

**KARADENİZ TECHNICAL UNIVERSITY
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES**

DEPARTMENT OF CIVIL ENGINEERING

**PERFORMANCE BASED DESIGN OPTIMIZATION OF STEEL MOMENT FRAMES
TAKING INTO ACCOUNT SOIL STRUCTURE INTERACTION**

DOCTORATE THESIS

Alexis NZAPFAKUMUNSI

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MAY GOD BLESS YOU ALL EXCEEDINGLY!!!

Alexis NZAPFAKUMUNSI

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THESIS STATEMENT

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Alexis NZAPFAKUMUNSI

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Ph.D. Thesis

SUMMARY

PERFORMANCE BASED DESIGN OPTIMIZATION OF STEEL MOMENT FRAMES TAKING INTO ACCOUNT SOIL STRUCTURE INTERACTION

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It is crucial to include the structure-soil interaction in the optimal design for a steel frame in order to produce a more accurate mathematical model and a structure that performs more realistically. In addition, performance-based design analysis must be performed to minimize the extent of structural damage stimulated by earthquake effects. On the other hand, optimum design methods should be used for the effective usage of materials. Metaheuristic techniques make it possible to include all of these factors and effects into each analytical iteration, despite the apparent complexity of the process. The performance of steel structural designs has recently been the focus of several studies. However, only a small fraction of these studies has used metaheuristic methods that account for soil structure interactions. This dissertation optimized the Performance Based Design (PBD) of steel moment-resisting frames (MRFs) using metaheuristic algorithm methodologies (Teaching Learning Based Optimization and Artificial Bee Colony). To properly account for soil structure interaction, an interactive computer program was developed to link with existing analytical software, SAP2000 OAPI, and deliver a more complex optimization solution. Using examples from the published literature, the efficacy of the established optimum design technique for multiple steel structural systems was evaluated.

Key words: Performance Based Design, Teaching Learning Based Optimization, Artificial Bee Colony, Soil Structure Interaction, Three-parameters elastic foundation model, Nonlinear Static Pushover Analysis, SAP 2000 OAPI

Doktora Tezi

ÖZET

ÇELİK MOMENT ÇERÇEVELERİNİN YAPI-ZEMİN ETKİSİNİ DE DİKKATE ALARAK PERFORMANSA DAYALI OPTIMUM TASARIMI

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Yapı zemin etkileşimini dikkate almak matematiksel modeli daha gerçekçi oluşturmak ve davranışı daha gerçeğe yakın modellemek için elde etmek için gereklidir. Ayrıca sismik etkiler altında yapılarda oluşacak hasarı azaltmak amacıyla performansa dayalı tasarımlar gerçekleştirilmektedir. Diğer yandan, malzemelerin etkin kullanımı için optimum tasarım yöntemleri kullanılmalıdır. Metasezgisel teknikler, sürecin görünen karmaşıklığına rağmen, tüm bu faktörleri ve etkileri her iterasyona dahil etmeyi mümkün kılar. Son zamanlarda, çelik yapıların performansa dayalı optimizasyonu alanında çok sayıda araştırma yapılmıştır. Ancak, metasezgisel yöntemler ve yapı zemin etkileşimini de sürece dahil etme fikri bu araştırmaların birkaçında kullanılmıştır. Bu çalışmada, çelik moment çerçevelerin performansa dayalı optimizasyonunu elde etmek için yapı zemin etkileşimini dikkate alan metasezgisel yöntemleri kullanılmıştır. Bu tezde, metasezgisel algoritma metodolojileri (Öğretme Öğrenmeye Dayalı Optimizasyon TLBO ve Yapay Arı Kolonisi ABC) kullanarak çelik moment çerçevelerinin performansa dayalı optimum boyutlandırılması gerçekleştirildi. Yapı zemin etkileşimini düzgün bir şekilde hesaba katmak için, mevcut analitik yazılım SAP2000 OAPI ile bağlantı kuran ve daha karmaşık bir optimizasyon çözümü sunan etkileşimli bir bilgisayar programı geliştirildi. Yayınlanmış literatürden örnekler kullanılarak, çelik yapı sistemleri için belirlenmiş optimum tasarım tekniğinin etkinliği değerlendirildi.

Anahtar kelimeler: Performansa Dayalı Tasarım, Öğretme Öğrenmeye Dayalı Optimizasyon, Yapay Arı Kolonisi, Zemin Yapısı Etkileşimi, Üç parametrelili elastik temel modeli, Doğrusal Olmayan Statik İtme Analizi, SAP 2000 OAPI

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ABBREVIATIONS AND SYMBOLS

2D	: Two Dimension
3D	: Three Dimension
ABC	: Artificial Bee Colony
ACI	: American Concrete Institute
ADRS	: Acceleration Displacement Response
ASD	: Allowable Stress Design
ATC	: Applied Technologies Council
CP	: Collapse Prevention
d	: displacement
D	: Dead load
DBYBHY	: Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik
E	: Young Modulus
FEMA	: Federal Emergency Management Agency
fit	: fitness function
GA	: Genetic Algorithm
HB	: Honey Bee
I	: Building importance factor
IO	: Immediate Occupancy
L	: Length
L	: Live load
lb	: lower bound
LRFD	: Loads Resistance Factor Design
LS	: Life Safety
MDOF	: Multi Degree Of Freedom
MPA	: Modal Pushover Analysis
M_u	: Ultimate moment
M_y	: Yield moment
N	: Number of members
newHB	: new Honey Bee
OAPI	: Open Application Programming Interface
PA	: Pushover Analysis
PBDO	: Performaance Based Design Optimization
pfit	: penalized fitness
SA	: Seismc Analysis
Sa	: Spectral acceleration

SDOF	: Single Degree of Freedom
SMFs	: Shear Moment Frames
SSI	: Soil Structure Interaction
T	: Period
TLBO	: Teacher Learning Based Optimization
ub	: upper bound
V	: Base shear force
W	: Building weight



1. INTRODUCTION

This chapter provides an introduction to the thesis and focus on the topic of interest, exploring its significance in the field of study. It outlines the general overview, statement of the problem, research objectives, scope of the study, performance based design and soil structure interaction. The background of each section is discussed along with its importance and relevance. Briefly, the chapter sets the stage for the rest of the thesis, providing a clear understanding of the research problem and the methodology used to address it.

1.1. General Overview

Since the beginning of human civilisation, shelter has been an essential need for survival. Five hundred thousand years ago, *Homo erectus*, an early ancestor of humans, constructed a building that is regarded as the world's oldest manmade structure. Japanese archaeologists discovered it on a hillside in Chichibu, north of Tokyo [1]. It was a simple building made of wood, herbs, and stones. A tremendous growth in the world's population over the past decades [2] has exposed the necessity to construct complex and tall structures to house these people. With the help of research and technological advancements, many structural systems are planned, designed, and constructed in a more complex fashion than ever before. However, such complexity is accompanied by a number of different engineering challenges, the most demanding and life-saving of which is the design of seismically resistant structural systems.

With the application of capacity design principles, the force-based elastic solution approaches for the design of low- and medium-rise residential buildings provide reliable results. Nevertheless, the dynamic forces, such as earthquake and wind, that affect structures horizontally can be analyzed more realistically using the nonlinear analytical methods that have been developed alongside the advances in computer technology.

1.2. Statement of the Problem

Throughout the seismic design process, there are often a large number of factors that affect the structural system's overall performance. —To guarantee that each design phase's performance objectives are met, an acceptability analysis is conducted. Acceptability checking procedures may use linear or nonlinear inelastic structural analysis techniques to compare the seismic responses of a structure to the design requirements for acceptability [3]. In this case, realizing that the existing design is the best option or that there is still a chance to find more cost-effective alternatives while complying to design code requirements is a challenging but vital assignment. Due to a rise in price and a speeding increase in demand for materials (particularly steel), one of the greatest challenges in the area of structural engineering is finding structural designs that are both inexpensive and have excellent performance. During the last 3 decades, numerous of structural optimization techniques have been developed in an effort to reach this objective.

The optimal design of structural systems may currently be achieved via the use of a range of optimization methods. In contrast to older methods such as optimality criteria and dynamic programming, the introduction of computer codes resulted to the creation of new and more complex optimization techniques. However, the rise in popularity of structural optimization using meta-heuristic search methods can be attributed to their general effectiveness in dealing with a broad range of practical and more challenging optimization problems, for which it is most often impossible to obtain the optimal solution using traditional deterministic design techniques [4], [5]. In the past three decades, engineers and designers have been able to use these new metaheuristic optimization methods to find the optimal design among thousands of possible designs alternatives.

The vast majority of these structure designs have some elements of structure with direct contact with the ground. When external loads, like wind or earthquakes, act on these structural systems, neither the structure nor ground displacements are independent. There is interaction between the soil and a structure constructed on it. Vibrational energy of the structure is transfered to soil via the foundation. Thus, the linked mechanism consisting of

the soil and the structure is evident. This process in which the structure's motion is influenced by how the soil responds and vice versa is known as soil-structure interaction [6], [7].

Soil Structure Interaction (SSI) refers to the relationship between soil and structure built on top of it. It is a significant factor to consider in the design of structures, especially those that are heavy or located in seismically active areas. SSI can have a major impact on how a structure responds to external loads, such as earthquakes. It is essential to consider SSI because behavior of the soil can significantly affect response of the structure. For instance, if the soil is soft, it may deform more under the weight of the structure, which can cause the structure itself to move or deform more. However, if the soil is stiff, it may be able to support the structure more effectively, resulting in less movement and deformation in the structure. Modern seismic design codes, such as ASCE 2007, ATC-55 2005, FEMA 2000, BCJ 2009, CEN 2004 among others [8]–[10], specify that the response of a structure has to be analyzed considering the entire structure (superstructure, foundation, and ground) to accommodate the impacts of SSI.

Within the scope of PBD, nonlinear analysis techniques (PA, time history analysis among others) are well-suited for analyzing the nonlinear behavior of structures subjected to seismic loads. Due to its relative simplicity of use in comparison to dynamic nonlinear methods [11], pushover analysis has become the preferred method of nonlinear analysis for many academics. It applies a controlled lateral seismic load pattern to the structure in small increments until an ultimate state is attained [11]. This analysis process typically employs a lumped-plasticity technique, which monitors the propagation of inelasticity via the development of plastic hinges at ends of the frame member under incremental loading. As results, a static pushover curve is generated [12] by graphing a force vs deformation. It is conceivable, for instance, to plot bending moment against plastic rotation or to relate the strength level obtained in particular portions to the lateral roof displacement of the structure. The results identify the failure mechanism, and deflection at which failure occurs, and provide insight on the structural system's ductility.

In order to improve or even replace the traditional process of designing structures to withstand earthquakes, optimization algorithms can be utilized to find the optimal design

that meets the necessary performance requirements while minimizing structural weight [13]–[15]. For this case, the pushover analysis is integrated in the optimization of structure and the process is enhanced by considering the interaction of the structure and the surrounding soil, SSI. Recently, many studies have been carried out on Performance Based Design (PBD) optimization of steel structures,[14], [16]–[18], however, only a few have utilized metaheuristic methods, PBD, and Soil Structure Interaction (SSI) [6], [7], [19], [20]. This study aims to evaluate SSI on the discrete optimization of steel moment resisting space frames for seismic performance through the use of metaheuristic optimization algorithms.

1.3. Research Objective

In order to stay on course with the trending performance-based design approach, much effort is needed to set a simple systematic comprehensive optimization method for the seismic design of multiple story steel structures. The technique would be able to automatically optimize both fixed and isolated base steel structures whilst simultaneously satisfying different practical seismic performance requirement levels with the probable minimum cost interpreted as structure weight. This design process will involve the coordinated applications of performance based design, soil structure interaction and metaheuristic design optimization; and will have the following specific objectives:

1. Define an adequate design objective function that perfectly illustrate the cost reduction (interpreted as structure weight).
2. Set an optimization solution technique capable of selecting the cross section profiles from the already available steel profiles list in compliance with serviceability and strength requirements for seismic steel structures.
3. Enhance the structural optimization technique to accommodate soil structural interaction.

4. Develop an interactive computer program which couples with existing analysis software (two ways data flow) for structural behavior evaluation and eases an integrated analysis and design environment.
5. Validate the effectiveness of the optimal design technique through various steel structure systems practical example

1.4. Scope of the Study

The scope of this study is to optimize steel structures whilst simultaneously satisfying different practical seismic performance requirement levels with the probable minimum cost interpreted as structure weight. It consists of three chapters: The first chapter is introduction which consists of general overview, statement of the problem, research objectives and scope of the study. In addition, Metaheuristic optimizations, seismic performance based concepts and soil structure interaction are discussed. The second chapter, describes the materials and methods used for successfully carrying out the research done in this thesis. The third chapter consists of conclusion and recommendations.

1.5. Metaheuristic Optimizations

It is in our human nature to always strive for perfection and excellence in many aspects of our lives. Cost minimization in a production line system, finding the shortest path from point A to B, selecting the ideal section profile in steel structure analysis, are often human desires from time to time. These problems can be very complex to deal with by the human brain but not with optimization methods. Nowadays, there are many optimization techniques that can be classified in two major categories: optimization by approximate and exact methods.

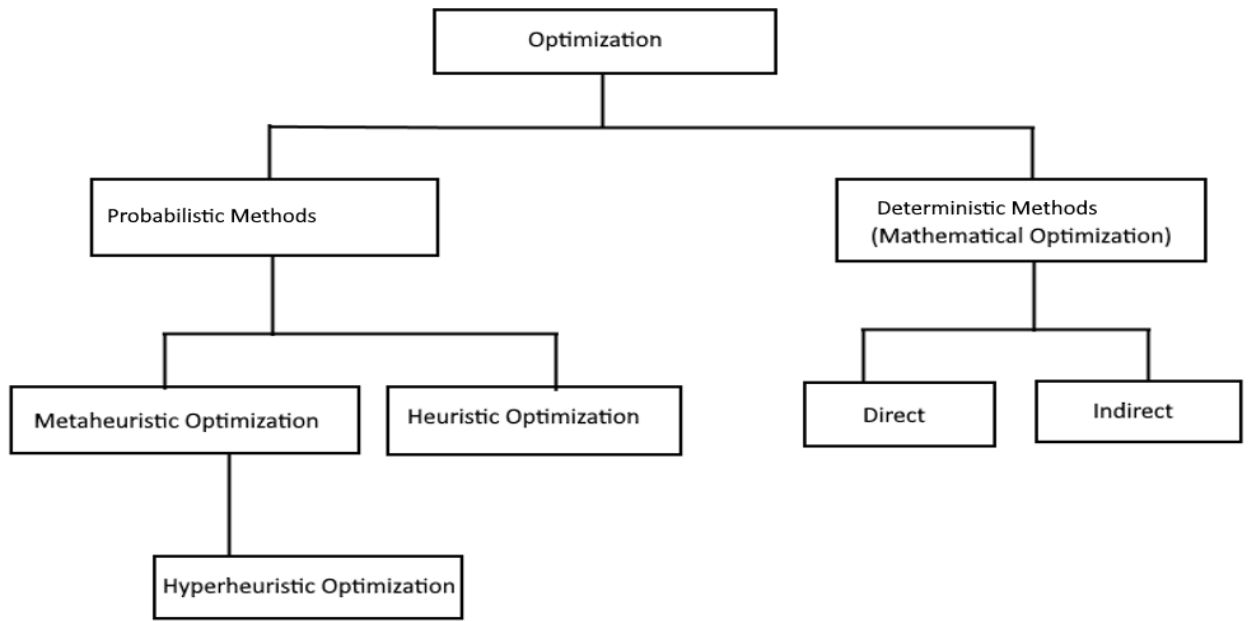


Figure 1.1. Optimization methods [21]

Deterministic optimization methods, also called mathematical optimizations, are a class of optimization techniques that can guarantee to find all optimal solutions to a given problem. These methods use mathematical algorithms and techniques to optimize a solution and can mathematically prove the optimality of the generated solution [21]. However, the computational time required to solve a problem using exact optimization methods enormously rises with the increase in size of the problem, making them unsuitable for complex and nonlinear problems. To overcome this problem, many researchers have accepted local optimum solutions with appropriate precision and optimization time, in other words approximate optimization methods [21].

In this dissertation, the focus is more on approximate optimizations, which are the primary choice over exact optimizations due to their rapid evolution and dual nature. Approximate optimization methods aim to find solutions to a problem in a more efficient manner, even if they might not always find the global optimum solution. The approximate optimizations have two major classes: Heuristic and metaheuristic. Not to mention hyperheuristic which is a combination of the two.

1.5.1. Heuristic, Metaheuristic and Hyper Heuristic

Heuristics are problem-specific techniques that often become caught in a local optimum and fail to identify the global optimum solution. Metaheuristics, on the other hand, are more flexible and adaptable problem-solving techniques that can be used as black boxes and are unlikely to become trapped in a local optimum [22]. Hyperheuristics take this idea a step further by selecting the most appropriate heuristic or metaheuristic for a given problem, but they also come with additional challenges in terms of evaluation and fine-tuning [23].

Metaheuristics, or optimization algorithms that are not based on exact mathematical analysis, have been proven to be effective and often higher to traditional techniques such as branched, or dynamic programming. They are certainly useful for solving complex problems or large instances and can provide a good balance between solution quality and computation time. Metaheuristics can be adapted to fit different optimization problems and do not have strict requirements for the formulation of the problem. However, they may require customization and parameter tuning to achieve optimal performance. Metaheuristics have ability to explore the search space more thoroughly and handle complex constraints, but they are not guaranteed to obtain global optimum solution and can be sensitive to parameter choices.

Numerous works of literature are available in this field: from theoretical concepts to computational improvement and PBD, in addition to a broad variety of applications in different domains. A large number of them are derivatives of the old famous algorithms like the evolutionary, particle swarm and harmony search algorithms among others [23], [24]. Recently, an increase in the development of novel efficient algorithms with different roots are frequently observed. The chronological order of methaheuristic optimization techniques as reported by Hojatollah et al. since 1965 are presented in Table 1.1. [21].

1.5.2. The Chronorogical Order of Metaheuristic Optimizations

The table below describes the methaheuristic optimization techniques evolution since 2010 to 2020.

Table 1.1. The chronological order of metaheuristic optimizations [21]

No	Algorithms	Developers	Description	Years
1	Consultant Guided Search (CGS)	Lordache	The algorithm uses exchange of information between people in a group to make decisions or solve problems. It involves gathering a diverse group of individuals with relevant expertise, allowing them to contribute their ideas and perspectives, and encouraging open discussion. The group then uses a voting system or other decision-making method to reach a consensus on the best solution. CGS is effective because it integrates different perspectives and expertise and promotes collaboration and communication within the group.	2010
2	Bat Algorithm (BA)	Yang	This algorithm is based on a way bats use sounds to locate prey, barriers, and other objects in the dark. It involves a group of bats that change their speed and position based on the loudness and pulse rate of their neighbors. The bats also have the ability to move together as a group towards the best detected location.	2010
3	Charged System Search (CSS)	Kaveh & Talatahari	In this algorithm, every agent (or "charged particle") is treated as a potential solution and its movements are guided by the laws of motion. The objective function and the distance between the agent and other agents are used to determine the force applied to every agent, which then help in determination of its new position, acceleration, and velocity.	2010

Table 1.1. Continued

4	Eagle Strategy Algorithm (ESA)	Yang & Deb	This algorithm combines random search and the firefly algorithm in two steps. It combines exploration of random search with exploitation of firefly algorithm for an efficient search.	2010
5	Group Counselin Optimization (GCO)	Eita & Fahmy	The algorithm imitates how people solve problems by consulting one other. The algorithm's iterations are compared to therapy sessions. Where members continually improve in these sessions, either with assistance from the counseling staff or from themselves.	2010
6	Social Emotional Optimization (SEO)	Xu et al.	This algorithm aims to find members of a population with the highest social status by considering their emotional scores and feedback from others. It involves initializing the emotional scores of each member, implementing a feedback system for interactions, updating social status based on emotional scores and feedback, and repeating these steps until the optimum solution is reached. The final solution is the members with the highest social status, who have gained it through their efforts and interactions with others.	2010
7	Spiral Dynamics Inspired Optimization (SDIO)	Tamura & Yasuda	The spiral search algorithm is based on the idea of a spiral movement, which can be seen in natural structures such as galaxies and tornadoes. It uses a multidimensional spiral pattern to explore a solution space, and has control parameters that can be adjusted to balance the variation and intensity of the search. The algorithm begins by setting the starting point at center of the search space and moving in a spiral pattern, increasing the distance from the starting point with each iteration. At each point, the solution space is evaluated to determine if a satisfactory solution has been	2011

Table 1.1. Continued

			found. If a solution is found, the search is terminated. If no solution is found, the spiral search continues until the entire search space has been exhausted. If the search is unsuccessful, the control parameters can be adjusted and the search can be repeated.	
8	Teaching Learning Based Optimization (TLBO)	Rao et al.	The teaching-learning based algorithm is a method for finding optimal solutions to problems by simulating the dynamic of a class where the members of the population act as students and the best solutions act as teachers. The process involves two phases: a learning phase where each member of the population is assigned a teacher and receives feedback and guidance to improve their solution, and an learning phase where the quality of the solutions (students) is assessed through students cooperation and the best ones help others to improve.	2011
9	Current Search (CS)	Sakulin & Puangdowa reong	This algorithm is inspired by the nature of current flow in electrical circuits. Typically, current will pick the path with the least amount of resistance out of all the possible ones.	2012
10	Water Cycle Algorithm (WCA)	Eskandar et al.	It is based on natural water cycle, where creeks flow into rivers and rivers flow into the sea. It involves evaluating the qualities of creeks and rivers, such as size, depth, and flow rate, and directing the flow of water towards the best solution, which is the sea.	2012
11	Walf Search Algorithm (WSA)	Tang et al.	This algorithm is based on behavior of wolves and their search for food and protection. It allows each agent to search independently while keeping track of their previous locations. If a new location is determined to be the best	2012

Table 1.1. Continued

			among all previously visited ones, the current agent will join with another agent at that location. This process continues until all agents have found the optimal position, which is the location with the most food and the least amount of predators.	
12	Mine Blast Algorithm (MBA)	Sadollah et al.	It is proposed based on a real world mine blast event. it aims to track the impact of an explosion and determine the most effective way to expand a mine based on that information. It does this by collecting data on the number of explosive bullets and any subsequent explosions, identifying the most damaging explosive component, and using that information to guide the expansion of the mine.	2012
13	Atmosphere Clouds Model (ACM)	Yan & Hao	This algorithm simulates behavior of clouds in nature to optimize a search space by mimicking the process of cloud generation, movement, and spread.	2013
14	Egyptian Vulture Optimization (EVO)	Sur et al.	The algorithm's design was influenced by the behaviors of the Egyptian vulture in its pursuit of food. The original intent of this algorithm was to solve hybrid optimization problems.	2013
15	Swallow Swarm Optimization (SSO)	Nerhal et al.	The algorithm is based on behaviors of a group of swallows and their motion. It involves three types of particles: explorers, aimless particles, and leaders. The explorers are responsible for finding and exploring new areas in the search space, while the aimless particles do not have a specific goal and just move randomly. The leaders have a specific goal in mind and try to guide the other particles towards it. If a leader finds a promising solution,	2013

Table 1.1. Continued

			it will try to guide the other particles towards it and become the new group leader.	
16	Grey Wolf Optimizer (GWO)	Mirjalili et al.	The GWO algorithm is based on the behavior of gray wolves and simulates their hierarchical hunting process. It involves four types of wolves, known as Alphas, Betas, Deltas, and Omegas, which represent different levels of leadership. The algorithm involves three main steps: searching for prey, surrounding the prey, and attacking the prey. After the hunting is complete, the wolves share the prey based on their rank in the hierarchy.	2014
17	Golden Ball (GB)	Osaba et al.	The algorithm is inspired by football game theory and involves the use of two populations of players, A and B, who compete against each other. The fitness scores of each player are calculated based on their performance in previous matches, and the top players from each population are selected to form the starting line-ups for the current match. Game theory strategies are then used to determine the best moves for each player during the match, and the performance of each player is evaluated and their fitness scores are updated after the match.	2014
18	Animal Migration Optimization Algorithm (AMOA)	Li et al.	The algorithm is based on idea of group migration in animals, where individuals move to other groups for better resources.	2014

Table 1.1. Continued

19	Soccer League Competition Algorithm (SLC)	Moosavian & Roodsari	The SLC algorithm, inspired by soccer leagues, is based on the competition among teams and individuals in a population. The approach involves forming teams of individuals with similar characteristics, such as skill level or ranking, and encouraging competition between these teams and within the teams themselves. The algorithm continuously adjusts the teams and individuals based on their performance to find the global optimum solution. This competition drives the optimization process, leading to faster and more accurate convergence towards the optimal solution in nonlinear systems.	2014
20	Chicken Swarm (CS)	Meng et al.	Algorithm simulates hierarchical behavior of a group of chickens, including roasters, hens, and chickens that are divided into groups. Each chicken is given a rank based on their age and size, with the oldest and largest chicken being the highest ranked roaster. The chickens interact and engage in behaviors such as pecking and scratching to establish their dominance over each other based on their rank. If a lower ranked chicken consistently challenges a higher ranked chicken and wins, they may eventually gain a higher rank. The algorithm also includes a mechanism for removing a chicken from the group if they consistently lose confrontations and fail to establish their dominance.	2014
21	Forest Optimization Algorithm (FOA)	Ghaemi & Feizi-Derakhshi	The algorithm is inspired by the process of planting seeds in a forest. it assumes that seeds planted directly under trees will not be able to grow, but seeds scattered in other areas may have the opportunity to grow and develop into new trees.	2014

Table 1.1. Continued

22	Heart Algorithm (HA)	Hatamlou	The algorithm approaches an optimization problem by considering best solution found so far as the heart of a system and all other potential solutions as blood molecules. The algorithm repeatedly moves these blood molecules toward or away from the heart based on their fitness in relation to the heart, with the goal of finding the optimal solution.	2014
23	Kaizen Programming (KP)	De Melo	Kaizen is a Japanese method of problem-solving that involves the contribution of ideas from various experts. These ideas are then combined to form an optimal solution, with the value of each idea being evaluated based on its contribution to the final solution. This approach is bit different to many evolutionary optimization methods where each agent is a complete solution.	2014
24	Exchange Market Algorithm (EMA)	Ghorbani & Bahaei	This algorithm is based on the idea of buying and selling stocks on the stock market and is designed to evolve over time to find the best possible investment strategy. It starts with a group of potential solutions (investment portfolios) and continually evaluates and selects the best performing ones to create new generations of portfolios through a process of combining and modifying existing ones.	2014
25	African Buffalo Optimization (ABO)	Odili et al.	The Africa Buffalo Algorithm is inspired by the behavior of African buffalo, who roam in search of green pastures. The algorithm uses a search function to track their movements and predict where they will go next based on various inputs such as weather and available resources. It also considers the social behavior of buffalo, who travel in herds, and uses an optimization function to determine the	2015

Table 1.1. Continued

			most efficient path for them to take to reach their destination.	
26	Elephan Herding Optimization (<i>EHO</i>)	Wang et el.	This algorithm is metaheuristic search algorithm based on swarm intelligence, that was inspired by the herding behavior of elephants.	2015
27	Lon Motion Algorithm (<i>IMA</i>)	Javidy at el	The algorithm uses the movement and forces of positive and negative ions to find the optimum solution to a problem. The ions are divided into two groups and are introduced into the search space to explore and find a solution. Positions of ions are updated based on forces between them and the surrounding medium. Once the ions reach a stable state, the algorithm selects the best solution from the candidates represented by the ions.	2015
28	General Relativity Search Algorithm (<i>GRSA</i>)	Beiranvand & Rokrak	The theory of relativity serves as the inspiration for this algorithm. The algorithm that uses the concept of general theory of relativity to model population members as particles in space that are only affected by gravity. These particles aim to reach a stable position and follow the shortest path possible. The algorithm determines the direction and length of steps for each particle based on its speed and the shortest path to the goal. It updates the positions of the particles iteratively based on the gravitational force acting on them and chooses the particle with the shortest distance to the goal as the best solution.	2015

Table 1.1. Continued

29	Jaguar Algorithm With Learning Behavior (<i>JALB</i>)	Chen et al.	This algorithm is based on the hunting techniques used by jaguars. It involves searching for prey within a designated area and moving towards it using a combination of exploration and exploitation. If the prey is caught, the algorithm ends. If the search area is exhausted without catching the prey, the algorithm also ends. The algorithm can be modified to include multiple jaguars working together and can take into account other factors such as the prey's movement or the presence of other predators.	2015
30	OPTICS Inspired Optimization (<i>OIO</i>)	Kashan	This algorithm uses the optics principles of convex and concave mirrors to search for a solution in the search space. It begins by selecting a starting point and reflecting the search space based on whether it is a peak or valley. The algorithm then continues to iterate through the search space, evaluating the objective function at each point and reflecting the search space based on the value of the objective function.	2015
31	Runner-Root Algorithm (<i>RRA</i>)	Merrikh-Bayat	This algorithm represents behavior of roots and runners in plants in order to achieve a specific goal. The runner moves in large steps to explore a wide area for resources, while the root searches for a small area with high concentrations of resources to exploit. The algorithm also includes functions for the runner to explore and the root to exploit. The plant's location is continually updated based on the results of the runner and root's searches, until the desired location is reached or resources are no longer available. The final location of the plant is then returned.	2015

Table 1.1. Continued

32	Vortex Search Algorithm (VSA)	Dogan & Olmez	The algorithm uses an adaptive step length to explore and exploit search space. It begins at a random point and adjusts the step length based on the gradient of the search space and its current position to find optimum solution.	2015
33	Stochastic Fractal Algorithm (SFS)	Salimi	This algorithm is based on concept of fractals, which are self-similar geometric patterns found in nature. It involves starting with a basic shape and dividing it into smaller copies of itself repeatedly to create a more complex pattern.	2015
34	Water Wave Optimization (WWO)	Zheng	The algorithm is a search method inspired by behavior of water waves, such as how they propagate, reflect, and break. It uses a set of initial wave fronts to explore a high-dimensional solution space, reflecting off of obstacles and breaking into smaller wave fronts to explore multiple paths simultaneously. When a wave front reaches a potential solution, it is tested and if successful, the algorithm stops and returns the solution.	2015
35	Bull Optimization Algorithm (BOA)	Findik	The algorithm changes way that members of a population are chosen for the genetic algorithm by only allowing the fittest members to participate in crossover. The list of members is sorted based on their fitness values, and the top members are selected for crossover.	2015
36	Elephant Search Algorithm (ESA)	Deb et al.	The algorithm is based on the concept of herd of elephants, with male members serving as exploring searching agents and female members serving as local searching agents. The males are responsible for finding new resources and potential mates, while the females focus on maximizing the resources within the herd. The male elephants act as	2015

Table 1.1. Continued

			explorers and innovators, and the female elephants act as caretakers in order to ensure the survival and growth of the population.	
37	Ant Lion Optimizer (ALO)	Mirjalili	This algorithm is based on behavior of ant lions and involves five steps: searching randomly for prey, setting a trap, waiting for the prey to fall into the trap, catching the prey, and rebuilding the trap.	2015
38	Lion Optimization Algorithm (LOA)	Yazdani & Jolai	This algorithm is based on the behavior of lions in a social group. It involves two types of lions, resident and nomad, who each have certain characteristics that help them search for solutions. The resident lions focus on a small area and use their characteristics to evaluate potential solutions, while the nomad lions explore a larger area and inform the resident lions of any potential improvements they find.	2016
39	Whale Optimization Algorithm (LOA)	Mirjalili & Lewis	The algorithm is based on the social behavior of whales and has three main stages: encircling the prey, attacking the prey, and searching for new prey. The group of whales surrounds and blocks the escape routes of the prey to increase the chances of capturing it. They then attack the prey and try to capture it using their collective intelligence and strength. If the prey escapes or is captured, the whales will search for new prey in different areas.	2016
40	Dynamic Virtual Bats Algorithm (DVBA)	Topal & Altun	This algorithm is inspired by the way bats use different wavelengths and frequencies to hunt. It involves continuously monitoring the environment and adjusting the frequency and wavelength to locate and capture prey more accurately.	2016

Table 1.1. Continued

41	Tag of War Optimization (<i>TWO</i>)	Kaveh & Zolghadr	The algorithm is inspired by the game of tag of war and involves evolving a group of candidate solutions through various operations in order to find the best possible solution.	2016
42	Virus Optimization (<i>TWO</i>)	Liang & Curvas Juarez	This algorithm simulates the process of a virus infecting cells and the immune system trying to control its spread. It starts by selecting a target cell and reproducing rapidly within it. The immune system then tries to reduce the virus population. This process is repeated until the virus population reaches a certain limit or the target cell is destroyed, at which point the algorithm either ends or moves on to a new target cell.	2016
43	Virus colony search (<i>VCS</i>)	Li et al.	The algorithm simulate way a virus infects and spreads in a host population. It starts by selecting a random cell and then looks for the closest unoccupied cell to infect. If no unoccupied cells are found, it continues to search in an outward spiral pattern until it finds one. Once it finds an unoccupied cell, the virus infects it and the process repeats until all cells in the colony have been infected.	2016
44	Crow Search Algorithm (<i>CSA</i>)	Askarzadeh	This algorithm is based on intelligent behavior of crows and is a population based technique. It involves creating a group of solutions, known as "crows," which represent potential solutions to a problem. During iterations, the crows may exchange information with each other, hide and find food, or even start over with a new solution.	2016

Table 1.1. Continued

45	Water Evaporation Optimization (WEO)	Kaveh & Bakhshpoori	The algorithm simulates the process of water molecules evaporating from a solid surface with varying characteristics. The solid surface is treated as solution space, and the wettability of the surface and other molecular properties are used as search parameters. The search parameters are adjusted based on the remaining water molecules on the surface, and the optimal surface properties for maximum evaporation are determined once the process is complete.	2016
46	Electro Search Algorithm (ESA)	Tabari & Arshad	The algorithm is inspired by orbital motion of electrons around an atom's nucleus.	2017
47	Grasshopper Optimization Algorithm (GOA)	Saremi et al.	This algorithm is designed to solve optimization problems by using a group of grasshoppers as a model. The young grasshoppers in this model take small steps slowly, while the adults take larger steps more suddenly.	2017
48	Sperm Motility Algorithm (SMA)	Raouf & Hezam	The algorithm is based on human reproductive system. It uses search agents that move randomly in the solution space, similar to the way that sperm cells move in the female reproductive system. The algorithm models the attraction of sperm cells to an egg cell, or ovum, which represents the optimal solution. The movement of the search agents is guided by a "chemical secretion" that helps them navigate towards the optimum solution.	2017

Table 1.1. Continued

49	Beetle Swarm Optimization Algorithm (BSOA)	Wang & Yang	This algorithm aims to improve the performance of swarm optimization by incorporating principles of how beetles search for food. It divides the search space into regions and assigns virtual beetles to each region to explore and find the optimal solution.	2018
50	Chaotic Bird Swarm Optimization Algorithm	Ismail et al.	This algorithm enhanced the quality of exploitation by integrating chaotically based approaches with foraging and privileged behavioral patterns.	2018
51	Butterfly Optimization Algorithm	Arora & Singh	This algorithm achieves optimal solution by simulating the foraging and breeding habits of butterflies.	2019
52	Chaotic Grasshopper Optimization Algorithm (CGOA)	Arora & Anand	This algorithm incorporates chaos theory into the GOA optimization process. On the chaotic maps, the balance between exploitation as well as exploration is efficiently maintained.	2019
53	Quantum Dolphin Swarm Algorithm	Qiao & Yang	In the algorithm, dolphin swarm algorithm incorporates quantum search algorithm to shift away from being trapped in local optimum.	2019
54	Emperor Penguins Colony	Harifi et al.	The algorithm takes its inspiration from the unique habits of the endangered Empiror Penguin. The penguins regulate the temperature using their own body heat and by moving in a circular pattern throughout the colony.	2019

Table 1.1. Continued

55	Darts Game Optimizer (<i>DGO</i>)	Dehghani et al.	This algorithm use principles of the Darts game to generate an algorithm to deal with optimization problems.	2020
56	Capuchin Search Algorithm (<i>CapSA</i>)	Braik et al.	The unusual habits of capuchin monkeys served as inspiration for this algorithm.	2020
57	Red deer Algorithm	Fathollahi-Fard et al.	The algorithm takes its inspiration from habits of Scottish red deer. The distinctive mating activity of Scottish red deer during the breeding season served as the primary inspiration for this algorithm.	2020

1.5.3. Description of Elements of Metaheuristic

There are several different classification criteria for metaheuristics, but one common way to classify them is based on whether they operate on one potential solution at a time or on a population of candidate solutions. Single solution based metaheuristics operate on one candidate solution at a time, iteratively improving it through the generation and replacement process until a termination criterion is reached [25].

On the other hand, population based metaheuristics operate on a population of candidate solutions simultaneously. They often involve some form of selection, reproduction, and crossover to evolve the population towards better solutions [23]. Some examples of population-based metaheuristics include genetic algorithms, evolutionary algorithms, and ant colony optimization. Both single-solution-based and population-based metaheuristics is used to solve optimization problems, but they have different characteristics and may be more suitable for certain types of problems. It's also worth noting that some

metaheuristics can be hybridized, combining elements from both single-solution-based and population-based approaches.

In the following sections 1.5.4 and 1.1.5 of this thesis the detailed mechanisms of the two metaheuristic methods Artificial Bee Colony algorithm (ABC) and Teaching Learning Based Optimizations (TLBO) are discussed.

1.5.4. Artificial Bee Colony Algorithm (ABC)

Some metaheuristics are inspired by the swarm intelligence and collaborative behavior of honeybees. In 2005, Karaboga introduced ABC algorithm in which was modeled after the foraging strategies of honeybees [26]. Thereafter, the ABC algorithm undergoes various changes and enhancements by Karaboga et al. In 2010, Hadidi and Kazemzadeh analyzed as well some of the algorithm's specific aspects [27].

The principles of this algorithm are notably drawn from an intelligence of honey bee colonies foraging behavior, and it has proven reliable search capabilities on a variety of optimization applications. The ABC algorithm makes an effort to resemble honeybees' normal foraging activity. Bees employ the wiggle dance as part of their search for the best food source and to discover new ones.

1.5.4.1. Colony of Honeybees

Honeybees are social insects living in colonies consist of one queen, male bees, and female worker bees. In the colony, all tasks are distributed between all bees in a very specific manner. The queen is responsible for reproducing and secreting pheromones that regulate the behavior of the other bees. The female workers carry out various tasks such as collecting food, cleaning the hive, and caring for the young [26]. After about three weeks, the female workers switch to a role of foraging for nectar and pollen. The male bees mate with the queen and then have no further role in the hive. All the bees work together to ensure the success of the colony [28].

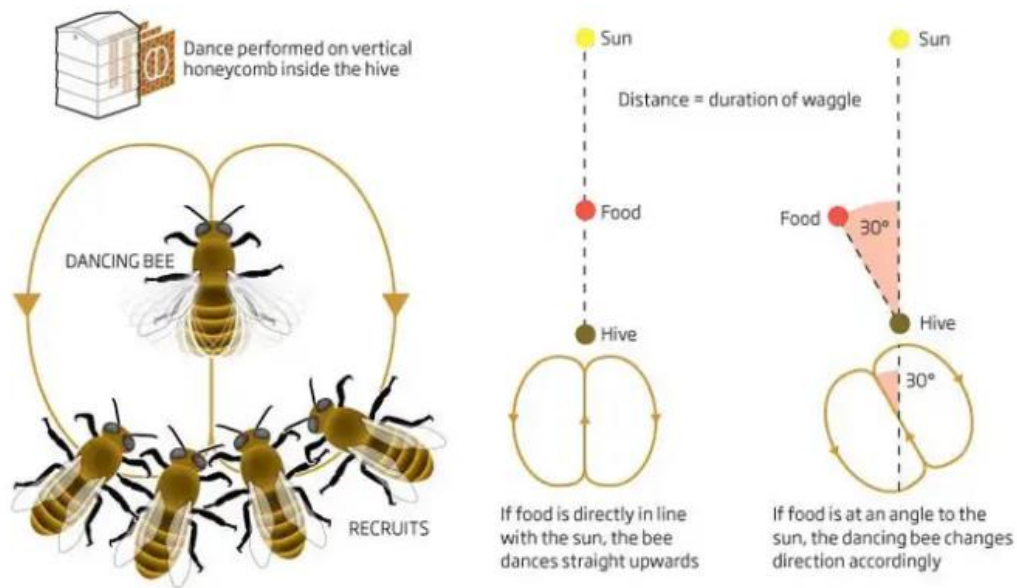


Figure 1.2. The wiggle dance of honeybees colony

These foraging bees are the same bees considered in ABC algorithm. Employed bees, look for potential nectar supplies from various flowers up to 12 kilometers away from the hive, but their typical flight radius is around 3 kilometers. While the employed bees find a food supply, they load up on nectar and return to the hive to discharge it. This interaction behavior of bees during food acquisition is depicted in Figures 1.2. [28].

By performing the waggle dance, employed bees successfully convey the information about food sources to their hive mates. Direction and duration of the dance convey the location of the food source relative to sun, while intensity of wiggles indicates the distance to the food source as shown in figure 1.2. This allows other bees in the hive to locate the food source and collect nectar. Some bees may also decide to leave the hive and search for food independently, rather than following the direction indicated in the waggle dance. These bees are known as scout bees.

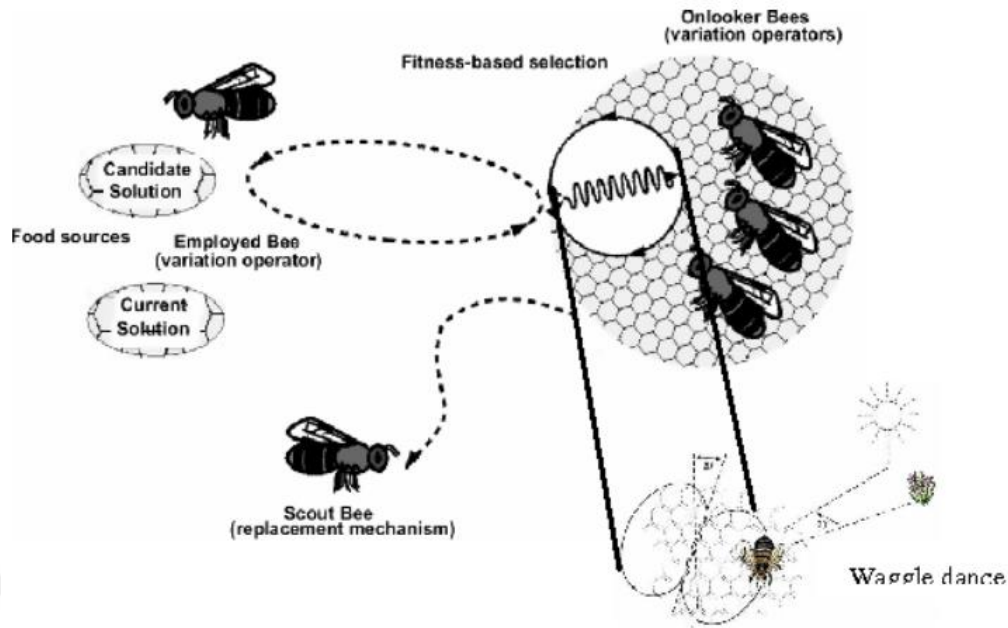


Figure 1.3. Graphical representation of the elements of the ABC algorithm [28]

1.5.4.2. Artificial Bee Colony (ABC) Concept

In the ABC algorithm, each potential solution is represented by a food source, with the quality of its nectar representing the solution's objective function. These food sources are continually altered by three types of honeybees as shown in figure 1.3. employed bees, scout bees, and onlooker bees [28].

The algorithm consists of 3 main elements: employed bees, onlooker bees, and scout bees. Employed bees are bees that are actively searching for food sources. They fly from hive to a food source, gather nectar, and then return to the hive. They then communicate the location and quality of the food source to other bees in the hive by performing a "waggle dance." Onlooker bees are bees that observe the waggle dance of the employed bees and use the information they receive to decide which food sources to visit. They select a food source based on the nectar's quality and its accessibility in terms of distance. Scout bees are bees that perform a random search near the hive. They are responsible for discovering new food sources and recruiting other bees to visit those sources.

The ABC algorithm works by iteratively searching for the optimum solution to an optimization problem. At the start of algorithm, a set of initial bees is randomly generated. The algorithm then iteratively goes through three phases until the stop criterion is met. In the first phase (the employed phase), the employed bees search for new food sources based on their individual knowledge. They fly from hive to a food source, gather nectar, and then return to the hive. They then communicate location and quality of the food source to other bees in hive by performing a "waggle dance." In second phase (onlooker phase), the onlooker bees watch attentively the waggle dance of employed bees and use information they receive to decide which food sources to visit. They select a food source based on nectar's quality and its accessibility in terms of distance. The most promising food source is selected using a probability based selection process like fitness proportionate selection process [25]. More onlooker bees are drawn to the best food sources. It's important to notice that both the employed and onlooker bee numbers are equal to the quantity of food sources in the vicinity of the hive.

The third and final phase of the ABC algorithm is the scout bees phase. This phase begins if a food source cannot be improved after a specific number of trials. In this case, the food source is neglected, and its coupled employed bee is transformed into a scout bee. The scout bee then searches for a new food source by randomly exploring the search space. If a best food source is identified, it is adopted as the new food source, and the process begins again with the employed and onlooker bees.

The flowchart shown in the figure 1.4 describes the pseudocode of the ABC algorithm [29].

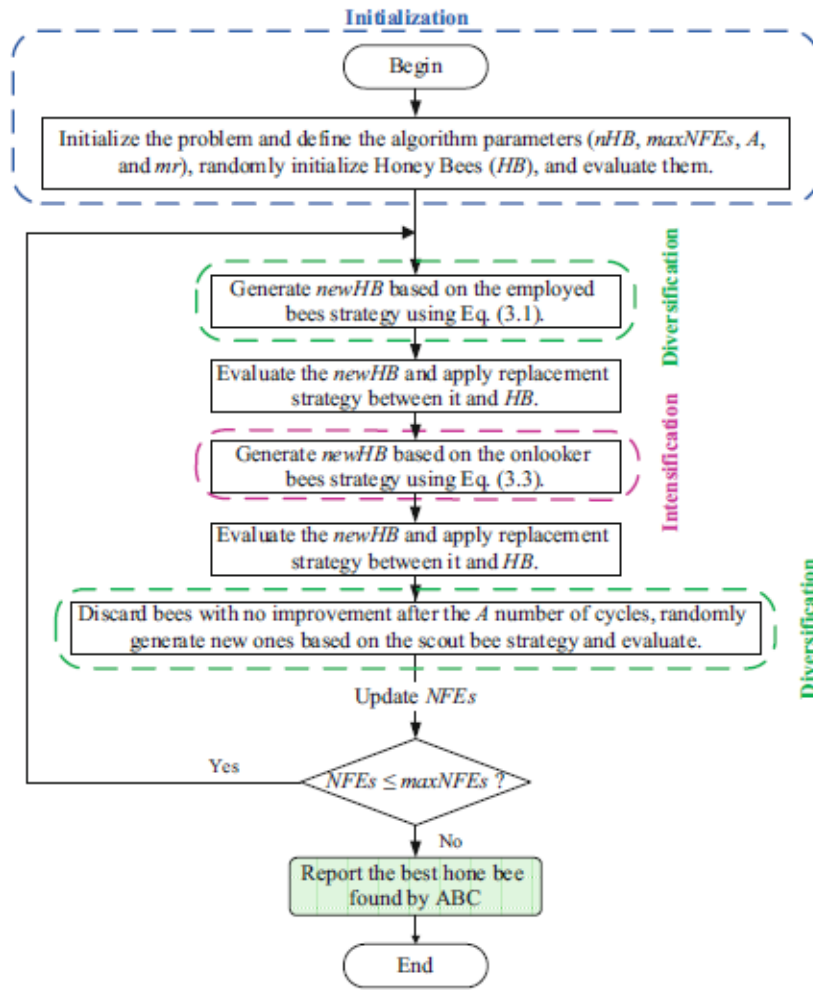


Figure 1.4. The Flowchart of the ABC algorithm [29]

The algorithm aims to obtain the global optimum of a problem by iteratively improving a set of solutions, called "food sources" or "artificial bees", through a process of exploration and exploitation. The (nHB , $maxNFEs$, A , and mr) parameters are specific to ABC algorithm and are used to control behavior of the algorithm [29]. The " nHB " parameter represents the number of food sources or artificial bees in the population. The " $maxNFEs$ " parameter represents the maximum number of function evaluations that can be performed before the algorithm terminates. The " A " parameter represents the maximum amount by which a food source can be modified during the exploration phase. The " mr " parameter represents the probability of choosing the exploitation phase over the exploration phase.

1.5.5. Teaching Learning Based Optimization Algorithm

The first version of the Teaching Learning Based Optimization (TLBO) algorithm was developed by Rao et al. in 2011 inspired by the teaching methodologies of the classical schooling system [30]. It is composed of two phases: Effect of the teacher on students — Teacher phase and effects of the students on one another — Learner Phase. The teacher guide the learning of students by sharing his knowledge and experience with them, and the students can also learn from each other by interacting with one another. The algorithm begins with a population of random solutions, called students, and at each iteration, the best solution among students is chosen as teacher. Students are then modified based on what they learn from the teacher and their interactions with each other.

In TLBO, a normal distribution of student grades is termed classroom performance in learning or instructor performance in teaching. The primary difference between 2 normal distributions is their mean value; for illustration, a better teacher instructs students with higher average test scores. Therefore, TLBO enhances the knowledge of other students throughout the teaching phase by comparing the teacher's knowledge to the average knowledge of all learners. Each learner acquires knowledge according to the place he or she occupies in the research space. In a classroom, learners also improve by interacting with each other after the teacher has finished teaching. In the learner phase, TLBO improves each student through the interaction of their knowledge with another randomly selected student. The pseudo codes below describes it better in a chronological order [31].

```

Input:  $N$ (number of learners),  $D$ (number of dimensions),  $\alpha$ (crossover probability),  $F$ (scale factor),
         $MFE$ (maximal function evaluations)
Output: Best fitness and best solution
1 begin
2   Set  $gBestFitness = Inf$ ,  $num\_eval = 0$  (function evaluation counter);
3   Randomly initialize the population  $P$  and historical population  $hP$ ,  $num\_eval += 2 * N$ ;
4   while  $num\_eval < MFE$  do
5     for  $i \leftarrow 1$  to  $N$  do
6       // Teacher phase;
7       Determine the teacher  $X_{teacher}^t$  and the mean  $X_{mean}^t$ ;
8        $T_F = round(1 + rand(0, 1))$ 
9       for  $j \leftarrow 1$  to  $D$  do
10         $newX_{i,j}^t = X_{i,j}^t + rand(0, 1) \cdot (X_{teacher,j}^t - T_F \cdot X_{mean,j}^t)$ 
11      end
12      Compute  $f(newX_i^t)$ ,  $num\_eval ++$ , and accept  $newX_{i,j}^t$  if  $f(newX_i^t) < f(X_i^t)$ ;
13      // Self-feedback learning phase
14      if  $f(X_i^t) < f(hX_i^t)$  then
15        for  $j \leftarrow 1$  to  $D$  do
16           $newX_{i,j}^t = X_{i,j}^t + rand(0, 1) \cdot (X_{i,j}^t - hX_{i,j}^t)$ 
17        end
18        Compute  $f(newX_i^t)$ ,  $num\_eval ++$ , and accept  $newX_{i,j}^t$  if  $f(newX_i^t) < f(X_i^t)$ ;
19      end
20      // Learner phase
21      Randomly choose a learner  $X_k^t$  ( $k \neq i$ );
22      if  $f(X_i^t) < f(X_k^t)$  then
23        for  $j \leftarrow 1$  to  $D$  do
24           $newX_{i,j}^t = X_{i,j}^t + rand(0, 1) \cdot (X_{i,j}^t - X_{k,j}^t)$ 
25        end
26      else
27        for  $j \leftarrow 1$  to  $D$  do
28           $newX_{i,j}^t = X_{i,j}^t + rand(0, 1) \cdot (X_{k,j}^t - X_{i,j}^t)$ 
29        end
30      end
31      Compute  $f(newX_i^t)$ ,  $num\_eval ++$ , and accept  $newX_{i,j}^t$  if  $f(newX_i^t) < f(X_i^t)$ ;
32    end
33    // Mutation and crossover phase
34    Generate  $rhP^t$  by shuffling  $hP^t$ , and  $map$  matrix according to  $\alpha$ ;
35     $newP^t = P^t + F \cdot map \circ (rhP^t - P^t)$ 
36    for  $i \leftarrow 1$  to  $N$  do
37      Compute  $f(newX_i^t)$ ,  $num\_eval ++$ , and accept  $newX_{i,j}^t$  if  $f(newX_i^t) < f(X_i^t)$ ;
38    end
39    // Update historical population;
40    for  $i \leftarrow 1$  to  $N$  do
41      if  $rand(0, 1) < 0.5$  then
42         $hX_i^t := X_i^t$ 
43      end
44    end
45    Update the best fitness and best solution;
46  end
47 end

```

Figure 1.5. The pseudocode of TLBO [31]

1.5.6. Conclusion

Research on metaheuristics has been criticized for its lack of an uniform design process, insufficient of scientific rigor in testing and comparing alternative implementation, and desire to develop methods with an enormous number of operators. Some authors also criticize the use of metaphors to generate "new" methods. [32].

Despite this criticism, arguing is hard. Metaheuristics solve the majority of the most critical real-world optimization problems in academic research and commercial applications, as they are able to find good solutions when other methods fail. As a result, some providers of commercial software use metaheuristics as primary optimization engines in production planning, vehicle routing, and nurse rostering software, in addition to general-purpose optimization and simulation software. [33].

1.6. Seismic Performance Based Analysis

Amid the structure analysis process, some structure members are expected to deform beyond the elastic limit when subjected to huge external loads or induced by severe earthquake or wind. In this situation, a nonlinear analysis is required for evaluation, since post-elastic behavior cannot be estimated simply using conventional elastic analysis techniques [34].

When the structure in question is induced by an earthquake, a seismic performance evaluation should be performed taking into consideration the post elastic behavior of structural members. In addition, the structure's maximum inelastic displacement demand has to be computed to sufficiently estimate the seismically induced demands on structure behavior in the inelastic range.

It is possible to determine the maximum inelastic displacement demand of structures using a range of nonlinear analytical techniques and approximations. However, the static pushover analysis which is extensively utilized, has been deemed an effective tool for nonlinear analysis due to the relative ease of its use in comparison to dynamic nonlinear

processes [35], [36], [37]. The main reason why it is adopted in this research as well. This magic nonlinear analysis tool identifies the process of the eventual collapse and the actual nature of building failure. By keeping track of the damage sequence of each structure component via the hinges they hold, the technique keeps track of potential weak zones in the structure during analysis [37].

Talking of hinges, also known as plastic hinges, are these points in a structure that are designed to deform under excessive loading, such as during an earthquake. These are areas where diagonal cracks appear in an existing building after a seismic quake. They are usually located at beam-column connections and at the ends of beams and columns. That is where designers more specifically structural engineers are supposed to put the hinges during the analysis of the specific computer analysis-model.

There are three main types of hinges that are commonly used in structural analysis: flexural hinges, shear hinges, and axial hinges [3]. Shear hinges mimic the behavior of beams and columns under shear forces, whereas flexural hinges simulate the behavior of beams and columns under bending forces. Axial hinges model the behavior of braces and truss members under axial loads. These hinges allow the structure to deform in a controlled manner, dissipating energy and preventing collapse.

The static pushover analysis and related research development are explored in further detail in the subsections that follow.

1.6.1. Concept of Pushover Analysis

Static pushover analysis method does not have a complete theoretical theory. It is fundamental concept is that the elastic and inelastic responses of the structure are led by the initial vibration mode, and this shape remains constant throughout the whole analysis process. This establishes the basis for transforming a dynamic problem into a static one. The features and advantages of the static pushover method include: access to the actual force acting on a single element, determination of the structure's collapse mechanism, determination of story drifts, joints deformation, determination of the performance capacity

of the elements and the structure. Due to its practicality and simplicity in the mathematical modeling of multi-degree-of-freedom (MDOF) systems has made it accepted as a structural analysis tool in various building codes: in the United States, (ASCE, 2007; ATC 55, 2005; FEMA, 2000); in Japan, (BCJ, 2009) and in Europe (CEN, 2004) [8]–[10].

1.6.2. Pushover Analysis Procedures

Pushover analysis uses a simplified nonlinear technique to calculate the force-displacement relationship also known as the capacity curve for a structure [37]. The analysis involves applying lateral loads to a structural model in a predicted manner. The load is increased monotonically to the structure, and some hinges started to form in the elements. Then the structural model changes depending on the nonlinear load-deformation diagrams in the members. The process continues until it reaches a limit or collapse status. As results, the pushover curves are determined by plotting the structure capacity, roof displacement and base shear graph as shown in Figure 2.6 [9].

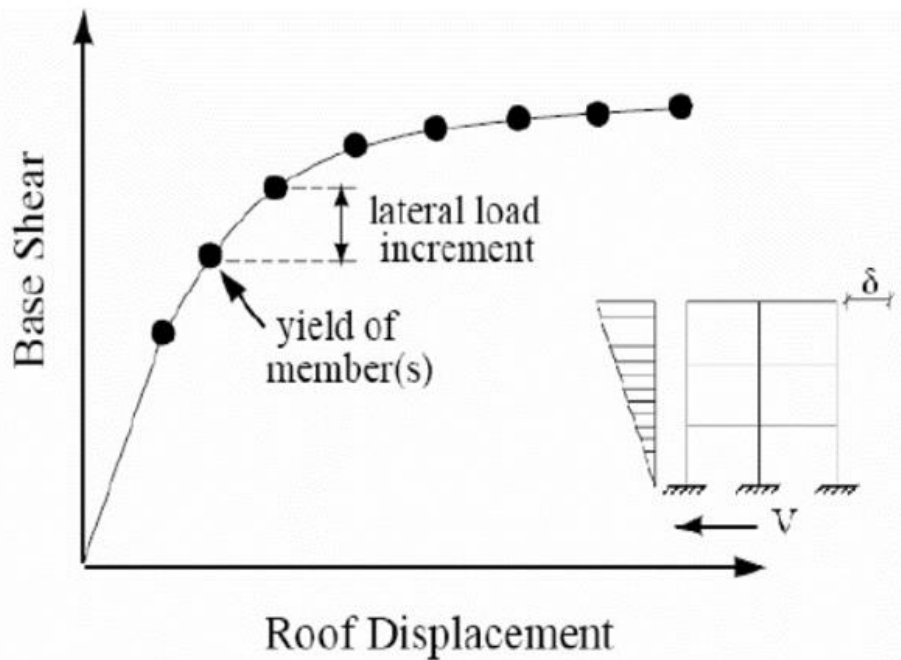


Figure 1.6. General pushover curve for a structure [9]

According to literature and depending on the parameters for which we want to control the pushover, it is possible to talk about 2 types of static pushover analysis. These are displacement controlled and force controlled pushover analysis methods. The force-controlled loading process is uncommon, and it is only used when the gravity load is known. The displacement controlled pushover analysis involves gradually increase of magnitude of the load distribution until the control displacement attained a desired value. Then, the internal forces, stresses and strains are computed at the desired value which is called target displacement.

In 1998, Krawinkler and Seneviratna stated that static pushover analysis is incredibly powerful for demonstration of the response of the structure that cannot normally be displayed by elastic or dynamic analysis [12]. It can provide valuable information about structural behavior and capacity under different levels of loading, as well as identify potential weaknesses or failure modes. However, the technique has some drawbacks that can affect accuracy of the results, such as its inability to consider torsional response, potential issues with target displacement estimation, and challenges with selecting the appropriate lateral load pattern and identifying failure mechanisms for higher vibration modes [37].

1.6.3. Conventional Seismic Analysis (SA) and Pushover Analysis (PA)

Pushover analysis concept would be best understood by comparing it with the conventional seismic analysis. Both PA and SA, use fundamental natural period of structure to estimate structure's maximum lateral loads [17], [36], [38]. While, in PA, the starting period is continually re-calculated along with the analysis, in SA the starting period is assumed to remain intact—equal to the initial value of the period [39].

When it comes to lateral loads applied to the structure both PA and SA use a pattern of predefined vertical loads distribution. In SA there exist two methods by which the lateral load can be vertically distributed to the structure: Seismic coefficient method —the lateral load is parabolically distributed and direct combination method of response spectrum — lateral load is distributed proportionally to modal combination [37]. However, distribution in PA is proportional to height raised to the power of 'k,' with k equal to 0, 1, and 2 for

uniform, inverted triangular, and parabolic distributions, respectively.[37]. Alternatively, k can be a computed value between 1 and 2; and the value of k is depending on the structure's period T , as specified in FEMA 356 [40]. It is also acceptable for the distribution to be proportional to either the first mode shape or a combination of modes.

Briefly, PA differs from SA as PA uses nonlinear models while SA makes use of the elastic model. In PA, materials or geometry nonlinearities of the structure are captured by inserting hinges into an otherwise linear elastic model produced by different structural analysis programs—SAP2000, Etabs, Opensees, or STAAD.Pro having facilities for PA.

1.6.4. Conventional Pushover Analysis Methods

FEMA has adopted three conventional pushover analysis methods:

- Capacity Spectrum Method (CSM)
- Displacement Coefficient Method (DCM)
- Modal Pushover Analysis (MPA)

1.6.4.1. Capacity Spectrum Method (CSM)

The capacity spectrum method is one of the ways to analyze response of a structure to an earthquake by creating a graphical illustration of the structure's force-displacement capacity curve and an earthquake response spectrum. This is done by converting the capacity curve and the response spectrum into a spectral acceleration (S_a) and spectral displacement (S_d) graph, which allows structure to be represented as an equivalent single-degree-of-freedom (SDF) system. The performance point, which describes the spectral displacement of the structure under a specific earthquake, can then be determined by using a trial and error process [37]. Steps for the recommended procedure are as follows:

1. Obtaining the pushover curve for the Multiple Degrees of Freedom (MDOF) system,
2. Idealizing capacity curve as a bilinear.

3. Converting the idealized curve to Acceleration Displacement Response Spectrum (ADRS) format; Using equations (1.1.) and (1.2.), find the capacity diagram as illustrated by Figure 1.7, [9].

$$S_a = \frac{\frac{V}{W}}{\alpha_1} \quad (1.1)$$

S_a : Spectral acceleration (meter per second squared)

W : Weight of the building (kN)

V : Base shear force (kN)

α_1 : First modal participation ratio

$$S_d = \frac{U_r}{\Gamma_1 \Phi_{r1}} \quad (1.2)$$

S_d : Spectral displacement (meter),

Φ_{r1} : Amplitude of the first mode at level i.

Γ_1 : Factor of modal participation for the first natural mode

U_r : Roof displacement (meter)

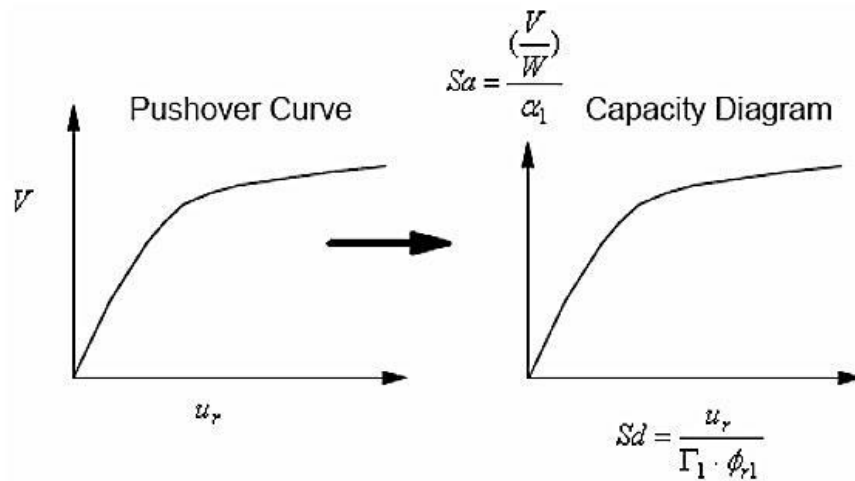


Figure 1.7. Pushover Curve and Capacity spectrum [9]

4. The ADRS spectrum of both the elastic and inelastic responses for a damping of 5% of different levels of ductility are shown in the Figure 1.8, [9].

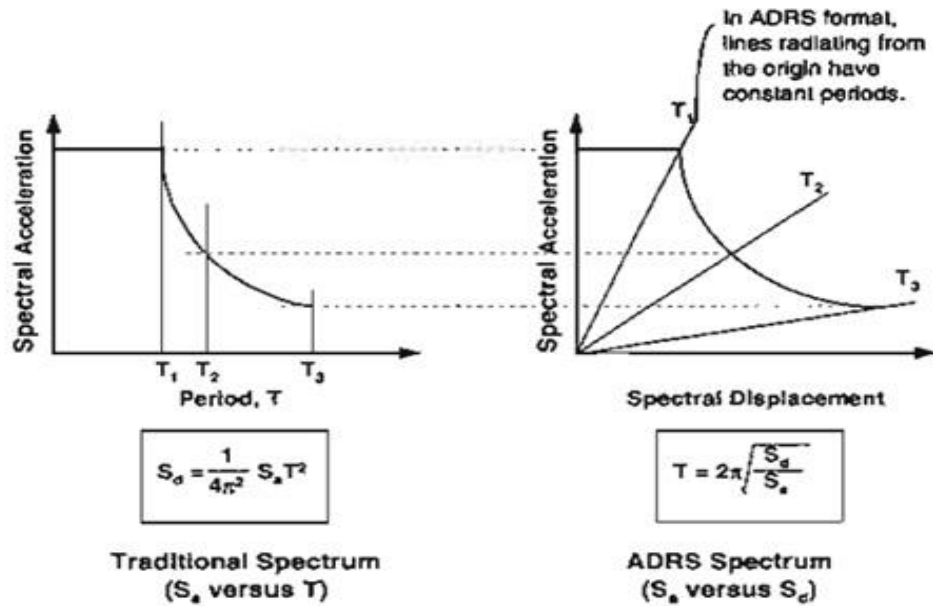


Figure 1.8. Conversion of conventional Spectrum to ADRS Spectrum [9]

The demand and capacity curves are then shown on the same graph as seen in Figure 1.9. below.

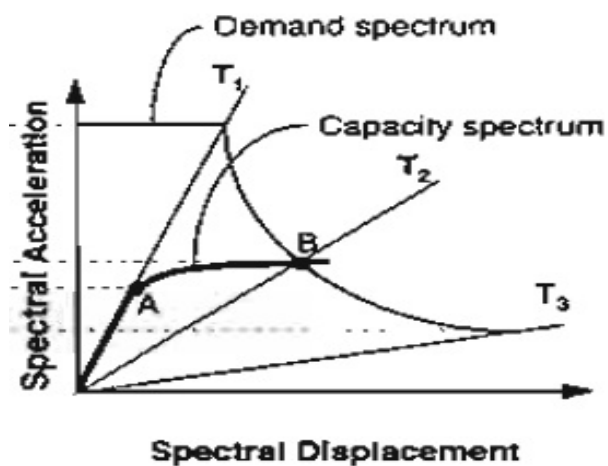


Figure 1.9. Capacity and Demand Spectrum on the same graph [9]

5. Calculate the elastic value at every point of intersection based on the demand and capacity curves. When the calculated elastic equals inelastic demand curve, as shown in Figure 1.10, [41], this point of intersection is called Performance Point.

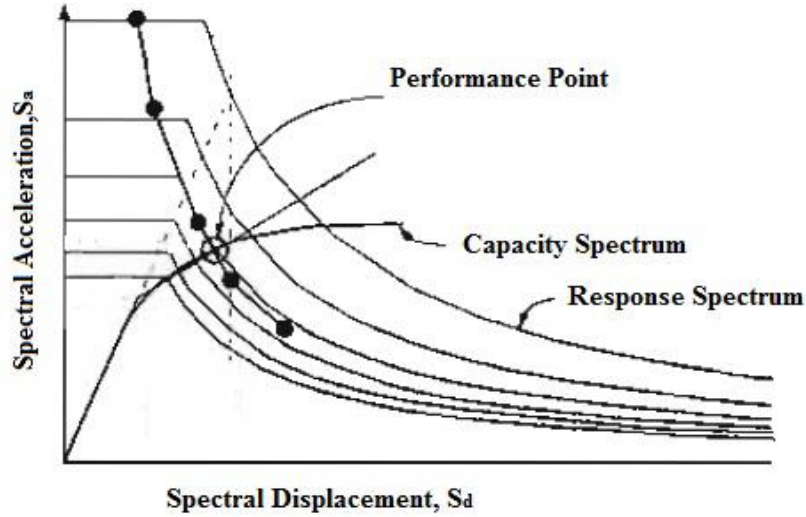


Figure 1.10. Performance Point using the Capacity Spectrum Method [41]

6. Using equation 1.3, we can convert the equivalent SDOF system's inelastic displacement demand into the MDOF system's response.

$$U_R = S_d \Gamma_1 \Phi_{r1} \quad (1.3)$$

1.6.4.2. Displacement Coefficient Method

Using the displacement coefficient approach, it is possible to directly determine the displacement demand numerically. There is no need to transform capacity curve into spectral coordinates. This coefficient method is commonly utilized to determine the target displacement (δ_t) [42]. The steps required for successfully carrying out displacement coefficient method are summarized below.

1. Create a bilinear design of capacity curve as demonstrated by the Figure 1.11, [9].

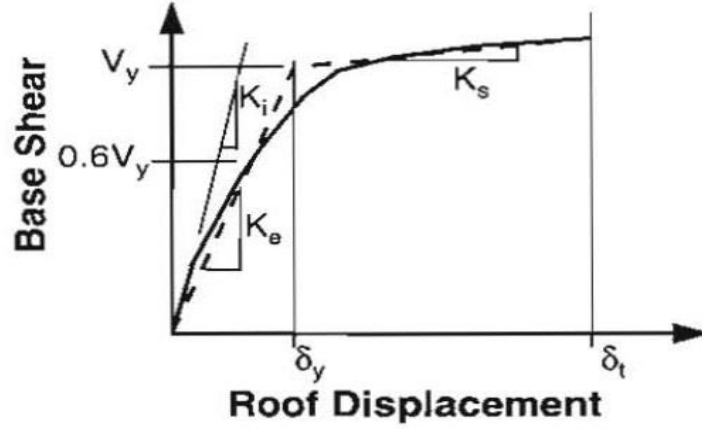


Figure 1.11. Bilinear Design of Pushover curve [9]

2. Using equation 1.4, the effective time period is calculated.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad (1.4)$$

K_i : Elastic lateral strength of the structure.

T_i : Elastic fundamental period (sec).

K_e : Effective lateral strength of the structure

T_e : Effective fundamental period (sec).

3. Determine target displacement by use of equations 1.5 and 1.6.

$$\delta_t = C_0 C_1 C_2 C_3 S_d \quad (1.5)$$

$$S_d = \frac{S_a T_c^2 g}{4\pi^2} \quad (1.6)$$

The estimated maximum inelastic displacement (δ_t) of the control point is given by the equation 1.5. For the considered ground motion, S_d is the elastic spectral displacement

with period T_e . The elastic spectrum displacement may be altered by adjusting the values of the coefficients C_0 , C_1 , C_2 , and C_3 . Below is an explanation of these coefficients [9].

C_0 : Is a factor of modification relating the spectral displacement of a multiple degree of freedom (MDOF) system to the displacement at the control point of the system. It is calculated by multiplying the first mode participation factor by the amplitude of the first mode vector at the control mode level.

C_1 : is a factor of modification relating the expected maximum elastic displacements of a structure to the displacements calculated from linear elastic spectral responses.

$$C_1 = (1 f_0 T_e \geq T_s) \quad (1.7)$$

$$C_1 = \frac{1 + \frac{(R-1)T_s}{T_e}}{R} \quad \text{for } T_e < T_s \quad (1.8)$$

$$R = \frac{S_a C_m}{\left(\frac{V_y}{W} \right)} \quad (1.9)$$

S_a : Is the acceleration of the response spectrum of the system at the effective fundamental period, taking into account the damping ratio.

R : Is ratio of the elastic strength demand to the calculated yield strength of the structure.

T_s : Is the characteristic period of the response spectrum, which is the period at which the spectrum transitions from the constant acceleration segment to the constant velocity segment.

V_y : Is yield strength of the structure, determined utilizing the results of a nonlinear static analysis for the idealized nonlinear force-deformation curve of the system.

C_m : Is the effective modal mass of the structure, determined using eigenvalue analysis for the fundamental mode.

W : Is the structure's weight.

C_2 : Is a factor of modification that represents the effect of trapped hysteretic energy, stiffness and strength degradation, on the maximum displacement response of a structure during an earthquake.

C_3 : Is a factor of modification that represents the increase in displacements due to P- Δ (moment-curvature) effects. If P- Δ effects are not taken into account, the value of C_3 is equal to 1. C_3 value can be calculated by using the equation 1.10.

$$C_3 = \frac{1 + |\alpha|(R - 1)^{1.5}}{T_c} \quad (1.10)$$

α : Is the ratio of post yield strength to effective elastic strength, as shown in Figure 1.11.

1.6.4.3. Modal Pushover Analysis (MPA)

Goel and Chopra presented modal pushover analysis (MPA) in 2001. The primary objective was to simplify the theory of structural dynamics by developing an analytical process that assumed a constant load pattern [43]. The MPA process is summarized by the phases listed below.

1. The first step is to calculate modes (Φ_n) and natural frequencies (ω_n) of the structure system using linear elastic analysis.

2. The second step in MPA is to develop pushover curve, which relate the base shear (V_{bn}) to the top displacement (U_m) of the structure system as a function of the applied force, as given in equation 1.11.

$$S_n^* = m \cdot \Phi_n^i \quad (1.11)$$

Where:

m : Mass matrix.

Φ_n^i : Mod shape of the i-th mode

3. The third step is the incremental application of the force distribution (S_n^*) to the building and record the base shear forces and relatedly roof displacements at each increment. In this case, the building has to be pushed well beyond the target roof displacement in the selected mode.

4. The fourth step is the idealization of capacity curve as a bilinear curve using the FEMA 273 method.

5. The fifth step is the development of the (F_{sn} / L_n) - D_n relationship as shown in Figure 1.8 by scaling the horizontal axis with $\Gamma_n \Phi_n$ and the vertical axis with M_n^* equal to $L_n \Gamma_n$. [9]

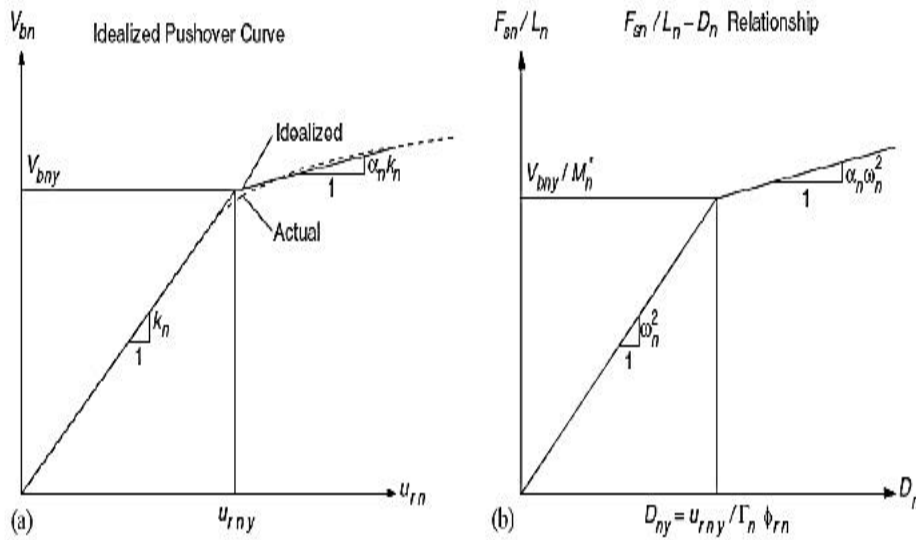


Figure 1.12. Pushover Curve (a) and SDF-System Curve (b) [9]

Figure 1.13. represents the idealized pushover curve of Chopra and Goel.

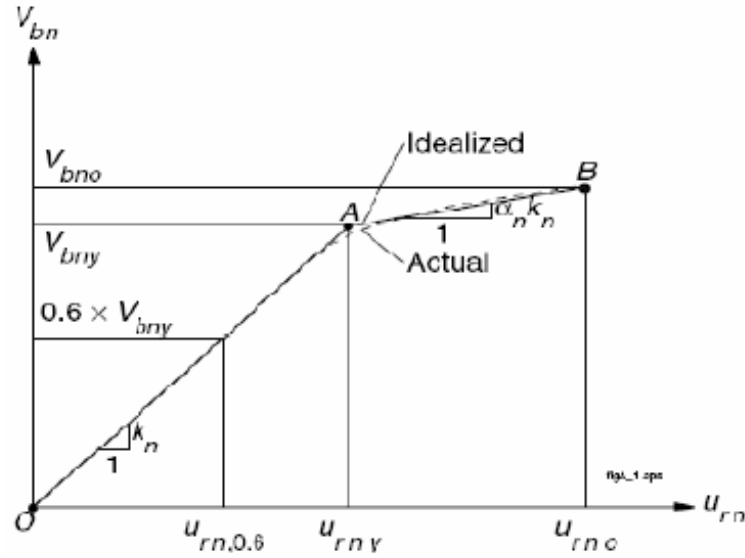


Figure 1.13. Optimizing the Repulsion Curve [9]

6. The sixth step is to find the maximum deformation of the n^{th} mode inelastic representative SDOF system under the selected ground motion excitation, D_n .

7. The seventh step is done by using Equation 1.12. to transform the SDOF system result to MDOF format.

$$U_{mo} = \Gamma_n \Phi_m^i D_n \quad (1.12)$$

Where:

Γ_n : Modal amplification factor for the n^{th} mode

Φ_m^i : Form factor at roof level.

D_n : Peak deformation of the representative SDOF system.

8. Step 8, retrieve the MDOF responses results from the pushover database.

9. Step 9, repeat steps 2 to 8 for different modes.

In most cases, the accuracy requirements can be met using just the first three modes. Typically, the first three modes are enough to fulfill the accuracy requirements.

10. Tenth step, usually to calculate the combined response of a MDOF system either the SRSS rule, that use the peak modal responses of each mode are squared and summed together, and then the square root of this sum is taken to obtain the combined response. Or, Complete Quadratic Combination (CQC) rule, is used. In Figure 1.14, the inelastic MDOF systems concept of MPA is schematically shown.

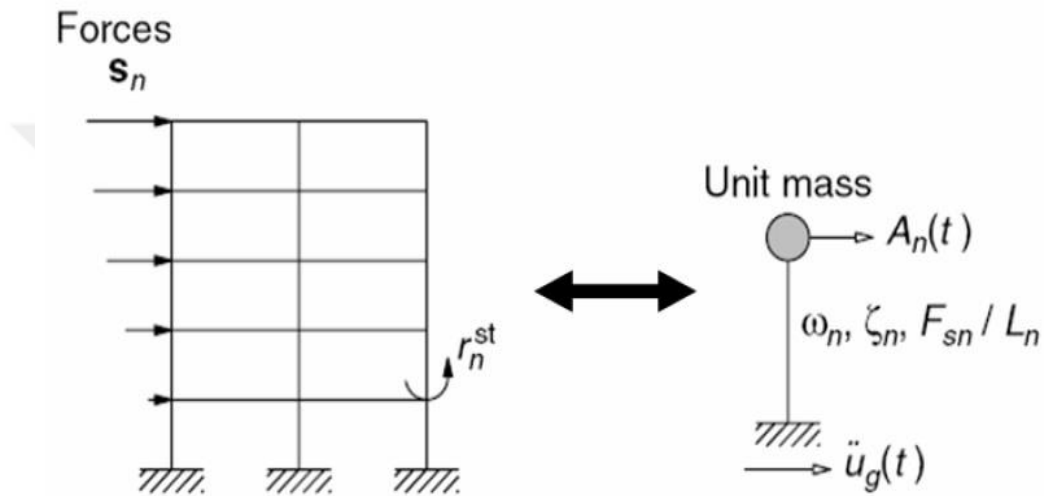


Figure 1.14. MDOF systems of MPA.

1.7. Soil Structure Interaction (SSI)

For many years, the analysis and design of numerous structural systems have been carried without considering the interaction between the soil and the structure (SSI). Later, with advance in research and technology it was found that the building behaviors under analysis is also affected by the underneath soil. Right after this approach is introduced, it gets a lot of attention and many researchers have developed numerous elastic soil structures models [44], [45], [46], [47], [48].

1.7.1. Chronological Evolution of Soil Structure Interaction Models

In the year 1867 Winkler created the first elastic soil model. It was simply composed of independent linear elastic springs that operate exclusively in the vertical direction [49]. Later, Gazetas developed the approach as a discontinuous elastic medium with three-dimensional independent springs. In both models, the spring stiffnesses are the solely soil factors. For better representation of the continuity of the soil, however, two-parameter soil models have been developed integrating the shear effect of the ground.

In the year 1940, Filonenko-Brodich studied the springs in the Winkler model and assume a connection of the springs by an elastic surface membrane at the top. In addition to the soil bearing coefficient parameter in the Winkler model, the constant tensile force obtained is the second ground parameter [49]. In the year 1946, Hetenyi also done some research on the Winkler model springs, he assumed the springs are three-dimensional and are connected by an elastic plate and the problem is assumed to be a 2 dimensional (2D) beam problem. For this model, the bending stiffness of plate or beam is taken as the second elastic soil parameter. In the year 1954, Pasternak examined the Winkler model assuming that the springs only undergo vertical displacement and considering a shear layer consisting of incompressible elements. Later, Vlasov and Leont'ev in 1966 developed that soil model considering the slippage in the soil layer [49].

Currently, numerous researchers provide a variety of models for accounting for effects of soil and building interaction during analysis and design. In 2010, Jialai and Chao studied the 3 parameters elastic foundation model for analyzing adhesively connected joints. They represent the adhesive layer as being made up of two regular spring layers joined by a shear layer. This novel 3 parameters, elastic foundation model allows the shear stresses along the two adherend interfaces of the joint to be different, and therefore, satisfies the equilibrium condition of the adhesive layer [50]. In 2020, Fathizadeh et al. examined the effects of SSI on performance based design optimization of MRFs [46]. They employed an engineered cluster-based genetic algorithm. In 2022, Requena Garcia et al. analysed SSI effects on the Lisbon mid-rise RC buildings' fragility to earthquakes [51]. They focus on evaluating the SSI impacts in seismic vulnerability evaluations of RC structures using two approaches:

Beam on Nonlinear Winkler technique and direct soil modeling, with a purpose of offering a way to effectively integrate the SSI effects and completely characterize the soil behavior.

Elastic foundation models can be grouped into three categories: Single, two and three parameters elastic foundation.

1.7.2. Single Parameter Elastic Foundation Model

This section provides an overview of the single parameter elastic foundation model, its assumptions and limitations in practical engineering problems.

1.7.2.1. Winkler Model

Winkler is the simplest model in which interaction between the ground and the building on it is taken into consideration. In this model, the ground is independent of each other, working only in the vertical direction expressed as a set consisting of linear elastic springs. The vertical ground displacement w is the result of a normal pressure acting only at that point. It is idealized by assuming that it depends only on the spring constant k . Based on this approach, the reaction of the subsoil underneath structure's elastic foundation function can be expressed as in Equation 1.13 [49].

$$q(x, y) = kw(x, y) \quad (1.13)$$

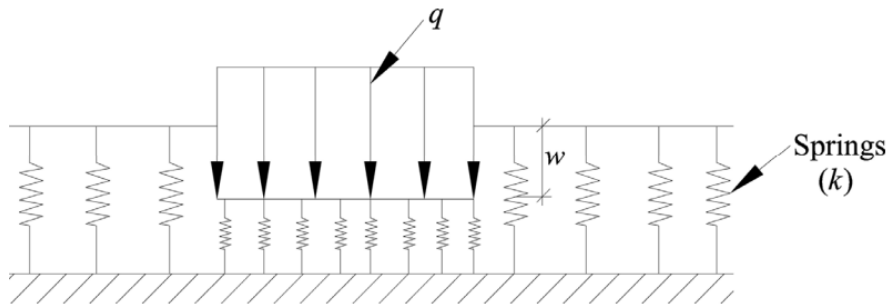


Figure 1.15. Winkler foundation Model [52]

Where:

q : Is the reaction of the soil underneath structure

k : Modulus of subgrade reaction

w : Displacement of subsoil surface

In this case, the foundation undergoes an apartialy surface distributed loading, q , and the springs can only be affected in the loaded area.

1.7.3. Two Parameters Elastic Foundation Model

In order to eliminate some of the limitations of single parameter soil models, two parameters soil models was introduced. These models have been chorolonologically discussed in the previous subsection 1.7 (*The Filonenko-Borodich model, the Hetenyi model, Pasternak model and the Vlasov and Leont'ev model*). In these models, the 1st parameter represents spring constant, as in the *Winkler model*, and 2nd parameter represents springs interaction.

In Filonenko-Borodich model, in addition to Winkler model, the upper part of the springs is assumed to have an elastic shell with a constant tension T on its surface. Thus, continuity between the springs is obtained [52]. The subsoil reaction equation for this model is given in Equation 1.14., Equation 1.15. It can be observed through equations, that the model represents the soil using k and T values where the term two-parameters soil model come from.

1.7.3.1. Filonenko-Borodich Model

$$p = kw - T\nabla^2 w, \text{ for both rectangular and circular foundation} \quad (1.14)$$

$$p = kw - T \frac{d^2 w}{dx^2}, \text{ for only strip foundation} \quad (1.15)$$

Where, $\nabla^2 \equiv$ Laplace operator $\equiv \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$ with T = tensile force

$$q(x, y) = kw(x, y) - T\nabla^2 w(x, y) \quad (1.16)$$

and $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$

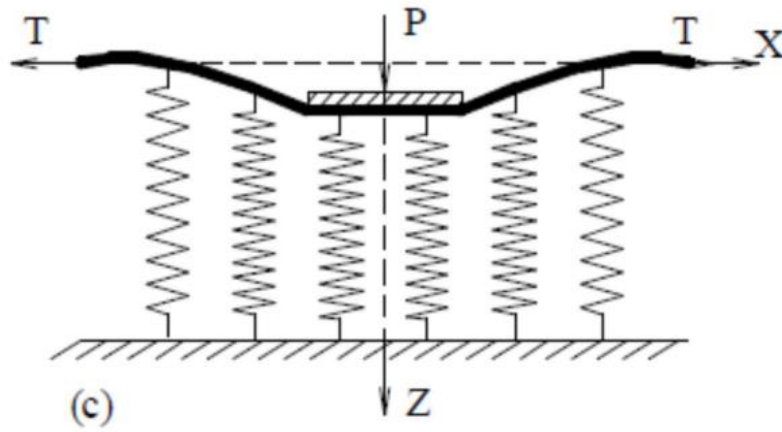


Figure 1.16. Filonenko-Borodich foundation Model [52]

1.7.3.2. Hetenyi Model

Literature indicates the Hetenyi model is a good compromise between the two hypotheses (*Viz Winkler foundation and isotropic continuum*). In the model, each spring interacts with every other spring via an elastic beam or plate that is subjected to deformation exclusively in the flexural direction. [49] [52].

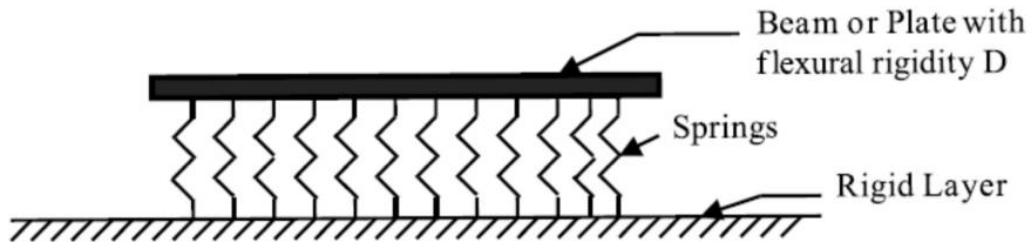


Figure 1.17. Hetenyi foundation model [52]

With

$$p = kw + D\nabla^4 w \quad (1.17)$$

$$\text{Where } \nabla^4 \equiv \frac{\partial^4}{\partial x^4} + \frac{\partial^4}{\partial y^4} + 2 \frac{\partial^4}{\partial x^2 \partial y^2}$$

D = Elastic plate flexural rigidity

$$= (E_p h_p^3) / (12(1 - \mu_p)^2) \quad (1.18)$$

p denotes the pressure at the plate-spring contact. E_p and μ_p are the Young's modulus and Poisson's ratio of the plate material, respectively, while h_p is the thickness of the plate.

1.7.3.3. Pasternak Model

The Pasternak model assumes the existence of a shear interaction between the spring components by connecting the ends of the springs to a beam or plate that undergoes only transverse shear deformation. [52]. In consideration of equilibrium in vertical direction of the shear layer, the relationship between load and deflection is calculated.

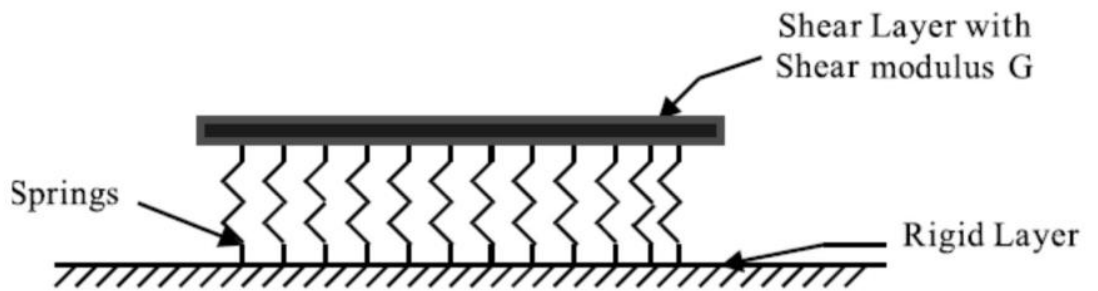


Figure 1.18. Pasternak foundation model [52]

Equation 1.18. expresses the relation between pressure and deflection.

$$p = kw - G\nabla^2 w \quad (1.19)$$

With: G , shear modulus of the shear layer.

This model's continuity is defined by presence of a shear layer. When compared to the Filonenko-Borodich model, it becomes clear that the two are physically equivalent (T has been replaced by G).

1.7.4. Three Parameters Elastic Foundation Model

For elimination of some of drawbacks of double parameters soil models, a novel three parameters elastic soil model has been adopted in this research. Its modeling in the SAP 2000 program was made by referring to the work of Hamarat [53]. The reaction of the subsoil underneath structure's elastic foundation is expressed as shown in the equation 1.19. below.

$$q_z = kw - 2t\nabla^2 w \quad (1.20)$$

Where:

w : Subsoil surface displacement

k : Modulus of subgrade reaction

$2t$: Soil shear parameter

Here both k and $2t$ are 1st and 2nd soil parameters respectively.

With:

$$k = \int_0^{H_s} \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)} \left(\frac{\partial \phi(z)}{\partial z} \right)^2 dz \text{ and,} \quad (1.21)$$

$$2t = \int_0^{H_s} G_s \phi(z)^2 dz \quad (1.22)$$

Where:

E_s : Subsoil modulus of elasticity

G : Shear modulus of subsoil

ν_s : Soil Poisson's ratio

$\Phi(z)$ is a mode shape function that describes vertical displacement change in the subsoil, as shown in the Figure. 1.19. It is expressed as shown by the equation 1.22.

$$\phi(z) = \frac{\sinh \gamma \left(1 - \left(\frac{z}{H_s} \right) \right)}{\sinh \gamma} \quad (1.23)$$

where:

H_s : Subsoil depth to the right base

γ : 3rd soil parameter and is calculated as

$$\left(\frac{\gamma}{H_s} \right)^2 = \frac{(1 - 2\nu_s)}{2(1 - \nu_s)} \frac{\int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} (\nabla w)^2 dx dy}{\int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} w^2 dx dy} . \quad (1.24)$$

In equation 1.22. the values of mode shape function $\phi(z)$ are computed such that $\phi(0) = 1$ and $\phi(H_s) = 0$ [48]. Key point in this case is to ensure that soil shear parameter, $2t$ and modulus of subgrade reaction, k both are dependent on the depth of the subsoil, H_s and mode shape function $\phi(z)$ as expressed in equation 1.20. and equation 1.21. Moreover, the 3rd parameter of the soil, γ , that describe profile of the vertical deformation in subsoil, varies with both displacement of the surface and depth of the subsoil [48][53].

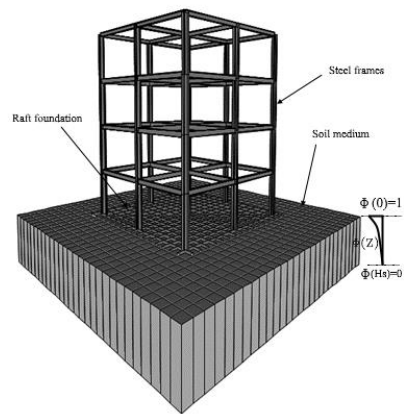


Figure 1.19. 3D geometry of a 3 parameters elastic foundation

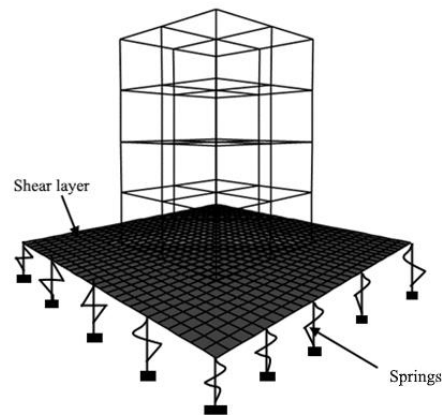


Figure 1.20. Mathematical model of a three parameters elastic foundation

2. MATERIALS AND METHODS

This chapter describes step by step the skills acquired in the provision of the studies carried out as well as the methods and materials used during the research. First, the metaheuristic optimizations were performed using only the Matlab programming interface. After becoming familiar with the MATLAB user interface, the optimization process was enhanced by integrating SAP 2000 software via OAPI. It helped a lot in visualizing the results as well as quickly creating the model using SAP 2000 features. To understand well and follow the procedure, one-, two-, and three-dimensional steel problems were solved. Later, the three-dimensional pushover analysis example using the 2018 TBDY is provided.

2.1. Metaheuristic Optimizations with MATLAB

This part of the study offers a comprehensive description of the MATLAB program, followed by the utilization of metaheuristic optimizations to solve benchmark problems using MATLAB program interface.

2.1.1. MATLAB Program

Since it became commercially available in 1984, MATLAB has been used extensively for research and teaching in business and education [54]. It is a high-level programming environment that allows for interactive numerical computation, graphical representation, and visualization. Modern programming software like MATLAB conveys problems and answers using well-known mathematical language. It has a simple editing system, strong data structures, customizable visuals, and debugging capabilities. It offers many additional benefits over other computer languages like Java, C, C++, FORTRAN among others for tackling technological problems, which makes it an excellent tool. It offers numerous built in functions and various toolboxes that have distinctive codes for tackling specified problems, such as statistics, data analysis, partial differential equations,

optimization, and many more disciplines of engineering and applied sciences. MATLAB may also be used to design algorithms, models, and applications.

MATLAB is an abbreviation for MATrix LABoratory. It was first created for computation of matrixes and convenient access to matrix application softwares EISPACK and LINPACK [54]. However, it has evolved into a broad tool over the years, although it began as a system for quickly and precisely solving matrices. All data are kept in a form of matrix, where each data set is represented in either a scalar, or a vector, or matrix form. This facilitates data handling, and it is from this basis that MATLAB derives its full potential. Due to the presence of exceptionally reliable and extensive predefined built-in functions, the MATLAB programming language is effective in tackling structural design problems using the finite element approach. Although MATLAB's mathematical functions are quite comprehensive, some finite element-specific tasks still need the development of a few extra functions.

2.1.2. Optimization of One Dimensional Problem

This part of the study deals with a simple spring loaded axially. Since the problem is one-dimensional, the axial stress component exists only in the x-direction of the structural body.

2.1.2.1. Case Study I: Spring

The compression tension spring in Figure 2.1. is a well-known engineering optimization problem and has been used to subjectively assess the efficacy of recently developed metaheuristics [29]. It is designed such that it must be able to withstand significant shear stress, frequent surges and minimal deflection while yet being lightweight. The design process is controlled by the three design variables: Coil diameter (D), diameter of the wire (d) and number of active coils (N), indicated by x_1 , x_2 , and x_3 respectively. The following objective function describes the problem $f(x) = (x_3 + 2)x_2x_1^2$ and has been minimized and constrained with the following inequality equations $g(i)$:

$$g_1(x) = 1 - \frac{x_2^3 x_3}{71785 x_1^4} \leq 0$$

$$g_2(x) = \frac{4x_2^2 - x_1 x_2}{12566(x_2 x_1^3 - x_1^4)} + \frac{1}{5108 x_1^2} - 1 \leq 0$$

$$g_3(x) = 1 - \frac{140.45 x_1}{x_2^2 x_3} \leq 0$$

$$g_4(x) = \frac{x_1 + x_2}{1.5} - 1 \leq 0$$

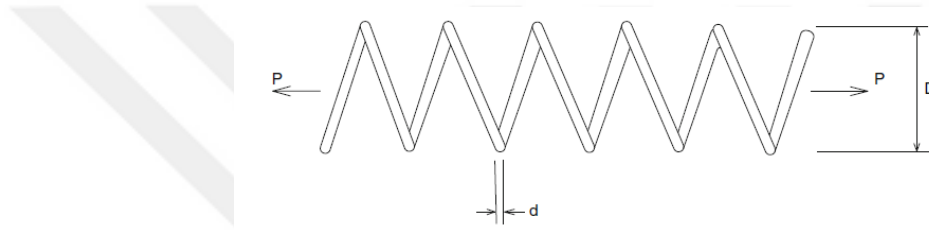


Figure 2.1. Geometry of the steel spring

The design space for each design variable is bounded by [0.05 2] for x_1 , [0.25 1.3] for x_2 and [2 15] for x_3 . The design variables number was taken as 3 while the number of honey bees used in analysis is 50. The number of iterations was set to be 50 iterations and a modification rate factor mr was set to be 0.8.

By using the concept for modeling a constrained optimization problem, the objective function is coded as: `function [x,fit,pfit] = fobj(x,lb,ub)`. With the help of the penalty function concept, a constrained problem can be transformed into an unconstrained optimization problem. `pfit = fit+penalty`

where:

`pfit`: the penalized fitness function

`fit`: is the fitness function and

`penalty = nou * g(i)` with: `nou`: is a big number and is taken as (10^6) in this study.

This Spring example has been solved for the sole purpose of understanding and becoming familiar with Artificial Bee Colony (ABC) procedures. The optimum results of ABC revealed that coil diameter (D) = x_1 = 0.3576, diameter of the wire (d) = x_2 = 0.0517 and number of active coils (N) = x_3 = 11.2445 with a minimum weight of 0.0126.

2.1.3. Optimization of Two-Dimensional Problem

This section briefly describes geometry and loading conditions of the 10 bar truss optimization problem. Since the problem is two-dimensional, the axial stress component exists in both the X and Y-direction of the structural body.

2.1.3.1. Case Study II: The 10-Bar Truss

In order to learn and understand well Teaching Learning Based Optimization (TLBO), a ten bar truss benchmark problem [55] was analyzed. As a well known simple optimization problem, the 10-bar truss design problem has been chosen subjectively to evaluate the effectiveness and robustness of the newly developed metaheuristics. In Figure 2.2, the 10 bar truss is shown schematically, where it is subjected to two dead joint loads of 100 kips (444.82 kN) each. It must be designed for a minimum weight and 2 in maximum joint deflection at node 2. The design variables of the optimization problem are the steel cross section profiles.

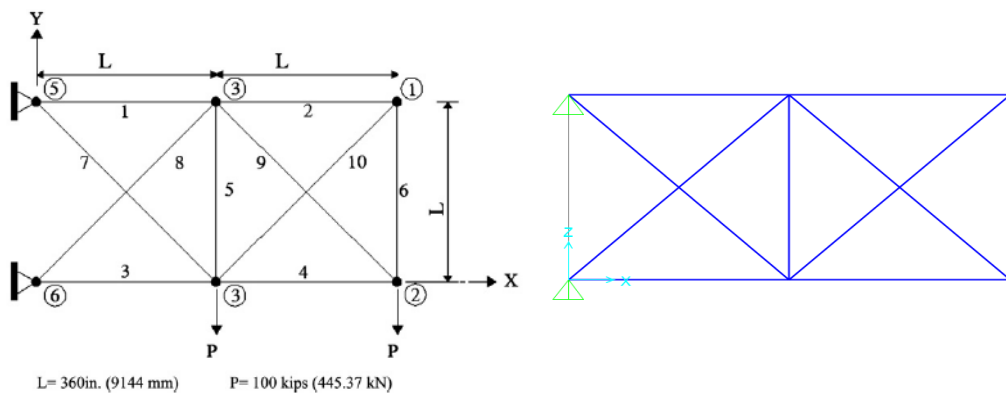


Figure 2.2. Ten bar truss [55]

The design that best minimized the cost expressed in terms of weight is 5016 ^{lb} (22.31) whereas the worse is 5149^{lb} (22.90 kN). 5063 ^{lb} (22.52 kN) and 5090 ^{lb} (22.64 kN) were also found in between the best and worse solution. In the Figure 2.3, optimum solutions are presented, where the vertical y axis denotes weight of the truss in pound (lb) and horizontal x axis the number of iterations.

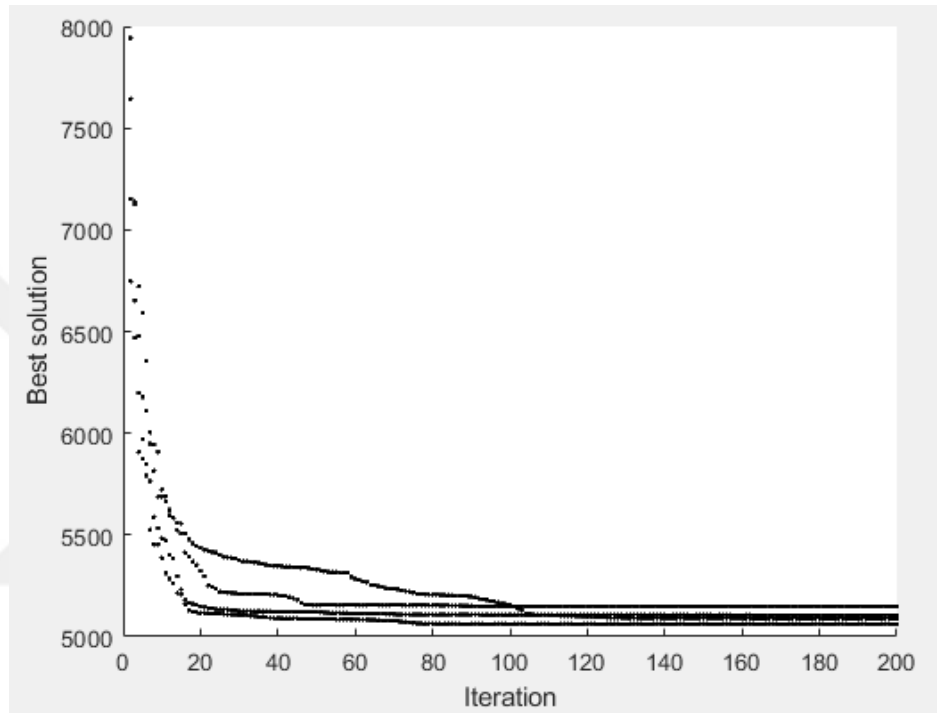


Figure 2.3. Comparison of 10 bar truss convergence curves by TLBO

This 10 bar truss example has been solved for the sole purpose of understanding and becoming familiar with Teaching Learning Based Optimization (TLBO). It has been found that the iteration time of TLBO is almost twice that of ABC. The detailed study of the 10 bar can be found in section 2.2.2.

2.2. Metaheuristic Optimization with MATLAB via SAP 2000 OAPI

Any structure design requires appropriate structure analysis methods in order to carry safely and economically the imposed loads as for its own weight. This was not an easy task as there were a necessity of more sophisticated calculations but with the advance in research and technology, numerous commercial structure analysis software are available now. SAP 2000 is one of them and is widely used by industry researchers and university institutions.

However, these commercially accessible software programs are not designed to be optimization tools. For overcoming this limitation, the Open Application Programming Interface (OAPI) was recently introduced by CSI, the developer of the finite element program SAP 2000. This grants access to the SAP 2000 advanced modules by using different programming languages, thus allowing pre and post analysis computations be efficiently programmed. Taking advantage of this, in this study, various steel structures were optimized by using either TLBO or ABC metaheuristic programs coded on Matlab interface program. The algorithms were able to obtain minimum weight — which is the objective of the problem by selecting suitable steel sections from the already existing sections list, under imposed constraints.

Normally, in most steel systems, the variable space is generally constituted with some wide flange sections offered by AISC steel construction guide [56]. These shapes neither their cross-section area nor moment of inertia or any other section parameters exist on continuous scale making deterministic gradient-based methods (exact methods) impractical. In addition, the optimization of structural systems is usually nonlinear, given that their responses are the result of complex interactions between many elements. Nevertheless, these obvious challenges, the development of novel stochastic techniques, as well as advancements in computer power have made it possible to obtain optimum designs for huge discrete structure optimization problems with a wide range of constraints at a reasonable computational cost.

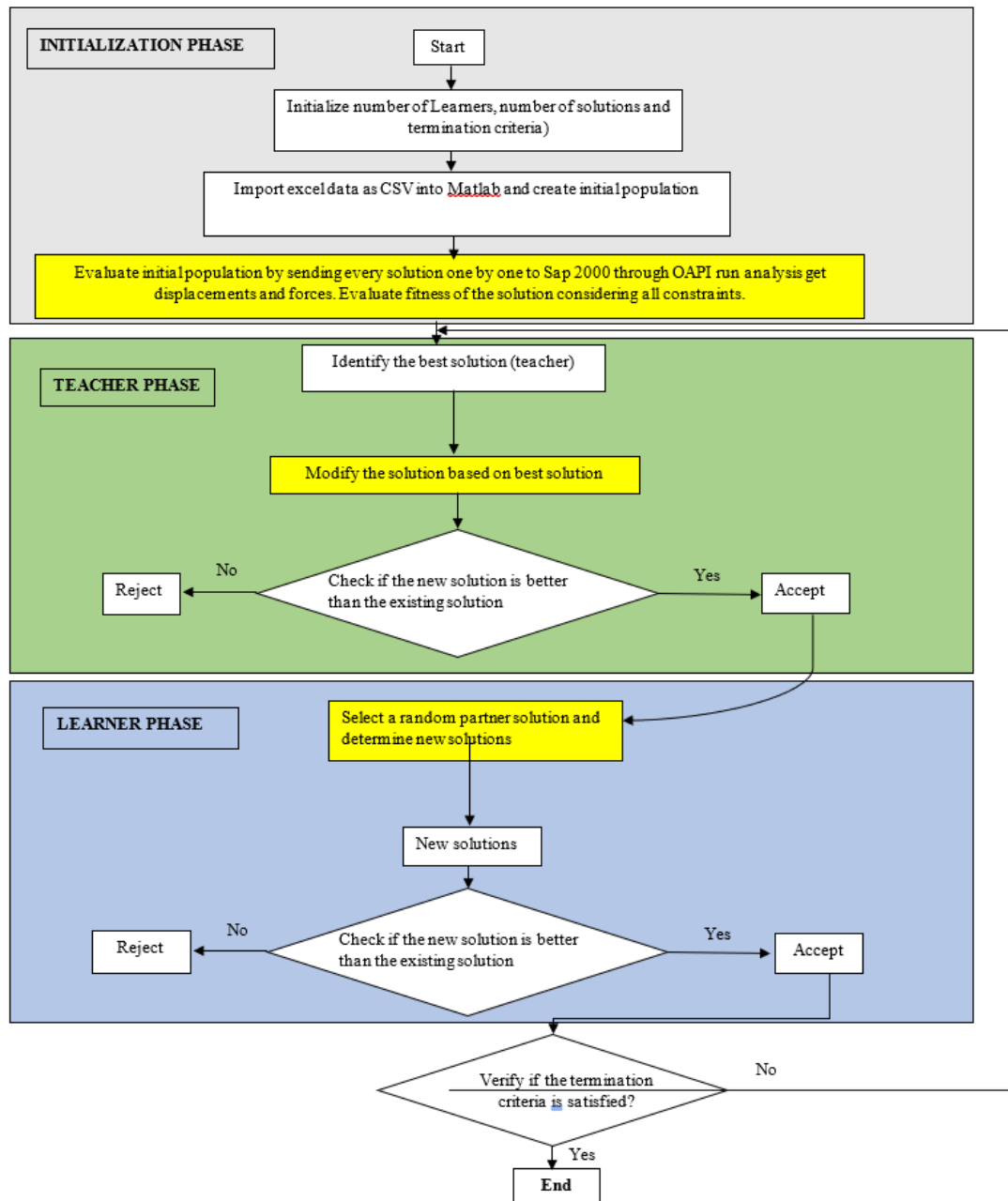


Figure 2.4. TLBO Algorithm involving SAP 2000 through OAPI

2.2.1. Optimization of Two-Dimensional Problem

This part of the study provides the optimization process of the 10 bar truss and the 2D two bays three storey frame with MATLAB via SAP 2000 OAPI. Since the problem is

two-dimensional, the axial stress component exists in both the X and Y-direction of the structural body.

2.2.1.1. Case Study III

The planar 10 bar truss schematically shown in Figure 2.2 is a benchmark problem used for various optimization methods for checking of their efficacy [55],[28]. The truss members has to be chosen from a list of 42 discrete sections 1.62, 1.8, 1.99, 2.13, 2.38, 2.62, 2.88, 2.93, 3.09, 3.13, 3.38, 3.47, 3.55, 3.63, 3.84, 3.87, 3.88, 4.18, 4.22, 4.49, 4.59, 4.80, 4.97, 5.12, 5.74, 5.94, 7.22, 7.97, 11.5, 13.50, 13.90, 14.2, 15.5, 16.0, 16.9, 18.8, 19.9, 22.0, 22.9, 28.5, 30.0, 33.5. Members material properties are: Young modulus (E) = 10000 Ksi (68947.57 MPa) and unit weight of 0.1lb/in³ (27.14 kN/m³). Truss is subjected to two forces of 100 kips (444.82 kN) each at nodes 2 and 3.

The goal is to decrease the truss weight such that the allowable stress in each member should not exceed ± 25 ksi (172.368 MPa) (both in compression and tension), and the permissible vertical displacement must be less or equal to 2 in (5.08 cm). The objective function that minized the truss weight was set as follows:

$$\text{Minimize } W = \sum_{i=1}^N \rho_i L_i A_i$$

Constained by:

$$\sigma^u \geq \sigma \geq \sigma^l$$

$$d^u \geq d \geq d^l$$

$$A^u \geq A \geq A^l$$

With:

ρ : Unit weight (lb/in³)

W : Net weight of steel truss (lb)

N : Total number of element

L : Length (in)

A : Cross sectional area (in²)

σ : Stress (Ksi)

δ : Displacement (in)

Table 2.1. Obtained results in comparison to the existing literatures

Members	Design results by Daloğlu & Toğan [55]	Design results by Sönmez [28]	Design results by Sousa & Takahashi [ref]	This Study
1	33.5	33.5	33.5	33.5
2	1.62	1.62	2.13	1.62
3	22.9	22.9	22	22.9
4	16	14.2	13.9	13.9
5	1.62	1.62	1.62	1.62
6	1.62	1.62	1.99	1.62
7	7.97	7.97	7.97	7.97
8	22.9	22.9	22.9	22.9
9	19.9	22	22.9	22
10	1.62	1.62	1.62	1.62
Weight (lb)	5448.62	5490.74	5525.04	5478.32
Max. deflection (in)	2.0173	-	-	2.048

The Table 2.1. above demonstrate a comparison of the optimum design results provided in the literature and this study. The optimum structure found by Toğan and Daloğlu [55] is the lightest of all of them.

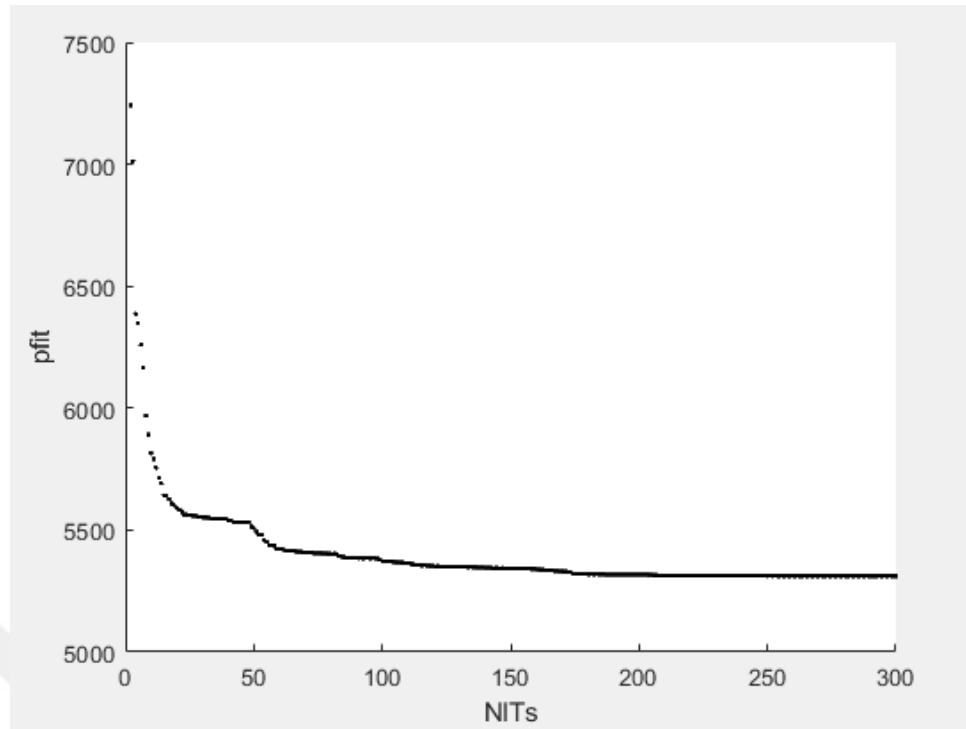


Figure 2.5. Design history for 10-bar truss

As shown by the graph above the algorithm converges to the solution at 212th iteration and kept the same design solution until the termination criteria was met which was set to 300 iterations.

2.2.1.2. Case study IV: 2D Two Bays Three Storey Frame

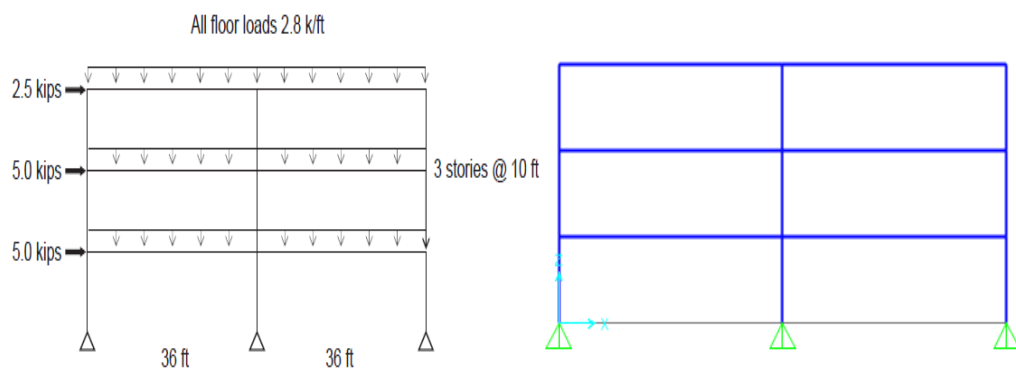


Figure 2.6. 2D two bays three storey frame [32]

Figure 2.6. illustrates the geometry and loading status for a 15 member 2-bays, three-storey frame (9 columns and 6 beams) [32]. Both columns and beams have to be selected from 267 predefined W-shapes with a manufacturing constraint requiring that all nine columns must be identical as for the six beams. Additionally, columns have to be selected from the first 18 W shapes of W10. The material has a Young's Modulus (E) of 29 000 ksi (199947.9615 MPa) with a yield stress $f_y=36$ ksi (248.21 MPa).

Frame has to be designed such that it satisfy the AISC-LRFD (serviceability and strength conditions) specifications while minimizing the weight of the steel structure frame. Objective function that minized the structure weight was set as follows:

$$\text{Minimize } W = \sum_{i=1}^{N_e} L_i w_n (\eta_i) \quad (2.1)$$

with:

L_i : is the nominal length of element i,

N_e : is the total number of elements in the frame,

w_n : is the unit weight of η_i steel profile for i^{th} element of sections AISC database.

AISC-LRFD (strength and serveability conditions) including allowable interstory drift, displacement limits were considered as constraints and applied on an unconstrained problem by penalizing the above mentioned objective function. The structure's penalized weight (pfit) was calculated by adding penalty to the fitness weight.

$$pfit = fit + penalty$$

where:

pfit: penalized weight

fit: is the fitness weight

$$\text{and penalty} = nou * g(i)$$

where:

nou: is a big number (10^9)

g(i) : AISC-LRFD constraints as discussed below.

The (AISC-LRFD) standard permits the following stresses for elements subjected to axial and bending force [56].

$$\text{For } \frac{P_u}{\phi P_n} \geq 0.2 \quad \frac{P_u}{\phi P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1 \quad (2.2)$$

$$\text{For } \frac{P_u}{\phi P_n} < 0.2 \quad \frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1 \quad (2.3)$$

Where:

P_u : Required axial strength (*Tension and Compression*)

P_n : Nominal axial strength (*Tension or Compression*)

M_{ux} : Required flexural strength about the major axis

M_{uy} : Required flexural strength about the minor axis (*as the frame is 2D the M_{uy} is equal to zero*)

M_{nx} : Nominal flexural strength about the major axis

M_{ny} : Nominal flexural strength about the minor axis

The factor of resistance for compression and tension, as well as the reduction factor for flexural, is equal to 0.9.

For roof and inter storey drifts constraints C_{d2} and C_{d1} was considered respectively, where both constants must be less or equal to 1. With:

$$C_{d2} = \frac{\delta_t}{\delta_{lt}} \quad \text{and} \quad C_{d1} = \frac{\delta_n - \delta_{n-1}}{\delta_{ls}} \quad (2.4)$$

Where:

δ_t : Displacement of the top storey

δ_{lt} : The Maximum allowable lateral displacement for the top storey ($H/300$) with H being total height of the building.

δ_n : Displacement of n^{th} storey

δ_{n-1} : Displacement of storey below storey n

δ_{ls} : The maximum allowable inter storey displacement ($h/300$) with h being the height of a typical storey.

Table 2.2. Obtained results in comparison to the existing literatures

Member	Design results by Camp et al. (ACO) [32]	Design results by Chen (GA) [57]	Design results by (SBO) [32]	This study (TLBO)
Beams	W24X62	W24X62	W24X62	W24X62
Columns	W10X60	W10X60	W10X60	W10X60
Weight (lb)	18792	18792	18792	18792
Mean Weight(lb)	-	22080	18792	-

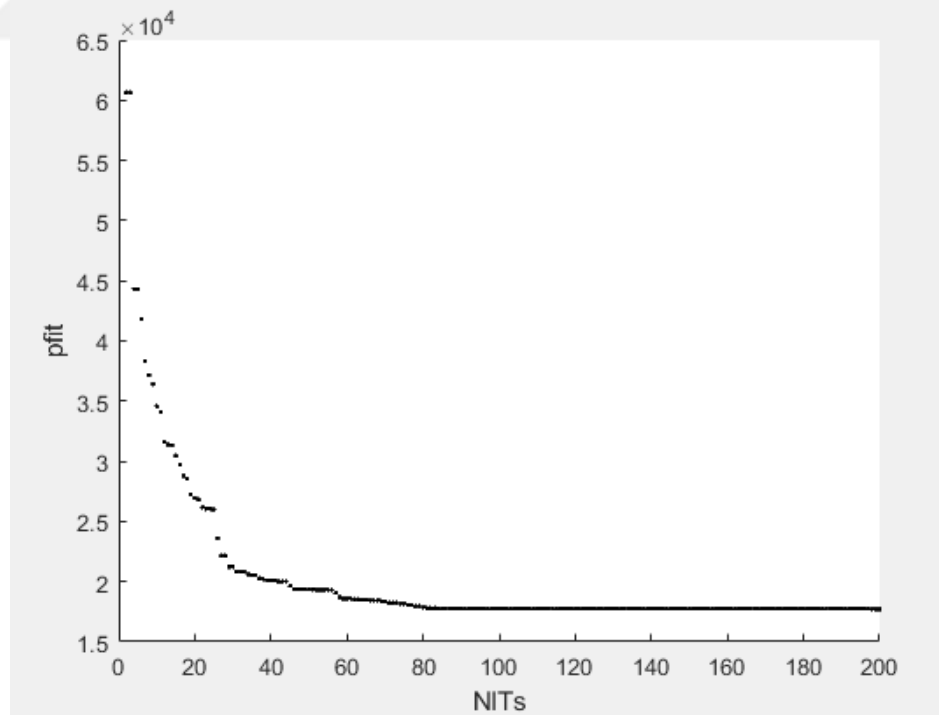


Figure 2.7. Design history for 2D frame

As shown by the graph above in Figure 2.7 the algorithm converges to the solution at 84th iteration and kept the same design values until the termination criteria was met which was set to 200 iterations.

2.3. Metaheuristic Optimizations and Pushover Analysis

Generally, code provisions allow structure and earthquake engineers analyze structures in the elastic range of their structural materials. However, with an increased number of structure failure due to hazardous phenomena like wind and earthquakes; some codes adapted to evaluating structure beyond elastic limit. As post-elastic behavior cannot be clearly characterized by commonly used elastic analysis methods, a nonlinear analysis approach is employed for evaluation purposes in this situation [58].

When the induced natural hazardous to the structure is an earthquake, a seismic performance evaluation should be performed considering the post elastic behavior of structural members (Which is the case for this research). Besides, the maximum inelastic displacement demand of structure must be computed in order to accurately estimate the seismically-induced demands on structure behavior in inelastic range.

For this purpose, there exists different nonlinear analysis procedures for estimation of structures' maximum inelastic displacement demand [36], [59]. However, pushover the performance-based nonlinear analysis and design procedure which was often used as the most effective technique for nonlinear analysis and design because of being more easy than dynamic nonlinear procedures is widely used [3], [60], [61]. The main reason why it is adopted in this research as well.

This magic nonlinear analysis tool demonstrates the real nature of gradual breakdown in structures and pinpoints the mechanism of ultimate collapse. Using a hinges-based approach, this technique examines the structure and identifies its potential weak areas by progressively assessing and recording the damage sequence of each element of the structure.

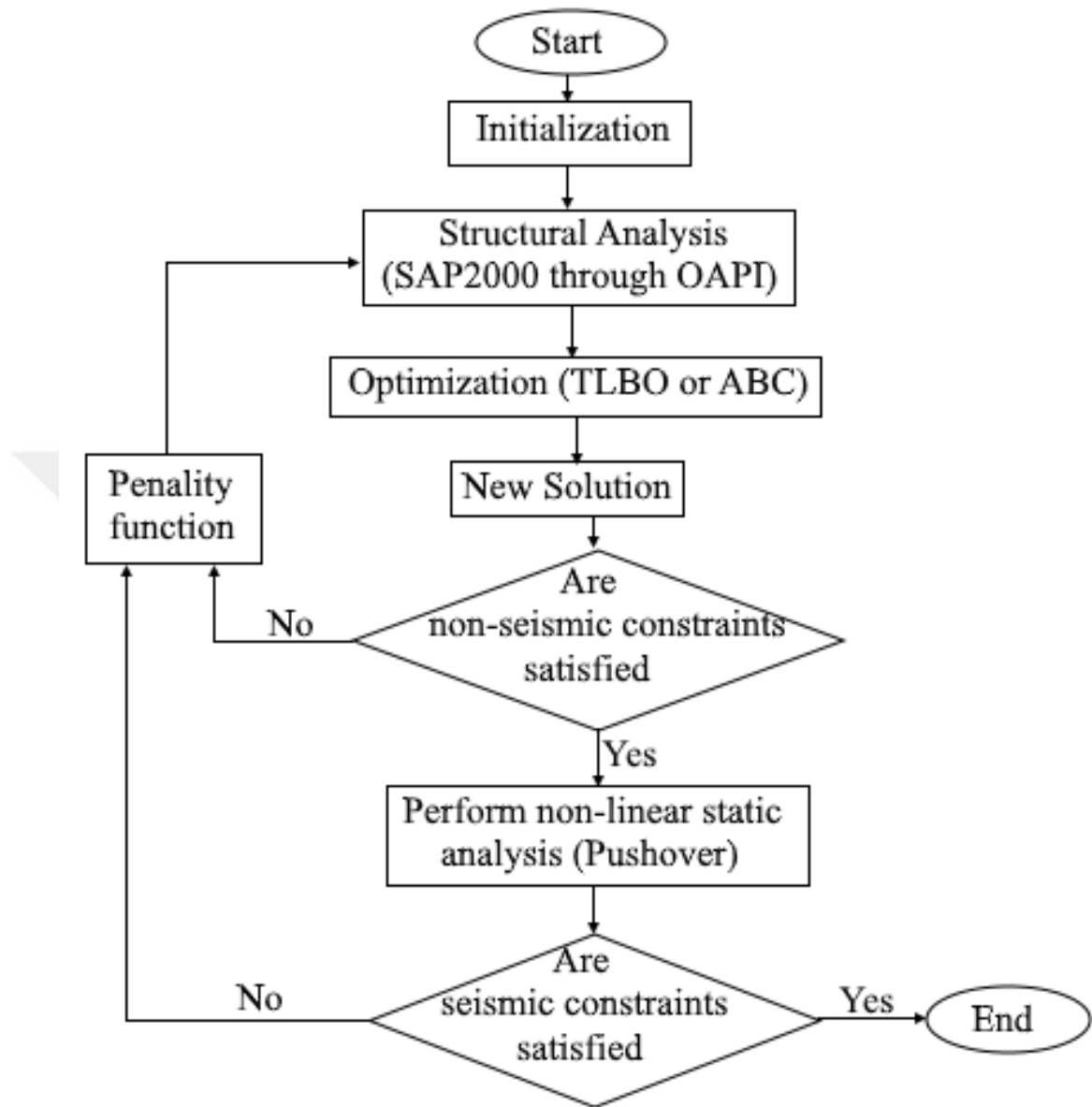


Figure 2.8. The pseudo-code of ABC and TLBO algorithms

2.3.1. Optimization of Two-Dimensional Steel Frames

In this section Seismic Performance Based Design (SPBD) of a 6-Story 2D Moment Resisting Steel Frame (MRSF) is performed using TLBO via SAP 2000 OAPI and the results found are compared to the literature. Since the problem is two-dimensional, the axial stress component exists in both the X and Y-direction of the structural body.

2.3.1.1. Case study V: 6-Story 2D Moment Resisting Steel Frame (MRSF)

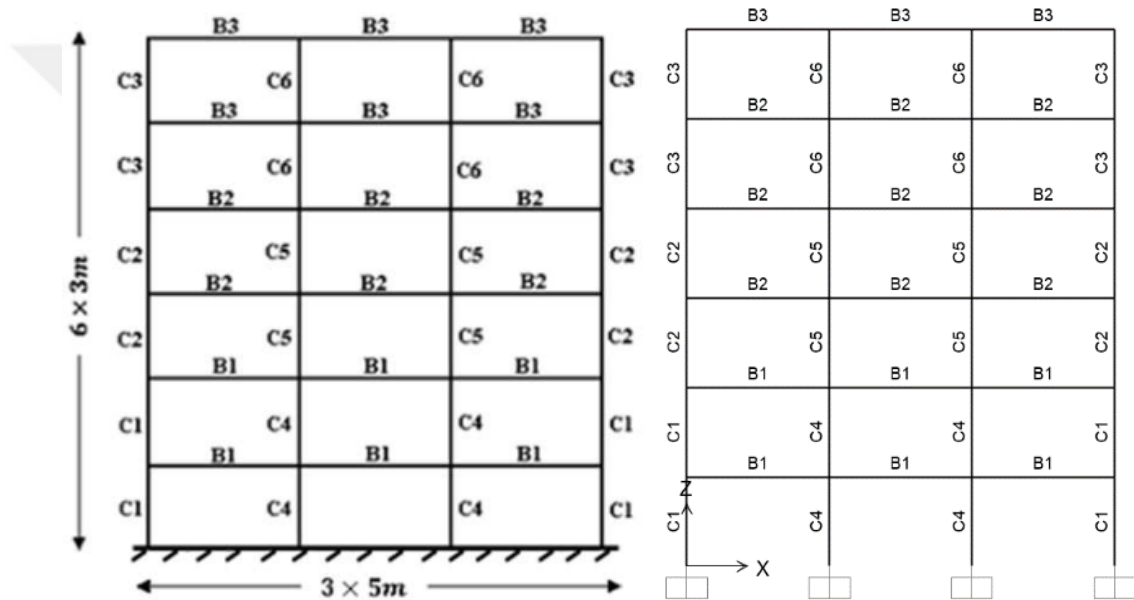


Figure 2.9. Six story moment resisting steel frame with a fixed base

The 6-story MRSF shown in Figure 2.9 above was studied by different researchers. Recently, Geno et al. [46] studied the same frame considering the bottom columns as both fixed and with the soil structure interaction. During analysis the frame members have been divided into nine groups ($C1, C2, C3, C4, C5, C6, B1, B2$ and $B3$) (as shown in the Figure 3.10), and a PBD with a desire for finding out how the structure will behave in four different performance levels were examined. The four considered Performance Levels are: *Operational (OP)*, *Immediate Occupancy (IO)*, *Life Safety (LS)* and *Collapse Prevention*

(CP). It is important to note that the risk levels shown in Table 2.3 below have been taken into account while determining these performance levels.

Table 2.3. Performance Levels and their corresponding Risk Levels [46]

Earthquake having probability of exceedance	Mean return period	Performance level
20%/50 year	225	IO
10%/50 year	474	LS
2%/50 year	2475	CP

Materials and applied loads of the studied steel frame systems are presented in Table 2.4 below:

Table 2.4. Material properties and loading conditions of steel frame systems [46]

E (MPa)	fy (MPa)	Poisson ratio	Dead load (kN/m)	Live load (kN/m)
210000	235	0.3	24.52	9.81

A dead load (SDL) of 24.52 kN/m and live load (LL) of 9.81 kN/m have been considered. For gravity loads combination Gravity (QG) = 1.1 SDL+1.1 LL were considered.

The lateral loads considered are defined as follows:

$$F_i = V \frac{W_i h_i^k}{\sum_{j=1}^n W_j h_j^k} \quad k = \begin{cases} 1 & T \leq 0.5 \\ \frac{T+1.5}{2} & 0.5 < T \leq 2.5 \\ 2 & T > 2.5 \end{cases} \quad (2.5)$$

Where :

F_i : Lateral load applied at at i^{th} storey

V : Base shear

W_i, h_i and W_j, h_j : are the weights and heights of i^{th} and j^{th} storey respectively.

T : Natural period of the structure system.

$$\delta_t = C_{00} C_{11} C_{22} C_{33} S_a \frac{T_e^2}{4\pi^2} g$$

$$T_e = T_i \sqrt{\frac{k_i}{k_e}}$$

The target displacement d_t was determined using the above equation according to FEMA 356 [9].

Where:

C_{00}, C_{11}, C_{22} and C_{33} : are modification factors.

S_a : Response Spectrum Acceleration (RSA) at effective fundamental period

T_e and T_i : Effective and elatic fundamental periods of the frame respectively.

K_e and K_i : Effective and elatic lateral stiffness of the frame structure respectively.

g : Earth gravity acceleration.

Table 2.5. Values of the modification factors considered for calculation of the target displacement

Parameters	Value
C_{00}	1.42
C_{11}	1
C_{22}	1.2
C_{33}	1

The design variables for frame optimization problem are the steel cross section profiles of the members chosen from the W-shaped American profiles, and objective function to minimize the structure weight is defined as:

$$\text{Minimize } W = \sum_{i=1}^N \gamma_i L_i A_i \quad (2.6)$$

Subjected to constraints of the problem R1, R2, R3,R4 as described below.

Where:

W : Weight of the structure

γ_i : weight of unit volume

N : Number of design variable

L_i : i th length of structure member

A_i : Cross sectional area

R1: cross sectional geometry constraint

The area A of the bottom column's cross section must be larger or equal to the area A of the top column's cross section.

$$R_1 = \frac{A_{i+1}}{A_i} - 1 \leq 0 \quad (2.7)$$

With i being the story number

R2: Strong column weak beam constraint

$$R_2 = \frac{\sum M_{pc}}{\sum M_{pb}} \geq 1 \quad (2.8)$$

Where:

M_{pc} : plastic moment of column

M_{pb} : plastic moment of beam

R3 Strength constraint

AISC 360-16 requires all structural components to be evaluated for 1.2D+1.6L, which is a non-seismic or gravity load combination [56].

Where:

D : is Dead load

L : is Live load

Referring to AISC standard, maximum allowed stress for elements susceptible to axial and bending forces are calculated using *Equation (2.2) and Equation (2.3)*.

R4: Drift constraints

Based on FEMA 356 each storey's maximum drift cannot exceed the permitted values shown in table 2.6 below.

$$R_4 = \frac{\delta_k^i}{\delta_{all}^i} - 1 \leq 0 \quad (2.9)$$

$i = IO, LS$ and $CP, k=1,2,\dots$

Table 2.6. Allowable drift at various Performance Levels (PL) [9]

Performance level	Allowable drift
IO	0.7%
LS	2.5%
CP	5.0%

Table 2.7. Optimal design results by Geno et al. (fixed base bottom columns) and this study (TLBO)

Members	Design results by Geno et al.[46]	This Study (TLBO)
C1	W21X48	W24X55
C2	W21X44	W18X55
C3	W21X44	W18X46
C4	W18X76	W21X57
C5	W21X62	W21X55
C6	W21X55	W18X46
B1	W21X44	W21X44
B2	W21X44	W18X50
B3	W16X26	W14X34
		11.35
Weight tons (kN)	11.01 (107.97)	(111.31)

Table 2.7. compares the optimum design results published in the literature to those found in this study using TLBO.

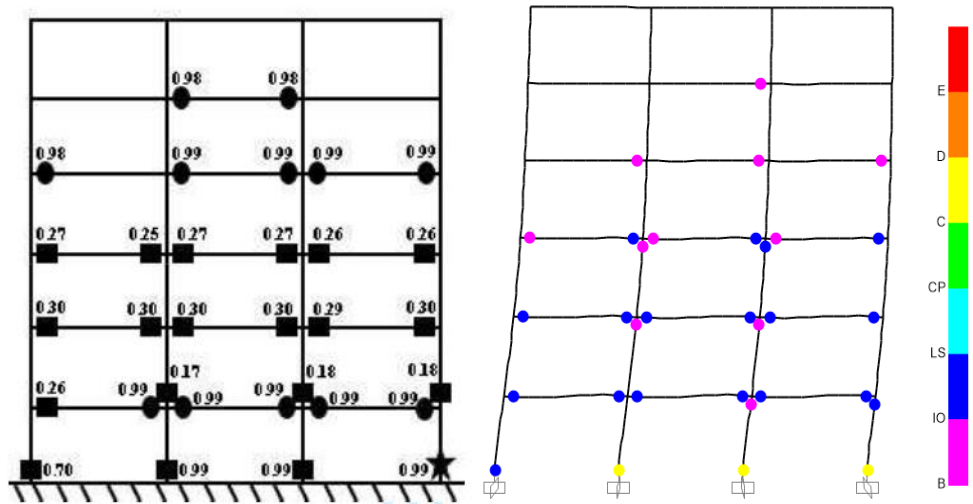


Figure 2.10. Hinge status for Geno et al. and this study [46]

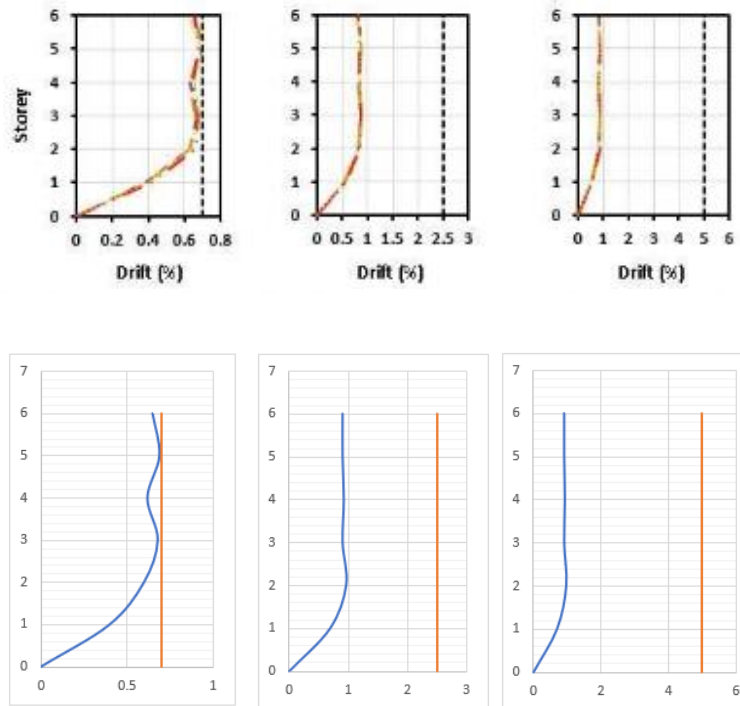


Figure 2.11. Inter story drifts at various performance levels for 6 story MRFs [46]

The inter story drifts at performance levels for the optimum 6 story moment resisting frame and the obtained story drift diagrams from PBD of the six story frame and the Performance Levels (*IO*, *LS*, and *CP*) are demonstrated in the Figure 2.11.

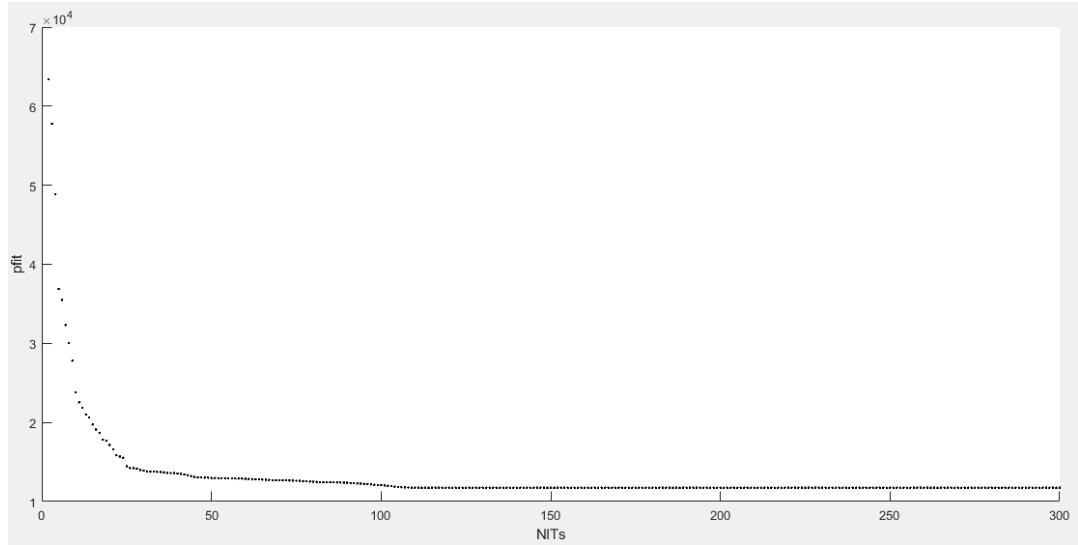


Figure 2.12. Design history for six story 2D MRSF

As shown by the Figure 2.12, the TLBO algorithm converges to the solution at 114th iteration and kept the same design solution until the termination criteria was met which was set to 300 iterations.

2.3.2. Optimization of Three Dimensional Problem

This section evaluates the Seismic Performance Based Design (SPBD) of a Four-story 3D Moment Resisting Steel Frame (MRSF) using TLBO via SAP 2000 OAPI. Since the problem is three-dimensional, the axial stress component exists in X,Y and Z-direction of the structural body.

2.3.2.1. Case Study VI: Four Story 3D Moment Resisting Steel Frames

The four story 3D MRSF shown in Figure 2.13 below was analysed for gravity and earthquakes loads by using the TBDY 2018. During analysis the bottom of base column were considered as fixed. The frame members were divided into ten groups ($G1, G2, G3, G4, G5, G6, G7, G8, G9$ and $G10$). The building was studied for PBD with a desire

of finding out how the structure will behave during earthquake and determine its performance level.

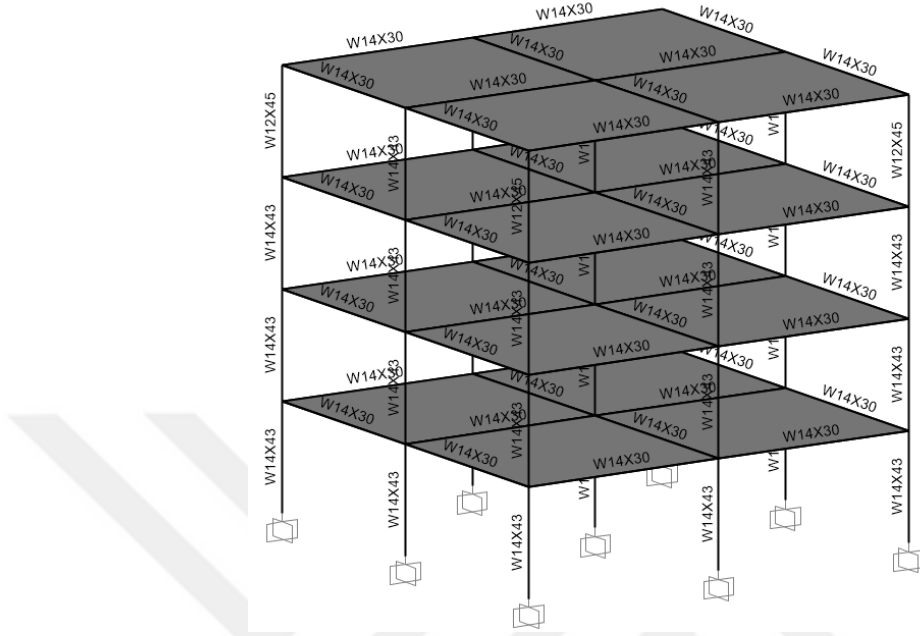


Figure 2.13. Four story 3D MRSF section profiles

2.3.2.1.1. Performance Objectives and Deformation Limits as Per TBDY 2018

The performance objectives and deformation limits for steel beams and columns as defined by TBDY 2018 were shown in the Figure 3.15. [62]

Tablo 5C.1. Çelik Kirişlerde Plastik Dönme Sınırları

Kiriş (Eğilme)	Şekildeğiştirme Sınırları		
	SH	KH	GÖ
Süneklik Düzeyi Yüksek (Bkz.Tablo 9.3)	10_y	60_y	90_y
Süneklik Düzeyi Sınırlı (Bkz.Tablo 9.3)	0.250_y	30_y	40_y

Tablo 5C.2. Çelik Kolonlarda Plastik Dönme Sınırları

Kolon (Eğilme)	Plastik Dönme Sınırları [rad]		
	SH	KH	GÖ
$P/P_c < 0.20$ (Bkz.Tablo 9.3)			
Süneklik Düzeyi Yüksek	10_y	60_y	90_y
Süneklik Düzeyi Sınırlı	0.250_y	30_y	40_y
$0.20 \leq P/P_c \leq 0.50$ (Bkz.Tablo 9.3)			
Süneklik Düzeyi Yüksek	$1.5 (1 - 1.66 P/P_c) \theta_y$	$9 (1 - 1.66 P/P_c) \theta_y$	$13.5 (1 - 1.66 P/P_c) \theta_y$
Süneklik Düzeyi Sınırlı	0.250_y	0.70_y	10_y
Kayma Bölgesi	10_y	90_y	120_y

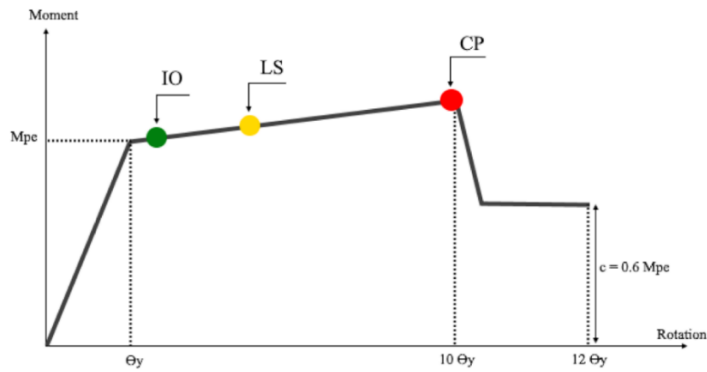


Figure 2.14. Steel structure performance levels and deformation limits as per TBDY 2018 [62].

Performance Levels	Deformation limits
<i>Immediate Occupancy (IO) = Sınırlı Hasar (SH)</i>	$\theta_p = \theta_y$
<i>Life Safety (LS) = Kontrollü Hasar (KH)</i>	$\theta_p = 6\theta_y$
<i>Collapse Prevention Level (CP) = Göçmenin Önlenmesi (GÖ)</i>	$\theta_p = 9\theta_y$

2.3.2.1.2. Modal Analysis of the Four Story 3D Moment Resisting Steel Frame

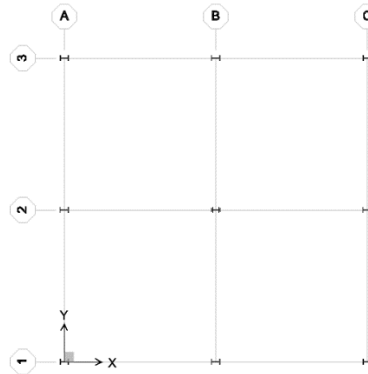


Figure 2.15. The four-story 3D MRSF plan view

The four story 3D MRSF can be analysed by using only the first primary mode as the mass participation ratio is greater or equal to 70% of total mass, and is geometrically symmetric in plan for both x and y directions.

Table 2.8. Four-story 3D moment resisting steel frame modal mass participation ratios

TABLE: Modal Participating Mass Ratios					
OutputCase	StepType	StepNum	Period	UX	UY
Text	Text	Unitless	Sec	Unitless	Unitless
MODAL	Mode	1	0.417534	0	0.85617
MODAL	Mode	2	0.339216	0	0
MODAL	Mode	3	0.265891	5.01E-20	0.01854
MODAL	Mode	4	0.22884	0.74308	0
MODAL	Mode	5	0.20221	4.05E-18	0

2.3.2.1.3. Hinges Definition and Assignment

The plastic hinges used in the model to get track of the local behaviors of the structure elements and overall performance of the four story 3D MRSF is shown in the Figure 2.16.

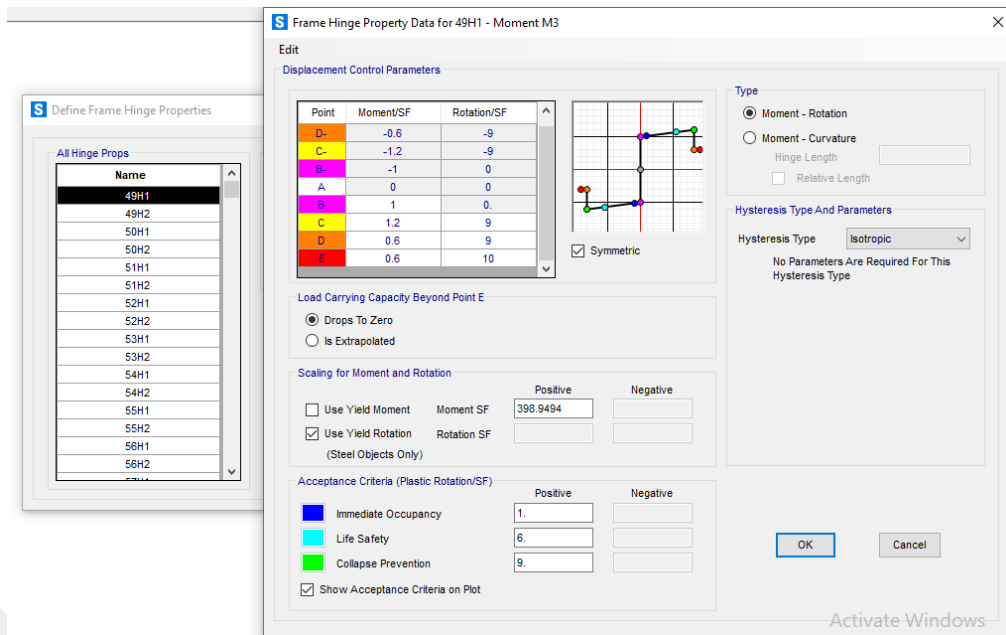


Figure 2.16. Definition of hinges used in model

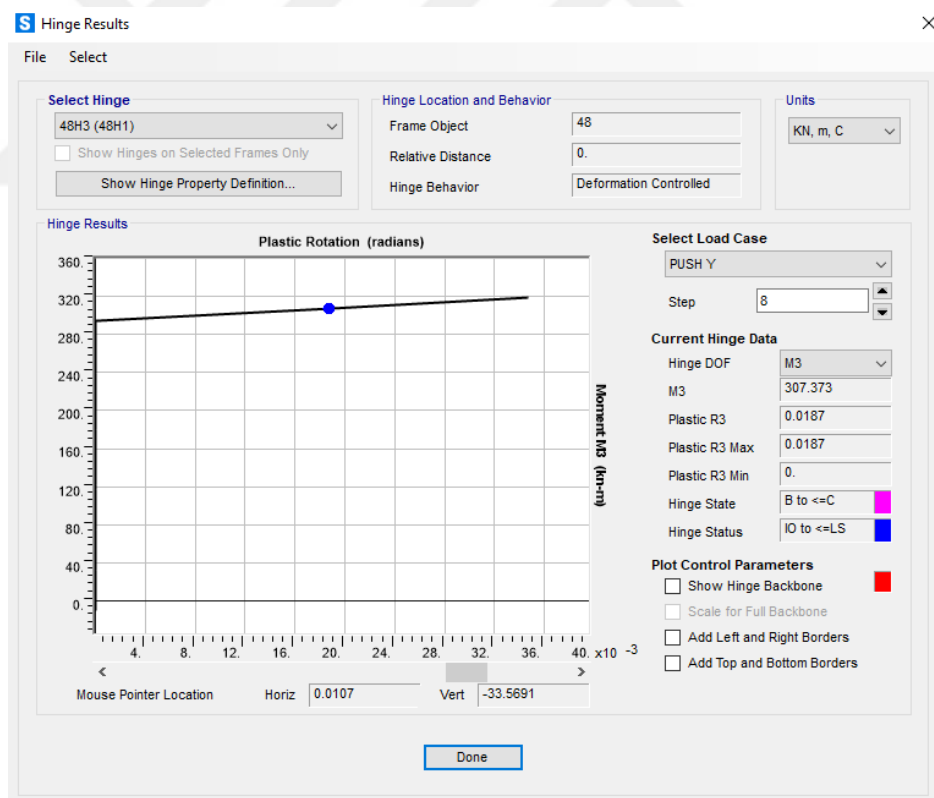


Figure 2.17. Hinge results

2.3.2.1.4. Four Story 3D Moment Resisting Steel Frame Design Spectrum

The earthquake ground motion level with a recurrence period of 475 years and a probability of exceedance 10% in 50 years was utilized.

1. Building category (Bina Kullanım sınıfı = BKS-1)
2. Earthquake design level was assumed to be DTS-1 a referred to the Table 3.2 of TBDY 2018 [63]

Tablo 3.2 – Deprem Tasarım Sınıfları (DTS)

DD-2 Deprem Yer Hareketi Düzeyinde Kısa Periyot Tasarım Spektral İvme Katsayısı (S_{DS})	Bina Kullanım Sınıfı	
	BKS = 1	BKS = 2, 3
$S_{DS} < 0.33$	DTS = 4a	DTS = 4
$0.33 \leq S_{DS} < 0.50$	DTS = 3a	DTS = 3
$0.50 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

3. Building height category (BYS -6)

The total height of the building is 12.4m. By means of Table 3.3 the building height category is classified into BYS - 6

Tablo 3.3 – Bina Yükseklik Sınıfları ve Deprem Tasarım Sınıflarına Göre Tanımlanan Bina Yükseklik Aralıkları

Bina Yükseklik Sınıfı	Bina Yükseklik Sınıfları ve Deprem Tasarım Sınıflarına Göre Tanımlanan Bina Yükseklik Aralıkları [m]		
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a	DTS = 4, 4a
BYS = 1	$H_N > 70$	$H_N > 91$	$H_N > 105$
BYS = 2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
BYS = 3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
BYS = 4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
BYS = 5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
BYS = 6	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
BYS = 7	$7 < H_N \leq 10.5$	$10.5 < H_N \leq 17.5$	
BYS = 8	$H_N \leq 7$	$H_N \leq 10.5$	

Based on all the data given above, elastic spectrum was determined as follows based on TBDY 2018.

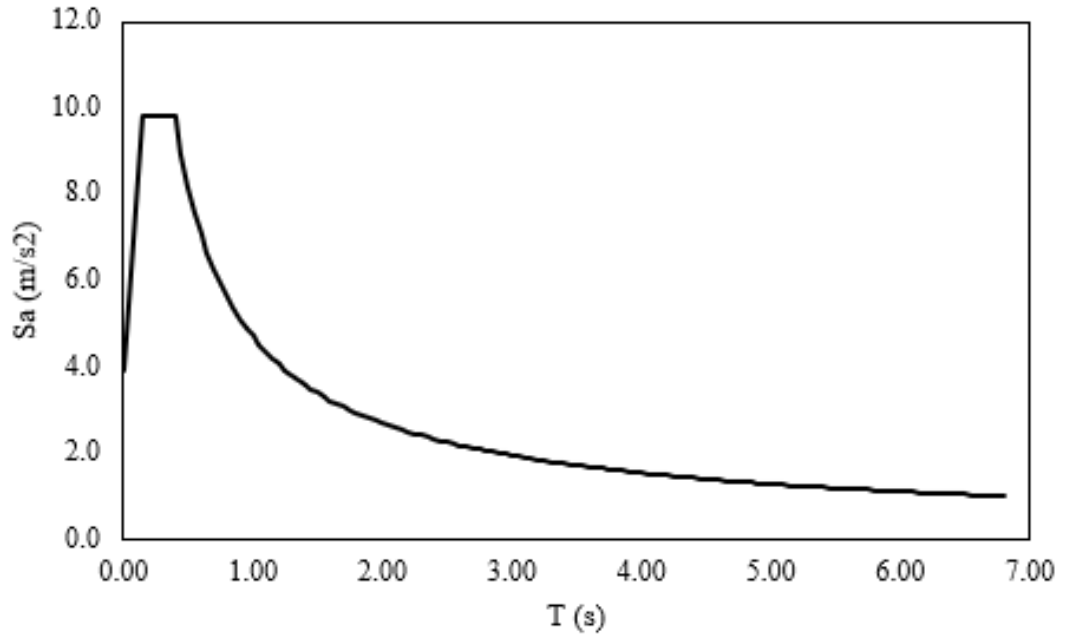


Figure 2.18. Elastic Spectrum

2.3.2.1.5. Performance Evaluations

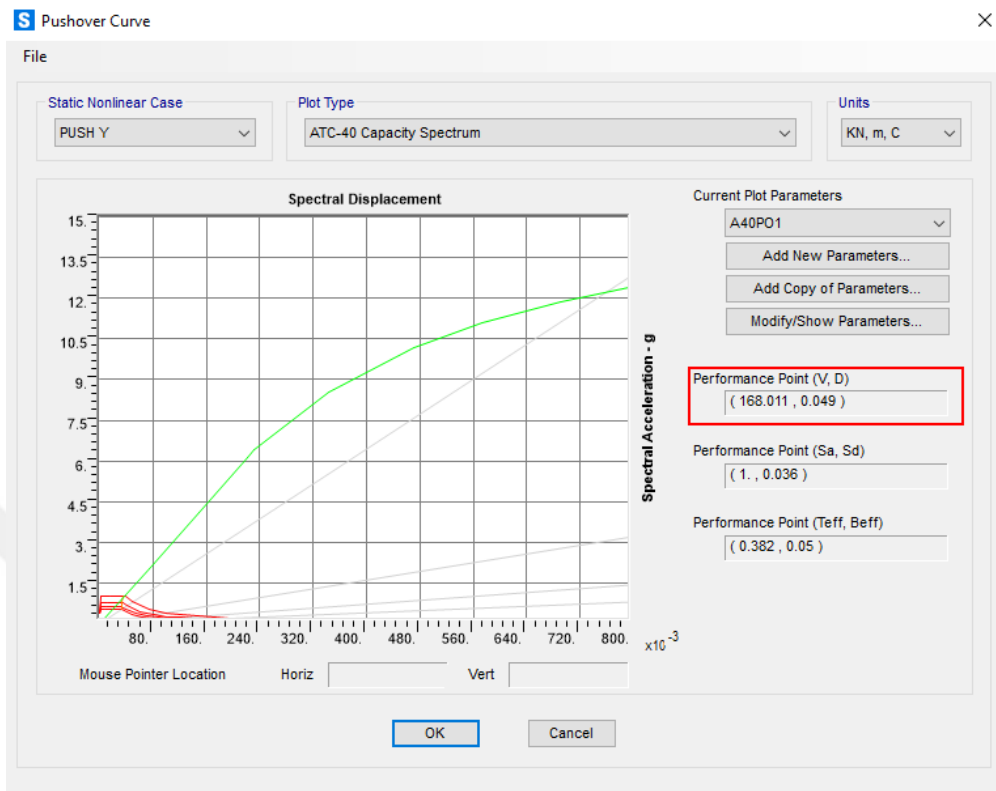


Figure 2.19. Pushover curve in Y- direction

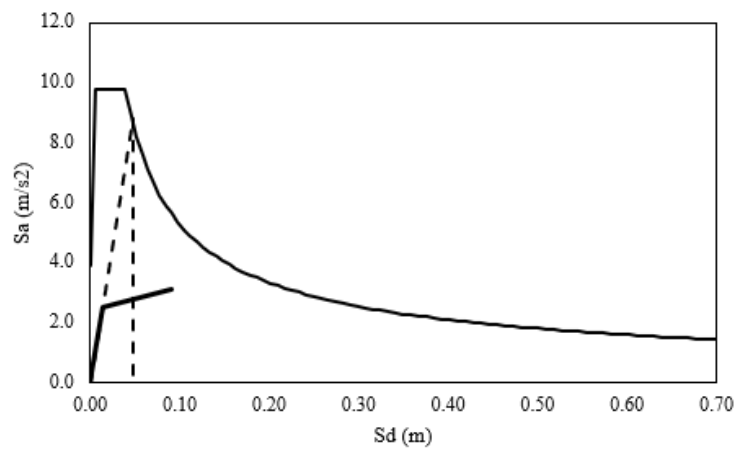


Figure 2.20. Excel Pushover in Y direction

The SAP2000 results were compared to the hand calculation done in excel in figure 2.20 above. The performance point is 0.049m and 0.047m in SAP2000 and Excel respectively.

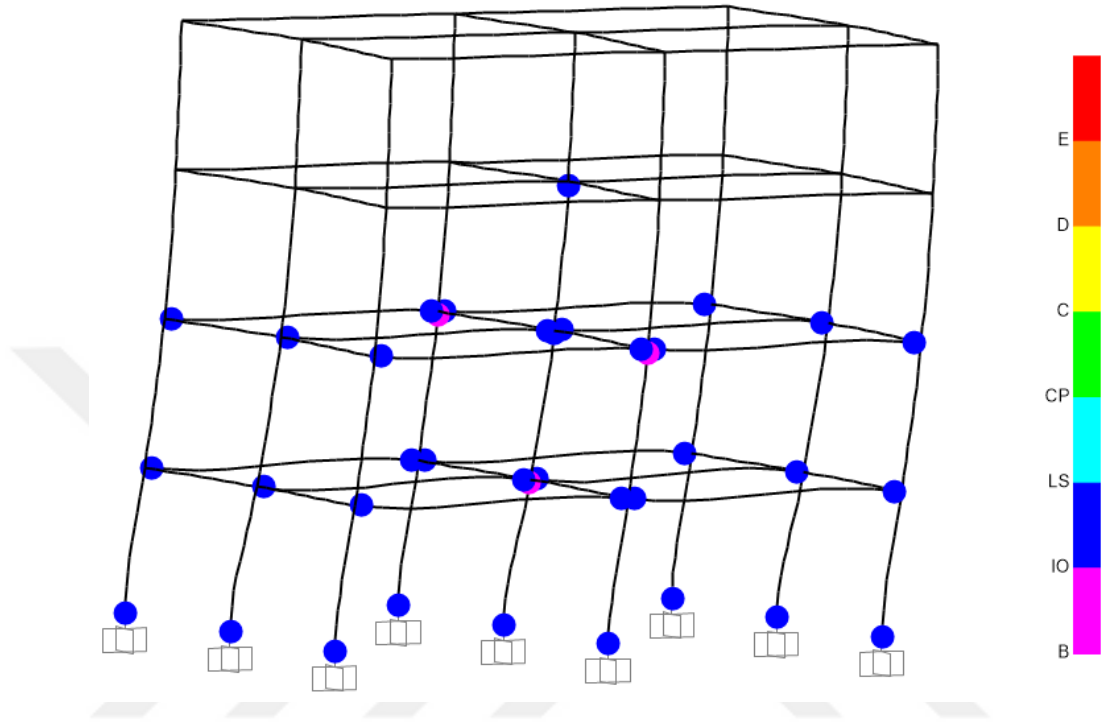


Figure 2.21. Performance point hinges state

The overall performance point of the building is Immediate Occupancy (IO)

2.4. ABC and TLBO for Discrete Seismic PBD Optimization of Planar SMFs via SAP 2000-MATLAB Integration

In this section, two problems of discrete PBD optimization of 3, and 6 story 2D MRSFs are designed. Cross-sections profiles of all elements are selected from the 267 W-shaped AISC section profiles. The modulus of elasticity for all elements is 210GPa and a yield stress of 235MPa. In addition to their own weight, all beams are subjected to a dead loads (DL) and Live Loads (LL) of 2500kg/m (24.52 kN/m) and 1000 kg/m (9.81 kN/m), respectively. TLBO and ABC coded using MATLAB program interface and connected to

SAP 2000 through OAPI were used for optimization and overall evaluation purpose. For both TLBO and ABC, the maximum iteration number of algorithm evaluation was set to a fixed number which is 300. The 3-, and 6-story planar SMFs optimum weights presented their nonlinear behaviors.

To model the planar frames this study used nonlinear static analysis (pushover analysis) the SAP 2000 software and following four assumptions were taken into consideration.

1. The *Uniaxial Steel* material model.
2. The *Nonlinear Beam Column* SAP 2000 auto built-in hinges have been used for simulating plasticity the structural elements' flexibility behavior.
3. The *P- Δ effects* have been taken into account in all analysis.
4. All frames were assumed to have fixed supports and rigid columns-beams connections

For this study, 3 performance levels were considered: *Immediate Occupancy (IO)*, *Life Safety (LS)* and *Collapse Prevention (CP)*. Inter-story drifts are allowed to be 0.7%, 2.5%, and 5% at the *IO*, *LS*, and *CP* performance levels, respectively. Table 2.9. displays Performance Levels (PL) and their respective risk levels.

Table 2.9. The Performance Levels and their corresponding Risk Levels [9]

Earthquake having probability of exceedance	Mean return period	Performance level
20%/50 year	225	IO
10%/50 year	474	LS
2%/50 year	2475	CP

The frames are designed to satisfy the AISC-LRFD (serviceability and strength conditions) conditions while minimizing the weight of the steel structure system. Objective function that minized the structure weight was set as follows:

$$\text{Minimize } W = \sum_{i=1}^{N_e} L_i w_n(\eta_i) \quad (2.10)$$

Where :

N_e : Number of element groups in the frame,

L_i : Length of element i,

w_n : Nominal weight of η_i steel profile for element i chosen from the AISC section library.

2.4.1. Case Study VII: The 2- Bays 3- Story Steel Smfs

The geometry and details on groups of the 2- bays 3- story steel SMFs are shown in Figure 2.23 below, the frame has 6 beams and 9 columns classified into 3 beam and 4 column groups, respectively.

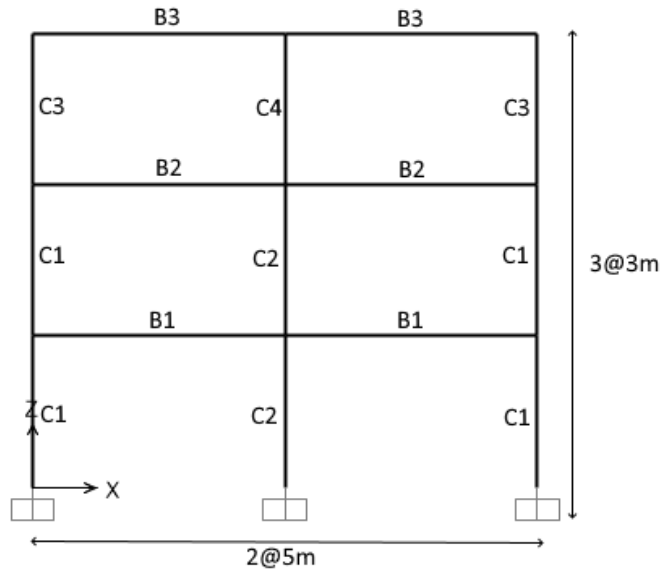


Figure 2.22. The 2- bays 3- story steel SMFs

Figure 2.23 and Figure 2.24 show the algorithms converge to the solution at 144th and 198th iterations for TLBO and ABC respectively. The results also indicated that, TLBO metaheuristic algorithms converged earlier to the best solution compared to ABC.

Table 2.10. Comparison of results to the existing literature

Design Variables	MFA [64]	ECBO [64]	NMA [64]	This study, TLBO	This study, ABC
C1	W14 x 38	W14 x 38	W14 x 38	W12 x 40	W14 x 43
C2	W16 x 45	W16 x 45	W16 x 45	W16 x 50	W16 x 50
C3	W12 x 30	W12 x 30	W12 x 30	W12 x 30	W12 x 35
C4	W16 x 45	W16 x 45	W16 x 45	W16 x 40	W16 x 45
B1	W18 x 35	W18 x35	W18 x 35	W18 x 35	W18 x 35
B2	W18 x 35	W18 x 35	W18 x 35	W18 x 35	W18 x 35
B3	W14 x 22	W14 x 22	W14 x 22	W14 x 22	W14 x 22
Minimum Weight kg (kN)	2919.6 (28.63)	2919.63 (28.63)	2907.5 (28.51)	2975.75 (29.18)	3098.1 (30.38)

For both cases the optimization results are presented in Table 2.10.

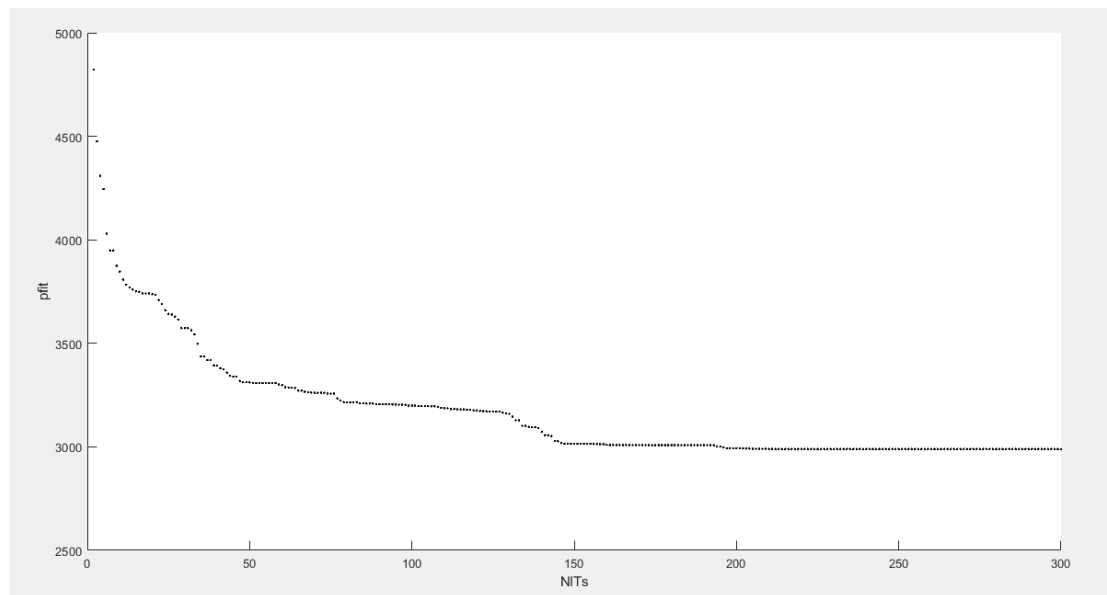


Figure 2.23. Design history of the 2- bays 3- story steel SMFs for TLBO

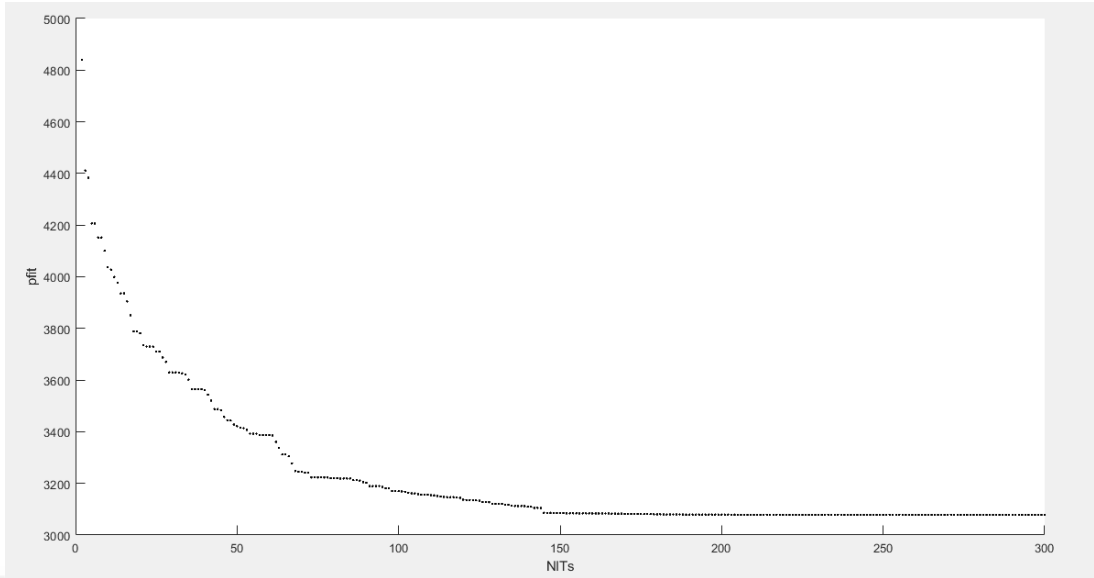


Figure 2.24. Design history of the 2-bays 3-story steel SMFs for ABC

Figure 2.26 presents the IO, LS, and CP drift profiles of the optimum solution obtained by TLBO and ABC, respectively. In this figure, the vertical axis denotes story number while horizontal axis represents inter-story drift in percentage. The results demonstrated that IO level constraints dominated the designs.

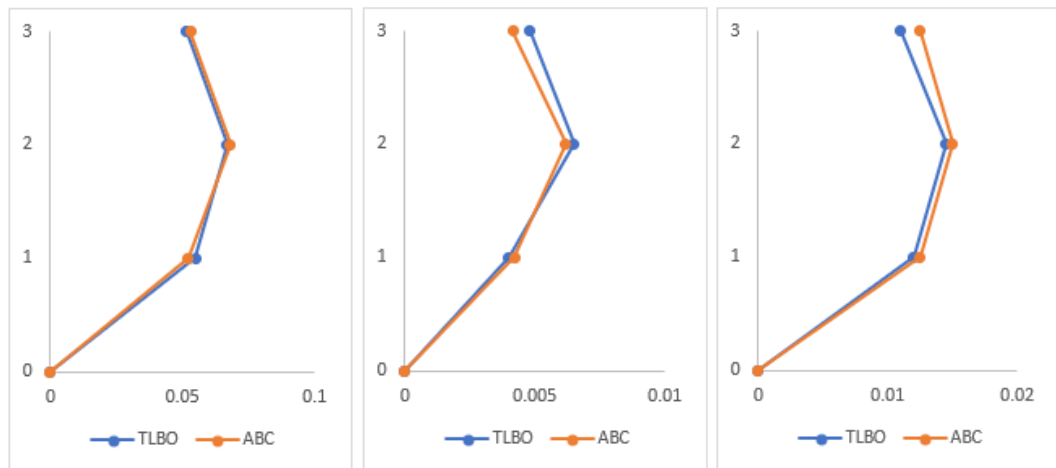


Figure 2.25. Inter story drift for IO, LS and CP Performance Levels (from left to right)

The 2 bays- 3-story frame plastic hinge dispersion of the optimum members size found by TLBO under the pushover loading condition at the *IO*, *LS* and *CP* Performance Levels are shown in Figure 4.4. Where all members' plastic hinge rotations did not surpass the plastic rotation specified range.

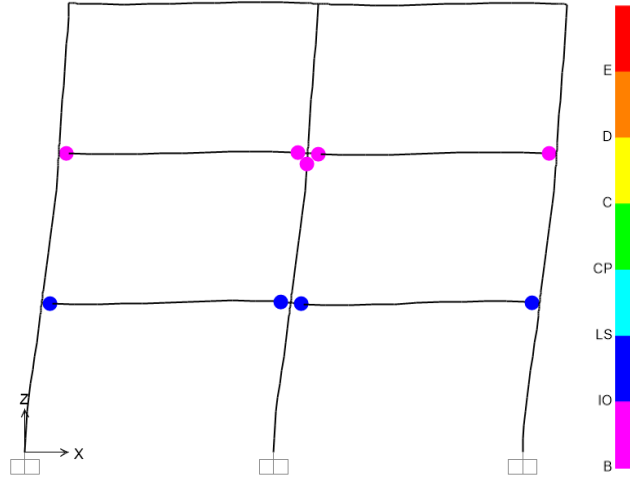


Figure 2.26. The 2 bays 3- story steel frame plastic hinges for six-story at IO level

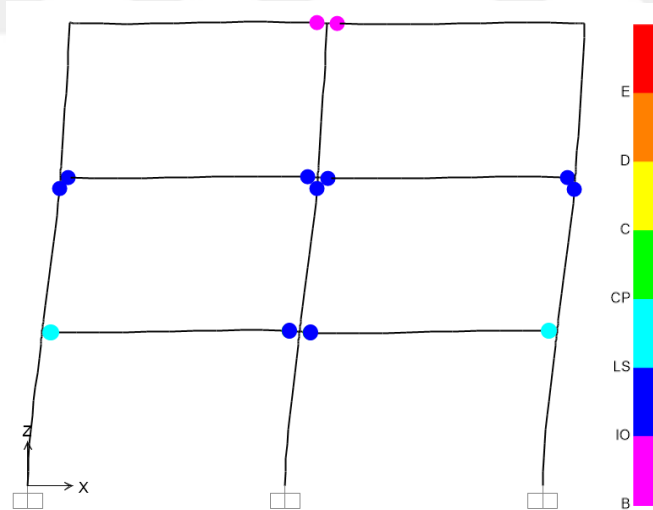


Figure 2.27. The 2 bays 3- story steel frame plastic hinges for six-story at LS level

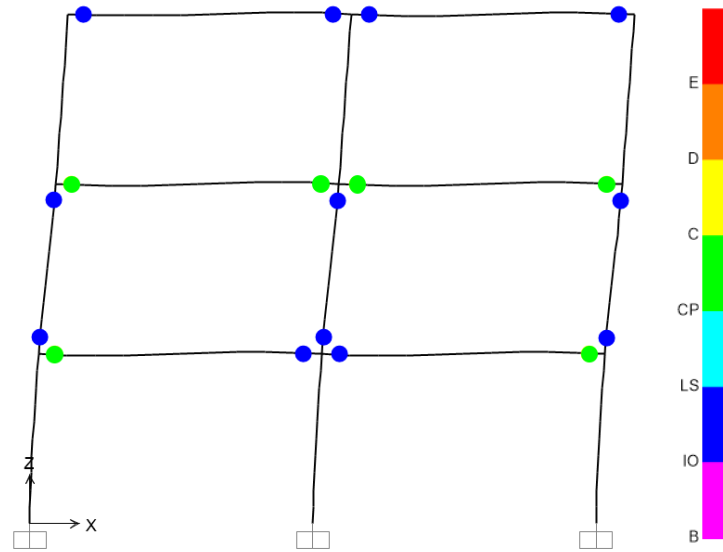


Figure 2.28. The 2 bays 3- story steel frame plastic hinges for six-story at CP level

2.4.2. Case Study VI: The 3- Bays 6- Story Steel Smfs

The geometry and details on groups of the 3- bays 6- story steel SMFs are presented in Figure 4.8. The 3- bays 6- story steel SMFs has 18 beams and 24 columns classified into three beam and six column groups, respectively.

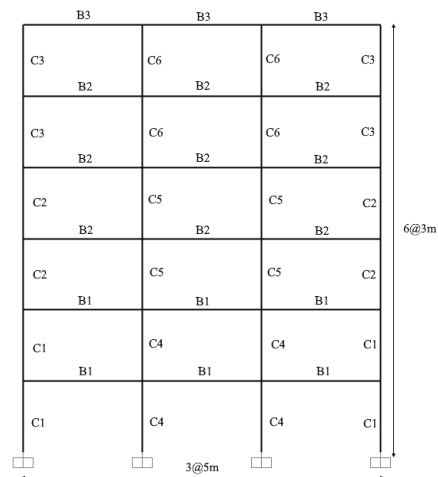


Figure 2.29. The 3- bays 6- story steel SMFs

Figure 2.30 and figure 2.31 show the algorithms converge to the solution at 107th and 238th iterations for TLBO and ABC respectively. The results also indicated that, TLBO metaheuristic algorithms converged earlier to the best solution compared to ABC.

Table 2.11. Comparison of results to the existing literature

Design Variables	GA [46]	MFA [46]	This Study (TLBO)	This Study (ABC)
C1	W21X48	W16X45	W21X50	W24X62
C2	W21X44	W16X45	W21X48	W18X60
C3	W18X76	W12X35	W21X48	W18X50
C4	W21X62	W16X50	W21X50	W21X62
C5	W21X55	W16X45	W18X76	W21X57
C6	W21X44	W16X45	W18X76	W18X55
B1	W21X44	W18X40	W21X48	W18X40
B2	W21X44	W18X35	W18X40	W18X35
B3	W16X26	W14X22	W16X31	W14X34
Weight kg (kN)	11010 (107.97)	9076.66 (89.01)	10553.66 (103.5)	11056.5 (108.43)

The optimization results for both cases are provided in Table 2.11 above.

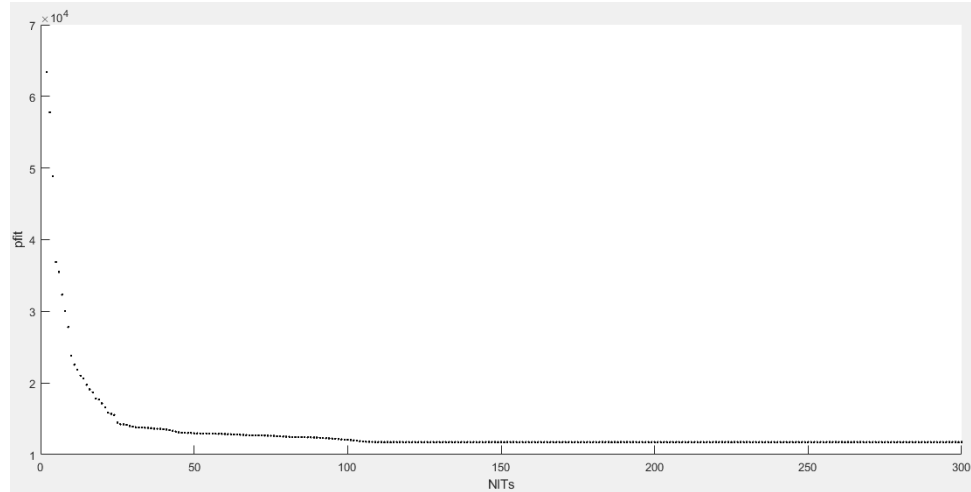


Figure 2.30. Design history the 3- bays 6- story steel SMFs for TLBO

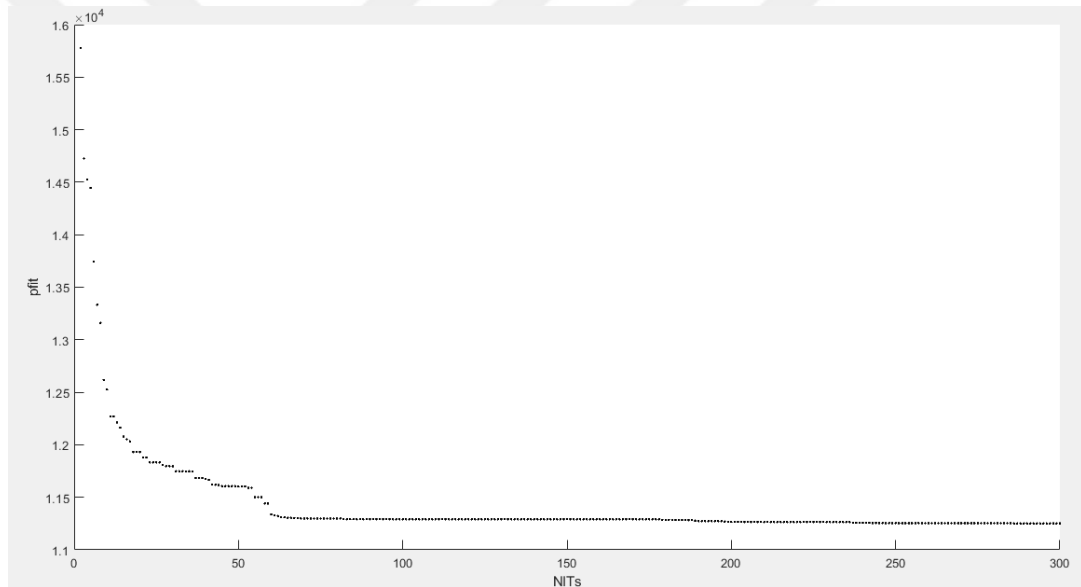


Figure 2.31. Design history the 3- bays 6- story steel SMFs for ABC

Inter story drifts of the best solution obtained by TLBO and ABC, at *IO*, *LS* and *CP* Performance Levels are illustrated in Figure 2.32. The vertical axis denotes story number while horizontal axis represents inter-story drift in percentage. The obtained results revealed that the design of 3- bays 6- story steel SMFs for both ABC and TLBO is controlled by the constraints of IO level.

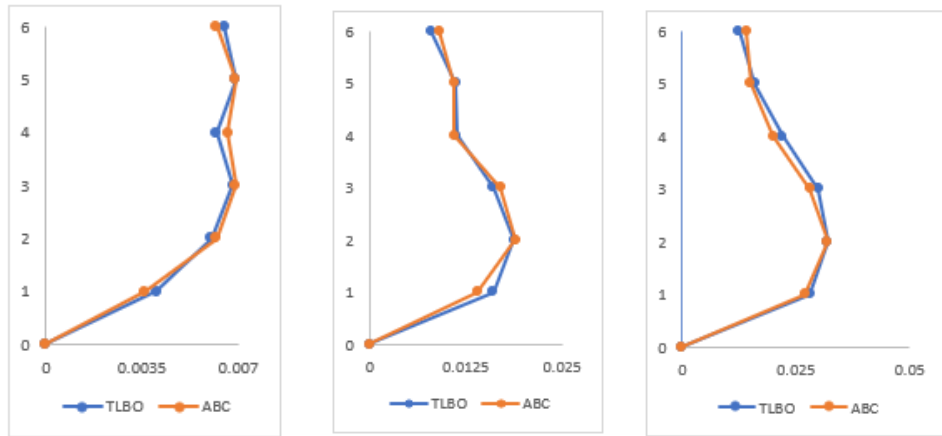


Figure 2.32. Inter story drifts for IO, LS and CP Performance Levels (from left to right)

The spread of plastic hinge for the 3-bay, 6-story frame subjected to lateral seismic loads at IO, LS, and CP Performance Levels are presented in Figure 4.12, Figure 4.13, and Figure 4.14, respectively. At which all elements' plastic hinge rotations are below the allowed plastic rotation.

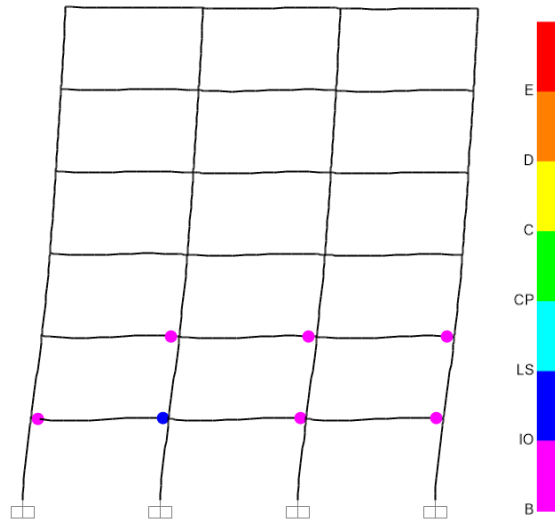


Figure 2.33. The 3 bays 6- story steel frame plastic hinges for six-story at IO level

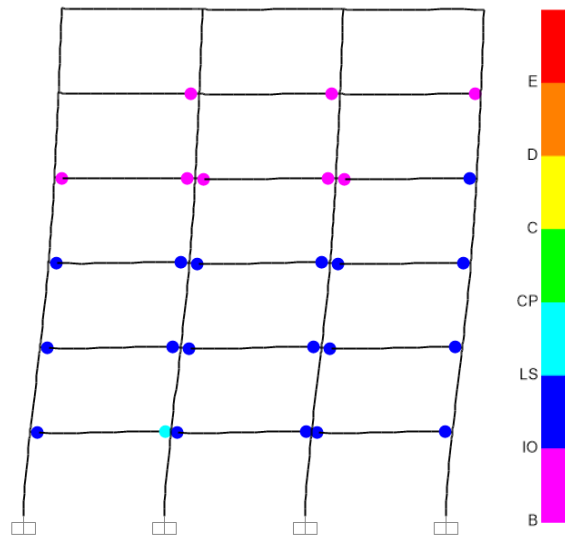


Figure 2.34. The 3 bays 6- story steel frame plastic hinges for six-story at LS level

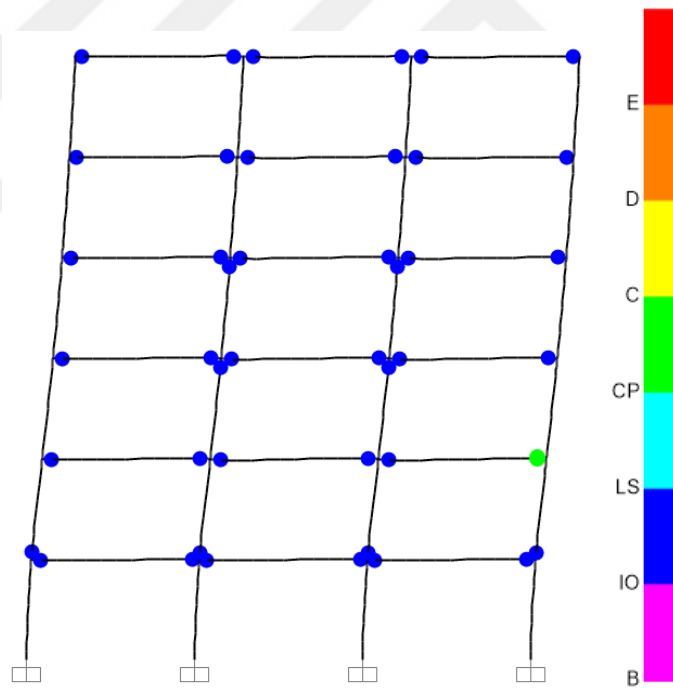


Figure 2.35. The 3 bays 6- story steel frame plastic hinges for six-story at CP level

2.4.3. Results and Discussions

In this section, discrete performance based seismic design optimization (PBDO) of 3 and 6 story 2D MRSFs was conducted. A static pushover nonlinear analysis to evaluate seismic capacity 3- and 6 story SMFs planar steel frames and metaheuristic optimizations to minimize the weight were performed. The objective function was the total weight of the 3 and 6 story MRSFs' planar steel frames, and the design variables were the grouped cross sections profiles of the frame's structural objects. A program in MATLAB was coded to perform task, applying both the metaheuristic and pushover analysis requirements in terms of constraints. For this purpose, a penalty function approach was employed to apply constraints on the problem: $pfit = fit + \text{penalty}$ and the penalty value was taken as a big number 1×10^9 . The developed program initiated and controlled the overall analysis process. However, the static analysis part of the process was performed by the program accessing SAP2000 features via OAPI. The results demonstrated ability of developed program to obtain optimum results compared to current literature. Moreover, the potential of TLBO in obtaining optimum results earlier compared to ABC. The results also indicated that, the best weights obtained using TLBO metaheuristic algorithms are 2% and 2.33% lighter than the best weight obtained by ABC, for 3- and 6 story SMFs respectively. This integration of metaheuristic into pushover analysis resulted into an automated procedure able to obtain optimum seismically safe steel structures.

2.5. Effects of Soil-Structure Interaction (SSI) on Seismic PBD

Structural and earthquake engineering community has been deeply affected by past severe earthquakes, which has changed the way structural and earthquake engineers approach their work globally today [65], [58]. Numerous seismic codes have integrated the PBD to conventional seismic design practices for the analysis of seismic structural systems [8]–[10]. In many parts of the world, structures are often analyzed for seismic loads with the premise that the base is either pinned or fixed. However, SSI system has a considerable impact on the behaviors of structures during earthquakes (SSI). Thus, for developing a more

realistic mathematical model and acquire more realistic behaviors of the structure, one must consider the interaction of soil and a structure built on it—*Soil Structure Interaction (SSI)* [20],[66]. Moreover, optimum design methods should be used for the effectiveness use of materials. It is a complicated task to take into account all these design variables, complex objective and constraints into each iteration during analysis, hence metaheuristic optimization techniques are commonly utilized.

The Figure 2.36 demonstrate a flow chart of the *Seismic Performance Based Design Optimization (SPBDO)* method used for structural analysis of the steel frames on elastic soil.



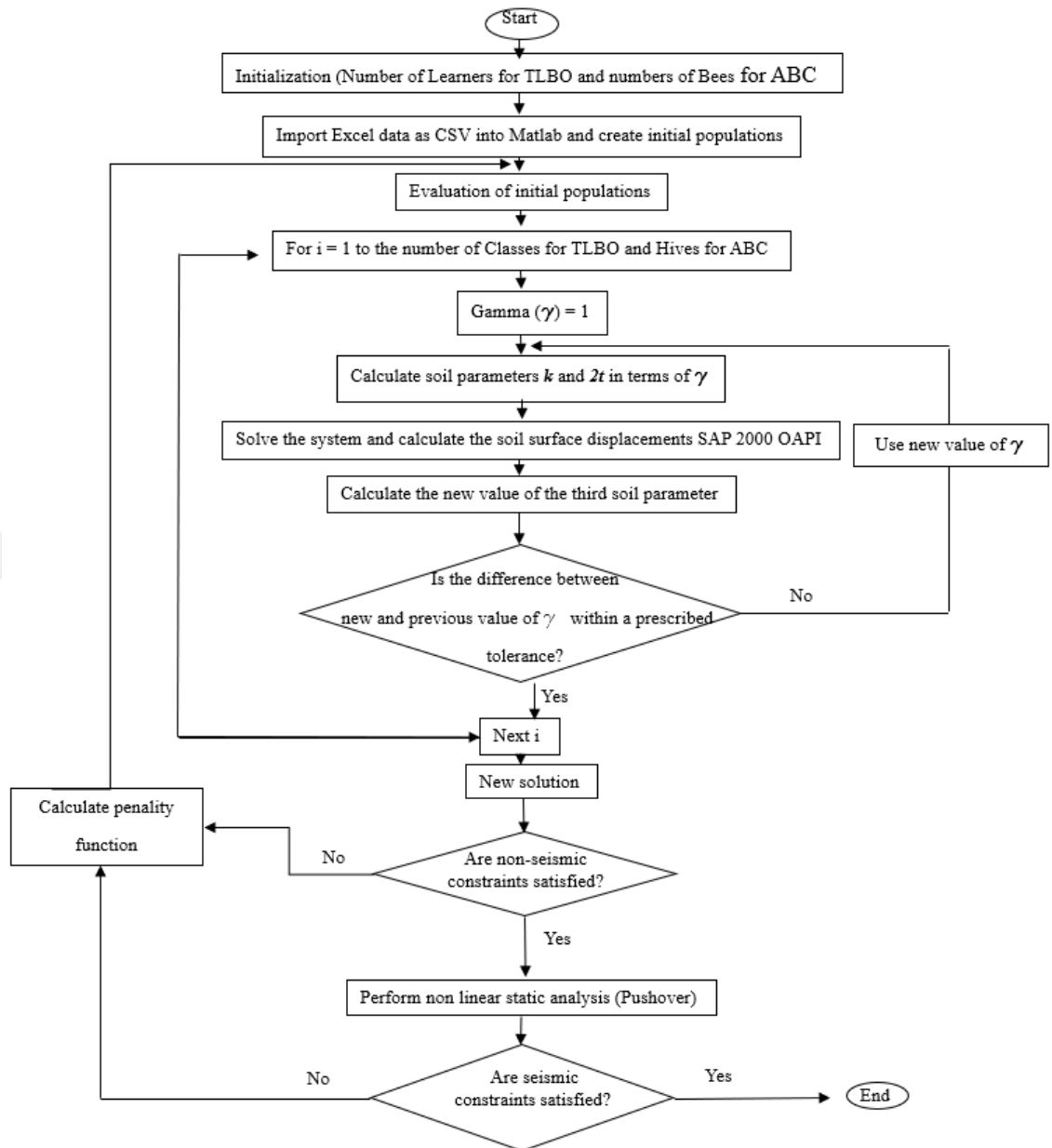


Figure 2.36. Flow chart of the SPBDO procedure for 3D steel frames on elastic soil

2.5.1. Soil –Structure Interaction Model

The 3 parameters elastic foundation model has been used as described in Chapter 1 Section 1.7.4. The reaction of the subsoil underneath structure's elastic foundation. The equation (1.20) has been used — $q_z = kw - 2t\nabla^2 w$. The key point in case is to ensure that soil shear parameter, $2t$ and modulus of subgrade reaction, k both are dependent on depth of the subsoil, H_s and mode shape function $\phi(z)$. Moreover, the additional parameter of soil, γ , that describe the vertical deformation in subsoil, changes with respect to the displacement of surface and depth of subsoil. As a result, an iterative approach is used to solve this complex problem of soil structure interaction [24]. To start iteration the value of γ , is initialized to 1. $2t$ and k , are then computed. Following structural analysis, a new γ is derived using displacements of soil surface. Thus, this process is iterated till both the most current and previous γ values are in the acceptable range.

For modeling the structure, a dual combination of both Matlab and Sap 2000 is utilized. Matlab is concerned with all coding while SAP 2000 software is used for the structural calculation part. The elastic surface spring in SAP 2000 represents the subgrade soil reaction modulus, k . However, because each spring functions individually, the interaction between the springs is neglected. The subsoil shell element that connects the tops of the springs induce their contact.

Using the bidirectional data flow feature of SAP 2000 OAPI, the coded Matlab program has been able to find soil parameters k and $2t$ in terms of γ (see Figure 2.37). As result, the dual system tool allows SAP 2000 to add *Soil Structure Interaction (SSI)* using a more realistic 3 parameters elastic foundation model. Where *SAP 2000 OAPI* provides two ways data flow between *MATLAB* and *SAP2000* for interactive structure analysis in an optimal design procedure.

2.5.2. Validation and Numerical Results

In this part, discrete SPBD problems of 2-, and 4-story space moment resistant frames are solved with and without taking into consideration of the SSI. These two examples are adapted from literature where they were studied for gravity and wind loads by Daloğlu et al. [24]. The 2-story space frame is solved to illustrate how a soil foundation and properties might change the optimum design of discrete SPBD of space frame. While on the other hand, the 4-story space frame is presented for the evaluation of soil depth. 4-story space frame is analyzed considering SSI for three distinct soil depths (5m, 10m, 15m). All cross section steel profiles of elements are selected from a collection of 267 W shaped sections in the AISC library [56]. Young's Modulus (E), yield stress (f_y) and steel density for all element members are 200GPa, 250MPa and 7.85ton/m³ respectively. TLBO and ABC coded using MATLAB program interface and connected to SAP 2000 through OAPI were used for optimization and overall evaluation purpose. For both TLBO and ABC, the total iteration number of algorithm evaluation was limited to a fixed number which is 300.

Throughout the course of analysis and modeling performed in SAP 2000 software, the following assumptions are taken into account. Beams and columns elements are modeled using force-based nonlinear fiber section representing the uniaxial steel material model and a 3% strain hardening. The “nonlinear Beam Column” fiber elements (P-M interaction) provided by SAP 2000 have been utilized in order to replicate the structural members plastic behaviors. All members are considered to be prismatic and compact. For all beams and columns are accounted for a fully rigidly connections. A rigid diaphragm connecting all elements of the same floor has been applied. Also, it has been assumed that panel zones have enough strength to prevent forming of plastic hinges in this zones. P-delta effects have been considered in all solved examples of this study while assuming soil has zero tensile strength.

2.5.2.1. Validation

With the purpose of validating the TLBO and ABC optimizations, a 4-story space SMFs as shown in figure 2.51 has been analyzed considering 15m subsoil depth. The loading patterns and cases as well as the material properties were adjusted to be same as those

considered by Daloğlu et al [24]. They found optimum design of the 4-story space MRFs steel using GA and HS salgorithms. For checking the efficiency and robustness of the presented algorithms, TLBO has been employed to resolve the same problem, the obtained results are given in Table 2.12. The table demonstrated that obtained optimum results from TLBO algorithm conform with the results of Daloğlu et al [24]. Moreover, the proposed TLBO has obtained lighter designs compared to the weights obtained by GA and HS. Therefore, its efficiency is obvious.

Table 2.12. Optimum designs for 4-story space MRFs for $H_s = 15\text{m}$

Group members	This study	Daloglu et al. [24]	
	TLBO ($H_s=15\text{m}$)	GA $H_s=15\text{m}$	HS ($H_s=15\text{m}$)
1	W10X22	W12X35	W8X28
2	W14X34	W18X35	W12X26
3	W8X28	W12X26	W12X30
4	W14X34	W16X26	W16X31
5	W10X39	W12X72	W10X33
6	W10X49	W18X50	W16X40
7	W10X39	W10X39	W14X34
8	W10X54	W14X38	W27X94
9	W10X39	W16X36	W10X33
10	W14X68	W27X94	W12X50
Best Weight (kN)	192.379	195.71	195
Maximum top story drift (cm)	3.34	3.32	3.31
Maximum inter- story drift (cm)	0.8	1.02	0.94

2.5.2.2. Numerical Results

In this part, discrete SPBD problems of 2-, and 4-story space moment resistant frames with fixed base or SSI are solved for seismic loads.

2.5.2.2.1. Case Study VI: The 2-Story Space Steel Frame

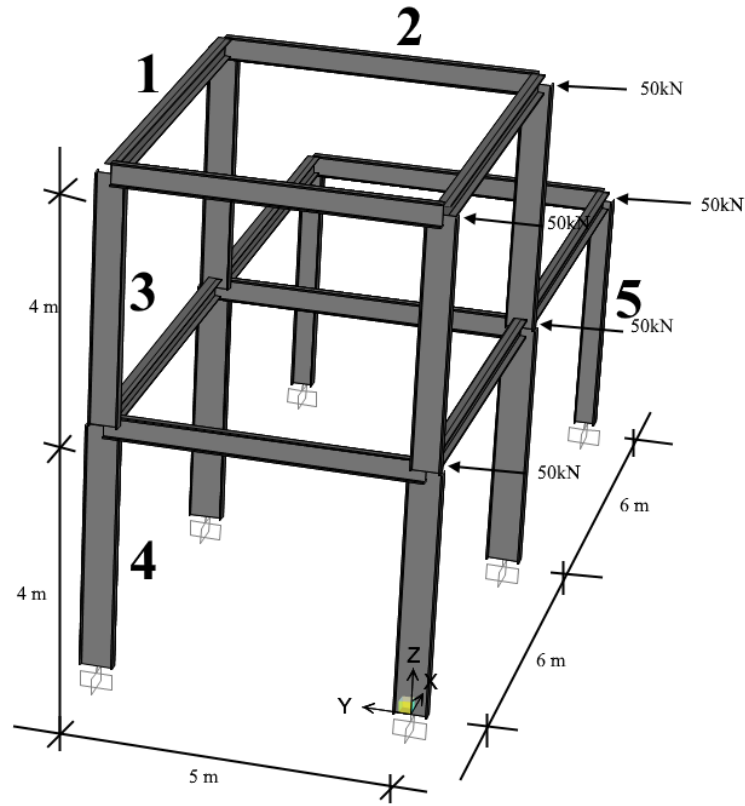


Figure 2.37. The 2-story space MRFs

To illustrate effects of elastic foundation in discrete SPBD optimization, the 2 story space frame studied by Daloğlu et al. using GA and HS algorithms is studied [24]. The 2 story space SMFs members are classified into five groups with 3 groups of columns and 2 groups of beams. The dead loads (DL) applied to all beams is 20 kN/m while every joint in y direction is subjected to a wind load of 50 kN as demonstrated in figure 2.37. Both the fixed base and the *Soil Structure Interaction (SSI)* are considered throughout the study.

For the purpose of simulating the SSI, soil A, which has a modulus of elasticity of $20,000 \text{ kN/m}^2$, soil B, which has a modulus of $68,950 \text{ kN/m}^2$, and soil C, which has a modulus of $120,000 \text{ kN/m}^2$, are utilized. These soil types replicate loose soil, medium dense soil and stiff soil, respectively. The soil depth $H_s=20 \text{ m}$, whereas the soil Poisson's ratio is

0.25. The foundation depth is taken as 30cm. Minimum weights, roof displacement, inter story drifts and optimum designs profiles are detailed in Table 2.14.

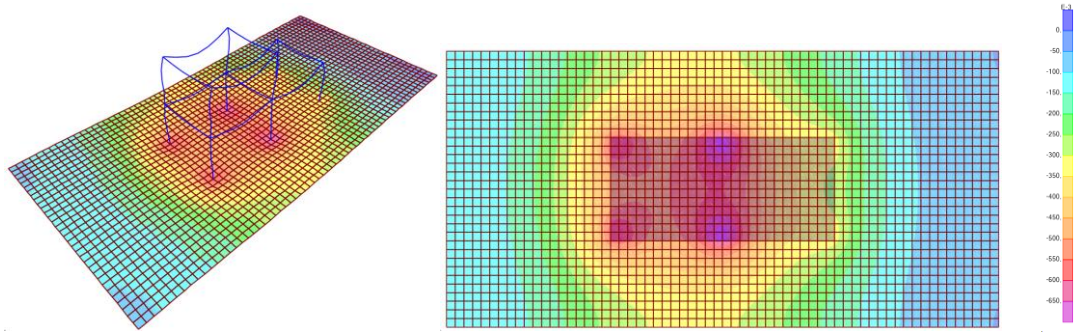


Figure 2.38. Soil surface settlements of Space MRFs for Soil A (cm)

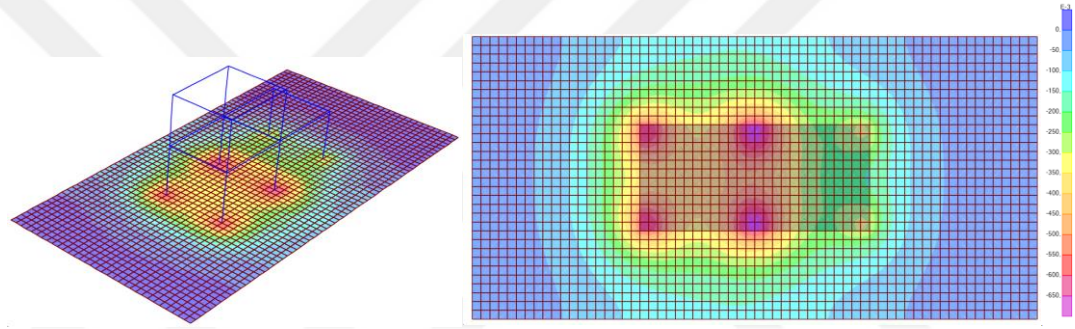


Figure 2.39. Soil surface settlements of Space MRFs for Soil B (cm)

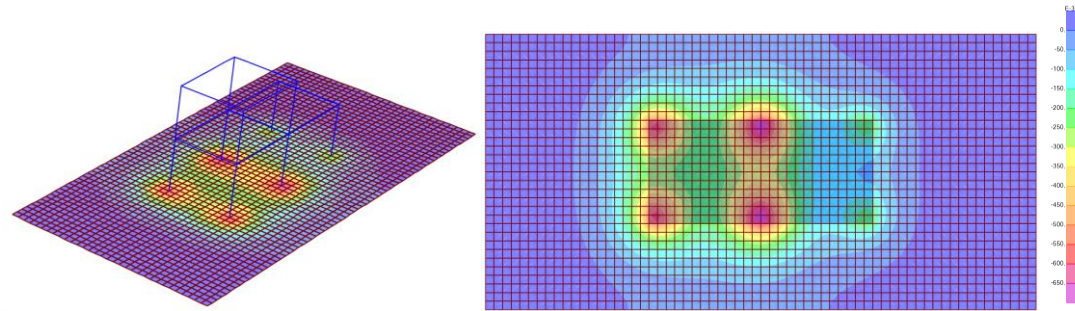


Figure 2.40. Soil surface settlements of Space MRFs for Soil C (cm)

Figure 2.41 and Figure 2.42 illustrate the history of design of 2 story, 21-members space MRFs for TLBO and ABC respectively. Whereas the soil surface settlements for various types of the soil for the optimum design are provided in Figure 2.38, Figure 2.39 and

Figure 2.40. The analysis were performed on intel Core i7 -10700F CPU, 32.00GB of RAM with a 64-bit operating system utilizing SAP2000 v23 and MATLAB R2017b. The Table 2.13 lists the final values of 3 parameters of elastic foundation for different types of soil used in the study.

Table 2.13. Soil parameters for 2 story, 21-members space MRFs

The soil types	γ	k (kN/m ³)	2t (kN/m)
Soil A	1.65452	1326.737	39420.483
Soil B	1.83726	4743.82	128473.51
Soil C	1.98238	8528.179	213648.843

Table 2.14 Optimum results by TLBO for the 2 story space MRFs on different foundation soil types

Group members	TLBO			
	Fixed	Soil A	Soil B	Soil C
1	W14X30	W12X40	W12X26	W12X40
2	W16X36	W21X50	W18X40	W14X26
3	W21X44	W14X48	W14X48	W18X46
4	W21X50	W14X48	W14X61	W21X50
5	W14X43	W12X30	W14X61	W14X48
Minimum weight (kN)	55.996	65.244	60.863	58.194
Maximum lateral roof displacement (cm)	2.4	3.65	2.68	2.81
Maximum inter-story drift (cm)	1.10	1.31	0.97	1.04

Table 2.15. Optimum results by ABC for the 2 story space MRFs on different foundation soil types

Group members	ABC			
	Fixed	Soil A	Soil B	Soil C
1	W18x35	W12x40	W14x34	W12x26
2	W21x50	W14x40	W18x40	W18x40
3	W12x30	W14x61	W14x53	W12x53
4	W14x48	W14x61	W21x62	W14x61
5	W12x26	W14x48	W16x36	W12x30
Minimum weight (kN)	57.89	66.129	63.674	58.436
Maximum lateral roof displacement (cm)	2.32	3.60	2.54	2.80
Maximum inter-story drift (cm)	1.07	1.3	0.94	1.01

The optimum design history of the 2- story MRFs with various types of the soil are shown in Figure 2.42 and Figure 2.43. From these figures, it is clear that for all three types of soils (soil A, B and C) the presented TLBO and ABC for performance-based design optimization of MRFs has converged to the optimum design solution.

In Table 2.15, the optimum section profiles, minimum weight, roof and interstory drifts for the 2-story space MRFs for different soil types are presented. The results demonstrated that soil type C has a negligible impact on the optimal weight compared to the fixed base and the optimal weight of the steel frame has only risen by 3.93% and 0.98% for TLBO and ABC respectively. The optimal weight for soil type B has risen by 8.69 % and 10.33% for TLBO and ABC respectively. However, the optimum weight of the 2- story MRFs is significantly affected by soil type A with an increase of 16.5 % and 14.7% for TLBO and ABC respectively.

It can be concluded that the optimal results of frame on soil type C and fixed based steel frame are practically very close to each other and is due to the greater stiffness of soil type C. The considerable low stiffness of soil type B, and notably soil type A, has promoted the swaying mode of foundation, which in turn has enhanced inter story drift and stories deflection. This phenomenon modifies the degree of structural performance, particularly in

strong earthquakes. Therefore, buildings on A and B soil types need soil-structure interaction modeling.

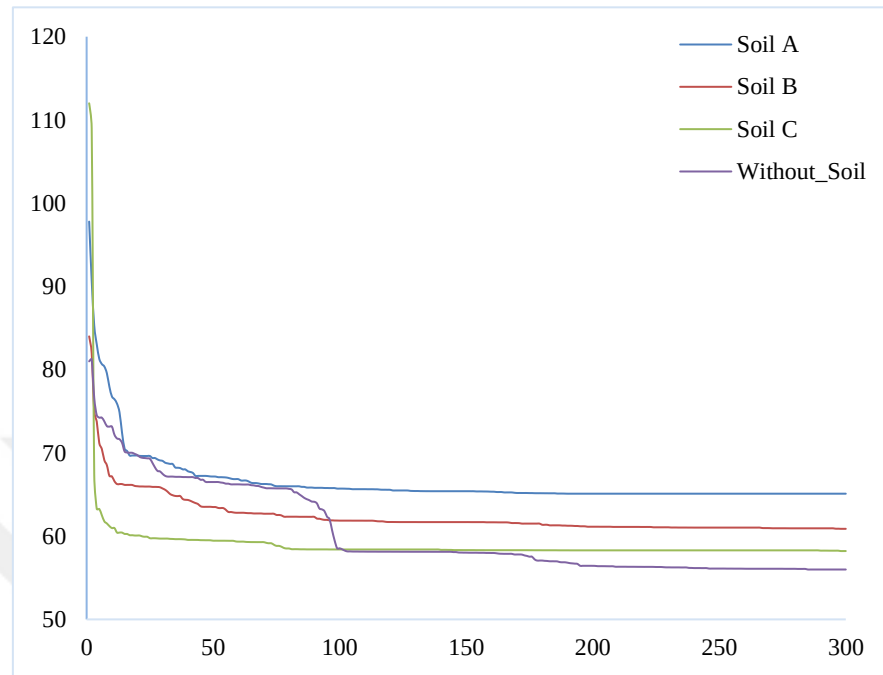


Figure 2.41. Design histories of 2-story space MRFs by TLBO for Soil A, B and C

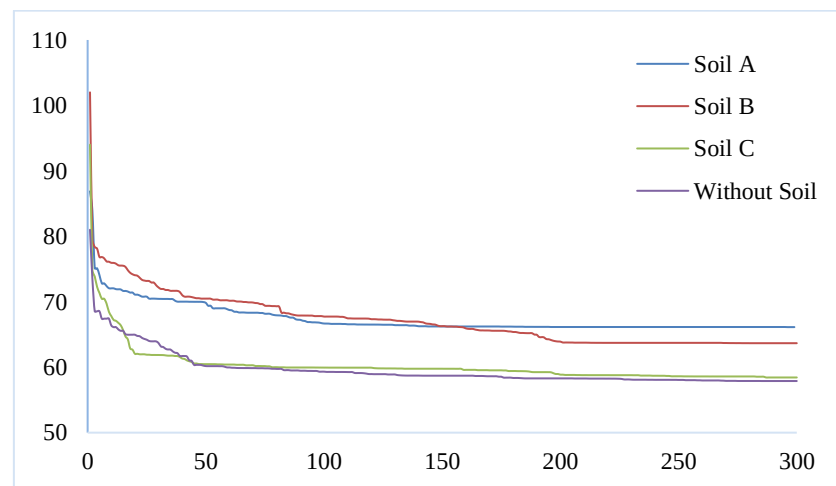


Figure 2.42. Design histories of 2-story space MRFs by ABC for Soil A, B and C

Figure 2.43, Figure 2.44 and Figure 2.45 show the story drifts of best solution derived by TLBO and ABC for soil type A, B and C at *IO*, *LS* and *CP* Performance Levels. In these figures, the vertical axis denotes story number while horizontal axis represents inter-story drift in percentage. The results showed that the drift constraints associated with different levels has been satisfied for all considered soil types.

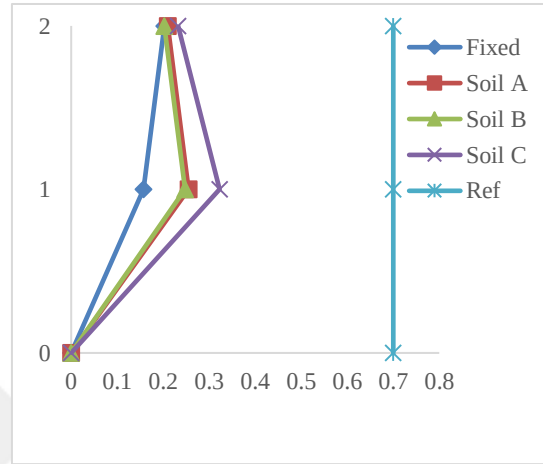


Figure 2.43. Story drifts from the PBDO of the 2 story space MRFs for all soil types and fixed base at *IO*

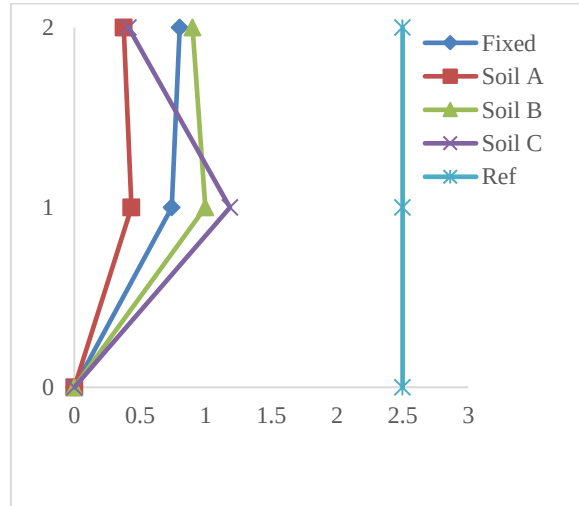


Figure 2.44. Story drifts from the PBDO of the 2 story space MRFs for all soil types and fixed base at *LS*

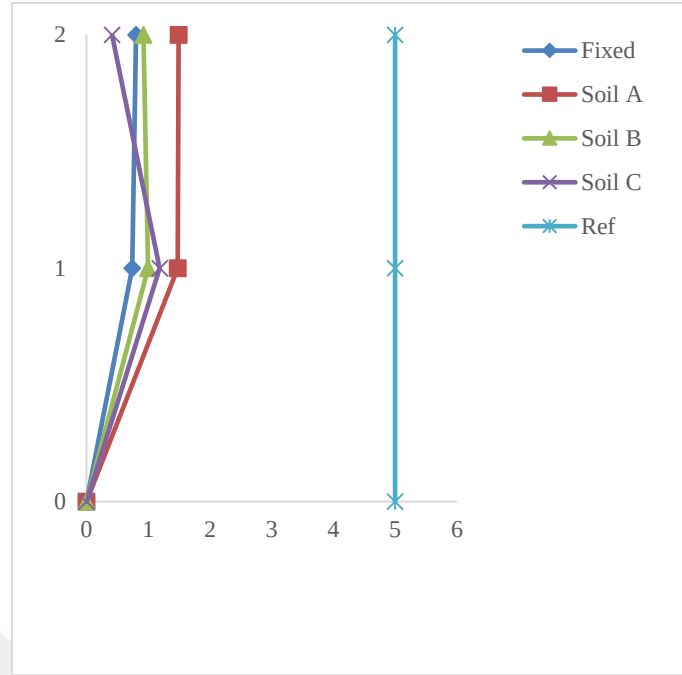


Figure 2.45. Story drifts from the PBDO of the 2 story space MRFs for all soil types and fixed base at CP

Figure 2.46, Figure 2.47, Figure 2.48 and Figure 2.49. illustrate the pattern of plastic hinges formation in the optimum 2-story space MRFs for three soil types —(Soil A, Soil B and Soil C). In these figures, the color of the hinges indicates percentage of joint rotation that is allowed within the range of rotation for a given performance levels. (i.e., the green color implies that the joint is operating at a LS Performance Level). The results show that, a number of plastic hinges transitioning from IO to CP Levels varies among soil types with less at IO and high at CP levels. Moreover, the weak beam-strong column restriction enables many plastic hinges occur at the ends of beams and base of columns on the 1st floor story. Comparing the hinge status at the bottom of 1st story columns, it is possible to see the influence of the SSI on the optimal results. The results demonstrated that members' plastic hinge rotations did not surpass the allowed range. Besides, the maximum permitted rotation and foundation vertical displacement are also within acceptable ranges.

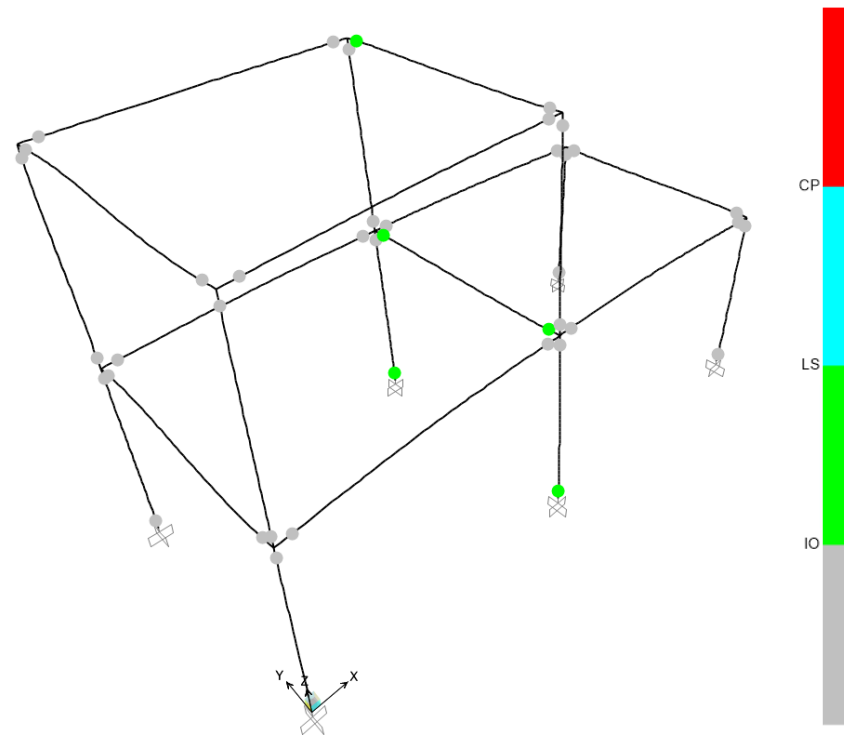


Figure 2.46. The plastic hinges distribution of optimum PBD of the 2-story space MRFs for fixed base

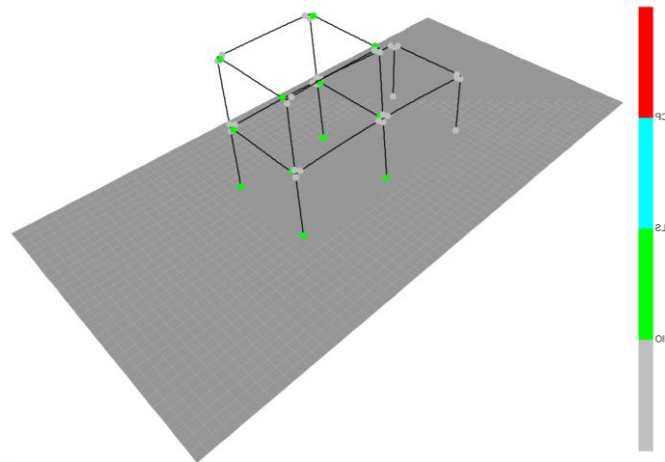


Figure 2.47. The plastic hinges distribution of optimum PBD of the 2-story space MRFs for Soil A

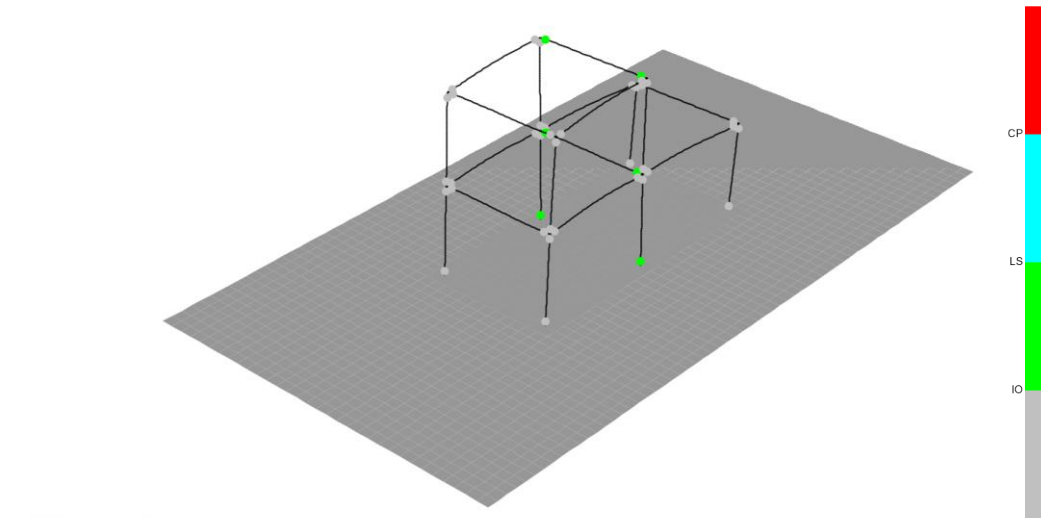


Figure 2.48. The plastic hinges distribution of optimum PBD of the 2-story space MRFs for Soil B

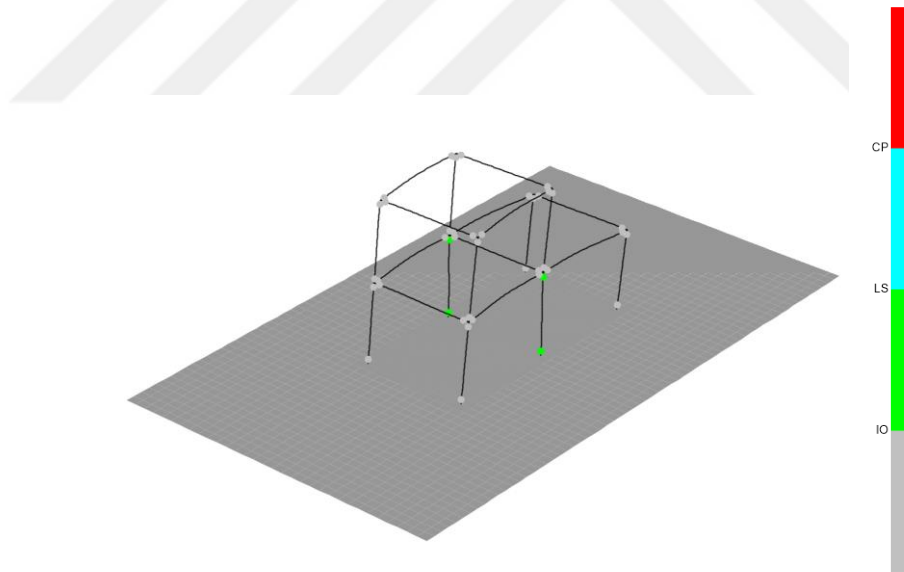


Figure 2.49. The plastic hinges distribution of optimum PBD of the 2-story space MRFs for Soil C

2.5.2.2.2. Case Study VII: The 4-Story Space Steel Frame

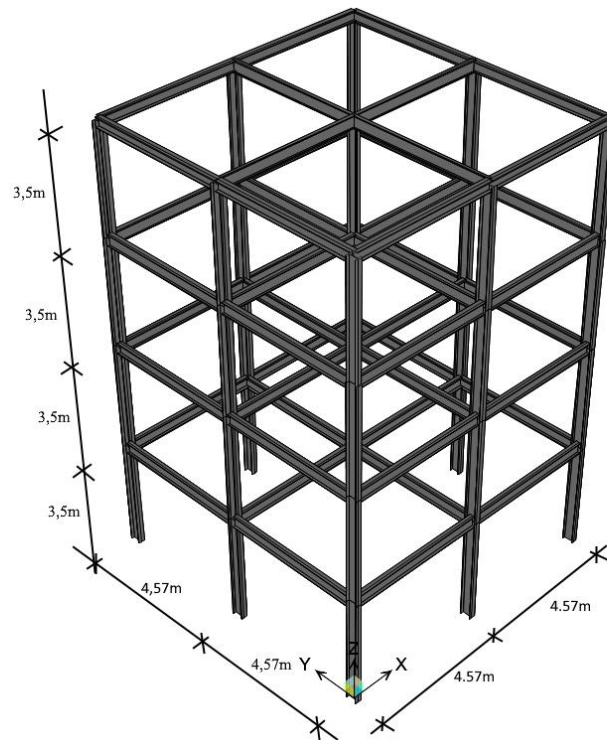


Figure 2.50. The 4-story space MRFs perspective view

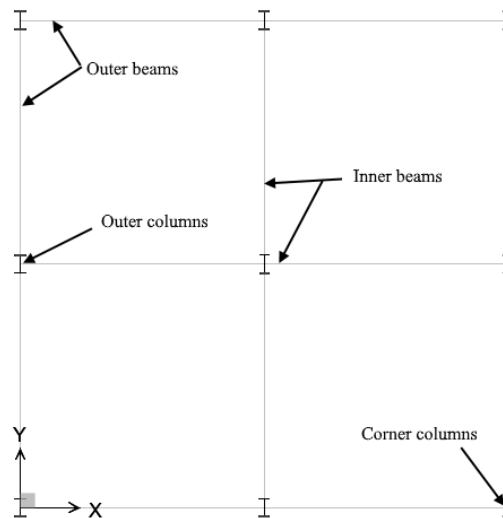


Figure 2.51. The 4-story space MRFs plan view

For the purpose of illustrating the impact of foundation and depth of the soil stratum in discrete SPBD optimization, the four-story 3D frame investigated by Daloğlu et al. [24] employing Harmony Search and Genetic Algorithms is taken into consideration. Members of the four-story 3D steel frame are broken down into the ten groups listed below. Beams are grouped as follows: (1) outer beams of the fourth floor; (2) outer beams of the third, second, and first floors; (3) inner beams of the fourth floor; (4) inner beams of the third, second, and first floors; (5) outer columns of the fourth floor; (6) outer columns of the third, second, and first floors; (7) outer columns of the fifth floor; (8) outer columns of the third, second, and first floors; (8) inner columns of the fourth floor; and ((See figure 2.51 and figure 5.52).

The dead loads (DL), live loads (LL) and roof live loads applied to all beams are 3.84 kN/m^2 , 2.4 kN/m^2 and 2.4 kN/m^2 , respectively. A wind pressure $p = C_e C_q q_s I_w$ with p the design wind pressure; C_e : The combined height, exposure and gust factor coefficient, C_q : The pressure coefficient; 0.8 is used for windward faces and 0.5 is used for leeward faces. q_s is wind stagnation pressure and equal to 0.785 kN/m^2 , and wind importance factor $I_w = 1$. These design parameters are the same for the 2-story steel frame space. The wind forces act on the building on the x-axis. Analysis is performed considering both fixed base and soil structure interaction (SSI).

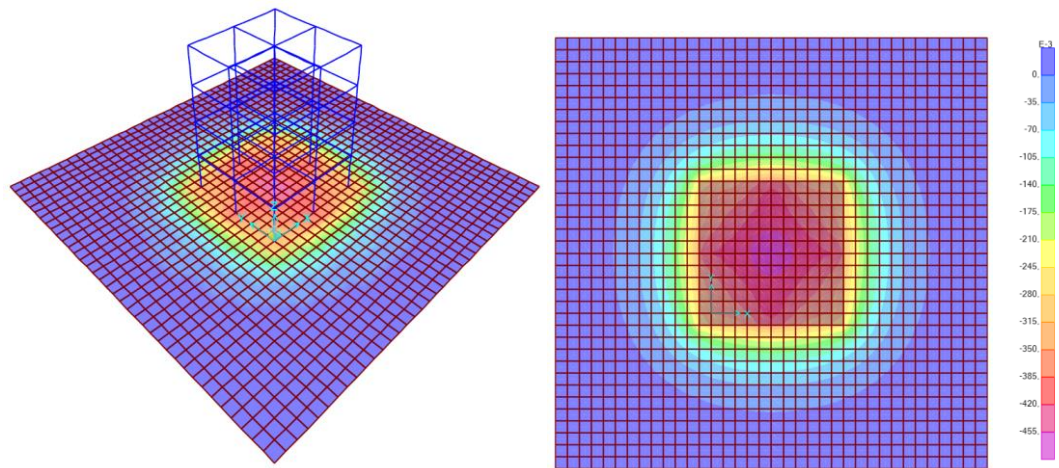


Figure 2.52. Soil surface settlement (cm) of the the 4 story MRFs for subsoil depth of $H_s = 5\text{m}$

This part of the research examines the impacts of three different subsurface depths on stiff bases: $H_s = 5\text{ m}$, $H_s = 10\text{ m}$, and $H_s = 15\text{ m}$. The ratio of Poisson for soil is 0.25, the heights are 5, 10, and 15 meters, and modulus of elasticity is $68,950\text{ kN/m}^2$. Table 2.17 displays the optimal inter-story drifts, roof displacement, steel profiles and minimum weights. Figures 2.55 and 2.56 depict the design histories of each case, with the population's best design being adopted in each iteration. Refer to Figures 2.52, 2.53, and 2.54, which represent the surface settlements, for the optimal design of the steel frames taking into consideration SSI.

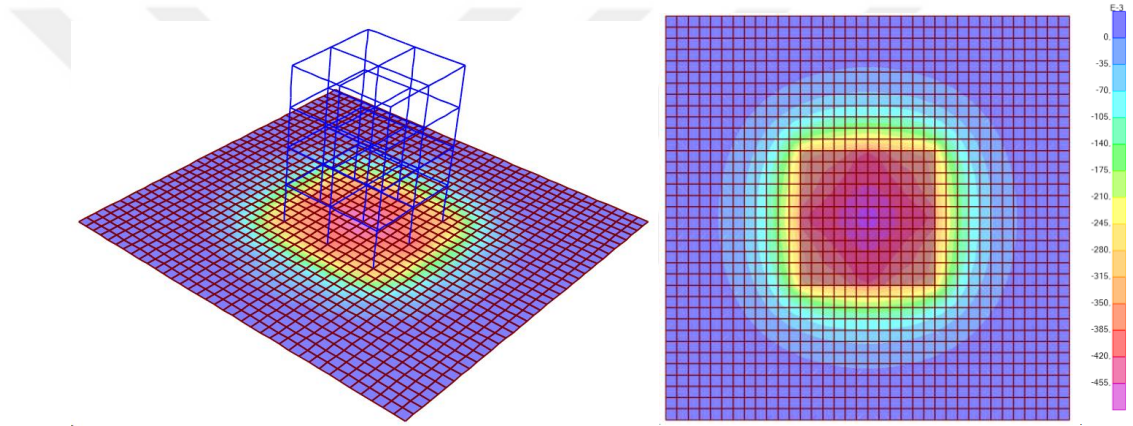


Figure 2.53. Soil surface settlement (cm) of the the 4 story MRFs for subsoil depth of $H_s=10\text{ m}$

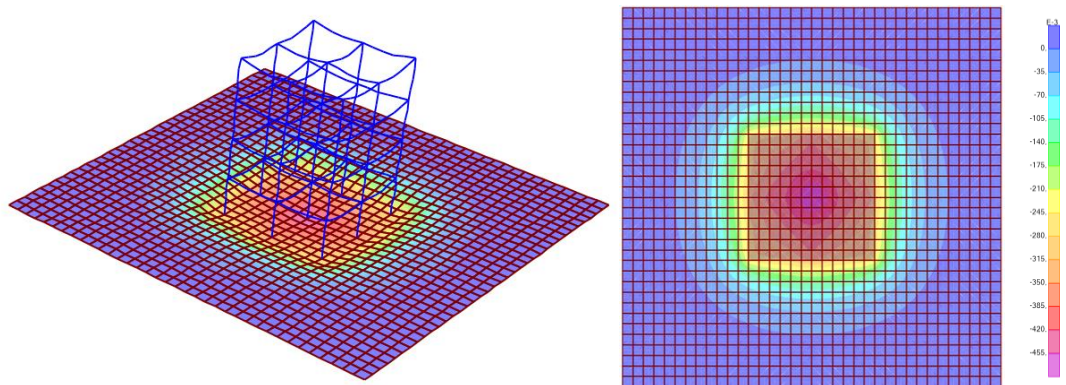


Figure 2.54. Soil surface settlement (cm) of the the 4 story MRFs for subsoil depth of $H_s=15\text{ m}$

Table 2.16. Parameters of the soil for the 4 story space SMF on elastic foundation

Depth of soil	γ	k (kN/m ³)	$2t$ (kN/m)
$H_s = 5$	1.69131	18422.619	33597.619
$H_s = 10$	3.2646	13801.431	41562.989
$H_s = 15$	4.882218	13481.712	42325.687

The optimum design history of 4 story space MRFs with different soil depths are given in Figure 2.55 and Figure 2.56. From these figures, it can be observed that for different depths of soils (5m, 10m and 15m) the presented TLBO and ABC for performance-based design optimization of MRFs converged to optimum design solution.

Table 2.17. Optimum results by ABC for the 2 story space MRFs on different foundation soil depths

Group members	ABC			
	Fixed	$H_s = 5m$	$H_s = 10m$	$H_s = 15m$
1	W16X31	W12X40	W16X31	W12X35
2	W18X35	W18X35	W18X35	W18X35
3	W14X26	W14X34	W14X34	W12X26
4	W14X30	W16X26	W12X26	W14X34
5	W12X45	W14X38	W10X26	W14X34
6	W18X40	W18X50	W18X40	W18X50
7	W10X39	W10X39	W10X39	W12X45
8	W14X38	W14X38	W24X68	W10X54
9	W16X36	W16X26	W14X48	W16X36
10	W27X94	W27X94	W30X108	W30X99
Minimum weight (kg)	185.171	193.098	201.37	205.739
Maximum lateral roof displacement (cm)	2.62	2.82	2.88	2.9
Maximum inter-story drift (cm)	0.82	0.88	0.91	1.08

Table 2.18. Optimum results by TLBO for the 2 story space MRFs on different foundation soil depths

Group members	TLBO			
	Fixed	Hs = 5m	Hs = 10m	Hs = 15m
1	W8X28	W12X45	W16X31	W10X22
2	W12X35	W14X30	W18X35	W14X34
3	W12X30	W14X22	W14X34	W8X28
4	W16X26	W14X34	W12X30	W14X34
5	W10X26	W12X45	W10X49	W10X39
6	W18X35	W14X43	W18X46	W10X49
7	W12X58	W14X43	W10X39	W10X39
8	W14X48	W14X43	W14X48	W10X54
9	W16X26	W12X35	W14X48	W10X39
10	W16X57	W14X34	W16X36	W14X68
Minimum weight (kg)	178.07	182.795	189.225	192.379
Maximum lateral roof displacement (cm)	2.27	2.98	3.06	3.1
Maximum inter-story drift (cm)	0.85	0.91	0.95	1.12

In Table 2.18., the optimum steel profiles, minimum weight, roof and interstory drifts for the 4 story space MRFs for different soil depths are presented. The results illustrate that the soil depth of 5m has a negligible impact on optimual weight compared to fixed base and the optimal weight of steel frame has risen by 8.44% and 14.16% for TLBO and ABC respectively. For soil depth of 10m, the optimal weight has risen by 19.92% and 28.93% for TLBO and ABC respectively. However, a soil depth of 15m has a huge impact on the optimum weight of the 2- story MRFs leading to an increase in weight of 25.55% and 36.73% for TLBO and ABC respectively.

Figure 2.52, Figure 2.53, and Figure 2.54 show that the greatest soil surface settlement occurs at $H_s = 15$ m, at about 0.45 cm, whereas at $H_s = 10$ m and $H_s = 5$ m, are around 0.35 cm and 0.3 cm, respectively. Table 2.16 displays soil parameters in relation to H_s .

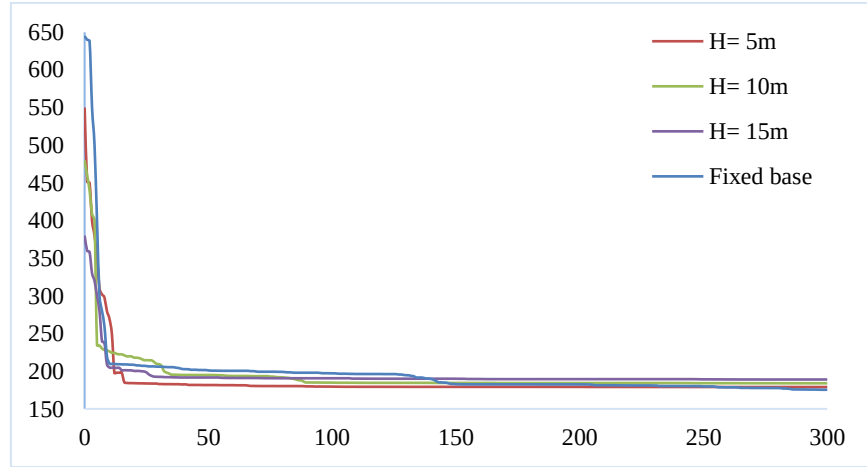


Figure 2.55. Design histories of 4-story space MRFs by TLBO for Soil depths 5m, 10m and 15m

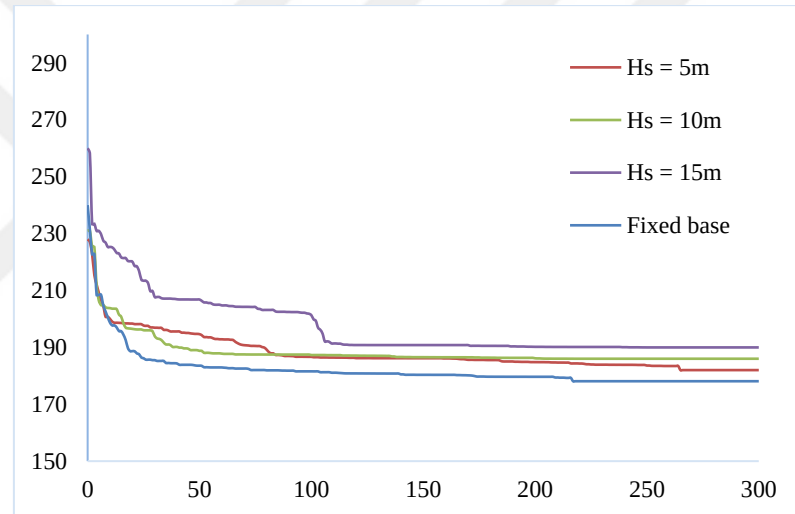


Figure 2.56. Design histories of 4-story space MRFs by ABC for Soil depths 5m, 10m and 15m

Figure 2.57, Figure 2.58 and Figure 2.59 show interstory drift of the optimum solution by TLBO at *IO*, *LS* and *CP* Performance Levels for soil depth $H_s = 5, 10$ and $15m$. In these figures, the vertical axis denotes story number while horizontal axis represents inter-story drift in percentage. The results showed that the drift constraints associated with different levels has been satisfied for all considered soil depths. Moreover, the results showed that designs were dominated by the constraints related to the *IO* level.

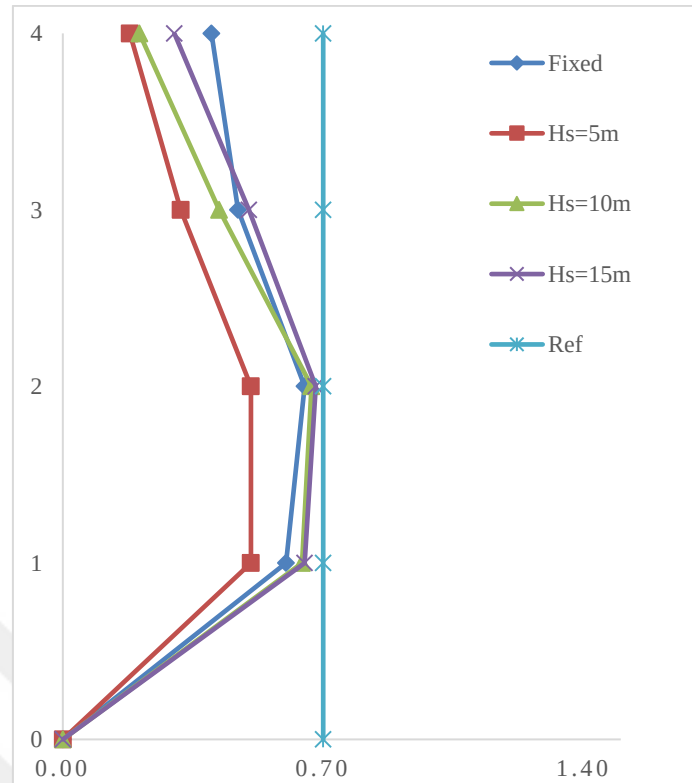


Figure 2.57. Story drifts from the PBDO of the 4 story space MRFs by TLBO for all soil depths and fixed base (IO)

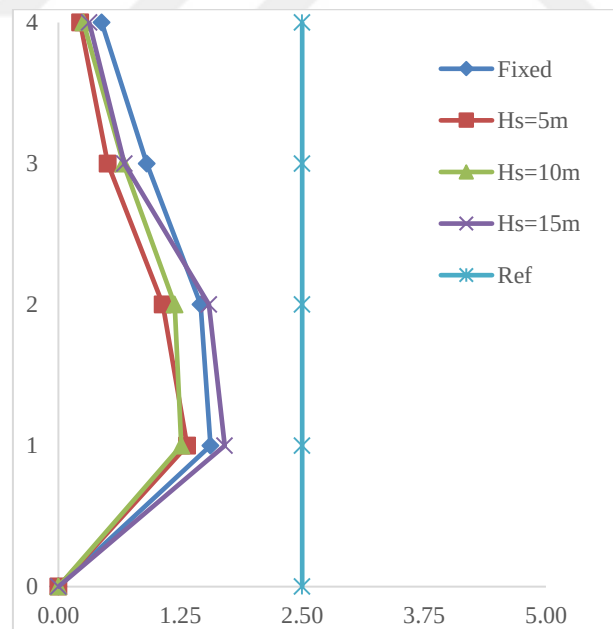


Figure 2.58. Story drifts from PBDO of the 4 story space MRFs by TLBO for all soil depths and fixed base (LS)

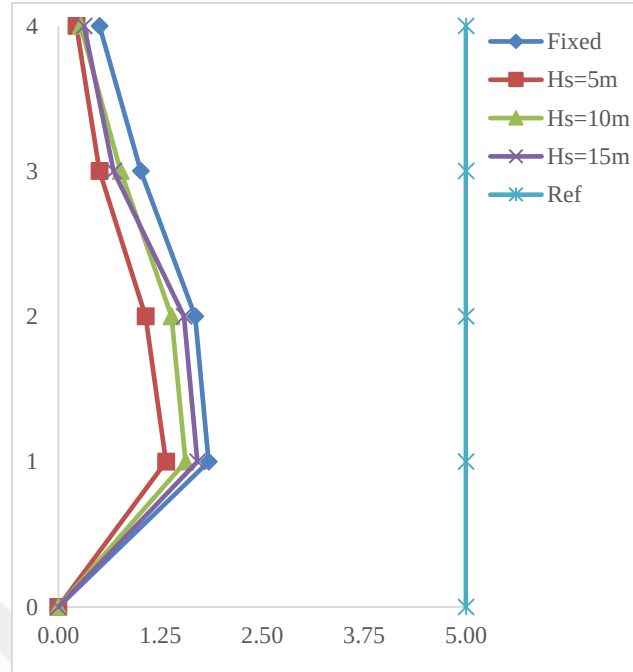


Figure 2.59. Story drifts from the PBDO of the 4 story space MRFs by TLBO for all soil depths and fixed base (CP)

Figure 2.60, Figure 2.61 and Figure 2.62. illustrate the pattern of hinges formation in the optimum 4-story space MRFs for different soil depth (5m, 10m and 15m). In these figures, the color of the hinges indicates percentage of the joint rotation that is allowed within the range of rotation for a given performance level as described in the introduction of this section. The figures show that, a number of plastic hinges transitioning from *IO* to *CP* Levels varies among soil depths with less at *IO* and high at *CP*. Moreover, the strong column weak beam restrictions enable most plastic hinges occur at the ends of beams and at the base of columns on 1st floor story. There is no hinges formation at either end of the columns elsewhere in the structure. Comparing the hinge status at bottom of the 1st story columns, it is possible to observe influence of the *Soil Structure Interaction—SSI* on optimum results. Additionally, another indicator that optimization problem was not only controlled by drift constraints at *IO* level but also the rotation of the inner column of the 1st story of the 4-story space MRFs is the red hinge on that joint. Furthermore, results demonstrated that the members' plastic hinge rotations did not surpass the allowed range. Besides, the foundation maximum permitted rotation and vertical displacement are also within acceptable ranges. All lateral displacements are less than the maximum allowed. However, inter story drifts are

nevertheless near the drift limits. This illustrates that optimum designs are mostly governed by inter-story drift limitations.

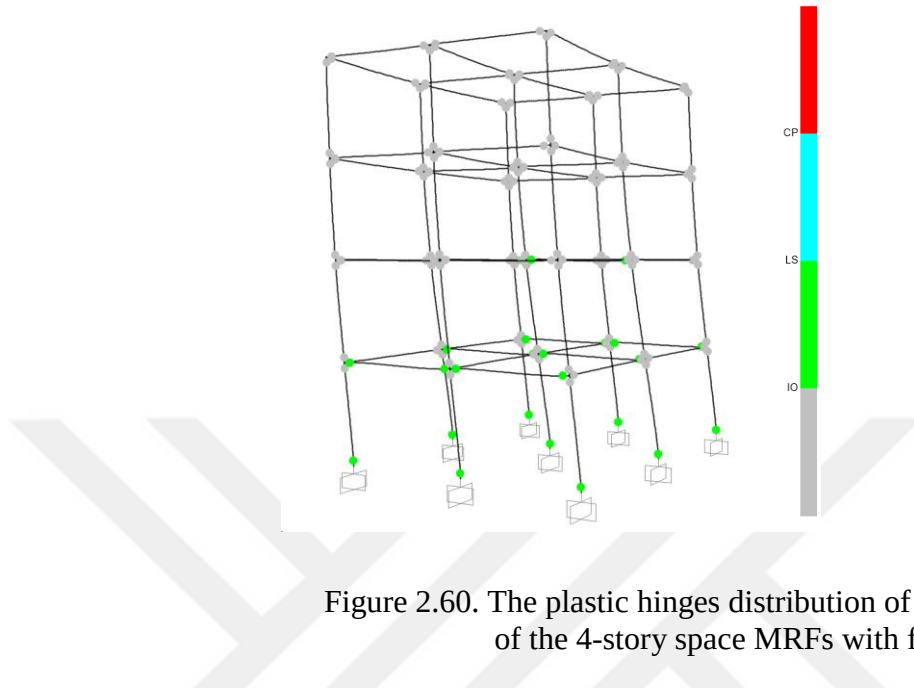


Figure 2.60. The plastic hinges distribution of optimum PBD of the 4-story space MRFs with fixed base

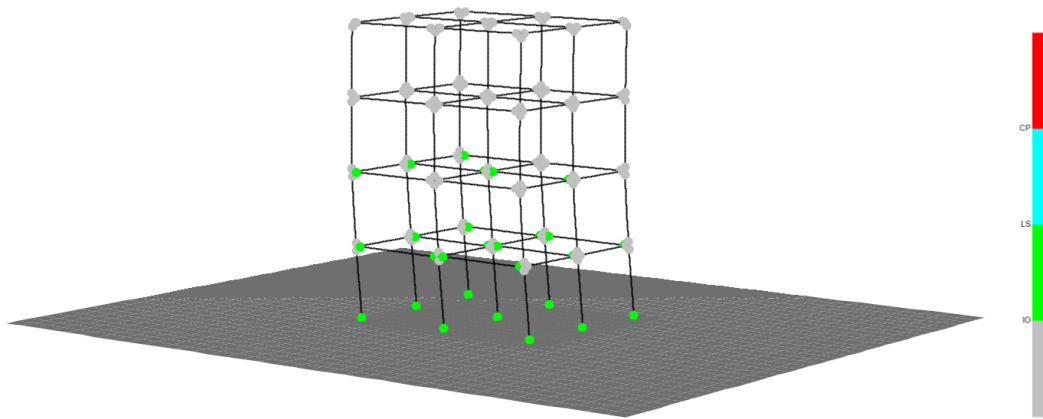


Figure 2.61. The plastic hinges distribution of optimum PBD of the 4-story space MRFs for soil depth $H_s = 5\text{m}$

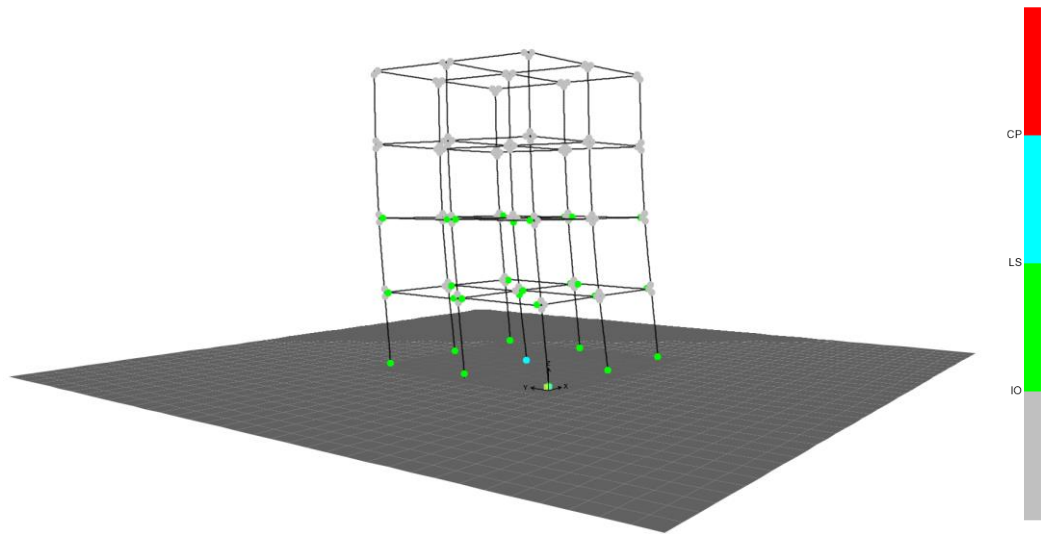


Figure 2.62. The plastic hinges distribution of optimum PBD of the 4-story space MRFs for soil depth $H_s = 10\text{m}$

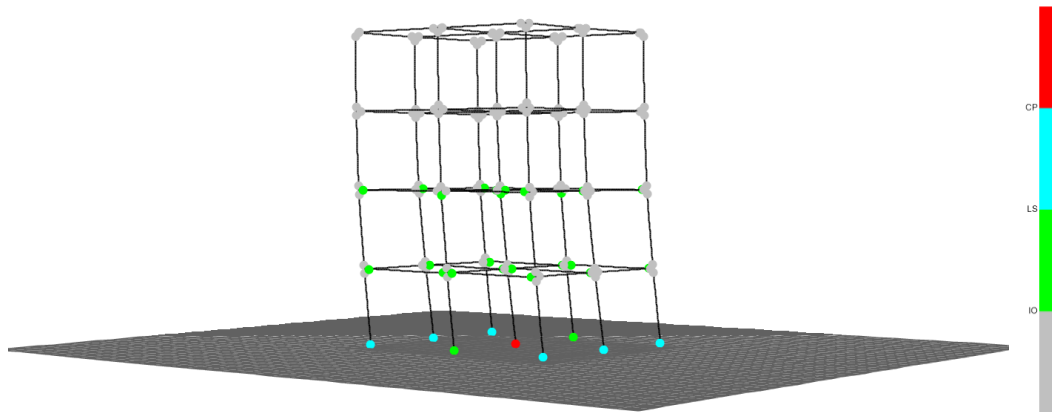


Figure 2.63. The plastic hinges distribution of optimum PBD of the 4-story space MRFs for soil depth $H_s = 15\text{m}$

3. CONCLUSION AND RECOMMENDATIONS

The objective of this study was to set a simple systematic comprehensive optimization technique involving the coordinated applications of Performance-Based Design (PBD), Soil-Structure Interaction (SSI), and Metaheuristic Design Optimization (MDO) for the seismic design of multiple story steel moment frames. For this purpose, a discrete Seismic Performance Based Design Optimization (SPBDO) of 2D and space Moment Resistant Steel Frames (MRSFs) was conducted. To evaluate the effects of *Soil Structure interaction—SSI* on the Performance Based Design Optimization (PBDO) of space Moment Resisting Steel Frames (MRSFs), a model of 3-parameters elastic foundation is utilized along with two metaheuristic algorithms (Teaching Learning Based Optimization—TLBO and Artificial Bee Colony Optimization—ABC).

LRFD-AISC strength and serviceability constraints as well as FEMA 360 rehabilitation requirements are applied on space moment resisting frames solved with and without consideration of *Soil Structure interaction—SSI*. To perform the task, a program coded in MATLAB was developed. The developed program initiated and controlled the entire analysis procedure. *Finite Element Analysis—FEA* part of the procedure on the other hand, was performed by SAP2000 via OAPI by a two-way data stream. Overall weight of the steel frame was taken to be an objective function in all analyses, and the grouped steel profiles composing the structural members of the steel frame were defined to be, problem design variables. Then, effects of considering interaction of soil and structure—*SSI* for different soil types and soil depths on optimal design of the MRSF space and their contribution on the occurrence of plastic hinges are investigated. Furthermore, vertical displacements of foundation and period of structural systems were evaluated.

Results demonstrated the ability of the developed program to achieve optimal design. Moreover, the potential of TLBO to obtain optimum results earlier compared to ABC was shown. The best weight obtained using TLBO metaheuristic algorithms are 2% and 2.33% lighter than the best weight obtained by ABC, for 3- and 6 story MRFs respectively. With regards to soil types, results show that soil type C (stiff soil) has a negligible impact on the

optimal weight of the 2- story MRSFs compared to the fixed base and the optimal weight of the steel frame has only risen by 3.93% and 0.98% for TLBO and ABC respectively. While the optimal weight for soil type B (medium soil) has risen by 8.69 % and 10.33% for TLBO and ABC respectively. However, the optimum weight of the 2- story MRFs is significantly affected by soil type A (loose soil) with an increase of 16.5 % and 14.7% for TLBO and ABC respectively. Briefly, the optimal results of frame on soil type C and fixed based steel frame are practically very close to each other and is due to the greater stiffness of soil type C. The considerable low stiffness of soil type B, and notably soil type A, has promoted the swaying mode of foundation, which in turn has enhanced inter story drift and stories deflection. For soil depths, results illustrate that the soil depth of 5m has a negligible impact on optimal weight compared to fixed base and the optimal weight of steel frame has risen by 8.44% and 14.16% for TLBO and ABC respectively. For soil depth of 10m, the optimal weight has risen by 19.92% and 28.93% for TLBO and ABC respectively. However, a soil depth of 15m has a huge impact on the optimum weight of the 4- story MRSFs leading to an increase in weight of 25.55% and 36.73% for TLBO and ABC respectively. Moreover, the greatest soil surface settlement occurs at $H_s = 15 \text{ m}$, of 0.45 cm, whereas at $H_s = 10 \text{ m}$ and $H_s = 5 \text{ m}$, are at 0.35 cm and 0.3 cm, respectively.

In optimal designs of steel structure with SSI, displacements of the soil surface change as a function of the soil modulus elasticity. As a result, it is possible to conclude that higher soil stiffness leads to smaller surface displacements. The analysis process was strongly dominated by the inter-story drifts limitations of IO level. However, the huge rotation of the first story bottom columns for the model with SSI (Soil type A and Soil depth $H_s=15\text{m}$), illustrates that optimization problems were not only governed by drifts constraints at the IO Performance Level, but also rotation of these columns.

This coordinated application of metaheuristic and soil foundation into pushover analysis resulted into an automated procedure able to obtain optimum seismically safe steel structures. It is clear that the method will not only better interpret the real performance of buildings under seismic effects (failure mechanisms, displacements, etc.), but also lead to more economical structure designs.

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