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Performance Evaluation of Web Servers using Response Time and Bandwidth

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Abstract: Computer system performance can be measured using the amount of useful work accomplished by that system when compared to the time and resources utilized. Useful work here means how well the computer is doing the work it is supposed to do. Most websites work under the support of web servers that usually include hardware (for instance CPU, RAM, Disk, and network) as well as software (web services). The alarming growth in web traffic has led to performance problems and has necessitated much research activities as ways to improve web server performance. In this paper, a web server performance evaluation system that evaluates a server based on time of file execution and bandwidth has been designed and developed. Access log data sets from University of Port Harcourt web site were used to study the system and develop software for checking time of server activity in any domain where the application is executed. The evaluation system so designed and developed can be deployed in any server and the values generated can be used in making useful decisions that will mitigate certain occurrences in the future and also offers site owners information that will be useful in forecasting users visiting behaviours and the effects on the server downtime.

Keywords: Bandwidth, performance evaluation, response time, web server, workload.

1. INTRODUCTION

Computer systems are like automobiles that need periodic check to run efficiently and serve user's need. A good performance evaluation of a web server may involve measuring one or more of the following parameters: throughput, bandwidth, response time and so on. As the number of web users are increasing every day, the web servers are required to serve millions of requests per second from multiple users. This requires upgrading of the server in both hardware platform or software architecture. It then becomes very important to evaluate the performance of a web server in order to tackle the bottlenecks and optimize resource usage. Generally, the performance of a web server can be evaluated in measurable terms using one or more of these parameters which include throughput, response time, bandwidth, e.t.c.

A server is a computer system that serves the needs or requests of other systems usually referred to as clients. A **web server** is a computer program that delivers content, such as web pages using the Hypertext Transfer Protocol over the World Wide Web. This means that it delivers HTML documents and any other additional content that may be included in a document, such as images, style sheets, videos and so on. If the performance of a web server is measured by different clients that are remote to the web server, the result may be different depending on the requirements and configurations of the clients. But in this paper, we are specifically concerned with the practical analysis of the activity log files obtained from a university website in Nigeria and how a web site behaves in terms of workload, throughput, response time and bandwidth usage at peak load during a three months period of intensive new students registration and e- learning on the web server. The analysis result was used to design a system that can check the server activity based on bandwidth and response time.

2. RELATED WORKS

Performance Evaluation and measurement of web servers has been the subject of many research. Many works have been done by researchers in order to evaluate the performance of web servers. Early works in the 1990s carried out workload characterization of web servers to obtain the invariants which were used to optimize server performance. Xue et al (2003) measured the performance of web applications and quantified web page attributes that affect response time. They discovered that the most important factor speeding up web page response times was to minimize the number of embedded objects. Caching and also transfer rate were able to improve response time in request bit per second.

Abbas and Kumar (2011) evaluated the performance of web servers as perceived by a client in cases when (i) there was no data flowing between a web server and a client and (ii) there was data flowing from the web server to the client. They focused on three parameters: round trip latencies, access rate and connection throughput.

Manjur (2017) measured the performance of a web server under virtual environment (that is, the web server was hosted on a virtual machine to measure latency). He compared the results obtained with that from a dedicated machine and from the result, it was found that the difference between two sets of results was largely negligible but in some areas one approach performed better than the other.

Nguyan (2017) presented an empirical analysis of several web servers including Apache, NodeJS and NginX to precisely figure out the trade – off between different software designs to tackle the performance bottleneck and resource consuming problems. Their result showed that the performance of the web servers is considerably improved with a good memory allocation.

Adepele et al (2006) evaluated the performance of web servers on the basis of the server load and the network load and it was discovered that to improve the performance of the websites, optimization of the load on the web server was needed which requires capacity planning strategies.

Another researcher (Lind, 2014) evaluated the performance of HTTP web servers on selected hardware platforms for embedded systems using load limits, performance characteristics and system resource usage. A simulated web application was used for the test and a total of five HTTP server software were tested. The overload behavior and efficiency of system resource usage differed greatly between the servers. The test results also showed that the performance varied significantly between HTTP server software running on the same hardware platform, and generally the software with limited feature sets performed best.

Jader et al (2019) reviewed different works that addressed web server performance and load balancing algorithms in the last half decades, to compare their capabilities and provide an efficient platform to build web-based system structures.

Xianghua et al (2013) presented web server performance evaluation model based on response time (MBRT) to evaluate the peak load of a Web server with given configuration. This was based on the special relationship between the response time and throughput when request rate was lower than peak load. They discovered that MBRT was simple when compared with other models and it has also been validated from both theoretical and empirical point of view in real environment.

3: RESEARCH DESIGN

3.1. Design Methodology

The design methodology adopted for the proposed system is the Water-fall Model. Water-fall model is a sequential model that divides software development process into different phases. Each phase is designed for performing specific activity during the Software Development Life Cycle (SDLC) and each phase must be completed before the next phase can begin without overlap between the phases.

3.2 Analysis of the Existing System

The World-Wide Web or the Web is a distributed hypertext-based information system. The client-server model, as it applies to the Web environment, is shown in Figure 3.1. A user accesses documents on the Web through a Web browser such as Google Chrome. The Web browser sends the user request to a Web server, which responds with the requested document. A Web server can respond to requests from multiple clients and this communication between a web server and client is always initiated by the client using Hyper Text Transfer Protocol (HTTP). Communication between a Web client and a Web server is carried out in the following manner: when a client has a request to make of a particular Web server, the client must contact that server. The server listens on a designated port for a request from a web client to establish a TCP connection. Once a TCP connection has been opened and the client has made its request, the server then parses the request and issues a response. The response includes a status code to inform the client if the request was successful or not. If the request is successful, a document is usually returned with the response. If the request is not successful, a reason for the failure is also returned to the client. Once the response is completed, the TCP connection between the Web client and Web server is closed. This process is repeated for each document that a client wishes to retrieve from a web server. The web server keeps activity log of various requests and responses issued at every given time which can be accessed for performance check of that system and for other purposes as the need arises.

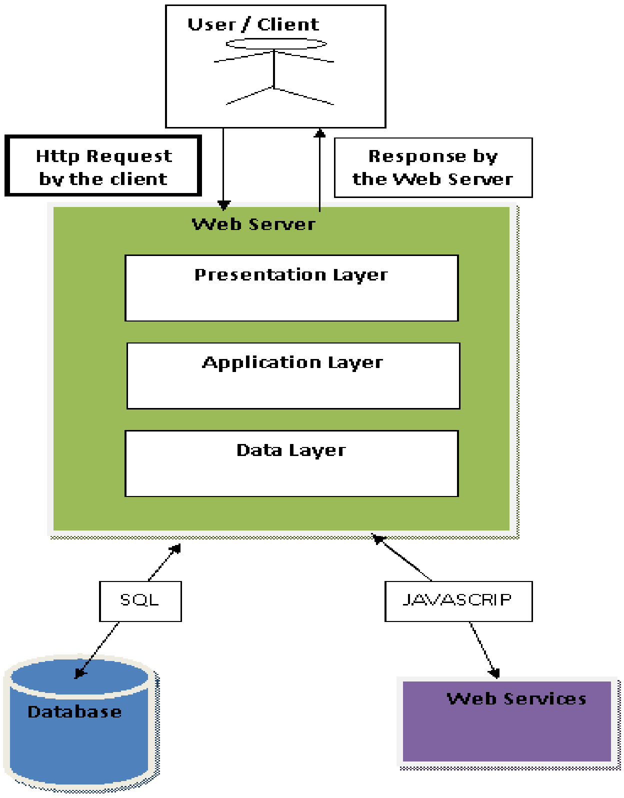


Figure 3.1: Existing System Architecture

3.2.1. Disadvantages of the Existing System

The existing system has the following drawbacks:
1. Web servers face a large number of users.
2. They must provide high availability services with low response time.
3. They must also guarantee a certain level of throughput.
4. The continuous increase of traffic on the web.

3.3 Analysis of the Proposed System

The major function of Web Servers is to provide documents to web clients that request them. In order to improve the performance of Web servers, knowledge of the workloads that these servers are required to handle is needed and also the variables that can affect the performance of these systems are also required. Knowledge of these workloads can be acquired by analyzing logs of web server’s activity. These logs are extremely valuable, because they provide a snapshot of actual requests to Web servers. In this research, log files obtained from University of Port Harcourt web server were examined to locate workload invariants. These variants were used to create a workload model of a web server, and also develop a

system that can check server activity in that domain to identify possible performance enhancements for web servers.
In the proposed system, web server evaluator (UPHevaluator) layer is introduced to configure and conduct evaluation test and to also provide real time information about the current test iteration results as shown in figure 3.2. This side is accessible from outside the web server to avoid traffic that may influence the test result.

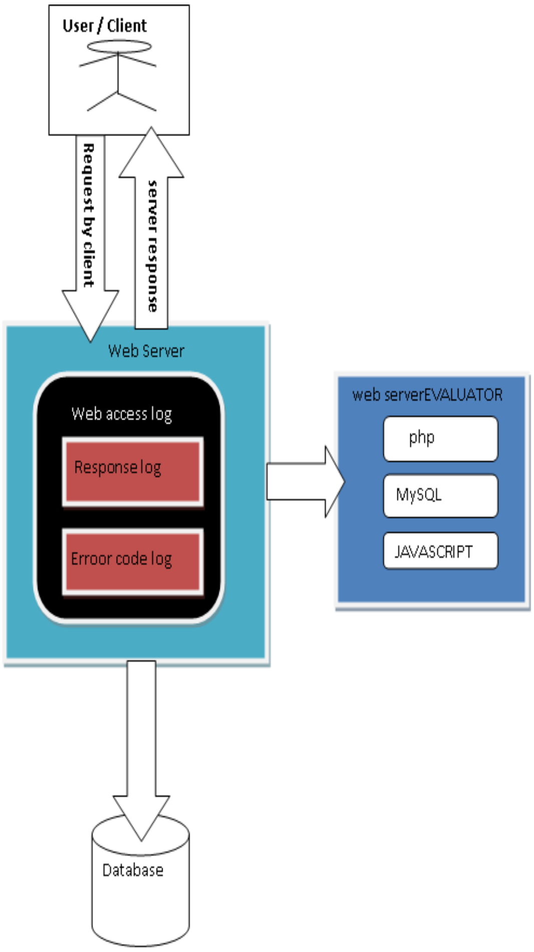
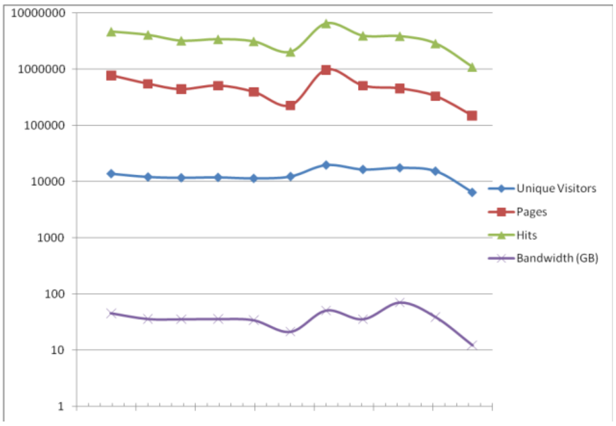


Figure 3.2: Proposed System Architecture

The plot of the raw data is shown in figure 3.3. From the graph in figure 3.3, it can be seen that the highest number of unique visitors, number of visit, pages and hits occurred during the month of July, 2019 while the highest bandwidth occurred in September,2019. It can be inferred that the bandwidth is not directly proportional to the number of unique visitors, their number of visits, hits and pages. The bandwidth depends on other factors not considered in this work.



Dec-18 Jan-19 Mar-19 May-19 Jun-10 Aug-19 Sep-19 Nov-19

Figure 3.3: Access Log Characteristic Graph

In figure 3.4, the graph of server response in relation to error logs is clearly shown; the highest error is that of partial content. It is much more likely to result from slow internet connection and file sizes of pages on the web server. This condition is understood since many students visited the site from their Phones and PDAs. Document not found error is about 100,000kb (100mb out of 400GB) in bandwidth and 79,565hits (approximately 80,000 hits out of 38,000,000 hits). The graph clearly shows falling of error on bandwidth and hits as other error parameters are evaluated.

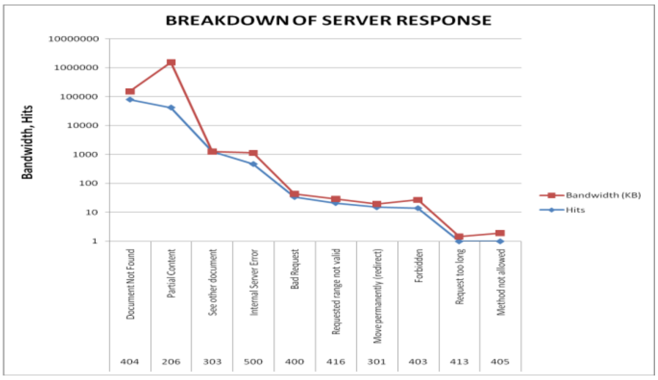


Figure 3.4: Breakdown of Server Response

3.4 Analysis of File Size Distribution

Images get more blurred; video gets less clear and often server variation restrict some of the file format from executing directly in them. For instance some web servers do not allow the execution of tiff image files but jpeg are allowed. Other file types are executed at variance depending on the web server and the application program powering it.
Apache web servers do not allow file name variation involving upper case and lower case mix up but internet information service (IIS) allows mix of lower case and upper case. Server users need to check some of this conditionality in their system.

From the graph of File Type Distribution in Figure 3.5, it is clear that the images generate the highest bandwidth on the site, followed by Acrobat files and finally java scripts, HTML files and Cascaded Style sheets. This indicates that the higher the file size, the higher the possible bandwidth the file requires.

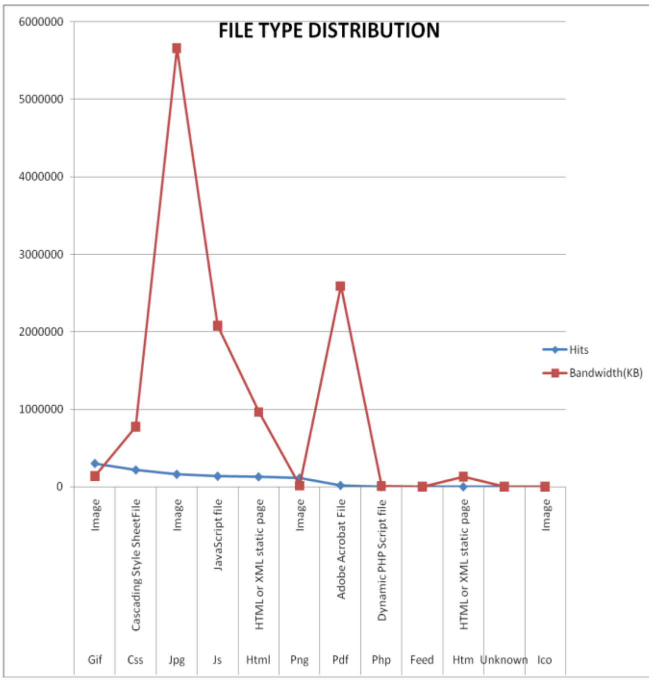


Figure 3.5: File Type Distribution Graph

3.5 Advantages of Proposed System

The proposed system has the following advantages:

1. The performance evaluator system stores all the measurements and parameters for measurement.

2. It provides a real time report on the web server being tested.

3. It evaluates a system using its own workload invariants which helps website owners to make informed decision concerning their own website or system.

3.6 Algorithm of the proposed system

The following steps are taken by the new system to evaluate the performance of web servers:

a. Set the server workload.

b. The parameter is checked based on the solution verified and selected for attention. This will lead to execution of the system process involved in the performance evaluation core using the parameters that have been fetched from the server.

c. Once the parameters are fully collected, the check will lead to decision whether the parameters are okay or not depending on the evaluation benchmark that is guided by what is core in the server.

d. If the parameters are Ok the server performance are evaluated. Otherwise the performance bottlenecks are identified and server tuning is done to enhance the server performance.

e. The server log data are fetched and new parameters generated and used in evaluating server performance.

f. The final performance solution is then presented for verification. If the verification is as expected a performance evaluation must have been concluded.

g. However, if the solution did not take into considerations all the factors required, the lapses are resubmitted for a fresh parameter checks and the cycle continues.

3.6.1 Overall System Flowchart of the Proposed System

The system flowchart of the proposed system is shown in figure 3.6.

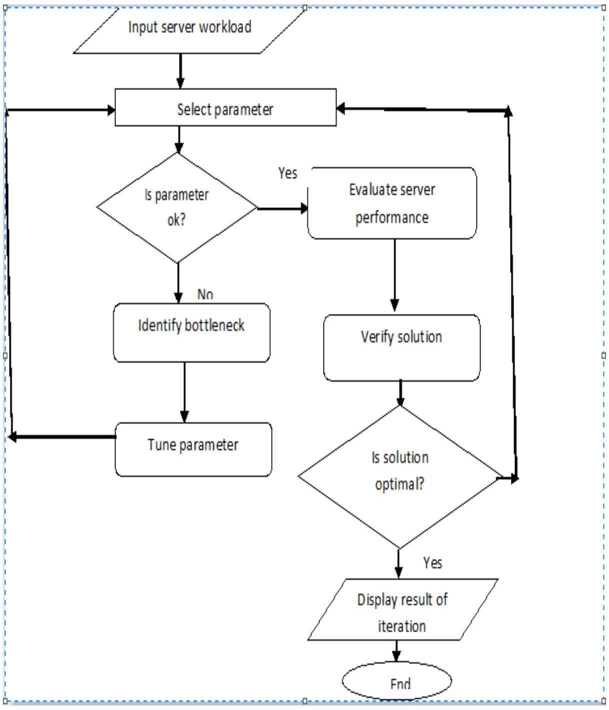


Figure 3.6: Flowchart of the proposed system

4. IMPLEMENTATION AND DISCUSSION OF RESULTS

Implementation process which is the coordination of the web log data collected from university of Port Harcourt web server and programming codes wer`e written to conform to the requirements set out in the system specification. Considering the fact that the project involves the use of interfaces for input of data and display of output, data generated from the web server for the purpose of the performance evaluation and for referencing the data for output, the system was developed using the following tools and languages. The programming languages selected for the purpose of implementation of the system developed in this project were HTML, PHP, and JavaScript. PHP was selected as the server side logic language while JavaScript was selected as the database server that will be used for the operation. HTML was selected as the language by which the web server and client browsers communicate for encoding information for web documents. **Apache Web Server** was employed to construct the web server and its associated data management system which was used as the relational database management system. Apache-HTTPD was used in this system to capture the timing required for execution of files and passed into the program for use in the computation of the response time required in the evaluation of the performance of the web server. Apache serves the PHP pages to the client browser for the display of the time used in the execution of each of the iterations. JavaScript was used mainly because the function calls provided an iteration that recalls the PHP code automatically whenever it has to recompute the server time. This system consists of three major modules, which are:

1. Parameter capture Module
2. Performance Evaluation Module
3. Result Data Module

Parameters capture Module

The Application module works directly with the PHP. It is an open source application module, which can be easily modified, and it is for capturing of server clock values before and after the execution of a single module in the system program. An application module consists of the response time capture and display offers The Application module works directly with the PHP. It is an open source application module, which can be easily modified, and it is for capturing of server clock values before and after the execution of a single module in the system program. An application module which consists of the response time capture and display offers the user the opportunity to view the activity time on their browser. All required information must be provided and displayed in the browser in order to successfully complete its execution.

Performance Evaluation Module

Flexibility and Availability in this application enables the user to view this interface and interact with this module in two separate interfaces before the access logs from the system can be used in the evaluation of the performance. This module allows the performance evaluator to examine the minimum time interval and the maximum time interval as well as the average time for all the execution done in the system. This generated information can be used to make decision on whether the server is performing well or not as shown in figure 4.1.

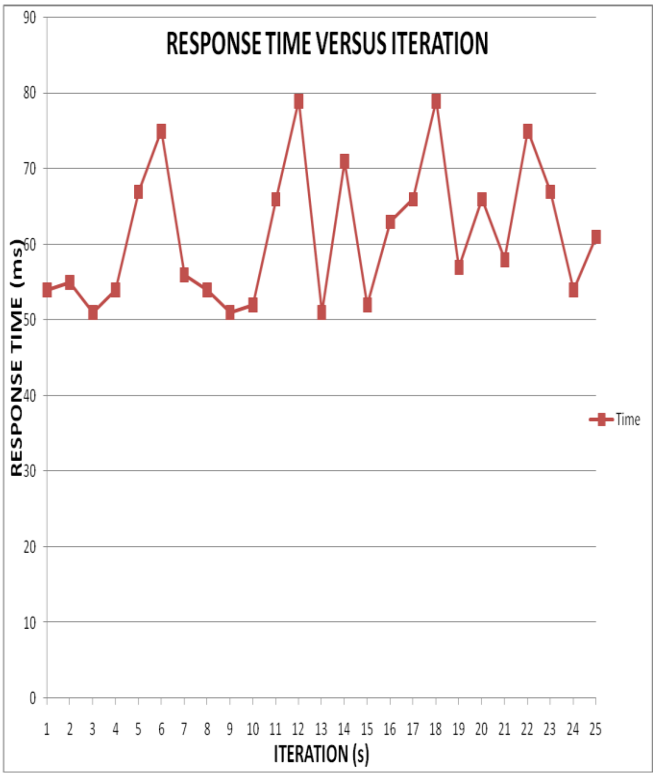


Figure 4.1: Performance Evaluation graph

Result Data Module:

This module is available in the data section of the system on the browser pages. When the system is running on a remote server, the performance result data can be displayed on the browser too showing the result data of the particular remote web server where it has been deployed. After clicking the button, a confirmation message will be displayed and the performance data generated will be displayed

5. CONCLUSION

The project has successfully studied the concepts of web server system and its associated parameter required for the evaluation of the system. We have also been able to develop a system that is useful for the evaluation of the web server using the access log data provided from the University of Port Harcourt web site. The access log takes care of logic and decision for use in the implementation of the service needed for the performance evaluation. It also offers us a more articulated direction on the possible users' behaviours on the site which is necessary to evaluate the system.
From the results obtained, it can be concluded that: the bandwidth size is not entirely a function of number of visitors, their number of visits, the pages visited and the hits, the breakdown of server response negatively imparts on the access log characteristics by reducing the size of the bandwidth and that the response time of the web server depends mostly on the bandwidth as well as the file types and sizes. From the result generated and plotted in the graph, it is clear that the server is performing well based on its response time and the low level of error generated from the access log within the period of investigation. It is also clear that the site designer needs to work on the images and pdf files so that the ratio of image bandwidth to other file size will be minimal. The evaluation system so designed and developed will be

deployed and the values collected used in making useful decisions that will mitigate against certain occurrences in the nearest future. It also offers site owners information that can be used in forecasting users visiting behaviours for instance how the server will fair during students registration or other related concerns.

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Earthquake Performance Analysis of Existing Reinforced Concrete Structure in the Istanbul Metropolitan Area Using Non-Linear Method

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Abstract: Many people suffer loss of life and property due to the earthquake disaster in Turkey. In order to minimize this disaster, the resistance of the structures to earthquake should be determined under the light of studies done before. Turkey is in one of the most dangerous seismic regions in the World. As a sub plate between the large Eurasian, Arabian and African plates, Turkey rests on the Anatolian tectonic plate. Arabian plate in the direction of pushing Turkey to the North, the Anatolian plate moves in the opposite clockwise. However, it is prevented to move in the North direction due to the Eurasian plate. In this case, it contains many active faults and Turkey's most populous city of Istanbul is in danger. In this study, it is aimed to perform a performance analysis according to the Turkey Building Earthquake Code 2018 in a six-story reinforced concrete shear wall-framed structure in Istanbul where active fault lines are located. This existing complex designed reinforced concrete building investigated in this study is in the city of İstanbul, Turkey. This city is under danger of approaching and inevitable Great Istanbul Earthquake likely greater than Mw 7. The nonlinear seismic behavior of a complex reinforced concrete (RC) residential building is investigated by and linear and the static pushover. The selected reinforced concrete structure was designed according to 1998 version of Turkish Earthquake Code (TEC-1998). In the earthquake engineering, performance-based design method is used to determine the level of expected performance of the structures under the earthquake effect. According to the code, the reinforced concrete shear wall building is not expected to satisfy controlled damage (life safety) performance levels under design earthquake
Keywords: Reinforced concrete shear-wall structure; failure analysis, performance-based design; Turkey Building Earthquake Code 2018.

1. INTRODUCTION

A large part of the territory of Turkey is facing earthquake hazards. Earthquake researches have been carried out in Turkey where earthquakes have caused loss of life and property in a very short time and as a result many earthquake regulations have come into force. Istanbul districts close to the Marmara Sea are at higher risk of earthquakes. Turkey's most populated city saw changes in terms of risk after an update of an earthquake risk map dating back to 1996. In the early hours of August 17, 1999, a 7.4 magnitude earthquake rattled the Marmara region east of Istanbul for 45 seconds. More than 18.000 people were killed, according to official numbers, with another 50.000 injured and nearly 300.000 left homeless. The Izmit earthquake and another deadly, quake three months later in the city of Duzce that killed nearly 900 people highlighted the loose construction standards across Turkey and the ill preparedness of emergency services. The Marmara Sea region housing one third of Turkey population is one of the most tectonically active regions in Eurasia (Kalkan et all., 2008).

However, the inadequacy of regulation contents and performance analysis in this context has not prevented major earthquake damages, continued to cause loss of life and property. Turkey Building Earthquake Code – 2018 (TBEC – 2018), which was formed as a result of performance analysis studies, came into force. In this context, performance analysis of an existing reinforced concrete shear wall structure within the borders of Istanbul province located in the dangerous earthquake zone was made according to TBEC-2018. Dya and Oretaa (2015) seismic vulnerability assessment of soft story irregular buildings using pushover analysis. Inel and Meral (2016) evaluated seismic performance of existing low and mid-rise reinforced concrete buildings by comparing their displacement capacities and displacement demands under selected ground motions experienced in Turkey. Jialiang and Wang (2017) the model with four-stories and two-bays was pseudo-dynamically tested under six earthquake actions whose peak ground accelerations (PGA) vary from 50 gal to 400 gal. Huang et all (2017) a linear analysis procedure was developed for accurate assessment of the seismic performance of buildings and the computation of direct and indirect economic losses resulting from earthquake shaking and is suitable for application to low- and medium-rise buildings of regular configuration.

The objective of this study is Perfomance based assessment of existing complex in plan shear-walled building with strength-based method, one of the linear method, and displacement-based approach, one of the nonlinear method. Determining seismic performance of existing building is important because of getting ready for probable earthquake to be occured.

The building is typical beam-column RC frame buildings with shear walls. The selected building was designed according to TEC-1998 considering both gravity and seismic loads. The nonlinear dynamic analysis is performed by using the finite element program SAP 2000. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. Seismic performance evaluation is carried out in accordance with the recently published TBEC-2018 that has similarities with FEMA-356 guidelines. This existing complex designed reinforced concrete building investigated in this study is in the city of İstanbul, Turkey. This city is under danger of approaching and inevitable Great Istanbul Earthquake likely greater than Mw 7. Thus, investigation of earthquake performances of this or similar complex buildings are very important.

2. THEORY

2.1. General Principles for The Evaluation and Design of Buildings Under Earthquake Effects According to TBEC–2018

As shown in Fig. 1a, five points labeled A, B, C, D, and E define force–deformation behavior of a plastic hinge. The values assigned to each of these points vary depending on type of element, material properties, longitudinal and transverse steel content, and axial load level on the element (ATC-40; FEMA-273, Çavdar et all.,2018).

Similar to ATC and FEMA, three limit conditions have been defined for ductile elements on the cross section in TBEC-2018. These are Limited Damage Reigion (SH), Marked Damage (KH) and Collapsing Limit (GÖ) (Figure 1b).

Performance Levels of Buildings to be Designed Under the Effect of Earthquake according to TBEC-2018 (Figure 1b):

a) Limited Damage Performance Level (SH); the building corresponds to the level of damage to the structural system elements, whereby a limited degree of damage or non-linear behavior occurs.

b) Controlled Damage Performance level (KH); in order to ensure life safety, the building seismic resisting system corresponds to the level of controlled damage which is not very heavy.

c) Collapse Prevention Performance Level (GÖ); the building corresponds to the to the pre-cash situation where severe damage to the structural system elements occurs. Partial or complete collapse of the building was prevented.

Turkish building earthquake code was also under the influence of two different design approaches and their corresponding calculation steps described.

2.1.1. Strength-based Design Approach

In the approach of Strength-Based Design, (a) reduced seismic loads is determined corresponding resisting system ductility capacity defined for a projected certain performance target (b) linear seismic calculation of resisting system is done under reduced seismic loads. Strength demands are obtained by combining reduced internal forces found from this calculation, if needed by taking into account the excessive strength, and the internal forces occurred from other loads. The Linear Earthquake Calculation (LEC) with Equivalent Seismic Load Method (ESLM) applicable buildings given in Table 1.

LEC with ESLM is done as follows:

LEC with ESLM is done as follows:

$$V_{tE}(X) = m_t S_{aR} (T_p^{(X)}) \geq 0.04 \, m_t I S_{DS} g \quad (1)$$

$$V_{tE}(Y) = m_t S_{aR} (T_p^{(Y)}) \geq 0.04 \, m_t I S_{DS} g \quad (2)$$

$$\Delta FN = 0.0075 \, N \, V_{TE} \quad (3)$$

$$F_{iE} = (VTE - \Delta FN) \frac{W_i H_i}{\sum_{J=1}^{N} W_J H_J} \quad (4)$$

Additional equivalent seismic load, Δ_{FN}, acting at the N'th storey (top) of the building shall be determined by Equation (3). Excluding Δ_{FN} , remaining part of the total equivalent seismic load shall be distributed to stories of the building (including N'th storey) in accordance with Equation (4).

In the Equation 1-4, V_{tE} is basement shear force, m_t is total mass, S_{aR} is reduced design spectral acceleration, I is building importance factor, S_{DS} is short period design spectral acceleration factor, the $T_p^{(X)}$ is the natural vibration period prevailing in the X direction of the building, $T_p^{(Y)}$ is the natural vibration period prevailing in the Y direction of the building, g is acceleration of gravity, F_{iE} is i. equivalent earthquake load acting on the center of mass.

In the 15th part of the TBEC-2018, the calculation of the Equivalent Seismic Load Calculation shall be made taking into calculation the following situation:

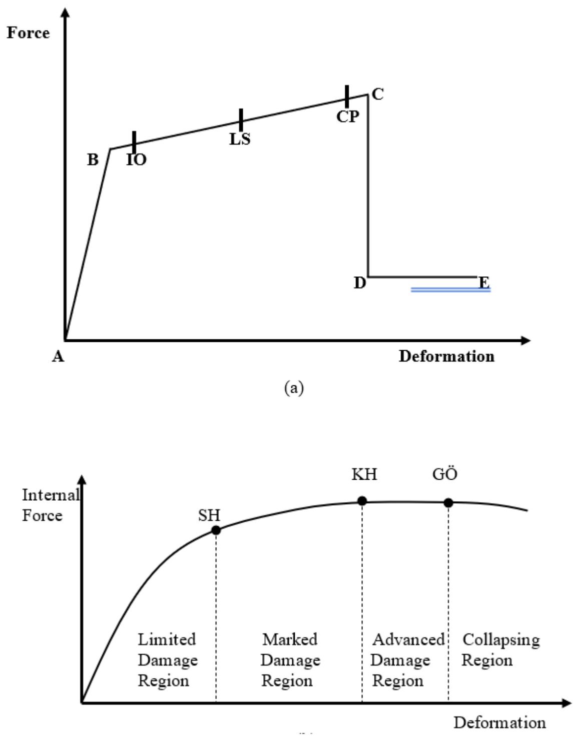
The buildings in which the ESLMcan be applied are given in Table 4.4. According to TBEC-2018, additional eccentricity will not be taken into calculation in the earthquake calculation of buildings. In the calculation of the total equivalent earthquake load (base shear force) according to Eq. (1) and Eq. (2) Ra = 1 will be taken.

In addition, in order to perform the calculation of the the equivalent static method, existing structure seems to meet the following boundary conditions:

The building height class is smaller than 5 (BHC <5).

Table 1. ESLM Applicable Buildings

Building Type	Allowed Building Height Class	
	EDC=1, 1a, 2, 2a	EDC=3, 3a, 4, 4a
Buildings where the torsional irregularity coefficient of each layer provides the condition ηbi≤2.0 and also the type B2 of irregularity	BHC ≥ 4	BHC ≥ 5
All Other Buildings	BHC ≥ 5	BHC ≥ 6



(b)

Figure 1. Building performance levels according to TBEC-2018.

2.1.2. Deformation-Based Design Approach

The Deformation-Based Design Approach (DBDA) is a modern design approach that is expected to be used more widely in the near future, allowing the modeling of the actual behavior of the seismic resisting system for a variety of performance goals and its nonlinear calculation (TBEC-2018).

In the Deformation-Based Design Approach, it is needed following steps according to the regulation:

- The internal force-deformation relationships of existing or previously designed structural system elements compatible with non-linear modeling approaches are determined.
- Under the earthquake ground motion selected in accordance with the performance target compatible with the Regulation, dynamic incremental methods are calculated in the static or time history of the seismic resisting system, deformation demands related to nonlinear ductile behavior and resistance to brittle behavior are obtained.
- The resulting deformation and internal force demands are compared with the deformation and strength capacities defined in accordance with the performance target specified in the regulation.
- For existing buildings, the assessment is defined by showing that the deformation and strength demands are below or exceed the deformation and resistance capacities corresponding to them.
- For the existing buildings to be newly constructed or reinforced, if the deformation and strength demands are below the corresponding deformation and resistance capacities, the deformation-based design is completed. Otherwise, the sections of the element are changed, and the calculation is repeated, and the deformation-based design is completed. (TBEC- 2018).

Deformation-based Design and Evaluations are taken into consideration as follows:

- Single Mode Pushover Methods can be used for buildings that have a Building Height Class (BHC) ≥ 5 and meet the conditions found in the regulation.
- Multi-Pushover Methods are available for all buildings with a BHC ≥ 2.
- The Nonlinear Time History Analysis Method can be used for the earthquake calculation of all buildings. This method is mandatory for high-rise buildings in Section 13 of TBEC-2018.

3. METHODS

3.1. Description of Investigated Reinforced Concrete Shear Wall Structure

The existing reinforced concrete framed building in Bahçelievler district of İstanbul consists of a basement floor, ground floor and 4 floors. The plan of the building is 19.04 m in X-direction and 8.81 m long in Y-direction. Each floor is in 2.80 m in height and it is totally 16.8 m in height. Seismic resisting system is frame-shear wall system. The concrete class is C30 and the reinforcement steel is S420. Typical geometry and reinforcing detail of column and beam cross-section areas are shown in Fig.2. In Fig. 2, the number before "Ø" is the number of bars, and after "Ø" is the diameter of bar in mm. The beam sizes in the building are designed as 25×50 cm and there are different column dimensions. Column dimensions are given Figure 2. The column dimensions in a defined position in the plan are the same in the other stories of the building. It is seen that Figure 2, longitudinal rebars are Ø16 for all columns. The longitudinal reinforcement ratio of these columns varies between 1.1% and 1.5%. The dimensions of all the beams in the building are the same as 25x50 cm. Beam longitudinal rebars are 3Ø16 on top and 3Ø16 in bottom for the residential building. Transverse rebars are Ø8/15 cm for columns and beams. Flexural rigidity is calculated for each member. Beams and columns were modeled as frame elements which were connected to each other at the joints.

The thickness of the slabs is the same throughout all floors, including two thicknesses of 12 and 13 cm. The basement floor of the building is designed as a shear-wall. This situation does not continue on the upper floors. ZC ground class is used in the district of Bahçelievler, Istanbul. A constant load of 1.5 kN/m2 and 2 kN/m2 as a moving load, except for its dead weight (TS 498, TS 500), are assigned to the building. A typical floor plan is shown in Figure 3. Three-dimensional finite element model of the residential building was prepared in structural analysis program (SAP2000) shown in Figure 4.

The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns and beams. Predominant mode periods of the building in X and Y directions are 0.904 and 0.713 s, respectively, based on cracked section properties.

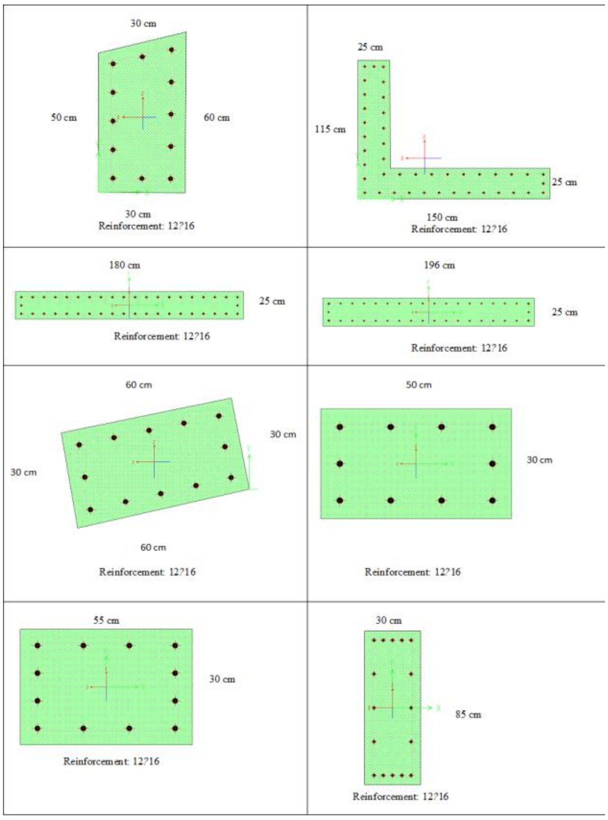


Figure 2. Dimensions and reinforcement of columns.

Figure 3. Typical floor plan of the building.

3.2. Determination and Assignment of Earthquake Loads

The floor weights required to calculate the earthquake loads shall be obtained from the sum of the mass values generated automatically by the program on the respective floor.

According to equivalent earthquake loads methods in TBEC-2018, the horizontal earthquake loads, +0.05 and -0.05 eccentricities are not distributed to the floor levels without considering the existing structures.

The structure was modeled by SAP2000 program and all the required loading was applied on the model according to the information obtained from the regulations and the analysis result was examined. As a result of the analysis, it was observed that the first natural vibration period in the X-direction of the structure (T1x) was T_{1X} = 0.904 s and the first natural vibration period in Y-direction (T1y) was 0.713 s. Natural vibration period; $T_{pA} = C_t\ HN^{3/4} = 0.1\times (16.8)^{3/4}$ = 0.8298 s according to section 4.7.3 in TBEC-2018. According to this, the maximum natural value of the structure should not be taken more than 1.4 times of the T_{pA} period (T_{pA} = 1.4×0.8298 = 1.162 s). Since the calculated dominant period values are smaller than this value, T_{1X} = 0.904 s will be based on calculated. The building is located in the district of Kocasinan, Bahçelievler district of Istanbul. It has been found to be 41.0036˚ North and longitude 28.8406˚ East. Accordingly, Turkey Earthquake Hazard Map of the S_S and S_1 values, respectively (AFAD,2018);

Map spectral acceleration coefficient for short period region S_S = 1.098 g. The map spectral acceleration coefficient for the 1.0 second period region is S_1 = 0.302 g.

The local soil class at the location of the building is defined as ZC. According to this, local soil impact coefficients using the earthquake regulation S_S = 1.098 and S_1 = 0.302 (F_S, F_1):

$F_S = 1.2$ and $F_1 = 1.5$

Design spectral acceleration coefficients:

$S_{DS} = S_S\ F_S = 1.098 \times 1.2 = 1.318$

$S_{D1} = S_1\ F_1 = 0.302 \times 1.5 = 0.453$

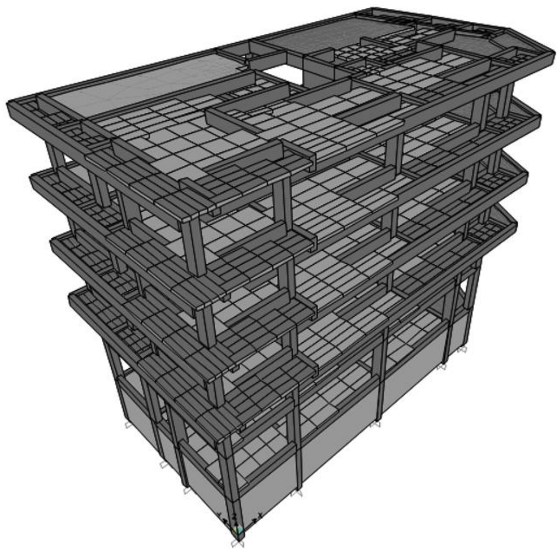


Figure 4. Three-dimensional finite element model of the residential building.

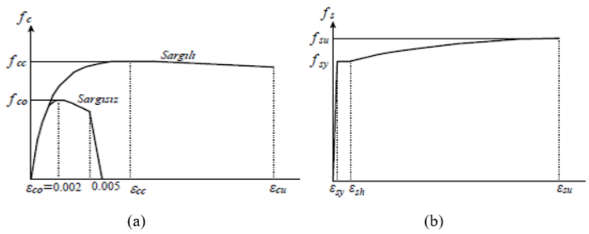


Figure 5. Stress-strain relationship of concrete (a) and reinforcing steel (b).

As the structure is used as a conventional building, the building usage class (BUC = 3) and the Building Importance Factor I = 1.0 (TBEC-2018, Table 3.1). Earthquake Design Class (EDC) is determined as EDC=1 using S_{DS} = 1.318> 0.75 and BUC = 3 values (TBEC-2018 Table 3.2). Building Height Class is determined as BHC = 6 using 10.5 m≤ H = 16.8 m ≤ 17.5 m and Eathquake design class (EDC) = 1 values (TBEC-2018 Table 3.3). Since the structure in question will be a cast-in-place (except for High-Rise Buildings) structures, for Earthquake Level DD-2 using the values of BHC = 6 ≥ 2, DTS = 1; Controlled Damage (CD) and design approach of the normal performance target were determined to be DBDA.

The remaining part of the total equivalent earthquake load ΔF_N is distributed to the building floors using the Equation 4. These values are given in the Table 2.

Table 2. Total Equivalent Seismic Loads

Floor	hi (m)	Hi (m)	Wi (kN)	Wi Hi	Ratio	F_{iX} (kN)	F_{iY} (kN)
4	2.80	16.8	1607.057	26998.566	0.2005	1651.010	2088.600
3	2.80	14	2684.503	37583.036	0.2791	1860.932	2360.062
2	2.80	11.2	2684.503	30066.429	0.2233	1488.746	1888.050
1	2.80	8.4	2684.503	22549.822	0.1672	1116.559	1416.037
Ground floor	2.80	5.6	1959.740	10974.543	0.0815	543.407	689.157
Basement	2.80	2.8	2315.058	6482.163	0.0481	180.023	180.023
Total:			12971.44	134654.559		6840.677(*)	8621.929(*)

4. RESULTS AND DISCUSSION

4.1. Calculation of Irregularity Types

4.1.1. A-1 Torsional Irregularity Calculation

The calculation steps for A-1 torsional irregularity are shown in the following tables. According to this calculation, torsional irregularity coefficient p_{bi} was greater than 1.2 was observed in the presence of torsional irregularity. The torsional irregularity ratio for each storey given in Table 3-4.

$$(\boldsymbol{\Delta_i})_{\mathbf{ort}} = \frac{(\boldsymbol{\Delta_i})_{\max} + (\boldsymbol{\Delta_i})_{\min}}{2} \tag{5}$$

$$\eta_{bi} = \frac{(\boldsymbol{\Delta_i})_{\max}}{(\boldsymbol{\Delta_i})_{ort}}$$

According to the data obtained from the calculations and the calculation steps taken from the above calculations (Equation 5) according to TBEC-2018. Torsional Irregularity was not found in the X and Y direction of the building.

Table 3. Calculating A-1 Torsional Irregularity in X Direction

Storey	dimin	dimax	(Δi) min	(Δi) max	(Δi) ort	ρbi
4	0.2584	0.2958	0.0494	0.0481	0.0488	0.99
3	0.209	0.2477	0.0571	0.0614	0.0592	1.04
2	0.1519	0.1863	0.0617	0.0714	0.0666	1.07
1	0.0902	0.1149	0.0564	0.0722	0.0643	1.12
Ground floor	0.0338	0.0427	0.0334	0.0423	0.0378	1.12
Basement	0.0004	0.0004	0.0004	0.0004	0.0004	1

Table 4. Calculating A-1 Torsional Irregularity in Y Direction

Storey	dimin	dimax	(Δi) min	(Δi) max	(Δi) ort	ρbi
4	0.2584	0.2958	0.0494	0.0481	0.0488	0.99
3	0.209	0.2477	0.0571	0.0614	0.0592	1.04
2	0.1519	0.1863	0.0617	0.0714	0.0666	1.07
1	0.0902	0.1149	0.0564	0.0722	0.0643	1.12
Ground floor	0.0338	0.0427	0.0334	0.0423	0.0378	1.12
Basement	0.0004	0.0004	0.0004	0.0004	0.0004	1

4.1.2. B-2 Interstorey stiffness irregularity (Soft Storey)

In the calculation of B-2 stiffness irregularity between interstorey stiffness, except basement floors, in the case of Stiffness Irregularity Coefficient p_{ki} > 2.0 defined by the ratio of the mean relative floor displacement rate on any floor to the ratio of the average relative displacement in an upper or a lower floor, the Stiffness Irregularity check between the neighboring layers was performed.

$$\eta_{ki} = (\Delta_i / h_i)_{ort.} / (\Delta_{i+1} / h_{i+1})_{ort.} > 2 \tag{6}$$

$$\eta_{ki} = (\Delta_i / h_i)_{ort.} / (\Delta_{i-1} / h_{i-1})_{ort.} > 2 \tag{7}$$

Table 5-6 shows the soft strorey irregularity calculations for X and Y directions. In accordance with the above calculations, no calculation was found that the average relative floor displacement rates on one floor were greater than 2. In other words, there is not type B-2 irregularity in the structure.

Table 5. B-2 calculation in X direction

Storey	hi (m)	di (m)	Δi (m)	(Δi/ hi)ort.	ɼki (12*)	ɼki (13*)
4	2.80	0.2785	0.0490	0.0175	-	0.83
3	2.80	0.2295	0.0591	0.0211	1.21	0.88
2	2.80	0.1704	0.0669	0.0239	1.13	1.04
1	2.80	0.1035	0.0647	0.0231	0.97	1.69
Ground floor	2.80	0.0388	0.0384	0.0137	0.59	-

Table 6. B-2 calculation in Y direction

Storey	hi (m)	di (m)	Δi (m)	(Δi / hi)ort.	ɼki (12*)	ɼki (13*)
4	2.80	0.1140	0.0241	0.00368	-	0.77
3	2.80	0.0899	0.0258	0.00479	1.30	0.83
2	2.80	0.0641	0.0263	0.00575	1.20	0.99
1	2.80	0.0378	0.0230	0.00579	1.01	1.49
Ground floor	2.80	0.0148	0.0141	0.00389	0.67	-

4.2. Determination of Earthquake Performance Pushover Analysis Method

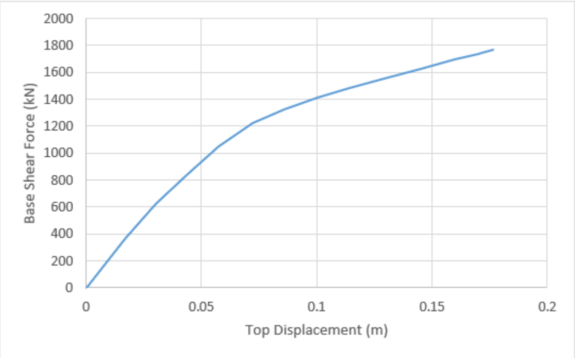
The Pushover Analysis method based on the performances of the structures under different earthquakes in order to control and strengthen the seismic resisting systems of existing structures. The based performance value is determined by the degree of damage that may occur in the structural and non-structural elements of the building.

This method of analysis is much more complex than the force-based calculations, however a more realistic method allows solutions to be more accurate and economical.

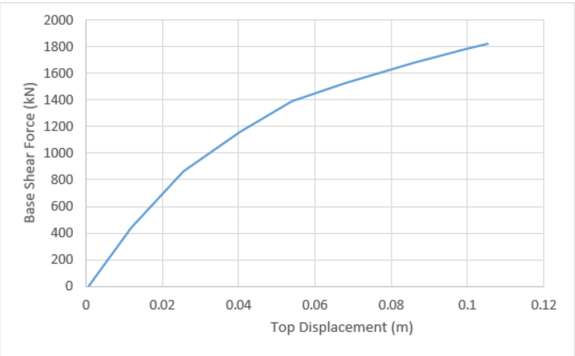
In this analysis method, the probability of exceeding the spectral magnitudes by 50 years was 2% and the corresponding repetition period was 475 years. In accordance with the information received from TBEC-2018, the status of Limited Damage (LD), Controlled Damage (CD), Pre-collapse Damage (PD) status of the existing structure were examined. In accordance with the information received from TBEC-2018.

It can be seen from the result under soil class ZC design earthquake of the pushover analysis through the X and Y direction (Figure 6) that building collapsed before reaching the push target. It is concluded from nonlinear static pushover analysis under design earthquake that according to displacement target of the building, the building not provided controlled damage performance level (CD) rating in the view of CD level targeted in TBEC-2018. According to TBEC-2018, the reinforced concrete shear wall building is not expected to satisfy CD performance levels under design earthquake.

Design earthquake is converted to spectrum curve and modal displacement demand is determined and performance points are determined by TBEC-2018 as seen in Figure 7. The plastic hinges are obtained by pushing again the bearing system up to this demand. It is seen in Figure 7 that, in case the incremental repulsion analysis is conducted via applying the Incremental Equivalence Seismic Load Method, the "modal capacity



(a)



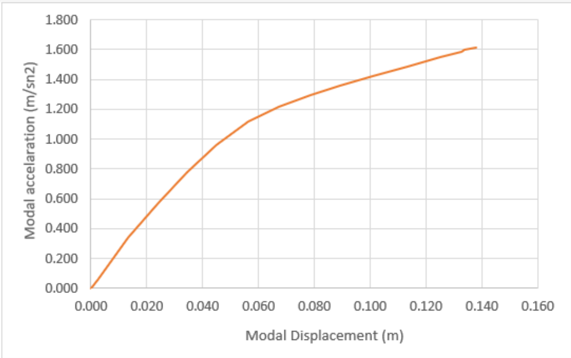
(b)

Figure 6. Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 6-story buildings

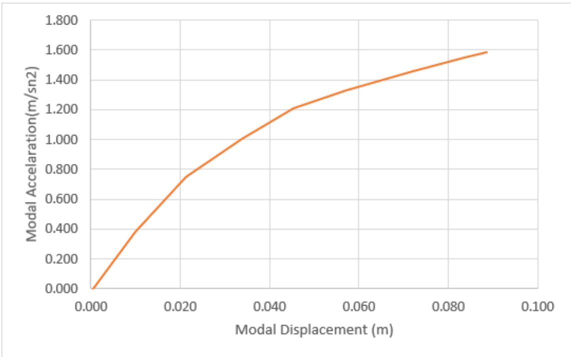
diagram" belonging to the primary (dominant) mode the coordinates of which are defined as "modal translocation – modal acceleration" shall be derived. The modal translocation volition belonging to the primary (dominant) mode shall be set taking the elastic behaviors spectrum and the modifications applied on this spectrum for different exceeding probabilities together with the mentioned diagram into consideration. In the final step, the translocation, plastic deformation (plastic rotation) and inner force volitions that corresponds to the modal translocation volition shall be calculated.

As shown in Figure 6, with the effect of horizontal displacements of the existing building located within the boundaries of Istanbul, where the fault lines are of great importance for the earthquake, the section damage of the sections with the result of the designation of hinges to the columns and beams according to the Pushover Analysis method regions are seen. After determination of damage regions of sections, the performance level of the building is controlled. It is seen from Figure 8 that the hinges through the X and Y directions of the structure after pushover analysis is under design earthquake (10% in 50-year hazard level). In the X-Direction of pushover analysis, no damage occurred in 182 (79.82%) of the 228 columns areas in total, 39 (17.10%) of them suffered minimal damage, and in 7 columns (3.08%) collapse occurred. In addition, there were no damage on 397 beams (68.92%) in 576 beam area, and minimum damage occurred on 179 beam (31.08%).

In the Y-Direction of pushover analysis, no damage occurred in 177 (77.63%) of 228 columns in total. 36 columns (15.79%) are in minimally damaged columns region. significant damage is occurred in 7 columns (3.07%), and in 8 columns (% 3.51) collapse is occurred. In addition, there were no damage on 343 beams (59.55%) and 576 beam girders, and 233 beams (40.45%) are minimum damage zone.

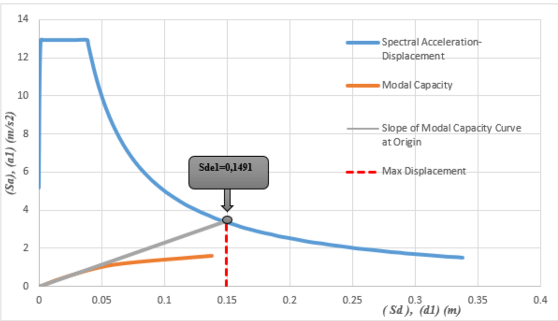


(a)

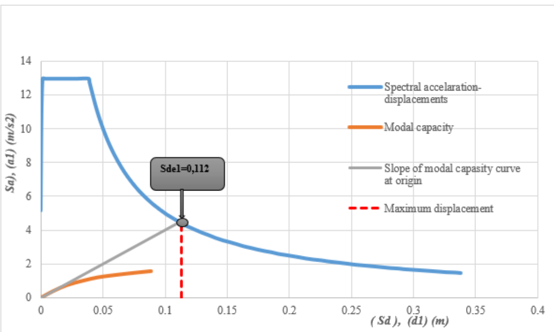


(b)

Figure 7. Modal Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 6-story buildings.



(a)



(b)

Figure 8. Spectral acceleration, spectral displacement, and modal capacity curves for X direction (a) and Y direction (b) by pushover analysis for 6-story buildings.

5. CONCLUSIONS

The purpose of performance-based earthquake engineering is to design and construct safe structures with seismic demands. The performance-based design method in earthquake engineering is used to determine the expected performance level under the effect of earthquake. For this purpose, different calculation methods have been developed and it is accepted by the scientific circles that the most reliable calculation methods are nonlinear calculation methods.

In this study, an existing building in Istanbul province is considered. One of the most important reasons for the selection of the existing structure in Istanbul is that the dangerous fault lines are present within the boundaries of this province. In line with this information, linear and nonlinear analysis of an existing structure according to TBEC-2018 was carried out. ESLM was used in linear analysis and pushover method was used nonlinear analysis method.

The determination of the building performance levels in the earthquake analysis is an important factor for the safe usability of the structures at the time of the earthquake. In the linear analysis, in accordance with the equivalent earthquake load, for the existing cast-in-place reinforced concrete (except for high buildings) BHC = 6 > 2, EDC = 1 using the class values of the normal performance target control for DD-2 Controlled Damage (CD). The design approach is determined as DBDA.

As a result of the analyzes, irregularity calculations were made according to the displacements in the X and Y direction:

In the analysis made according to X direction of the structure, only torsional irregularity is seen on the ground floor. In the Y direction, torsional irregularity is found on normal floors.

No discontinuity was found in the structure in any way to prevent the operation of the structural system element.

A-3 irregularity is observed in Y direction due to the fact that the right appearance of the structure does not provide a complete symmetry.

As a result of calculations in the X and Ydirections B-1 irregularity in the structure and as a result of the displacements as a result of the rigidity irregularity was not found.

As a result of the pushover analysis, it is seen that in the evaluation of X Direction line in ZC local floor class design earthquake, the Controlled Damage Performance Level which is the target performance for the buildings is provided. However, the same cannot be achieved for Y Direction. Because, since more than 35% (40.45%) of the beams were passed to the Advanced Damage Area according to the regulations of the existing building, the Controlled Damage Performance Level could not be achieved. In this respect, the building was evaluated according to the Level of Performance of Collapse Prevention.

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Stochastic Finite Element Analysis of Composite Structures with Partially Restrained Connections under Earthquake Loads

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Abstract: The stochastic analysis of composite structures with partially restrained (PR) connections under seismic loads present some interesting and challenging issues to practicing engineers. This paper proposes an efficient, robust, and accurate method for stochastic finite element analysis of concrete–steel composite structures allowing for PR connections. These are followed by suitable numerical example which indicates that employment of such a stochastic finite element analysis. The Kocaeli earthquake in 1999 is considered as a ground motion. The connections parameters and material properties are random variables. It is essential to properly consider the PR connections in the stochastic dynamic analysis and design of the steel-concrete composite structures since design forces change significantly. The assumption that the connections are rigid, which is routinely used in the application, is not proper. The effect of the variability connection stiffness on the composite structures responses is sufficiently important for consideration in structural safety.

Keywords: Stochastic finite element analysis; Composite Structure; PR connections, Stiffness matrix; Monte Carlo simulation**.**

1. INTRODUCTION

This paper presents the effect of the variability in connection stiffness and material (elastic module) properties on stochastic responses of a composite structure modeled with PR beam-to-column connections by using stochastic finite element method (SFEM) and Monte Carlo simulation (MCS) method. A computer program for PR connections composite 3-D frame systems was developed in FORTRAN language and incorporated into a general-purpose computer program [1, 10] for dynamic deterministic and stochastic analysis of medium and large-scale three-dimensional frames. Then, this program is combined to MCS method. Firstly, the stochastic dynamic analysis results acquired from all the random variables (elastic module and connection rigidities) are compared with each other separately, and secondly the efficiency and accuracy of the proposed are validated by comparison with results of MCS method. Finally, the results of stochastic finite element analyses of the composite structure with fully rigid joints have been compared with the results obtained from two type PR connections. Elastic module and connection rigidities are chosen as random properties. This means these values changes in the borders of a standard deviation. In spite of extensive research into deterministic analysis of PR connections composite construction [2-9], the stochastic dynamic analysis and design of this type of construction form cannot be fully utilized by engineers unless an efficient, robust and accurate method of analysis is available.

2. Theory

The stiffness matrix formulation of composite 3-D frames with type PR connections and PSFEM dynamic analysis formulation are given references [10].

2.1. Stiffness matrix formulation of 3-D composite frame with PR connections

The stiffness matrix formulation of composite system with type PR connections is given according to References [10-13].

In the stochastic finite element method (SFEM), the deterministic finite element formulation is modified using the perturbation technique or the partial derivative method to incorporate uncertainty in the structural system. Since the basic variables are stochastic, every quantity computed during the deterministic analysis, being a function of the basic variables, is also stochastic. Therefore, the efficient way to arrive at the stochastic response may be to keep account of the stochastic variation of the quantities at every step of the deterministic analysis in terms of the stochastic variation of the basic variables.

There are two fundamental ways to solve the stochastic problem (i) analytical approach and (ii) numerical approach. Among analytical approaches, the perturbation method is widely used because of its simplicity. Numerical method such as Monte Carlo Simulation is generally applicable to all types' stochastic problems and is often used to verify the results obtained from analytical methods. A detailed discussion of these methods is presented below:

2.2. Perturbation based stochastic finite element method (PSFEM) formulation

The perturbation method is the most widely used technique for analyzing uncertain system. This method consists of expanding all the random variables of an uncertain system around their respective mean values via Taylor series and deriving analytical expression for the variation of desired response quantities such as natural frequencies and mode shapes of a structure due to small variation of those random variables. The basic idea behind the perturbation method is to express the stiffness and mass matrices and the responses in terms of Taylor series expansion with respect to the parameters centered at the mean values [1].

2.3. Monte Carlo Method (MCS)

The Monte Carlo Simulation generates a set of random values of X according to its probability distribution function. The set can be written as $X = \{x_1, x_2, \ldots, x_N\}$, where N is the number of simulation. For each values of X, the stiffness and mass matrices are computed. At the end of N simulations, we have a random set of displacement and stress values $\{\{q_\beta\}_1, \{q_\beta\}_2, \{q_\beta\}_3, \ldots, \{q_\beta\}_N\}$, $\{\{\sigma\}_1, \{\sigma\}_2, \{\sigma\}_3, \ldots, \{\sigma\}_N\}$ for X^i [14]. From this finite set of solutions, the expected values of displacement and stress are computed using the following formulas;

$$\mu_{\{q_\beta\}} = \frac{1}{N}\sum_{i=1}^{N}\{q_\beta\}_i$$

(1)

$$\mu_{\{\sigma\}} = \frac{1}{N}\sum_{i=1}^{N}\{\sigma\}_i \tag{2}$$

3. Numerical Example

Variations in material and geometrical properties and capacity of connections affect the uncertainty in structural response of steel-concrete composite systems. Therefore, the attentions should be focused to compare the stochastic dynamic responses of structural systems made of composite cross-sections for different random variables. An eight-story composite residential building was considered in this study. A typical floor plan is shown in Fig.1. The composite residential building has 8 stories and typical floor height is 3.0 m. Framing of the building is irregular in plan where there are 7 axes in X-direction and 3 axes in Y-direction. In order to study the stochastic dynamic response of the steel-concrete composite beam and column with type PR connections shown in Fig. 2, the elastic module of the material characteristic, E and the connection stiffness k are modelled as random variables. The composite columns (Fig. 2c) and composite beams (Fig. 2d) are consisting of a concrete part (Ec = 3.0x107 kPa, Gc = 1.25x107 kPa, $\rho_c = 2500 kg/m^3$, $\upsilon = 0.20$) stiffened by a steel one (Es = 2.1x108 kPa, Gs = 8.75x107 kPa, $\rho_s = 7850 kg/m^3$) (reference material). The column has a box shaped closed composite cross section as shown in Fig. 2c. The cross-section properties are computed as AE =AG = 0.0251 m2, Iy = 5.364x10-4 m4, Iz = 1.986x10-4 m4, It = 8.295x10-4 m4. The composite beams are formed as a box shaped composite cross section, with uniform Poisson's ratio $\upsilon = 0.20$ and damping ratios $\xi = 0.05$. The cross-section properties is computed as AE =AG = 0.147 m^2, Iy = 0.0014 m4, Iz = 0.0064 m4, It = 0.063 m4 (Fig. 2d). The shear deformation coefficient for two sections is selected as ay=az=0.

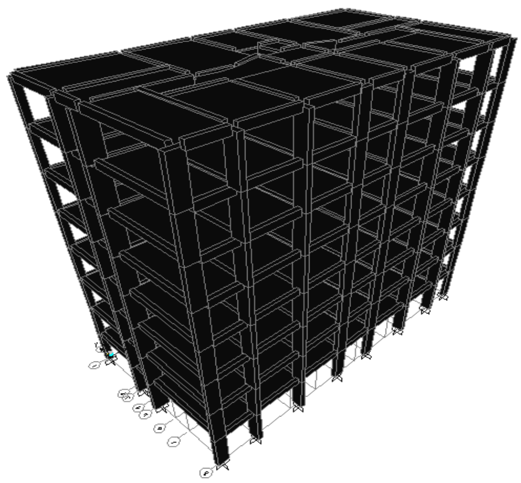


Figure 1.Three dimensional finite element model of building

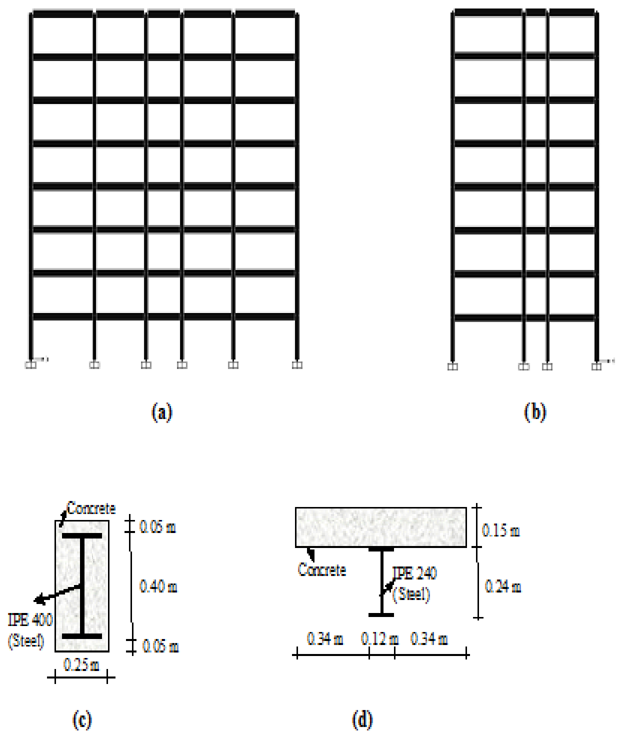


Figure 2. (a) Typical X-Z sectional view (b) Typical Y-Z sectional view (c) The dimensions of composite column cross-section. (d) The dimensions of composite beam cross-section.

The probability density function of random variables is assumed as normal (or Gaussian) distribution. The Mode Superposition Method considering the Wilson-ϴ algorithm is used for solving the dynamic equilibrium equations. 1999 Kocaeli earthquake is the largest natural disasters of the 20th century in Turkey after 1939 Erzincan earthquake. For the Kocaeli earthquake, the official death toll was more than 15 000, with approximately 44 000 people injured and thousands left homeless. For that reason, YPT330 component of Yarimca station records of 1999 Kocaeli Earthquake (Fig. 3) is utilized as ground motion [15]. This ground motion continued up 35.0 s is applied to the system in a horizontal direction. The dynamic responses of the composite structure are obtained for a time interval of 0.005 s. The composite residential building is modeled by equal lengths of 1840 stochastic finite elements. MCS method was simulated for

10000 simulations. Mean of maximum displacements and internal forces are determined according to PSFEM and MCS method for composite system. The stochastic finite element results obtained from Case A, Case B are compared with each other.

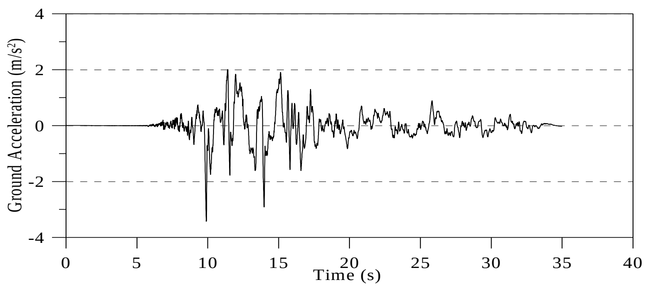


Figure 3. Acceleration time history of Kocaeli earthquake (YPT330), 1999 [15].

Case A. Elastic module of material properties is chosen as random variable for composite structure. The other variables are considered as deterministic for steel-concrete composite system. The elastic module of composite elements is chosen as reference material's (steel) elastic module. This random variable is assumed to follow a normal distribution with the coefficient of variation 0.15. The respective expectation and correlation function and coefficient of variation [1] for the elastic modulus E_ρ are assumed as follows:

$$E\,[E\rho]= 2.1x108\ kPa \quad \lambda=10 \tag{3}$$

$$\mu(E_\rho, E_\sigma) = \exp\left(-\frac{|x_\rho - x_\sigma|}{\lambda l}\right) \qquad \rho,\sigma = 1,2,\ldots,72 \tag{4}$$

$$\alpha = 0.15 \tag{5}$$

where x_ρ, l and λ are ordinates of the element midpoints (n random variable, $\rho,\sigma = 1,2,\ldots,n$), structural member length and decay factor.

Case B. The variation in connection rigidity, k, was modeled as random variables for steel-concrete composite structure. Different approaches in literature are used to model the connection. This study includes representing the connection by a rotational spring attached to each end of the connecting member. A non-dimensional fixity factor is used to characterize the connection behavior. Two types of semi-rigid beam-to-column connections were considered. The first of these connections is relatively stiff (k_1) and the second is rather weak (k_2). For comparison, the same structure with rigid joints (k_0) was analyzed.

The respective expectation and correlation function and coefficient of variation for the connection rigidity $k\rho$ are assumed as follows:

$$E1\,[k_1] = 1013796\ kNm/rad \qquad \lambda = 10 \tag{6}$$

$$E2\,[k_2] = 337932\ \ kNm/rad \qquad \lambda = 10 \tag{7}$$

$$\mu(E_\rho, E_\sigma) = \exp\left(-\frac{|x_\rho - x_\sigma|}{\lambda l}\right) \qquad \rho,\sigma = 1,2,\ldots,72 \tag{8}$$

$$\alpha = 0.15 \tag{9}$$

3.1. Responses of Composite structure with PR Connections

The composite structure is assumed to be subjected to the ground motion shown in Fig. 2. The stochastic finite element analysis of the composite structure with various connection types according to random connection stiffness, material and geometrical properties has been carried out. Characteristic results of the lateral displacements along story height of composite structure as well as bending moments, shear forces and axial forces at the base of the columns for the various types of connections are presented.

3.1.1. Horizontal Displacements

Firstly, the accuracy of PSFEM is tested with MCS method. For this aim, these two methods are compared with each other for the horizontal displacement values (Fig. 4) and other internal forces (shear forces, bending moment and axial forces) are given for only PSFEM.

The mean of maximum horizontal displacements along the right border of composite system according to PSFEM and MCS methods are presented in Fig. 4 for Cases A-B. The overall horizontal displacements values according to PSFEM of composite structure subjected to ground motion are greater than those of the MCS method for all random variables. However, the displacement values obtained from the perturbation method are generally close to those calculated using MCS method, as shown in Fig. 4. At the top of composite system where maximum horizontal displacement takes place, it can be observed that the maximum differences between PSFEM and MCS method are 5.5% and 4.4%, respectively for Cases A-B.

It can be seen from Fig. 4 that the structure with PR connections has a greater lateral displacement than the other one with fully rigid connection. These differences increase with decrease in the connection stiffness.

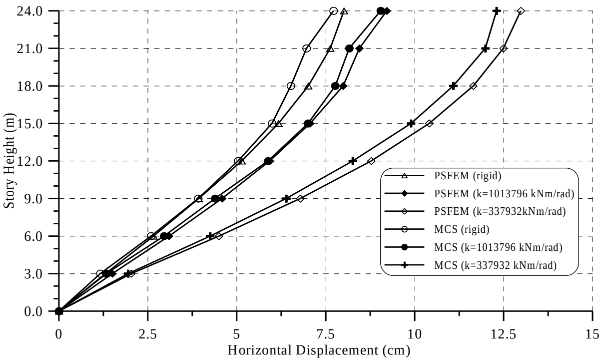
3.1.2. Internal Forces

The maximum shear forces at the top joint of columns in every floor for the residential composite structure are plotted in Fig. 5 for Cases A-B. It is seen from Fig. 5 that the composite structure with PR connections has a smaller shear forces (k1) when compared with the fully rigid connection (k0). These differences increase with decrease in the connection stiffness. The maximum differences between k0 and k2 according to k1 connections in the shear forces at the base of the composite structure with rigid joints and PR type of joints are 19.3% and 8.6% for Cases A-B, respectively. Changes in the bending moments are similar to shear forces for full rigidity and PR connections of Cases A-B (Fig. 6).

The last comparison for internal forces is about axial forces obtained from this structural system. Fig. 7 presents maximum axial forces of the columns in every floor for the structure. It can be seen from Fig. 7 that the composite structure with type PR connections has smaller axial forces when compared with the fully rigid connection.

These differences increase with decrease in the connection stiffness. Consequently, the difference in maximum axial force at the column of the composite system with rigid joints (k0) and PR type of joints (k_1) are 48.5 % and 11.9% for Case A, and 49.9% and 10.5% for Case B according to $k_{2,}$ respectively

It is obvious that there is a significant difference between the results obtained for the composite structures with rigid joints (k_0) and the structures with PR (k_1 and k_2) connections especially in the case of the weak connections types (k_2).



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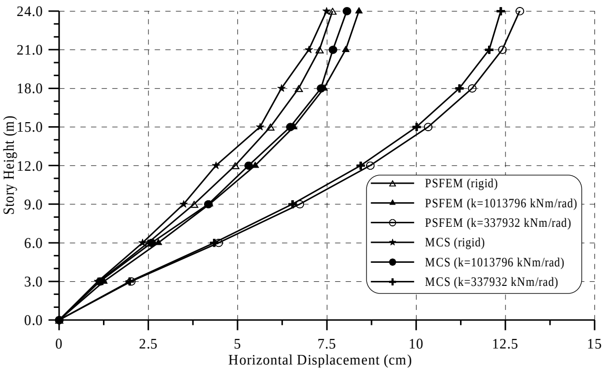
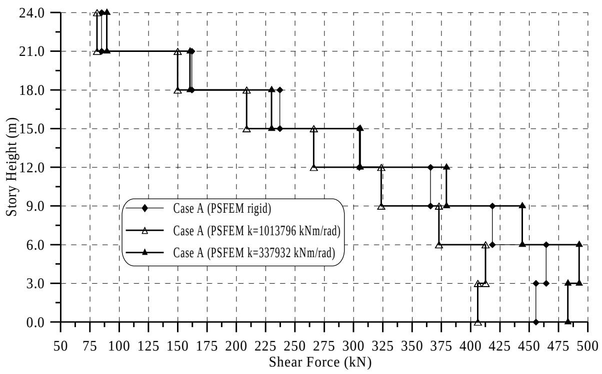


(b)

Figure 4. Mean of maximum horizontal displacement along the story height of the composite structure for Case A (a), Case B (b).



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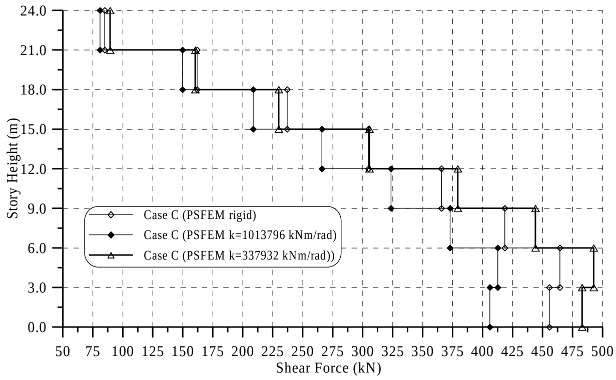


(b)

Figure 5. Mean of maximum shear forces at the top joint of the columns along the story height of the composite structure for Case A (a), Case B (b).

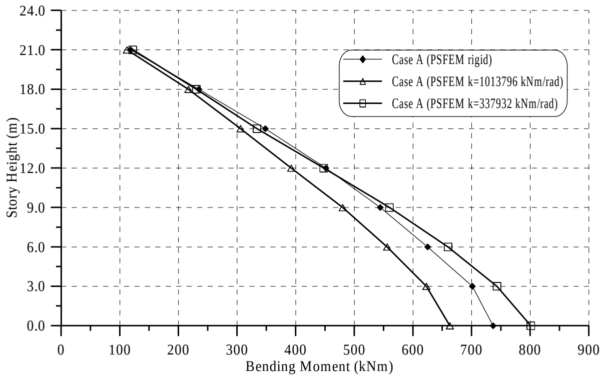
3.1.3. Limitation in the PSFEM

The horizontal displacements determined by PSFEM and by MCS method were compared for 0.05. The maximum horizontal displacements at top of the composite structure are given in Figs. 8(a) and (b). These figures show that the differences between maximum displacements obtained from PSFEM and MCS remains acceptable level if α is less than 0.20.

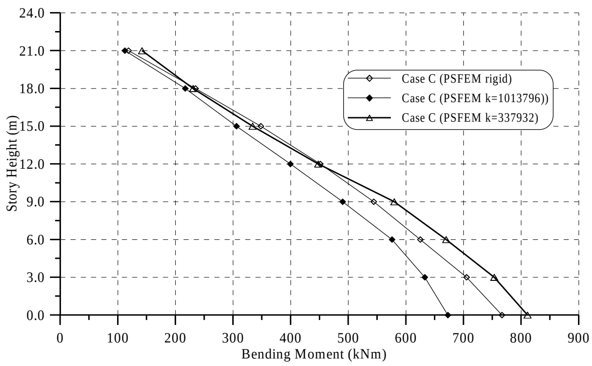
If it is mentioned the results obtained from this example; for the analysis of this composite structure (Fig. 1-2) presented its numerical properties, it needs about twenty seconds for perturbation based stochastic analysis, however, it needs about fourteen hours for MCS analysis with the PC which have Intel Pentium (R) 2.40 GHz CPU and 768 MB RAM.

The accuracy of the obtained results compared with those acquired from MCS method solution is remarkable. However, as the number of degrees of freedom of the structure and the number of uncertain parameters increase, the structural analyses based on MCS becomes very heavy from a computational point of view, and, in some cases, the computational effort makes them inapplicable.

It can be seen from these figures that the maximum values of dynamic responses from the three random variables are very similar to the result from the MCS method. For accurate dynamic responses, it is necessary that the analysis technique incorporate the effect of structural parameter randomness. This is of special importance for accurate stochastic dynamic analysis of composite systems, which exhibit wide dispersion in structural parameters.

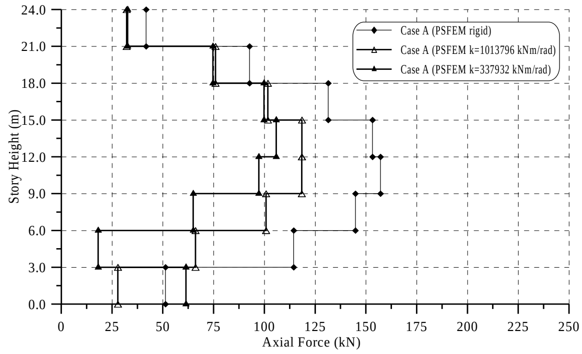


(a)

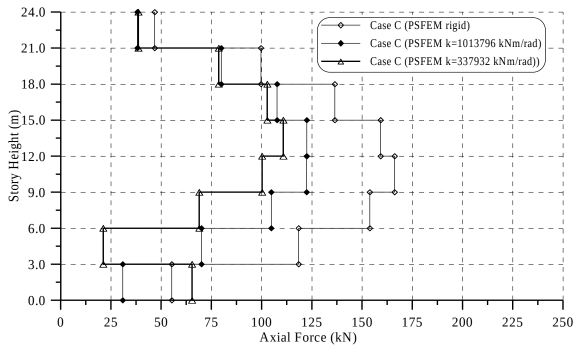


(b)

Figure 6. Mean of maximum bending moment at the top joint of the columns along the story height of the composite structure for Case A (a), , Case B (b).



(a)



(b)

Figure 7. Mean of maximum axial force at the top joint of the columns along the story height of the composite structure for Case A (a), Case B (b).

4. CONCLUSIONS

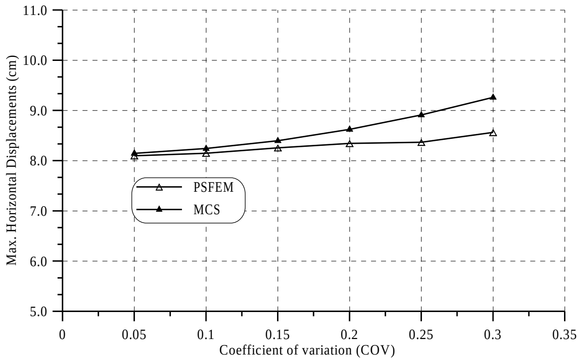
In this paper, the effect of variability in elastic module and initial connection stiffness on the stochastic responses of the composite structure with type PR connections subjected to ground motion is investigated using PSFEM and MCS method. The complex dynamic stiffness matrix for a prismatic composite beam with rotational springs at its ends was obtained in an explicit form. The stiffness matrix was based on the analytical solutions for stochastic finite element analysis of steel-concrete composite 3-D frame with type PR connections and some conclusions are drawn for the systems as follows:

On the bases of the above theoretical considerations and the results of the applied numerical analysis, it is evident that the PR connections greatly influence the dynamic behavior of steel-concrete composite structure. The connection flexibility may significantly alter the response of structure.

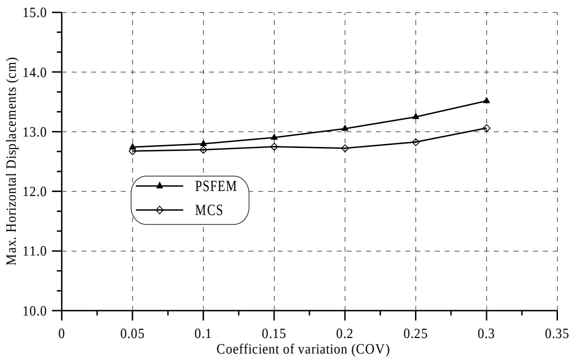
From the results of numerical example, it can be concluded that the stochastic structural responses of the composite structure with PR connections and the composite structure with conventional type of connections (rigid) are considerably different. It shows that the stochastic effect of PR connections on structural response is significant. Therefore, the stochastic variation of PR connections should be used in design and response analysis of real composite structures.

These numerical conclusions show that displacements and internal forces are close to all random variables (elastic module and connection stiffness) for PSFEM and MCS method. The stochastic finite element response values obtained for the random variable connection stiffness are generally higher than those of the other random variables for chosen composite structure.

The connections are vital structural components that are very often responsible for the behavior and safety of structures subjected to strong dynamic (seismic) loads. Overestimating the connection restraint can result in larger lateral displacements than what was predicted. Therefore, connection design and modeling have a great practical importance.



(a)



(b)

Figure 8. Comparison of the PSFEM with MCS method for composite structure with PR connection (a) k_1= 1013796 kNm/rad (b) k_2=337932 kNm/rad.

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Investigation of Wet-Dry Cycle Effect on Swelling Behavior of Stabilized Expansive Soils

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Abstract: Expansive soils pose a significant hazard to civil engineering structures due to its high swelling and shrinkage potential. Cyclic wetting–drying phenomena can cause progressive deformation of expansive soils, which may affect building foundations, drainage channels, and liner and cover systems in waste containment facilities. Therefore, it is essential to modify these soils by stabilization techniques at the geotechnical applications. In this study, expansive soil samples were stabilized by using pine tree sawdust waste material and the effects of wetting and drying cycles on swelling behavior of stabilized expansive soils were investigated under laboratory conditions. The results indicated that conducting consecutive drying and wetting causes a considerable reduction in the swelling potential of soil samples prepared with different qualities of pore water. Consequently, it was concluded that pine tree sawdust can be successfully used to improve the swelling behaviors against the wet-dry cycle.

Keywords: Expansive soil; pine tree sawdust; soil stabilization; swelling pressure, swelling potential

1. INTRODUCTION

Expansive soils are considered a worldwide problem because they cause extreme damage to civil engineering structures such as embankment dams, water conveyance structures, irrigation and drainage channels, retaining walls, small buildings, tunnels, highways, roads and pavements (Nelson and Miller, 1992). These soils are prone to large volume changes (swelling and shrinkage) which are directly related to changes in water content. Expansive soils are found in many parts of the world particularly in arid and semi-arid regions (Gourley et al.,1993) and can be identified assoils which form deep cracks in drier seasonss (Soltani and Estabragh, 2015).

Despite being classified as a problematic and destructive soil, expansive soils are widely utilized for civil engineering projects and landfilling of municipal, industrial and radioactive wastes (Komine and Ogata,1994; Pusch, 2001; Siddiqua et al., 2011). Many researchers have proposed that cyclic drying and wetting conducted on expansive soils will likely cause reduction in swelling potential and hence, can be adopted as a simple and economic method to reduce the damage caused by swelling in arid and semi-arid regions (Ahmadi et al., 2012; Soltani and Estabragh, 2015).

Seasonal changes cause moisture variations of soil water, increasing in the wet season and decreasing in the dry season. Evaporation of soil water during drying period causes volume shrinkage and desiccation cracks to occur. Swelling and shrinkage are the most important parameters in the investigation of volume change properties of expansive soils. Several factors affect shrinkage and cracking such as clay mineralogy, clay content, compaction conditions, temperature changes, cyclic wetting and drying, soil particle orientation, moisture and density conditions. Temperature changes have great impact on the behavior of compacted soil, mainly when landfill barrier materials are subjected to elevated temperatures due to bio-chemical reactions of the waste contained (Öncü and Bilsel, 2020).

Recently, there have been many researchers investigating the influence of cyclic wetting and drying on the swelling behavior of natural clayey soils. Some researchers found out that swelling potential decreases when expansive clayey soils are repeatedly subjected to swell then allowed to dry to their initial water content (Al-Homoud et al., 1995; Basma et al., 1996; Day, 1994; Dif and Bluemel, 1991; Osipov et al., 1987; Rao et al., 2000; Rao and Revanasiddappa, 2006). On the other hand, several researchers have examined the influence of cyclic wetting and drying on the swelling behavior of expansive soils modified by lime. It is observed that the swelling potential of expansive soils modified by lime increases when it is subjected to the cyclic wetting and drying (Guney et al., 2007; Rao et al., 2001; Yong and Ouhadi, 2007).

Several soil stabilization methods are available for stabilization of expansive clayey soils. These methods include the use of chemical additives, rewetting, soil replacement, compaction control, moisture control, surcharge loading, and thermal methods (Chen, 1988; Nelson and Miller, 1992; Yong and Ouhadi, 2007). Many investigators have studied natural, fabricated, and by-product materials and their use as additives for the stabilization of clayey soils.

All these methods may have the disadvantages of being ineffective and expensive. Therefore, new methods are still being researched to increase the strength properties and to reduce the swell potential of expansive soils (Akbulut et al., 2007; Al-Rawas et al., 2005; Asavasipit et al., 2001; Bell, 1996; Cetin et al., 2006; Guney et al., 2007; Kalkan and Akbulut, 2004; Kolias et al., 2005; Miller and Azad, 2000; Moavenian and Yasrobi, 2008; Prabakar et al., 2003; Puppala and Musenda, 2002; Senol et al., 2006; Sezer et al., 2006; Kalkan et al., 2019; Mohamedgread et al., 2019; Yarbaşı and Kalkan, 2019; Kalkan, 2020; Kalkan et al., 2020).

In this study, the pine tree sawdust as an additive waste material was used to stabilize the expansive soils in terms of swelling behavior. The expansive soil samples stabilized with pine tree sawdust were exposed to the wetting-drying cycles and then tested their swelling behaviors. Also, obtained results of stabilized expansive soil samples with pine tree sawdust were presented and discussed.

2. MATERİLA and METHODS

2.1. Expansive Soil

The clayey soil material was supplied from the clayey soil deposits of Oltu-Narman sedimentary basin, Erzurum, NE Turkey. The clayey soil samples were taken 0,75 m deep. According to the United Soil Classification System, clayey soil are inorganic clays of high plasticity (CH). These soils have high expansion potential as a result of over consolidation, high-very high plasticity and montmorillonite content (Kalkan, 2003; Kalkan and Bayraktutan, 2008). The grain-size distribution of clayey soil was given in Figure 1.

2.2. Pine Tree Sawdust

Wood cutting factories, generates a by-product known as sawdust. The pine tree sawdust waste material was obtained from the carpenters in the industrial zone of Oltu (Erzurum), NE Turkey. The pine tree sawdust is an organic waste resulting from the mechanical milling or processing of timber (wood) into various standard shapes and useable sizes. Consisting of soil-like particulate materials that are lighter than soil, sawdust inexpensive and environmentally safe (Rao et al., 2012; Oyedepo et al., 2014). The grain-size distribution of pine tree sawdust was illustrated in the Figure 1.

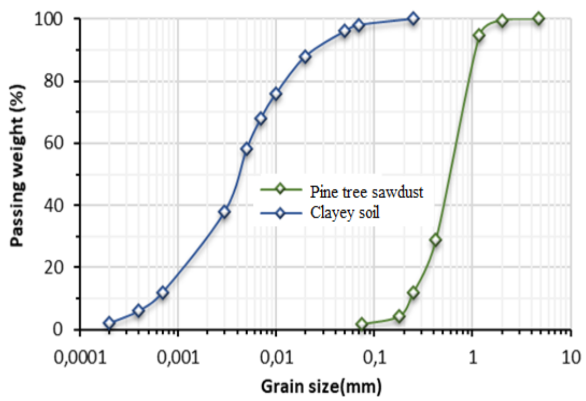

Figure 1. The grain-size distributions of clayey soil and pine tree sawdust

2.3. Preparation of the Samples

Under dry condition, expansive soil and pine tree sawdust materials were mixed to prepare mixtures of expansive soil-pine tree sawdust. The amounts of pine tree sawdust were selected to be 0,5%, 1% and 1,5 % of the total dry weight of the mixtures (Table 1). The dry mixtures were mixed with the required amount of water recognized to give the optimum water content. All mixing was done manually and proper care was taken to prepare homogeneous mixtures at each stage.

Table 1. Expansive soil and pine tree rates of samples

Samples	Expansive soil	Pine tree sawdust	Total
MIX0	100	-	100
MIX1	99,5	0,5	100
MIX2	99,0	1,0	100
MIX3	98,5	1,5	100

2.4. Swelling Pressure Test

This test was performed in the standard one-dimensional oedometer apparatus in accordance with ASTM D 4546. The sample was confined in the consolidation ring of 74 mm diameter and 20 mm high, and water was allowed to flow into the sample. The samples were submerged in water. The deflection of the dial gauge was set to zero. As a result, when the samples showed no further tendency to swell, the maximum surcharge load, P_{MS}, at that point was used for the calculation of the swelling pressure. The swelling pressure can be expressed as;

$$S_{PR} = P_{MS} / A \qquad (1)$$

where S_{PR} is the swelling pressure in kPa, P_{MS} is the maximum surcharge load on the sample in kN, and A is the area of sample in m^2.

2.5. Swelling Pressure Test

This test was carried out in a similar way as swelling pressure test. However, the sample was allowed to swell under a small load. The samples were loaded to a static pressure of 0.7 kPa. The samples were submerged in pure water. The samples were allowed to swell under the initial seating load. The dial gauge readings were recorded periodically until there were no further changes in swelling. The swelling percentage can be expressed as;

$$S_{PT} = H_{MS} / H_{OT} \qquad (2)$$

where S_{PT} is the swelling percentage, H_{MS} is the axial expansion in mm, and H_{OT} is the original thickness of the sample in mm.

2.6. Wet-Dry Cycle Test

In this test, all samples were submerged in tap water allowing the samples to swell fully over 48 h. Water was then drained and the consolidation cell with wetted samples were transferred from the odometer apparatus in to a test room. The wetted samples were then allowed to air-dry to their initial water contents at 22 °C. The drying of the samples required about five days. After all dried samples were carefully weighed, the dried natural and stabilized samples within the consolidation cells were again placed in the odometer apparatus and these samples were wetted by allowing them to swell for 48 h. A wet-dry cycle test was completed with the saturation of samples within the consolidation cell and their drying in an air-dry environment. In these tests, the natural and stabilized samples within consolidation cells were subjected to 5 cycles of alternate wetting and drying.

3. Results and Discussion

3.1 Change in Swelling Pressure and Swelling Potential with Pine Tree Sawdust

The change in the swelling pressure and swelling potential values with the pine tree sawdust additive material in the stabilized expansive soil samples were presented in Figures 2 and 3. The swelling pressure and swelling potential steadily decreased with increasing pine tree sawdust content in the expansive soil material. The decrease in the swelling pressure and swelling potential of the stabilized samples was attributed to the addition of low-plastic materials and the interaction between clay and pine tree sawdust particles (Kalkan, 2006). The maximum values for the swelling pressure and swelling potential were obtained from the samples of MIX2 and MIX3. It is known from literature that the engineering properties of expansive soils are controlled by the CEC, SSA and pH (Eades, 1962; Erzin and Erol, 2007). With the addition of pine tree sawdust, these parameters of expansive soil changed

(Churchman and Burke, 1991; Locat et al., 1984; Ohtsubo et al., 1983; Sridharan et al., 1988).

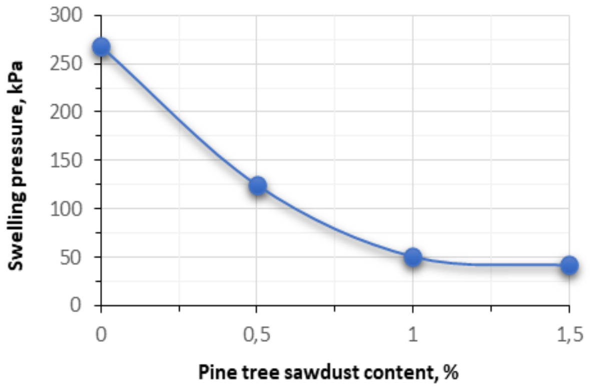

Figure 2. Change in swelling pressure with pine tree sawdust

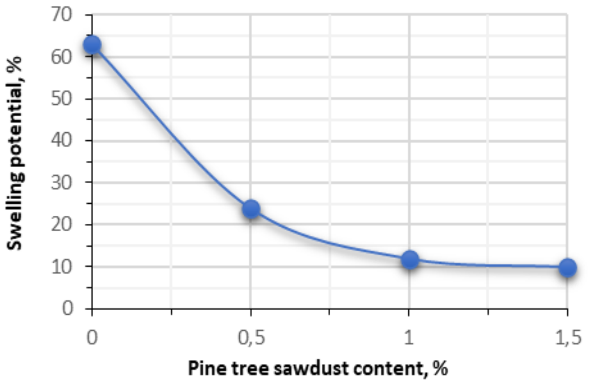

Figure 3. Change in swelling potential with pine tree sawdust

3.2 Change in Swelling Pressure and Swelling Potential with Wet-Dry Cycle

The change in the swelling pressure and swelling potential with wet-dry cycles in the stabilized expansive soil with pine tree sawdust were determined by the wet-dry cycle tests. The results of wet-dry cycle tests were presented in Figures 4 and 5. It was seen from the experimental test results that both the swelling pressure and swelling potential decrease with increasing wet-dry cycles. The most reduction in the swelling pressure and swelling potential values were observed within the first 3 wet-dry cycles.

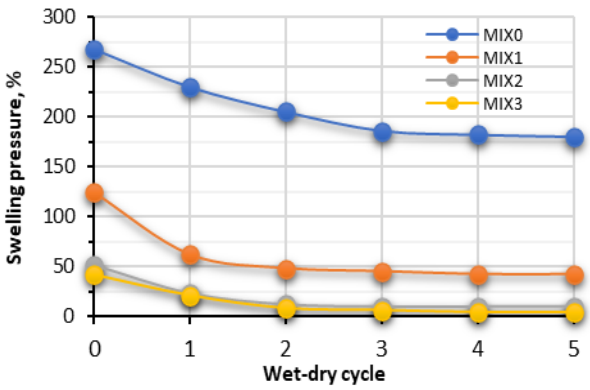

Figure 4. Change in swelling pressure with wet-dry cycle

It was observed that the pine tree sawdust additive material played an important role on the changing of swelling pressure and swelling potential values with the wet-dry cycles. It is an important finding that with the addition of pine tree sawdust, both the swelling pressure and swelling potential decrease with increasing wet-dry cycles. However, the swelling pressure and swelling potential increase when a clayey soil is modified by lime and was subjected to wet-dry cycles (Guney et al., 2007; Rao et al., 2000; Rao et al., 2001; Yong and Ouhadi, 2007).

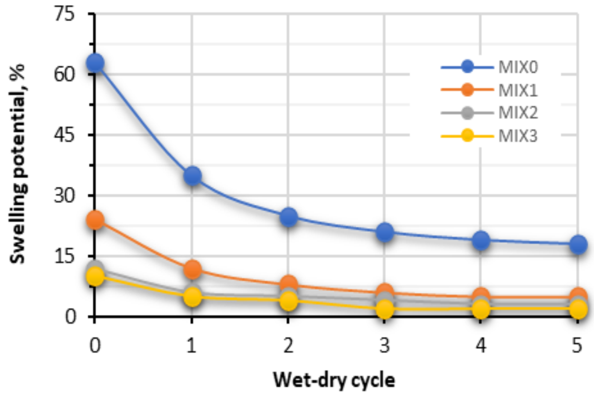

Figure 5. Change in swelling potential with wet-dry cycle

For the natural and modified clayey soil samples, both the swelling pressure and swelling potential decreased with increasing wet-dry cycles and they reached equilibrium at the end of fifth wet-dry cycle. The decrease in the swelling pressure and swelling potential is attributed to a gradual destruction of the matrix of the clay structure brought about by the cyclic swelling process (Kalkan, 2011).

4. CONCLUSIONS

In this study, the effect of wet-dry cycles on the swelling pressure and swelling potential behavior of expansive clayey soils stabilized with the pine tree sawdust waste material was investigated. The swelling pressure and swelling potential values steadily decreased with increasing pine tree sawdust content. Both the swelling pressure and swelling potential of natural expansive soil samples decreased with increases in the wet-drying cycles. It is an important finding that with the addition of pine tree sawdust waste material, both the swelling pressure and swelling potential decreased with increases in the wet-dry cycles compared with that of the natural expansive soil samples.

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Freeze-Thaw Resistance of Fine-Grained Soils Stabilized with Waste Material Mixtures

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Abstract: This paper evaluates the use of waste material mixtures including marble dust and scrap tire rubber the stabilization of fine-grained soils in order to remove the effects of freeze-thaw cycles. In this study, a fine-grained soil material was stabilized by using waste material mixtures. Natural and stabilized fine-grained soil samples were subjected to freeze-thaw cycles under different curing periods. After the freeze-thaw cycles, compressive strength tests were performed to investigate effects of waste material mixtures on the freeze-thaw resistance of fine-grained soil samples. The experimental results showed that the samples of fine-grained soil stabilized with waste material mixtures have high freeze-thaw durability as compared to unstabilized fine-grained soil samples. Consequently, we conclude that waste material mixtures including marble dust and scrap tire rubber, can be successfully used as an additive material to enhance the freeze-thaw durability of fine-grained soils for soil stabilization in the geotechnical applications.

Keywords: Fin-grained soil; soil stabilization; freeze-thaw; marble dust; scrap tire rubber

1. INTRODUCTION

Fine-grained soil (FGS) is generally classified as expansive soil. This means that some clays will tend to expand as they absorb water and will shrink as water is drawn away. These FGSs contain clay minerals that have the potential for swelling and shrinkage under changing moisture contents. Clay minerals could originate from the weathering of shale, slate, sandstone, and limestone. Another source is the diversification of volcanic ash deposited under marine conditions during geologic times, settled alone or mixed with shale or limestone (Grim 1968; Kalkan and Bayraktutan, 2008). Expansive soils are known to cause severe damage to structures resting on them. However, these soils are very important in geology, construction, and for environmental applications, due to their wide usage as impermeable and containment barriers in landfill areas and other environmentally related applications (Erguler and Ulusay, 2003; Harvey and Murray, 1997; Kayabali, 1997; Keith and Murray 1994; Murray, 2000; Sabtan, 2005). Safe and economic designs of foundations on FGSs and performance of compacted clayey soils for geotechnical purposes require the knowledge of swelling characteristics such as swelling pressure, swelling potential and swelling index. Cyclic drying and wetting phenomena can cause progressive deformation of expansive clayey soils, which may affect building foundations, drainage channels, buffers in radioactive waste disposals, etc. (Guney et al., 2007; Nowamooz and Masrouri, 2008; Rao et al., 2001).

Construction of buildings and other civil engineering structures on weak or soft soil is highly risky because such soil is susceptible to differential settlements due to its poor shear strength and high compressibility. Expansive clays are dominated by clay minerals with the potential for crystalline swelling, such as minerals of the smectite group. These are recognized as having very small particles, even among the clay minerals (Meunier, 2006; Fityus and Buzzi, 2009). Expansive soils are highly plastic that typically contain clay minerals such as montmorillonite that attract and absorb water. If clayey soils contain montmorillonite or a certain type illite, they will have significant swelling potential when wetted and tend to influence their engineering behavior (Shi et al. 2002; Sabtan 2005). Reaction of an expansive soil to changed environmental conditions is to swell or exert large pressures against non-yielding structures; but it may also exhibit a high degree of shrink-swell reversibility with changes in moisture content, leading to deformation and damage to buildings (Popescu 1979; Mohan et al. 1973; Mitchell 1993; Bell and Maud 1995; Du et al. 1999; Abdullah and Al-Abadi, 2010; Kalkan, 2012; Kalkan and Yarbaşı, 2013; Mohamedgread et al., 2019; Yarbaşı and Kalkan, 2020).

In cold regions, soils in areas with seasonal frost are exposed to at least one freeze-thaw cycle every year. In the freezing period, subsoil moisture moves towards the frozen layer because of a temperature gradient. Void spaces of soil gradually increase due to frost heave and moisture moves to the interstices of the soil and then freezes. In the thawing period, thawing of the frozen layer begins from the top and the bottom at the same time. The maximum soil moisture content appears above frozen layer and becomes temporarily-perched water. Additionally, the soil moisture content under frozen layer is more than it was during the prefrozen period (Zhang and Shijie, 2001; Yarbaşı et al., 2007).

Nowadays there are various alternatives available to increase the strength and stiffness of the weak soil and to improve the behavior of soil under various loading and environmental conditions (Parihar et al., 2015). Many earth structures such as liners of waste landfills, levees and dams are constructed of FGSs. Also, excavated FGSs might be reused as fill material in some earth structures. In these kinds of applications, there could be a tendency for characteristics of the soils (e.g. strength, volume change and mechanical characteristics) to vary over time. One possible solution to these problems is the use of randomly distributed tensile reinforcement elements in the soil. Such elements are available as polypropylene fibers (Yetimoğlu et al., 2005; Akbulut et al., 2007; Zaimoğlu, 2010; Zaimoğlu and Yetimoğlu, 2012; Yarbaşı and Kalkan, 2020).

Several stabilization methods are available for stabilizing expansive soils. These methods include stabilization with chemical additives, rewetting, soil replacement, compaction control, moisture control, surcharge loading, and thermal methods (Chen, 1988; Nelson and Miller, 1992). All these methods may have the disadvantages of being ineffective and expensive. Therefore, new methods are still being researched to increase the strength properties and to reduce the swell behaviors of expansive soils (Puppala and Musenda, 2002). Many investigators have experienced on natural, fabricated, and by-product materials to use them as stabilizers for the modification of clayey soils (Aitcin et al., 1984; Sandra and Jeffrey, 1992; Kayabali, 1997; Asavasipit et al., 2001; Prabakar et al., 2003; Kalkan and Akbulut, 2004; Cetin et al., 2006; Kalkan, 2006; Akbulut et al., 2007; Kalkan, 2020; Kalkan et al., 2020; Yarbaşı and Kalkan, 2020).

The waste of marble dust (MD) can cause environmental problem and economic loss if the waste is not used. Leaving the waste material to the environment directly can cause environmental problems. Therefore, many countries have still been working on how to reuse the waste material so that they give fewer hazards to the environment. The MD is settled by sedimentation and then dumped away which results in environmental pollution, in addition to forming dust in summer and threatening both agriculture and public health.

Therefore, utilization of the MD in various industrial sectors especially the construction, agriculture, glass and paper industries would help to protect the environment (Karasahin and Terzi, 2007). Wastes can be used to produce new products or can be used as admixtures so that natural sources are used more efficiently and the environment is protected from waste deposits. The MD is generally used as reinforcement material or raw material in various areas and applications (Davini, 2000; Arslan et al., 2005; Acchar et al., 2006; Akbulut and Gurer, 2007; Karasahin ve Terzi, 2007; Saboya et al., 2007; Hwang et al., 2008; Pereira et al., 2008; Aruntas et al., 2010; Celik and Sabah, 2008; Demirel, 2010).

The concept of soil reinforcement with natural fiber materials originated in ancient times. Randomly distributed fiber-reinforced soils have recently attracted increasing attention in geotechnical engineering (Yetimoglu and Salbas, 2003). The concept and principle of soil reinforcement was first developed by Vidal (1969). He demonstrated that the introduction of reinforcement elements in a soil mass increases the shear resistance of the medium. The primary purpose of reinforcing soil mass is to improve its stability, increase its bearing capacity, and reduce settlements and lateral deformation. There are several researches investigating the utilizability of scrap tire rubber as low-cost additive material for the soil stabilization (Hausmann, 1990; Prabakar and Sridhar, 2002; Yarbaşı et al., 2007; Akbulut et al., 2007; Zaimoğlu, 2010; Zaimoğlu and Yetimoğlu, 2012; Kalkan, 2013; Yarbaşı and Kalkan, 2020).

The main objectives of this research are to investigate the utilizability of waste material mixtures including MD and scrap tire rubber (STR) for stabilization of FGSs in geotechnical applications. Also, to test the strength performance of FGSs stabilized with waste material mixtures. To accomplish these objectives, natural FGS samples were stabilized by using different contents of waste material mixtures. The stabilized FGSs obtained by the compaction process were subjected the unconfined compression tests after exposing the freeze-thaw cycles and the results obtained were compared with that of natural FGSs.

2. EXPERIMENTAL MATERIALS

2.1. FGS

The FGS used in this experimental study was supplied from the clay deposits of Oltu Oligocene sedimentary basin, Erzurum, NE Turkey. This soil with green color and high plasticity is over-consolidated and it has clayey-rock characteristics in natural conditions. It is defined as a high plasticity soil (CH) according to the Unified Soil Classification System (Kalkan, 2003; Kalkan and Akbulut, 2004; Kalkan and Bayraktutan 2008). The chemical composition and engineering properties of the FGS are summarized Tables 1, and 2, respectively.

Table 1. Chemical properties of FGS and MD

Components	FGS	MD
SiO_2	46.83	0.36
Al_2O_3	15.35	0.28
Fe_2O_3	6.81	0.04
CaO	11.02	54.98
MgO	4.52	0.62
Na_2O_3	0.92	0.03
K_2O	1.23	0.07
TiO_2	0.81	-
SO_3	-	0.06
CaO_2	-	43.56

Table 2. Some properties of FGS and MD

Properties	FGS	MD
Density, g cm^{-3}	2.63	2.75
Sand (%)	2	-
Silty (%)	66	-
Clay (%)	32	-
Liquid limit (L_L, %)	73	-
Plastic limit (P_L, %)	35	-
Unit volume weight (g/cm^3)	37	-
Porosity (%)	-	0.2

2.2. MD

The waste MD was obtained in wet form as an industrial by-product directly from the deposits of marble factories, which forms during the sawing, shaping and polishing processes of marble in Afyon (Turkey) region. The wet marble sludge was dried up prior to the preparation of the samples. The dried material was sieved through a 0.25 mm sieve to remove the coarse particle (Kalkan and Yarbaşı, 2013). Ths generally used as reinforcement material or raw material in various areas and applications. The chemical composition and physical properties of MD are given in Tables 1 and 2, respectively.

2.3. STR

The STR fibers were supplied by local recapping truck tires producer in Erzurum, Northeast Turkey. When the tread on truck tires down, it is more economical to stave off the old tread and replace it than to purchase brand new tires. The tire is shaved off into 150 mm and smaller strips using a sharp rotating disc. These strips are then ground into scrap rubber (Pierce and Blackwell, 2003; Akbulut et al., 2007). The STR fibers used in this study has length of 1.18 mm. The engineering properties of the STR fiber were given in the Table 3.

3. EXPERIMENTAL PROSEDURE

3.1. Preparation of sample mixtures

The FGS used in this study was dried in an oven at approximately 65 °C and then ground before the preparation of mixtures. The required amounts of FGS, MD and STR were prepared and then blended together under dry conditions. As the waste STR fibers tended to lump together, considerable care and time were spent to get a homogeneous distribution of the waste STR fibers in the mixtures. The weights of the mixtures were determined according to the formula below;

$$W_{MIX} = W_{FGS} + W_{MD} + W_{STR} \quad (1)$$

where W_{MIX}, W_{FGS}, W_{MD}, W_{STR} are the total dry weights of sample mixtures, FGS, MD and STR, respectively. The component of the samples used in the experimental studies is summarized in Table 4.

Table 3. Some properties of FGS and MD

Parameters	Value
Density (mg/cm^3)	1.153-1.189
Elastic modulus (MPa)	1.97-22.96
Tensile strength (MPa)	28.1
Extent at failure (%)	44-55
Softening temperature (°C)	175

Table 4. Some properties of FGS and MD

No	Sample	Materials (%)			Total (%)
		CS	MD	STR	
1	MIX1	100.00	-	-	100
2	MIX2	94.5	5	0.5	100

3.2. Compaction tests

To prepare the samples for unconfined compression tests, Standard Proctor tests were performed in accordance with ASTM D 698. Each material was evaluated at six different water concentrations in three steps. To ensure uniform compaction, the required quantities of FGS-waste material mixtures were placed inside mold-collars assemblies and compressed alternately in three steps from the two ends until the samples reached the dimensions of the mold.

3.3. Unconfined compression tests

The unconfined compressive strength (UCS) values of natural FGS and stabilized FGS samples were determined from the unconfined compressive tests in accordance with ASTMD 2166. This test is widely used as a quick and economical method of obtaining the approximate compressive strength of the cohesive soils. In this study, three cylindrical samples were prepared and tested for each combination of mixtures. The unconfined compressive tests were performed at a deformation rate of 0.16 mm/min.

3.4. Freeze-thaw tests

The freeze-thaw tests were performed by a programmable freezing apparatus. The natural and stabilized FGS samples were subjected to freeze-thaw tests in accordance with ASTM C 666. All samples were placed in the freezing apparatus and conditioned at −18 °C for 2.30 h. During the freezing process, the cylindrical samples were insulated by 50 mm polystyrene to obtain one-dimensional freezing. After the freezing was completed, the samples were transferred from the freezing apparatus into a test room at +20 °C for 2.30 h. This freeze-thaw cycle was repeated 20 times and then these samples were subjected to the unconfined compression tests.

4. RESULTS AND DISCUSSION

4.1. Effect of mixtures on the UCS

The effect of waste material mixtures on the UCS values of the FGS was obtained by performing the unconfined compression tests under laboratory condition. The natural and waste material mixture-stabilized FGS samples were cured for 1, 7 and 28 days and then all samples were subjected to the tests at the end of curing periods. The test results showed that the waste material mixtures improved the UCS values of FGS samples. The increase in the UCS value of stabilized FGS with waste material mixtures was obtained at each different combination (Figure 1). However, the maximum values of the UCS was observed at the stabilized samples with waste material mixtures including 5% MD and 0.5% STR wastes. Less and more from optimum content of MD and STR caused the decrease in the UCS values of stabilized FGS.

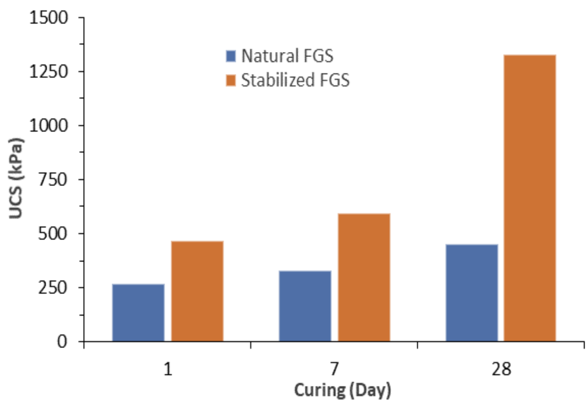

Figure 1. The effects of waste material mixtures on the UCS

The curing time played an important role on the increase of UCS values for stabilized FGS samples when compared that of fresh stabilized FGS samples. It was seen that with the increase in the curing time, UCS values of stabilized FGS samples also increased and maximum UCS values obtained at the 28-day curing time (Fig. 1). The same results were obtained from some experimental studies carried out in the past (Ranjan et. al., 1996; Prabakar and Sridhar, 2002; Akbulut et al., 2007; Zaimoglu, 2010; Hejazi et al., 2012; Kalkan, 2013; Muntohar et al., 2013; Lv and Zhou, 2019; Benziane et al., 2019).

Both the MD and STR contents play an important role in the development of UCS of stabilized FGS. The results indicate that MD-STR mixtures have more effect on the UCS than that of the MD or the STR. The improve in the UCS values of stabilized FGS samples was attributed to the changes in the composition of material mass. The addition of waste material mixtures including MD and STR wastes caused the change in the composition of stabilized FGS. It was noted in literature that the addition of additive changed the composition, mineralogy and particle size distribution of clayey soil (Gillot, 1968; Ola, 1978; Kalkan and Akbulut, 2004).

Similarly, the STR fiber played an important role in the increase in the UCS values of stabilized FGS. The increase in the UCS might be due to the bridge effect of fiber which can efficiently impede the further development of failure planes and deformations of the soil (Maher and Ho, 1994; Tang et al., 2007; Zaimoğlu and Yetimoğlu, 2012).

4.2. Effect of mixtures on the freeze-thaw resistance

The effects of waste material mixtures on the freeze-thaw resistance of FGS samples were investigated under laboratory conditions. The results obtained from the experimental studies showed that the waste material mixtures including MD and STR wastes increased the UCS values of stabilized FGS. As compared to the unstabilized FGS samples before freeze-thaw cycles, the UCS value of the stabilized FGS sample contents of 5% MD and 0.5% STR wastes and at 28-day curing period was the maximum level. It can also be seen that the stabilized FGS with the MD and STR wastes exhibit more ductile behavior than the unstabilized FGS samples. Similar results were also obtained for granular soils modified with waste additives (Akbulut et al., 2007; Yarbasi et al., 2007; Zaimoğlu, 2010; Kalkan, 2013).

The UCS values of all samples were decreased after freeze-thaw cycles (Figure 2). It may be attributed to the fact that pore water freezes and forms ice lenses in the pore space between the soil particles; then these ice lenses expand in volume and push particles of the soil and act like springs, increasing gaps among soil particles (Tunç, 2002; Isik et al., 2020). However, the decrease in the UCS values of stabilized FGS samples were lower level than that of unstabilized FGS samples after freeze-thaw cycles (Figure 3). The more UCS values of stabilized samples brought the more freeze-thaw resistance against to the freeze-thaw cycles (Yarbaşı and Kalkan, 2019).

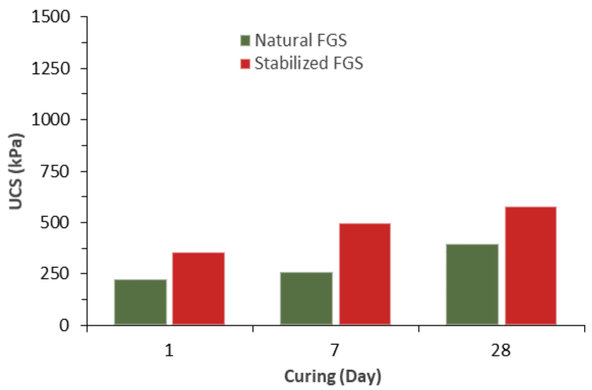

Figure 2. The effects of waste material mixtures on the freeze-thaw cycles

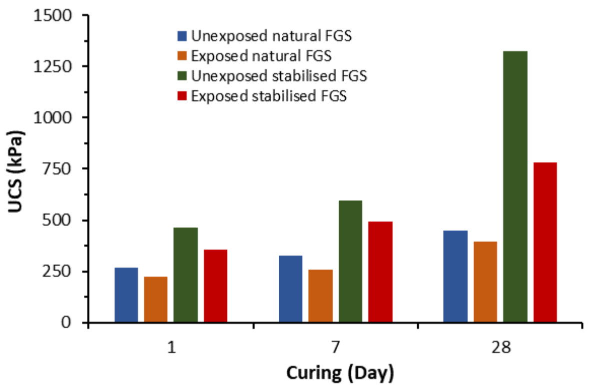

Figure 3. Effects of waste material mixtures on the freeze-thaw cycles

5. CONCLUSIONS

In this study, the effect of waste material mixtures including MD and STR wastes on the strength behavior of stabilized FGS samples was investigated and some conclusions were drawn. It was seen from experimental results that the addition of waste material mixtures increases in the UCS values of stabilized FGS samples. Meanwhile, the stabilized FGS samples were had more freeze-thaw resistance when compared with that of unstabilized FGS samples. It was concluded that waste material mixtures including MD and STR wastes can be used to improve the strength properties of FGS. In addition, this mixture can potentially reduce stabilization costs by utilizing wastes in a cost-effective manner.

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