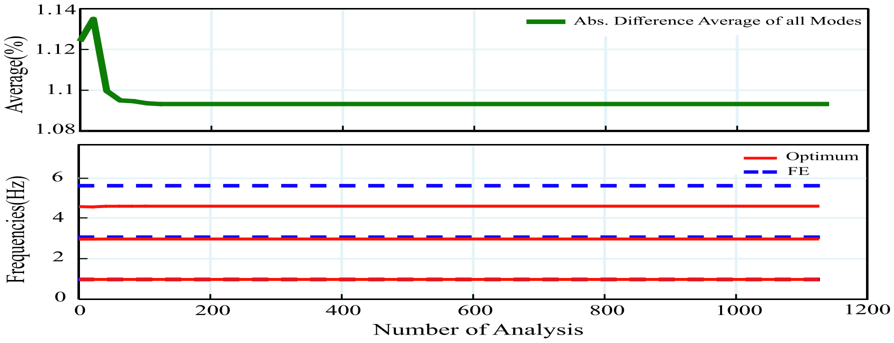

Figure 6. Iteration history of the optimization process in optimized Model 2 of the 3-story structure

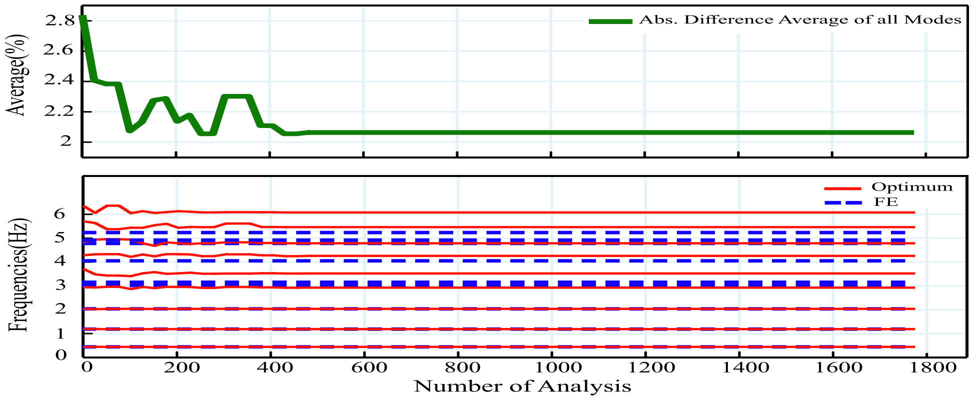

Figure 7. Iteration history of the optimization process in optimized Model 2 of the 9-story structure

This comparison approach is applied to roof responses, including displacement, velocity, and absolute acceleration, which are analyzed, with the results shown in Tables (4,5). Optimized Model 2 demonstrates significantly higher accuracy for both buildings, showing more minor discrepancies than optimized Model 1. The results reveal that the most minor discrepancies in both Peak and RMS occur for displacement. In contrast, the most significant discrepancies are observed in absolute acceleration, particularly under the Kobe and Northridge earthquakes. The large discrepancy in Peak response is due to the rapid and intense oscillation in the absolute acceleration response curve, with slight misalignment in time between the FE and optimized models. The Peak responses in the FE and optimized models do not coincide, resulting in a slight delay and contributing to more significant Peak discrepancies in absolute acceleration at the roof story. For example, in the Model 2 of 3-story structure under the Northridge earthquake, the Peak absolute acceleration in the FE model is 17.85 m/s². In contrast, in optimized Model 2, it is 16.20 m/s² with a difference of 1.65 m/s², and the Peak discrepancy in response to the proposed comparison approach is 10.03 m/s².

If the comparison had been limited to evaluating the differences in Peak and RMS values without utilizing the proposed comparison approach, achieving accurate results from the time-history analysis regarding the precision of the optimized models would not have been possible. Figure (8) presents the roof displacement time-history response of optimized Model 2 for the 3-story structure under the

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

Elcentro earthquake. It shows excellent alignment with the FE model time-history response and demonstrates the high accuracy of optimized Model 2.

Table 4. Statistical comparison of response discrepancies between optimized models of the 3-story structures to the FE model

3-Story		**Model 1**		**Model 2**	
Earthquake	**Response Type**	**Statistical**		**Statistical**	
		Peak	**RMS**	**Peak**	**RMS**
Elcentro	Displacement(m)	0.174	0.047	0.014	0.002
	Velocity(m/s)	1.100	0.302	0.172	0.025
	Abs. Acceleration(m/s^2)	7.528	1.993	3.969	0.636
Hachinohe	Displacement(m)	0.171	0.071	0.013	0.004
	Velocity(m/s)	1.089	0.455	0.123	0.036
	Abs. Acceleration(m/s^2)	6.822	2.947	2.896	0.711
Kobe	Displacement(m)	0.568	0.148	0.047	0.010
	Velocity(m/s)	3.563	0.935	0.431	0.082
	Abs. Acceleration(m/s^2)	25.720	6.041	9.724	1.694
Northridge	Displacement(m)	0.246	0.054	0.033	0.004
	Velocity(m/s)	1.620	0.341	0.389	0.069
	Abs. Acceleration(m/s^2)	10.180	2.301	10.03	1.531

Table 5. Statistical comparison of response discrepancies between optimized models of the 9-story building to the FE model

9-Story		**Model 1**		**Model 2**	
Earthquake	**Response Type**	**Statistical**		**Statistical**	
		Peak	**RMS**	**Peak**	**RMS**
Elcentro	Displacement(m)	0.235	0.102	0.028	0.006
	Velocity(m/s)	0.714	0.284	0.154	0.040
	Abs. Acceleration(m/s^2)	3.110	0.950	2.338	0.384
Hachinohe	Displacement(m)	0.249	0.100	0.020	0.005
	Velocity(m/s)	0.733	0.268	0.115	0.028
	Abs. Acceleration(m/s^2)	2.416	0.827	1.572	0.292
Kobe	Displacement(m)	0.310	0.128	0.068	0.012
	Velocity(m/s)	1.101	0.386	0.519	0.092
	Abs. Acceleration(m/s^2)	7.954	1.805	6.072	0.918
Northridge	Displacement(m)	1.026	0.435	0.060	0.015
	Velocity(m/s)	2.830	1.182	0.434	0.064
	Abs. Acceleration(m/s^2)	7.901	3.322	5.043	0.708

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

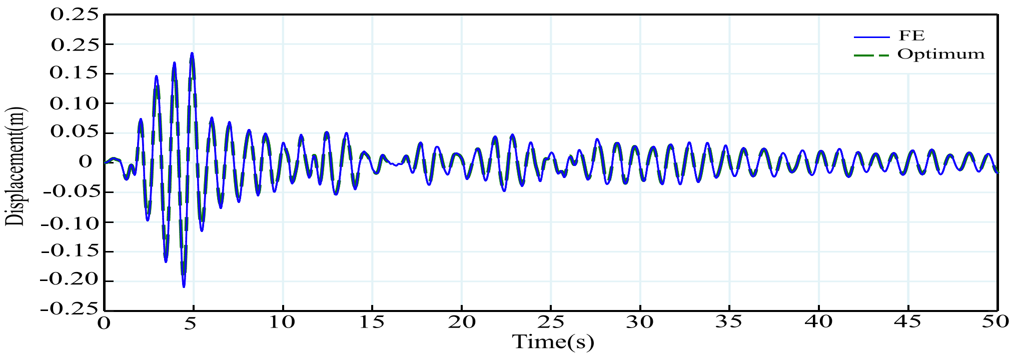

Figure 8. Comparison of roof displacement response history between the FE model and optimized Model 2 of the 3-story structure under the Elcentro earthquake

5.3. Comparison to Other Articles

This section compares the optimized shear-type Model 2 in the 9-story building with the model from Cimellaro et al. [10]. The authors did not find a linear shear-type model for the 3-story structure. The comparison focuses on dynamic parameters and time-history responses. Figure (9) displays the frequencies and mode shapes of the first to the third modes of the FE model, optimized Model 2, and the model from [10]. Optimized Model 2 demonstrates higher accuracy across most frequencies, particularly at lower frequencies, while exhibiting nearly identical accuracy in mode shapes compared to Model [10]. The first three frequencies of the optimized Model 2 show no deviation in the first and second frequencies and only a slight difference of 0.147% in the third frequency compared to the FE model. In contrast, Model [10] exhibits more significant deviations of 3.393%, 8.094%, and 1.820% in the first, second, and third frequencies compared to the FE model. Furthermore, comparing the mode shapes for the first three modes of the optimized Model 2 and Model [10] comparative to the FE model suggests that both models achieve nearly identical overall accuracy.

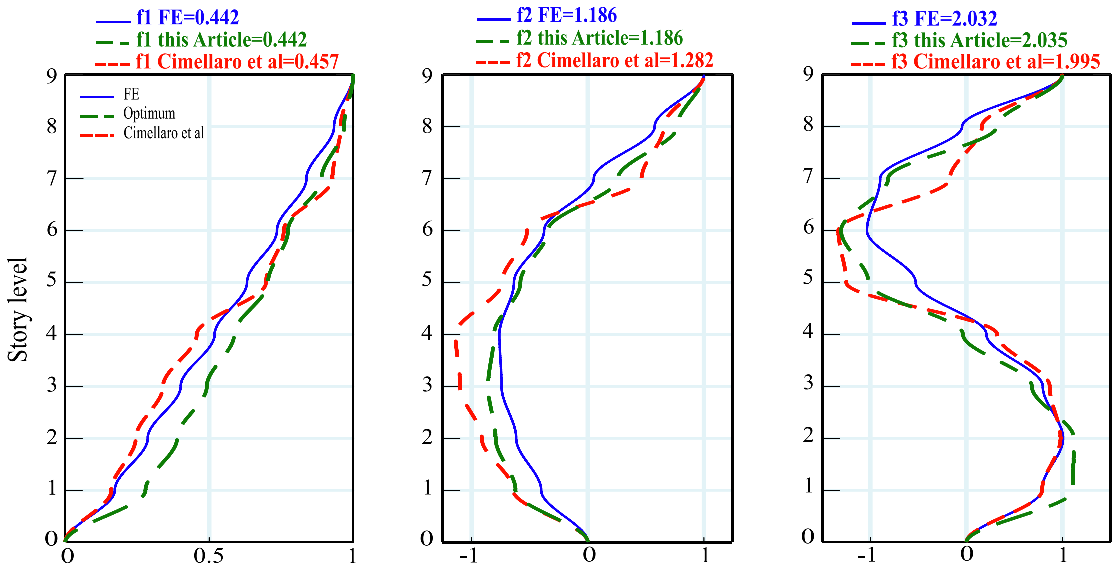

Figure 9. Comparison of frequencies and mode shapes of the optimized Model 2 and the Cimellaro et al. model with the FE model of the 9-story structure

Additionally, Figure (10) presents the roof displacement time-history response under the Elcentro earthquake for the three models: the FE model, the optimized Model 2, and the model from [10]. The analysis results indicate that optimized Model 2 achieves a much higher accuracy in this study.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

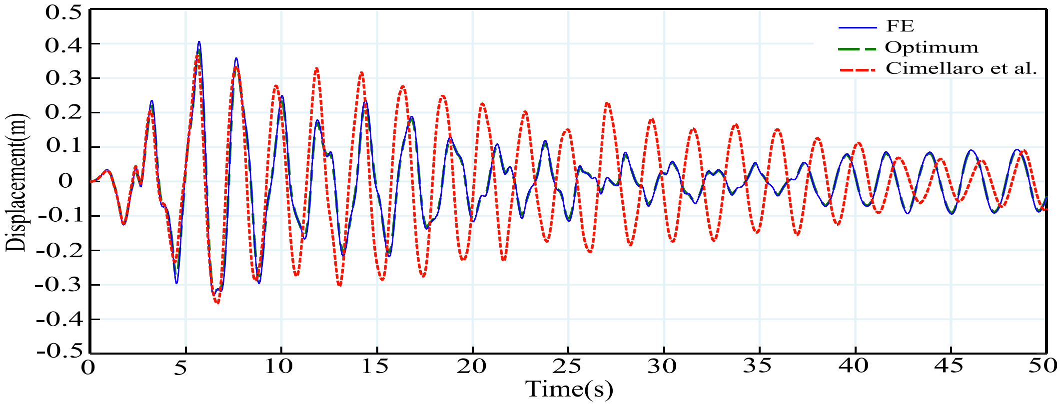

Figure 10. Comparison of roof displacement response time-history between the FE, optimized Model 2 and Cimellaro et al. model of the 9-story structure under the Elcentro earthquake

6. Conclusion

This article addresses the reduction of degrees of freedom in 3 and 9-story benchmark structures to reduce computational time, cost and effort. An optimized shear-type model was developed for these structures using a genetic algorithm, exhibiting high accuracy compared to finite element models. Initially, a preliminary shear-type model was created for each structure by adjusting the coefficient in the stiffness equation for each structure, ensuring that the fundamental frequency matched that of the finite element models.

An optimization problem is defined to enhance the accuracy of these preliminary models, which is solved using a weighted sum approach. The design variables were the story stiffnesses of the preliminary shear-type model. Two objective functions were defined: one minimizes the difference between the frequencies of the FE model and the preliminary shear-type model with equal weighting, and the other uses modal participation percent weighting. By solving the optimization problem, the optimal stiffness for each story was determined according to the defined objective functions within the search space. Frequency comparison between the finite element and the optimized shear-type models showed that the optimal model obtained using the weighted objective function showed significantly higher accuracy, demonstrating that the optimization process improved the accuracy of the preliminary shear-type model.

For time history analysis, the finite element and optimized shear-type models were subjected to four benchmark earthquake excitations, and the displacement, velocity, and absolute acceleration responses at the roof story were compared. Peak and RMS differences in these responses by the proposed comparison approach were evaluated, showing that the optimized Model 2 had much more minor discrepancies than optimized Model 1 in both structures. Furthermore, the optimized Model 2 of the 9-story structure was compared to another model from a different study in terms of dynamic parameters and time-history responses, and it was discovered to have significantly higher accuracy. The optimized Models 2 of the 3 and 9-story benchmark buildings presented in this article highly represent finite element models. They can be used for dynamic analysis and other structural and earthquake engineering purposes.

References

[1] Koutsovasilis, P.(2009): Model order reduction in structural mechanics: coupling the rigid and elastic multi body dynamics. Technische Universität München.

[2] Clough, R.W.(1960): The finite element in plane stress analysis. *Proc. 2^< nd> ASCE Confer. On Electric Computation, 1960*.

[3] Yoo, E.J.(2010): Parametric model order reduction for structural analysis and control. Technische Universität München.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.15

[4] Besselink, B.,Tabak, U.,Lutowska, A.,Van de Wouw, N.,Nijmeijer, H.,Rixen, D.J.,Hochstenbach, M.and Schilders, W.(2013): A comparison of model reduction techniques from structural dynamics, numerical mathematics and systems and control. *Journal of Sound and Vibration*, 332(19), 4403-4422. DOI: 10.1016/j.jsv.2013.03.025.

[5] Wu, L.(2018): Model order reduction and substructuring methods for nonlinear structural dynamics. DOI: https://doi.org/10.4233/uuid:f9736e8b-d00f-4e25-b456-48bd65b43788.

[6] Guyan, R.J.(1965): Reduction of stiffness and mass matrices. *AIAA journal*, 3(2), 380-380. DOI: https://doi.org/10.2514/3.2874.

[7] Qu, Z.Q.,Shi, Y.and Hua, H.(2001): A reduced‐order modeling technique for tall buildings with active tuned mass damper. *Earthquake engineering & structural Dynamics*, 30(3), 349-362. DOI: https://doi.org/10.1002/eqe.10.

[8] Liu, W.,Tong, M.and Lee, G.C. (Year): Simple procedure for preliminary design of structural damper. in *Proc. 13th World Conference on Earthquake Engineering*. 2004. pages.

[9] Bagchi, A.(2004): A simplified methd of evaluating the seismic performance of buildings. *Earthquake Engineering and Engineering Vibration*, 3(223-236.

[10] Cimellaro, G.,Soong, T.and Reinhorn, A.(2009): Integrated design of controlled linear structural systems. *Journal of Structural Engineering*, 135(7), 853-862. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0000032.

[11] Larki, M.H.A.A. (2011): Physically simplified model of multi-story buildings for their quick dynamic analysis. in *3rd International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*. 2011 of Conference. Corfu, Greec. 12 pages.

[12] Alonso-Rodríguez, A.and Miranda, E.(2018): Assessment of effects of reductions of lateral stiffness along height on buildings modeled as elastic cantilever shear beams. *Journal of Earthquake Engineering*, 22(4), 553-568. DOI: 10.1080/13632469.2016.1244132.

[13] Nitti, G.,Lacidogna, G.and Carpinteri, A.(2021): An analytical formulation to evaluate natural frequencies and mode shapes of high-rise buildings. *Curved and Layered Structures*, 8(1), 307-318. DOI: https://doi.org/10.1515/cls-2021-0025.

[14] Gharra, K.,Khanlari, K.and Marnani, J.A.(2022): A Simplified Matrix Analysis Approach to Multi-story Buildings Involving a Friction Damper. *Periodica Polytechnica Civil Engineering*, 66(1), 75-85. DOI: 10.3311/PPci.18558.

[15] Ahmadie Amiri, H.,Hosseini, M.and E. Estekanchi, H.(2022): Efficient seismic risk assessment of irregular steel‐framed buildings through endurance time analysis of consistent fish‐bone model. *The Structural Design of Tall and Special Buildings*, 31(2), e1901. DOI: https://doi.org/10.1002/tal.1901.

[16] Fiore, I.,Greco, A.and Pluchino, A.(2022): On Damage Identification in Planar Frames of Arbitrary Size. *Shock and Vibration*, 2022(1), 4736992. DOI: https://doi.org/10.1155/2022/4736992.

[17] Ohtori, Y.,Christenson, R.,Spencer Jr, B.and Dyke, S.(2004): Benchmark control problems for seismically excited nonlinear buildings. *Journal of engineering mechanics*, 130(4), 366-385. DOI: https://doi.org/10.1061/(ASCE)0733-9399(2004)130:4(366).

[18] Gupta, A.(1999): Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures. *The John A.*

[19] Farzam, M.F.and Kalajahi, T.D. (2023): STATISTICAL SEISMIC PERFORMANCE ASSESSMENT OF VISCOUS DAMPER IN BENCHMARK BUILDINGS UNDER FAR-FAULT AND NEAR-FAULT EARTHQUAKES. in *2nd Croatian Conference on Earthquake Engineerin, Conference*. 2023 of Conference. 11 pages.

[20] Darwin, C.(2007): On the origin of species (1859). *Evolution and creationism*, 43-50.

[21] Golberg, D.E.(1989): *Genetic algorithms in search, optimization, and machine learning*. 1st ed. Addion wesley. Vol. 1989. 372, Addison-Wesley Professional.

[22] Chopra, A.K.(1998): *Dynamics of structures*. fourth ed., earson Education India. .

[23] Marler, R.T.and Arora, J.S.(2010): The weighted sum method for multi-objective optimization: new insights. *Structural and multidisciplinary optimization*, 41(853-862. DOI: 10.1007/s00158-009-0460-7.