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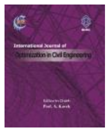

SELECTION OF SUITABLE RECORDS FOR NONLINEAR ANALYSIS USING GENETIC ALGORITHM (GA) AND PARTICLE SWARM OPTIMIZATION (PSO)

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ABSTRACT

This paper presents a suitable and quick way to choose earthquake records in non-linear dynamic analysis using optimization methods. In addition, these earthquake records are scaled. Therefore, structural responses of three different soil-frame models were examined, the change in maximum displacement of roof was analyzed and the damage index of whole structures was measured. The soil classification of project location was divided into 4 different types according to the velocity of shear waves in the Iranian Code for Seismic Design. As a result, 8 frame models were considered. The selection and scaling were carried out in 2 stages. In the first stage, the matching with design spectrum was carried out using genetic algorithm in order to achieve the mean of structural response. In the second stage, the matching with average of structural responses were carried out using PSO to achieve 1 or 3 accelerograms with related factors in order to be used in structural analysis.

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KEY WORDS: non-linear analysis; PSO; genetic algorithm; matching range; damage index.

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1. INTRODUCTION

Selecting suitable records for dynamic analysis of time history and scaling factor for this purpose has turned out to be one of the most important branches of the science of civil engineering and structural analysis. At present, we can predict which type of earthquake will possibly take place by analyzing previous events and their effects by existing methods. These are divided into groups in this discussion:

1- Methods which are based on earthquake parameter.

2- Methods which are based on earthquake parameters as well as the response of non-linear dynamic response.

The first method uses some important earthquake parameters like earthquake magnitude (M), the distance of source to site I and standard deviation in the attenuation relationship (ε) and it examines the previous earthquakes. These parameters can be considered on their own such as earthquake magnitude (M) or in pairs such as magnitude and the distance of the source to site (M, R) or all three together (M, R-ε). In some methods, the scale of earthquake intensity will be also added to these three parameters in order to achieve an analysis of possible damage and losses of structure. In addition, we can use record parameters like PGA, PGV, EPA, EPV, Ia, focal distance, and earthquake duration which has been discussed in [1-2]. Also, matching with the design spectrum is one of the most important factors for selection and scaling.

The second method recommends using not only important parameters of the earthquake but also premium software equipped with non-linear dynamic analysis in order to achieve structural responses, which makes results dispersion for getting closer to minimum level [3]. The second method appears to be the most complete one since structure performance would not be taken fully into consideration in the first group methods. It is possible to design a structure with an excess capacity but it would lead to a non-economic project or the dispersion of responses of structures.

The extent of destruction can be measured using the scale of damage as a qualitative parameter, which also takes into consideration previous records in the classification methods, but not as a quantitative one. However, in the second method, it is possible to determine all kinds of structural responses in the quantitative way using non-linear dynamic analysis [3].

In the second method, the records should be initially classified according to required parameters and the precision which is necessary and then the value of needed structural responses will be achieved for the considered site region using nonlinear analysis.

2. PROPOSED METHOD

In Iranian Code for Seismic Design [4] it is also emphasized to use the second method because this code clearly deems it is necessary to use 7 pairs of accelerograms in order to achieve the average of their dynamic response for the structure. Alternatively, one can use structure's maximum dynamic response with 3 pairs of accelerograms. Either of the methods needs to be considered with some essential characteristics of record and their matching with design spectrum in the specific limit. In the direction of this process, in this paper it is trying to present a method for selection and scaling of existing records for using in the non-linear dynamic analysis of structures by the optimization methods.

In the literature, it has been shown that structural optimization is recognized as a practical design tool which can be applied to that of the realistic buildings. In addition, the scope of the optimal design methods is extended to the numerical optimization procedures where it can be incorporated with pushover analysis to automate the pushover drift performance design of reinforced concrete structures [5].

One of the most important ways of selection and scaling is matching with the design spectrum. This matching has already been done by inflexible ways like the square root of the sum of squares and has some disadvantages but new methods have been recently created for selection and scaling using optimization methods. In this paper genetic algorithm is recognized as a practical method for solving this problem. In genetic algorithm the final solution has a particular algorithm which is searched by considering some conditions. This algorithm has no limitation for scaling factors and we can have the variety of values. The main advantage of using this method is its much increased efficiency in obtaining the solution.

Finally, the main aim is to obtain statistics that are the best fit and process them in order to get a procedure which produces results. This matter is carried out by various methods which can apply optimization methods like PSO.

3. GENETIC ALGORITHM (GA)

3.1 Basic of genetic algorithm

The genetic algorithm is a statistical method for optimization and search. The genetic algorithm is a small part of evolutionary calculations which itself is a part of artificial intelligence. A particular feature of this algorithm is the reason why it is not considered as a simple random searcher. In fact, the primary idea of this method has been derived from Darwin’s evolutionary theory and its application is formed on the basis of natural genetic. In genetic planning, the aim is to find an algorithm which can find the response of any problem. In this method, we should define desirability to understand which algorithm is better. The important property of a genetic algorithm is its resistance, which means there is a flexible balance between its efficiency and necessary properties for survival in various environments and conditions. Generally, the higher the resistance of an artificial system to environmental conditions, the less its re-design cost will be. It will also be able to delete certain inadequate answers. In fact, when the compatibility of a system increases, the system is able to work longer and more efficiently. In biological systems, the amount of flexibility, resistance and efficiency is enormously great. Compatibility, survival, self-regeneration, conductivity and reproduction are among other properties of natural and biological systems, so engineers plan to imitate them in artificial systems. However, where the resistant application is needed, nature will work better. The genetic algorithm has been used in various applications like function optimization, system's recognition and image processing [6-8].

4. PARTICLE SWARM OPTIMIZATION (PSO)

PSO algorithm is a social search algorithm which has been modeled by social behavior of bird flocks. At first, this algorithm has been applied in order to discover those models which are dominant on the simultaneous flight of birds and their sudden change in direction as well as the premium form changing in the group. In PSO, particles are flowing in the search atmosphere. The location change of particles in search atmosphere is affected by their own and neighbors' knowledge and experience. Therefore, another position of Swarm particles can affect the procedure of searching a particle. The result of modeling this social behavior is a search process in which the particles are inclined to the successful regions and learn from each other in SWARM .The principle of PSO is based on this maxim that in every minute, each bit modifies its own location in search atmosphere due to the best place which has been there up to now and it finds the best location which is available in its neighborhood.

One of the main features of PSO method is the emphasis on group intelligence and thinking. The great difference between these methods with other algorithms like genetic algorithm or simulated annealing is that society members are aware of the other members' situation or the best member of society and they'll consider such a result which has been achieved by them in their own decision-making. Likewise, the members keep their best own result in their mind during the algorithm execution and they always try to mix it with their decisions. That is why if a mistake happens and they make a bad decision, they can soon compensate it. So, the society members are able to search the limits around them without being worried about making the result tougher. If the new decisions are good, they'll be accepted and if they're bad, the algorithm can compensate the mistakes which happen. The effect-receiving of people from other members of society will be determined by some coefficients known as learning coefficients.

The analysis results show that if learning parameters would be increased, they always make algorithm converge later. Furthermore, the results indicate that the average population around 32 to 64 particles is appropriate for suitable performance of algorithm in functions' optimization.

The PSO algorithm has been widely used in optimization problems [9-11].

5. NON-LINEAR HYSTERESIS MODEL

To use nonlinear analysis, hysteretic model has a particular importance in the exact prediction of structural dynamic response. The selected model has to create a behavior similar to the real hysteretic behavior of various elements and some factors like stiffness degradation, strength deterioration and pinching behavior being affected by alternative loading can be taken into account. With respect to the number of elements of a building which would be modeled, the model of hysteresis loops should be simple as far as possible not to let the calculations size be much.

The three- parameter model is one of the models which are appropriate for many elements of building [12]. This three- line model can be identified with 3 parameters α, ß and γ in which introduce the characteristics of stiffness degradation, strength deterioration and the pinching behavior, respectively. The coefficient of α stiffness degradation states the decrease of the level of hysteresis loops' package. The parameter ß as the strength deterioration states the velocity of strength reduction and the coefficient of γ pinching behavior leads to decrease in the level of hysteresis loops as well as decrease in the energy loss. Having changed these three parameters, we can create many various hysteresis curves. These three parameters will be achieved while we do many alternative loading experiments over the reinforced concrete members.

It is needed to have more researches in the definition of three- line model parameter to make assessments which have more matching with reality. However, this model presents a wide range over inelastic behavior modeling of reinforced concrete elements.

According to observed damages in a reinforced concrete building, the index of calibrated damage for nominal strength deterioration, park et.al offered ß=0. 1. In this model, three indices of damage would be calculated because the nominal strength deterioration ß=0.1 has been considered in the index of Park-Ang damage, so it was realized that three parameter hysteresis loop of Takeda model is more appropriate for taking into account non-linear behavior of reinforced concrete in the analysis procedure [12, 13].

$A=2.0$, $\beta=0.1$, $\gamma=\infty$.

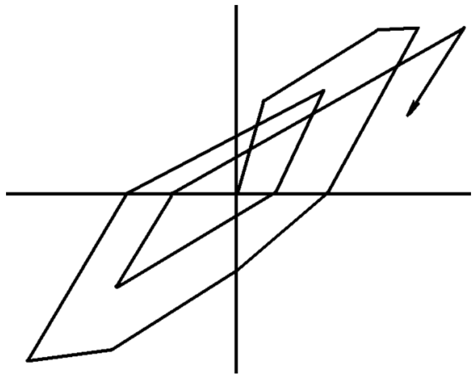

Figure 1. Takeda hysteresis model [12, 13]

6. DAMAGE INDEX

6.1 Review of damage index

In an assessment of damage in concrete structures, the way we properly face with earthquake and also the exact prediction of its effects on structures enjoys a great importance in civil engineering. In the recent decade, the development of study methods and laboratory facilities has proven that increasing the structure's stiffness as a one-parameter of design in conventional method is not able to provide the sufficient safety and subsequently decrease the structural damage. Nowadays, one of the parameters which are taken into consideration in modern attitude of researchers toward structural behavior is the concept of energy in structures. Hysteretic energy which is wasted after the occurrence of giving up in hysteresis loops has a leading effect on the structural damage of system and it is the most important part of an equation of energy which is given to structures. So, controlling this energy value can lead to control the structural behavior and its damage. The amount of hysteretic energy in a structure can be an index of the given damage level or its flexibility.

Park-Ang and Van (1984) presented a well-known relationship for damage index called Park-Ang as a method in the direction of structural design. In this method, the most important initial parameters are design, section cutting and index of earthquake intensity by which the structure ductility will be achieved [12]. Aki Yama in 1985 has presented a way for designing structures in his book which is based on the spectrum of input energy and the premium distribution of damage in the whole of the structure [14] .Shen and Ekbas (2000) introduced a method with respect to design according to the performance of a new damage index in which input energy, wasted energy and structural properties of a building such as the relative replacement of floors and ductility have been taken into account [15].

6.2 Park-Ang model for calculating damages in the reinforced concrete frame

In IDARC software, an index based on a model which is presented by Park et.al (1984) is being used in order to calculate the damage.

In this model, the damage size would be considered in the collective form in all elements, floors and the whole of the structure. The structural damage will be defined by Damage index (D) which indicates the damage arising from an earthquake in the linear combination of maximum ductility δ_m and absorbed Hysteresis's energy $\int dE$ .The damage index of Park-Ang for one structural element will be defined as the following [14]:

$$D = (\delta_m / \delta_u) + (ß / \delta_u P_y) \int dE \quad (1)$$

where δ_m is changing the form of maximum arising from earthquake load, δ_u is final tolerable changing in the form of an element, P_y is stiffness of giving up limit, ß is parameter of reducing stiffness and $\int dE$ is absorbed hysteresis energy by element during the history of response.

The index of floor damage and the whole of the structure will be calculated by the sum of the partial damage index (D_i) like the following relationship:

$$D = \Sigma \lambda_i D_i , \lambda_i = E_i / \Sigma E_i \quad (2)$$

where λ_i is energy weight coefficient, E_i is absorbed energy in every element.

Based on observed damage in 9-story reinforced concrete building, the damage index is calibrated [16].

Park et.al offered ß=0.1 in order to nominal strength deterioration. Having used this model, three damage indices will be calculated.

Damage index of member: beams and columns

Damage index of story: horizontal and vertical elements

Overall damage index of structure

In IDARC software, first beams and columns damage of each floor will be calculated as the elements of that floor for calculating the damage in all floors and then the damage of the floor and the whole of the frame will be calculated using the weight coefficients which are based on absorbed hysteresis energy in elements and floor elevation [14]. In table 1 the calibration of damage index for building is presented.

Table 1: calibration of overall damage index in building [13]

Building Status	**Damage index**	**Building appearance**	**Damage value**
Building collapse	> 1.0	- part or total collapse of structures	Collapse
Non-repair	0.4 – 1.0	- severe buckling and crushing concrete reinforcement in columns	Intensive
Be repaired	< 0.4	- creating large cracks concrete in weak members	Intermediate
		- create small cracks and minor crushing concrete in columns	Minor
		- create cracks scattered in structures	Weak

7. PROCEDURE OF PROPOSED METHOD

Initially, 2 different kinds of reinforced concrete frames with a medium ductility were modeled mathematically. Then, 4 different types of soil in the Iranian Code for Seismic Design [4] were considered for each of these frame models. As a result, 8 frame-soil models were taken into consideration in the mathematical modeling. There follow these models' specifications, loading and other needed parameters for analysis and design:

Model A- 5-story frame MRF in three spans, length of each span 4m

Model B- 15-story frame MRF in five spans, length of each span 4m

Each story was considered to be 3 meters high. The width of all frames spans was equal to 4 meters where the dead load was 600kg/m^2 and live load was 200kg/m^2. Concrete was assumed to have a stress of 25 N/mm^2 with an elasticity modulus of 25000 N/mm^2. The longitudinal bars and stirrups were assumed to be AIII. Base acceleration of design and behavior factor, were assumed to be 0.35g and 7, respectively.

In this method, the scaling of world earthquakes' records which were studied in this research work, are classified based on occurrences in the location of 4 different types of the soil. The information on earthquake records is derived from a data base acquired by Berkley University [17]. All of the used records have at least 15 km distance from an earthquake source which means these records are far from the earthquake source.

Table 2: Number of records for each soil type [17]

Soil Type	1	2	3	4
Vs (m/s)	> 750	375-750	175-375	<175
Number of Records	199	344	337	209

Stage 1: Design and Analysis of Models

ETABS software was used for all the 8 models. These are analyzed by linear static method while the ACI 318-05 code was used as the design code.

Table 3: Coefficient of Base Shear I

Type MRF	5				15			
Soil Type	1	2	3	4	1	2	3	4
C	0.14	0.17	0.19	0.19	0.08	0.1	0.13	0.17

After designing by ETABS, the sections' dimensions and the value of longitudinal reinforcement rebar and stirrup, were used as inputs for IDARC software. All IDARC data inputs are based on (N-mm); as a result, the displacement outputs are based on millimeters.

Stage 2: Selecting 20 records with corresponding scale factor

These 8 records and 20 scale factors should be chosen in such a way that the difference between the area measurements of under the graph, which is scaled with the spectrum of the Iranian Code for Seismic Design [4] in the pre-determined limit according to the alternation period and the record spectrum, should be the least. The boundary of matching would be interpreted according to structure behavior in Elastic and plastic stages. i.e. that of the minimum and maximum alternation period in which could occur in the structure, would be taken into consideration. The maximum of a structure’s alternation period is the alternation of collapse threshold which is different in various references [1]. In this study, the maximum alternation period has been regarded as 150% of the experimental alternation period according to the Iranian Code for Seismic Design [4] and it is considered to be $T= 0.07*H^{3/4}$ for reinforced concrete frames. The minimum of structure alternation period was supposed to be 50 % of the experimental alternation period according to the Iranian Code for Seismic Design [4]. Therefore, the limitation which is going to be surveyed for matching is 0.5- 1.5 times of period of structure alternation.

Table 4: Period of MRF and rang of matching

Type of MRF	Height (H) m	Period (T)s	(T_0)s	(T_n)s
5	15	0.5335	0.2	0.8
15	45	1.216	0.2	1.8

In order to achieve our goal, a program was created based on MATLAB software. In this program, initial core had the capability of calculating and drawing the acceleration spectrum (the acceleration spectrum would be calculated using Duhamel integration). The acceptable error was 5.5 % and the spectrum of selected record response was taken with 5 % reliance. MATLAB software consists of the materials for genetic algorithm; therefore it was not necessary to write a program for this algorithm. However, it was needed to define the fitness function so that it performs at its most optimum level. The fitness function is shown in the following expression:

$$\text{Fitness Function} = \Sigma\{\sqrt{(} \Sigma_{i=1-20} [S_i.\underset{T=T0}{\overset{Tn}{SA_i}}(T)]^2 / \Sigma_{i=1-20} [S_i]^2) - F_T(T) \}*\{\Delta t/2\} \qquad (6)$$

Acceptable scale factor range: 0.5-2.5

Population in each generation: 300

Number of generations studied: 200

Crossover ratio: 0.65

Mutation ratio: 0.025

F_T (T) is the value of the design spectrum at point T, S_i is the value of scaling factor of record *i*, SA_i (T) is the value of accelerogram spectrum at point T which is calculated by the computer program which was specifically written for this purpose by the authors. Δt is the distance between the spectrum reading . For each frame type, Duhamel integral was calculated 60000 times in the determined limit. All the results were checked 200 times within the allocated time frame to make sure that the most optimized algorithm was achieved. These are shown in the diagrams in the appendix and reference [18]. After obtaining the scale factors, records were entered in IDARC v6.1 program. Structural responses were obtained by Takeda Hysteresis Model using the historical non-linear dynamic analysis as shown in the tables in the Appendix and reference [18]. Then, the mean of the responses was calculated, i.e. the maximum displacement of roof and index of the whole damage of structures as shown in table (2) of appendix. Therefore, the main aim of this stage is to obtain the mean of structural responses which matches the design spectrum and this is needed for Stage 3.

Stage 3: Selecting 1 or 3 records with corresponding scale factor

At this stage, 1 or 3 records with the related scaling factors are selected in such a way that structural responses of non-linear dynamic analysis for 3 records with their factors, as well as the mean of achieved structural response, should have the least difference in comparison to Stage 2. Therefore, a new program was written with the MATLAB language using a bird's algorithm so that the cost function could be defined and optimized by using the following function:

$$\text{Cost function} = 0.9*(| U_r - U_r | / U_r) + 0.1*(| D_t - D_t | / D_t) \qquad (7)$$

where U_r is the mean maximum displacements of roof from table (2) of appendix, U_r is the mean maximum displacements of roof from 1 or 3 selected records, D_t is the mean of index of whole damage from table (2) of appendix, D_t is the mean of index of whole damage from 1 or 3 selected records. As it is observed here, the weight of displacement response is taken into consideration as 9 times of weight of damage response.

Other existing parameters in the algorithm are as the following:

c_1= 2 and c_2=2, the population size is 30 and the number of tries to reach the minimum is 100 times. Therefore, IDARC V6.1 software runs 3000 times for every frame type to optimize the function. The acceptable error is 10% in this bird's algorithm as it is less precise than genetic algorithm but its computational operation is faster than that of the genetic algorithm. The scaling factor is infinite in the higher extremity but its lower extremity is considered from 0.5. This stage's results are given in the appendix and reference [18].

Finally, the structural engineer is able to utilize results to apply the considered records with the related scaling factors to analyze and design software programs for structural engineering in order to be used in the non-linear dynamic analysis depending on the project type and their importance.

Overall in this paper, the programs were run for 356 hours and performed 120000 non-linear analyses for the models with 1 and 3 records.

Example

A building has a natural time period (T) of 1.2 seconds and it is built on soil type 2 that is assumed to have a range of 0.2 to 1.8 seconds for matching. This building is regarded to have the same time period as a 15-story structure as in this paper (see table 4). The number of generations studied is 200 and the population in each generation is 300 in genetic algorithm. A crossover ratio of 65.0% and a mutation probability of 2.5% were performed. The acceptable range of scale factors was from 0.5 to 2.5. The genetic algorithm selected 20 records and the corresponding scale factors as shown in Table 5 were regarded as representing the best match.

Table 5: 20 records selected and scaled for 15-story MRF on soil type 2

File Name	Scale Factor	Earthquake Name (station And Component)	Max Displacement of Roof	Overall damage
13	0.660619	CAPE MENDOCINO 04/25/92 1806, SHELTER COVE AIRPORT, 000 (CDMG STATION 89530)	5.5463	0
14	2.479517	CAPE MENDOCINO 04/25/92 1806, SHELTER COVE AIRPORT, 090 (CDMG STATION 89530)	22.128	0
17	0.515388	CHI-CHI 09/20/99, CHY022, E (CWB)	3.1497	0
23	1.827706	CHI-CHI 09/20/99, CHY050, N (CWB)	27.4757	0
24	0.501896	CHI-CHI 09/20/99, CHY050, W (CWB)	2.5911	0
91	2.499500	CHI-CHI 09/20/99, TCU018, N (CWB)	45.0423	0.013
114	1.404131	CHI-CHI 09/20/99, TCU087, W (CWB)	36.9554	0.009
118	2.496973	CHI-CHI 09/20/99, TCU094, W (CWB)	96.9066	0.026
121	0.913352	CHI-CHI 09/20/99, TCU100, N (CWB)	57.8551	0.014
125	0.507837	CHI-CHI 09/20/99, TCU105, N (CWB)	28.3812	0
149	0.501255	COYOTE LAKE 08/06/79 1705, SJB OVERPASS BENT 5 G.L., 337 (CDMG STATION 1492)	0.5151	0
178	2.496896	IMPERIAL VALLEY 10/15/79 2316, SUPERSTITION MTN CAMERA, 045 (USGS STATION 286)	11.4096	0
193	2.054910	KOCAELI 08/17/99, MECIDIYEKOY, 000 (ITU)	5.7179	0
194	2.497677	KOCAELI 08/17/99, MECIDIYEKOY, 090 (ITU)	26.5586	0
254	0.510700	LANDERS 6/28/92 1158, MISSION CREEK FAULT, 090 (USGS STATION 100)	0.4205	0
257	1.808231	N. PALM SPRINGS 07/08/86 09:20, RIVERSIDE AIRPORT,	0.7129	0

		180 (CDMG STATION 13123)		
258	1.532803	N. PALM SPRINGS 07/08/86 09:20, SAN JACINTO SOBOBA, 090 (CDMG STATION 12204)	4.4804	0
259	1.645596	N. PALM SPRINGS 07/08/86 09:20, SAN JACINTO SOBOBA, 000 (CDMG STATION 12204)	2.8352	0
269	0.506480	MORGAN HILL 04/24/84 04:24, GILROY GAVILAN COLL, 337 (CDMG STATION 47006)	0.6702	0
270	0.505290	MORGAN HILL 04/24/84 04:24, GILROY GAVILAN COLL, 067 (CDMG STATION 47006)	0.7218	0

Acceptable percentage error in the time period of range 0.2 to 1.8 seconds is 2.02%. This represents an excellent match which can be observed in Fig. 2 where the spectrum of individual scaled records is shown with narrow lines.

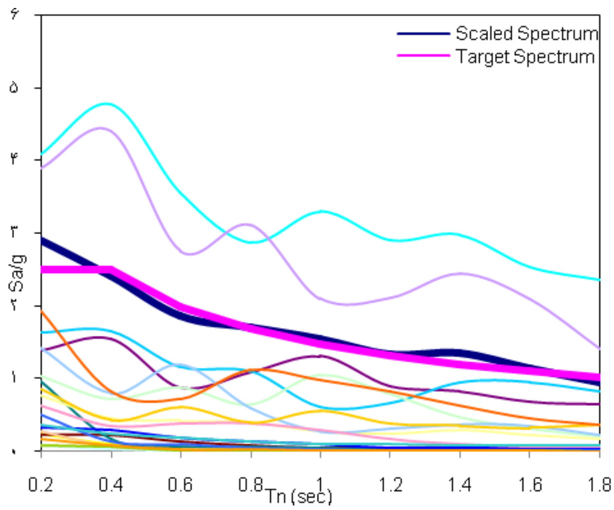

Figure 2: Target spectrum, scaled spectrum and 20 records used for 15-story on soil type 2

Two measure sensitivity scales can be used for non-linear time history dynamic analysis for this building; one with 1 record and the other with 3 records as shown in Tables 6 and 7 with corresponding scale factors.

Table 6 demonstrates a 15-story frame type in soil type 2 for 1 record with corresponding scale factor:

Table 6: Part of result: For 1 record and corresponding scale factor

Frame type	Soil type	File name	Scale factor	Earthquake Name (station And Component)	Error
15	2	236	1.25	LANDERS 6/28/92 1158, VILLA PARK - SERRANO AV, 000 (USC STATION 90090)	0.99

Table 7 demonstrates a 15-story frame type in soil type 2 for 3 records with corresponding scale factor:

Table 7: Part of result: For 3 records and corresponding scales factors

Frame type	Soil type	File name	Scale factor	Earthquake Name (station And Component)	Error
		124	1	CHI-CHI 09/20/99, TCU104, W (CWB)	
15	2	233	1	LANDERS 06/28/92 1158, BAKER FIRE STATION, 140 (CDMG STATION 32075)	0.01
		51	1	CHI-CHI 09/20/99, HWA022, W (CWB)	

The execution algorithm flowchart is shown in Fig. 3 to demonstrate how it works.

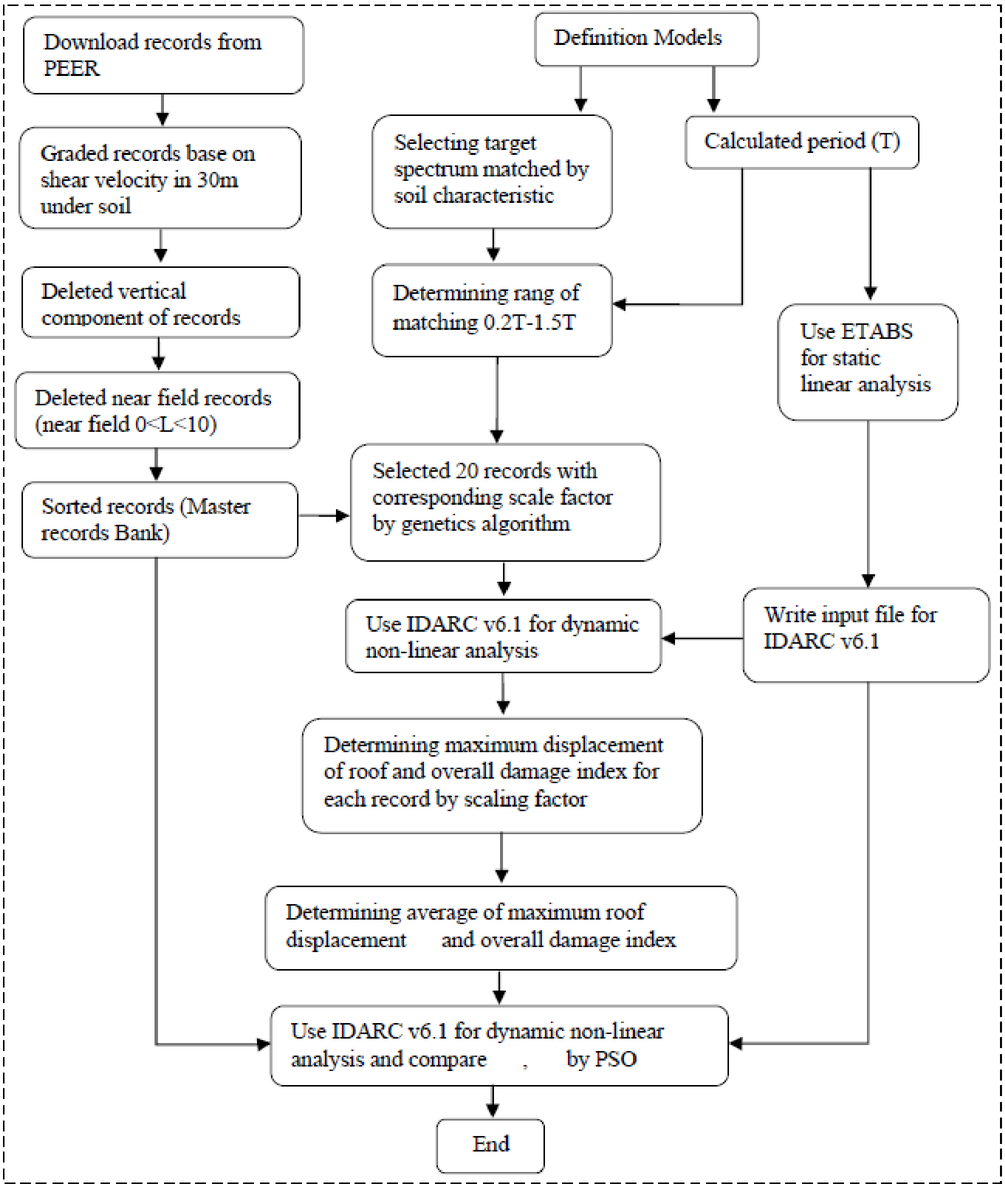

Figure 3. Procedure of research and program flowchart in MATLAB

8. CONCLUSION

The results of these methods utilized in this study are not conclusive. However, this paper suggests one method for analyzing non-linear dynamic for design structure. The structures must have regularity in their height and plan. These kinds of records with their suggested scale factor produce the mean overall damage and roof displacement for zone’s spectral response which the structural engineers can certainly use. It must be noted that such answers are not unique but they are very good approximate results for difficult questions. In other words, they are a good combination of records by scale factor. This conclusion has been arrived by performing non-linear dynamic analysis 3000 times for each model. In other words, 120000 non-linear dynamic analyses were conducted for the models with 1 and 3 records.

It can therefore be seen that to perform non-linear time history analysis one only require to know the time period of the structure and the type of soil of the location. By these parameters it is easy to select 1 or 3 records depending on the computational effort which is suitable. The result of the analysis is very close to the result of analysis of 20 records.

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APPENDIX

Table 1: 20 records selected and scaled for 5-story MRF on soil type 1

File Name	Scale Factor	Earthquake Name (station And Component)	Max Displacement of Roof	Overall damage
10	0.501299	CHI-CHI 09/20/99, HWA056, N (CWB)	1.465	0
23	0.500876	CHI-CHI 09/20/99, ILA052, W (CWB)	0.973	0
26	2.498427	CHI-CHI 09/20/99, ILA063, N (CWB)	9.1577	0
27	2.5	CHI-CHI 09/20/99, ILA063, W (CWB)	10.8641	0
54	0.510971	CHI-CHI 09/20/99, TAP036, W (CWB)	1.2368	0
61	1.081825	CHI-CHI 09/20/99, TAP065, W (CWB)	6.7964	0
73	0.617431	CHI-CHI 09/20/99, TAP079, W (CWB)	1.7889	0
82	0.976319	CHI-CHI 09/20/99, TAP060, W (CWB)	4.3473	0
88	0.654069	WHITTIER NARROWS 10/01/87 1442, MT WILSON , 090 (CDMG STATION 24399)	0.262	0
91	0.500189	SAN FERNANDO 02/09/71 14:00, LAKE HUGHES #9, 021 (USGS STATION 127)	2.1801	0

103	0.500231	MORGAN HILL 04/24/84 04:24, GILROY ARRAY #1, 320 (CDMG STATION 47379)	0.2986	0
104	0.50005	MORGAN HILL 04/24/84 04:24, GILROY ARRAY #1, 230 (CDMG STATION 47379)	0.4629	0
106	0.5016	PALM SPRINGS 07/08/86 0920, ANZA FIRE STATION, 225 (USGS STATION 5160)	0.1108	0
107	1.924798	PALM SPRINGS 07/08/86 0920, RED MOUNTAIN, 360 (USGS STATION 5224)	0.6113	0
109	0.500022	N. PALM SPRINGS 07/08/86 09:20, MURRIETA HOT SPR, 090 (CDMG STATION 13198)	0.1158	0
111	2.463169	N. PALM SPRINGS 07/08/86 09:20, SILENT VALL POPPET F, 000 (CDMG STATION 12206)	1.7437	0
135	0.501812	LOMA PRIETA 10/18/89 00:05, POINT BONITA, 207 (CDMG STATION 58043)	1.8201	0
144	0.627029	LANDERS 06/28/92 1158, TWENTYNINE PALMS, 090 (CDMG STATION 22161)	2.6838	0
148	2.499998	CHI-CHI 09/20/99, TTN016, W (CWB)	3.8196	0
162	2.499999	KOCAELI 08/17/99, GEBZE, 000 (ERD)	50.1722	0.024

The error percentage of scaled spectrum and target spectrum is equal to 1.82 which is acceptable.

Table 2: Average of maximum roof displacement and Average of overall damage

Frame type	Soil type	Average of maximum roof displacement	Average of overall damage
5	1	3.9376	0.0001
	2	8.0697	0.0015
	3	22.001	0.007
	4	10.032	0.0023
15	1	8.357	0.0001
	2	19.004	0.0031
	3	33.488	0.0087
	4	22.348	0.0035

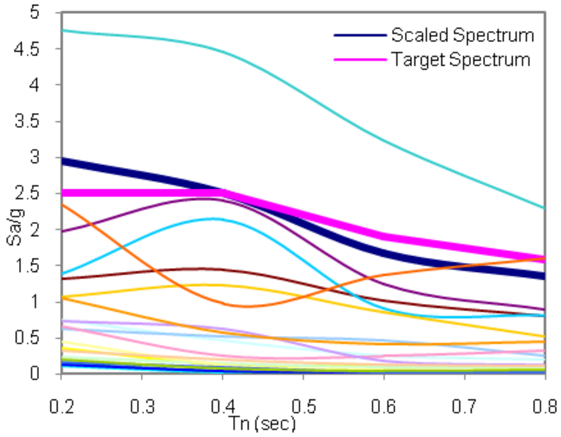

Diagram 1: Target spectrum ,scaled spectrum and 20 records used for 5-story on soil type 1

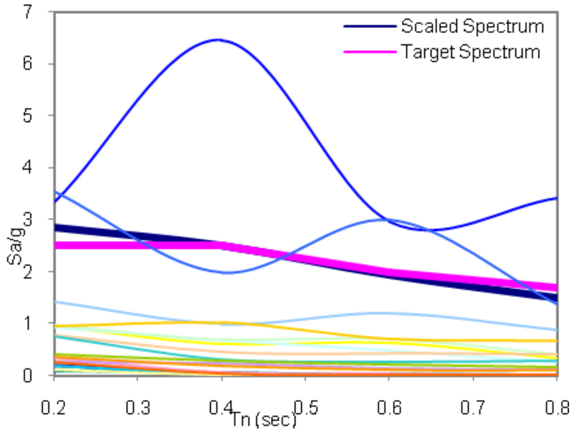

Diagram 2: Target spectrum ,scaled spectrum and 20 records used for 5-story on soil type 2

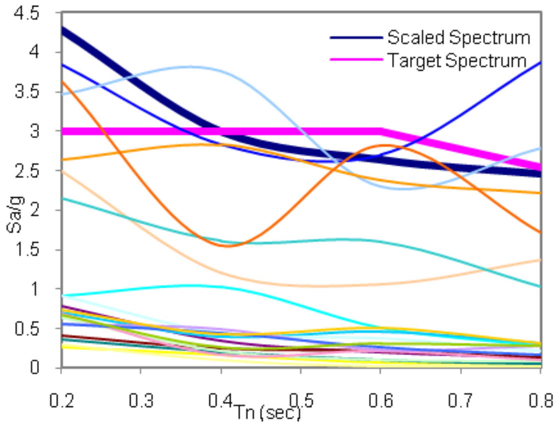

Diagram 3: Target spectrum ,scaled spectrum and 20 records used for 5-story on soil type 3

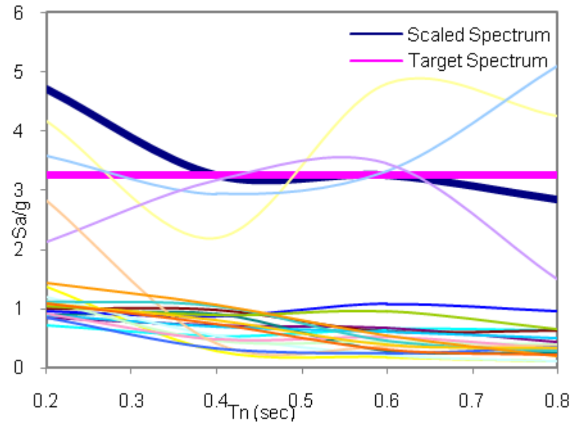

Diagram 4: Target spectrum ,scaled spectrum and 20 records used for 5-story on soil type 4