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Prediction of force reduction factor (R) of prefabricated industrial buildings using neural networks

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Abstract. The force (load) reduction factor, R, which is one of the most important parameters in earthquake load calculation, is independent of the dimensions of the structure but is defined on the basis of the load bearing system of the structure as defined in earthquake codes. Significant damages and failures were experienced on prefabricated reinforced concrete structures during the last three major earthquakes in Turkey (Adana 1998, Kocaeli 1999, Duzce 1999) and the experts are still discussing the main reasons of those failures. Most of them agreed that they resulted mainly from the earthquake force reduction factor, R that is incorrectly selected during design processes, in addition to all other detailing errors. Thus this wide spread damages caused by the earthquake to prefabricated structures aroused suspicion about the correctness of the R coefficient recommended in the current Turkish Earthquake Codes (TEC - 98). In this study, an attempt was made for an approximate determination of R coefficient for widely utilized prefabricated structure types (single-floor single-span) with variable dimensions. According to the selecting variable dimensions, 140 sample frames were computed using pushover analysis. The force reduction factor R was calculated by load-displacement curves obtained pushover analysis for each frame. Then, formulated artificial neural network method was trained by using 107 of the 140 sample frames. For the training various algorithms were used. The method was applied and used for the prediction of the R rest 33 frames with about 92% accuracy. The paper also aims at proposing the authorities to change the R coefficient values predicted in TEC - 98 for prefabricated concrete structures.

Keywords: neural network; force reduction factor; prefabricated industrial buildings.

1. Introduction

Designing structures to resist anticipated earthquake loads has always been a very challenging work for structural engineers. During the design procedure, engineers use building codes to

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calculate earthquake loads acting on the structure. Modern codes require the structures to withstand frequent earthquakes of minor intensity without damage and to resist moderate earthquakes without structural damage but possibly with some nonstructural damage, and major earthquakes without collapse but possibly with some structural as well as non-structural damage. For this purpose earthquake effects are converted into equivalent external loads to be applied on structures. The loads defined in seismic codes are a function of the seismicity of the region, the geometry of the structure, the importance of the structure, the soil conditions and the dynamic properties of the structure. The design earthquake force is calculated in accordance with all those parameters and decreased by a value of an R factor which is called as the Force (Load) Reduction Factor or Response Modification Factor. Many investigators such as Uang (1991), Miranda and Bertero (1994), Kappos (2001), Elneshai (2001), Mwafy and Elneshai (2002), and Maheri and Akbari (2003) have discussed the proper selection of the R factor so far.

During the past earthquakes in Turkey, it was observed that prefabricated constructions appeared to face serious damages even though they have many advantages over the monolithic constructions. The field observations showed that especially the seismic performance of single storey prefabricated structures used for industrial purposes was very poor. It was also observed that the prefabricated constructions with an inappropriate selection of R value from the code faced total collapse. Many researchers agreed that selecting a wrong R value had caused total or partial collapse of many prefabricated concrete buildings, like Arslan, Korkmaz and Gulay (2006), Korkmaz and Tankut (2005), Iverson and Hawkins (1994) and Tezcan (2005a, 2003b).

Artificial Neural Network (ANN) is one of artificial intelligence (AI) applications that have been implemented by engineers to perform specialized design tasks since 1970s. Engineers often deal with incomplete and noisy data which is one area where neural networks (NN) are most applicable. NNs are one of the practical AI tools to find out and generalize some problems from some examples and experienced data to produce meaningful solutions to difficult problems. This practical approach works even when the input data contains errors or is incomplete or fuzzy, which is often typical of some design process. These characteristics of ANNs make them a promising candidate for modeling some of the real engineering problems. The neural network approach is rapidly establishing a useful position in structural engineering. Some recent developments include structural analysis and structural damage assessment with neural network approaches like Kaltakcı and Dere (1997), Jewkins (1993) Kassim and Topping (1987) and Rafiq, Bugmann and Easterbrook (2001).

In this study, artificial neural networks (ANN) will be used to obtain an appropriate force reduction factor, R. The paper will endeavor to introduce neural network application of force reduction factor, R, for single bay prefabricated structures which experienced heavy damages or complete failures during the last earthquakes in Turkey.

2. Seismic design of building structures according to the codes

The format of all seismic codes is usually set for the building to be "strong" enough to resist a static lateral force, namely the base shear, V, which is computed as a fraction of the total weight (Eq. (1)), W. The fraction is known as the base shear coefficient C represented as (Eq. (2))

$$V = C \times W \tag{1}$$

The C coefficient accounts for the period of vibration of the building, T, the effective ground acceleration coefficient, A_o, the importance factor, I, the spectral coefficient, $S(T)$, and the force (load) reduction factor (or response modification factor), R.

$$C = \frac{A(T)}{R(T)} = \frac{A_o \times I \times S(T)}{R(T)} \tag{2}$$

The seismic load reduction factor R varies from 1.5 to 8 and is tabulated in the Turkish Earthquake Code-1998 [TEC-98, hereafter](1998). The maximum value of R depends on the assumed ductility level (high or normal or low) of the system and varies between 3 and 8. As given in Table 1, for prefabricated concrete structures, the value of R ranges from 3 to 6 according to the ductility level of the system.

2.1 Introduction to R

The concept of force reduction factor is based on the premise that a well-detailed seismic framing system could sustain large inelastic deformations without collapse (ductile behavior) and develop lateral strength in excess of the structures design strength (reserve strength). Thus the earthquake loads are reduced by dividing them into "R" factors. This R factor represents the ratio of the force that would develop under the specified ground motion if the framing system were to behave entirely elastic under the specified design forces at the strength level. In addition to that, through the use of R factor, the designer makes a significant assumption, by estimating the non-linear response of a structure through utilizing linear analysis.

The R factor was introduced for the first time in 1959 at the "Recommended Lateral Force Requirement" prepared by the Structural Engineers Association of California (SEAOC), (http:www. seaoc.org). It was named as K factor at that time. The same K factors were adopted at the seismic provisions of 1961 UBC (Uniform Building Code) from the above mentioned requirements Then, the structural response modification factor, R was first introduced in ATC 3-06 (1978). The R factors are intended to reflect reductions in design force values that were justified on the basis of risk assessment, economics and non-linear behavior. The aim was to develop R factors that could be used to reduce the expected ground motions presented in the form of elastic response spectra in order to lower design levels by introducing modern structural dynamics into the design process.

Table 1 R factors for prefabricated concrete structures in TEC-98

Building structural system	Systems nominal ductilty level	Systems high ductilty level
Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	6
Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases	------	5
Buildings in which seismic loads are fully resisted by prefabricated solid structural walls	------	4
Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	5

Table 2 Codes and force reduction factors

Code	Shortcut	Name
TEC-98	R	Force Reduction Factor
Eurocode 8	q	Behaviour Factor
UBC, FEMA-273/356	R	Response Modification Factor
NBCC, 1995	Sp	The Structure Performance Factor
NZS 1992	1/Ds	Ductility Factor
IAEE	R	Force Reduction Factor

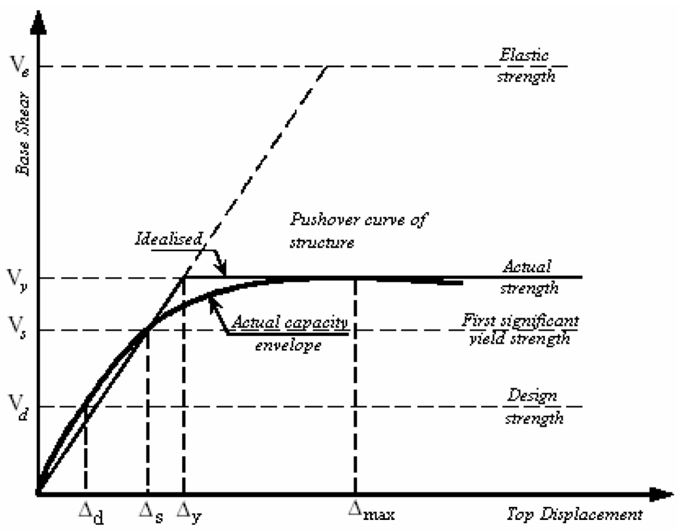


Fig. 1 Load displacement curve of a frame (Elneshai 2001)

R is expressed in the form of the behaviour factor q in Eurocode 8 (EC8, 1994), as the response modification factor R in the US codes and guidelines (Uniform Building Code "UBC", 1997, NEHRP Provisions "FEMA 273" 1997 and "FEMA 356" 2000), as the force modification factor R, in the National Building Code of Canada (NBCC, 1995), as the structure displacement ductility factor μ, and as the structure performance factor S_p, in the New Zealand Loading Standart (NZS 1992) and as ductility factor $1/D_s$ in the Japanese Building Standard Law (IAEE 1992) and finally the structural behaviour factor or force reduction factor R in the TEC-98 (Table 2).

The force reduction factor, R, can be obtained from the force-displacement relationship of the structure that can be determined either experimentally or analytically. Since the experimental evaluation of the R factor for a broad range of structures under a realistic set of excitations is extremely costly, the only alternative left is inelastic pushover analysis of the investigated systems. (Nonlinear static analysis involves the incremental loading of the mathematical model of the frame using a predetermined force profile. The pushover curves that relate to the base shear force and the roof displacement are obtained by these analyses.

The R factor in all seismic codes serves to reduce the elastic base shear (V_e) to design base shear level (V_d) as shown in Fig. 1. The main function of this parameter is to reduce the elastic seismic force to a realistic force level to be used in design work.

$$R = \frac{V_e}{V_d} \tag{3}$$

Many investigators have discussed this parameter defined simply in Eq. (3). for different types of structures, however, so far, there has not been many investigations on R value of Precast Concrete Structures.

2.2 Prefabricated construction

With the boosting of industrialisation in Turkey that affected the construction sector, and in early 1960's "prefabricated construction" became the prominent sub-sector of the construction industry. The fundamental reasons lying beneath the preferentiality of the prefabricated construction can be enumerated as;

a) Shortness of the construction period
b) Confidentiality of the quality-check of the member fabrication in the factory environment
c) Increased rate of production and economy compared to the conventional construction techniques.

Contrary to the superiorities that the prefabricated technology provides, the failures and damages have been observed in the structures erected using this technique during intensively destructive earthquakes that hit the country especially in the last 10 years, such as the 1992 Erzincan (M_w 6.8), 1996 Adana-Ceyhan (M_w 6.3), 1999 Adapazarı-Izmit (M_w 7.4), 1999 Düzce (M_w 7.2) earthquakes. These damages brought up the need for the re-examination of the criteria recommended in the code, namely TEC-98.

Fig. 2 shows the view of one of the prefabricated building two months before and after the Kocaeli Earthquake. The style of construction is known as lambda type, i.e., reinforced concrete prefabricated production industrial structure possessing two 20-meter transverse spacing, 6-meter height with rigid moment transferring connections. The Fig. 3 is the picture of a factory of a prefabricated construction company situated in the earthquake region of Turkey (Adapazari).

Following the 1999 Marmara earthquake, numerous totally failed or many highly damaged prefabricated buildings were reported by Tankut (2000) and Toniolo (2002). An inventory illustrating the damage status of the buildings constructed in the region by the Turkish Prefabricated

Fig. 2 The state of the building before and after the Kocaeli Earthquake-1999 (Arslan 2006)

Fig. 3 Overall collapse of the prefabricated systems

Table 3 The distribution of damaged structures after the 1999 earthquake

	Members of TPCA	Non members of TPCA
Total number of precast buildings	481	129
Heavily damaged	17	53
Partially damaged	14	27
Non-damaged	450	49

Construction Association (TPCA) member firms is given in Table 3, (Atakoy 1999). Even though the number of damaged structures seems to be very low, the actual number is higher than the values given in Table 3. Tens of buildings constructed by the firms which are not supervised by the Prefabricated Construction Association failed during the earthquakes and were excluded from these records. Those structures were constructed with serious mistakes at both design and application stages.

3. The multi-layered perceptron neural network structure and training

In this study, a three-layered feed-forward NN is used and trained with the error back propagation for the determination of R factor. Fig. 4 shows a general structure of the NN.

The cost function utilized in back-propagation algorithm is (Eq. (4))

$$\varepsilon(n) = \frac{1}{2}\sum_{k=1}^{N_0} e_k^2(n) \tag{4}$$

where $\varepsilon(n)$ represents the instantaneous cost function at iteration n, $e_k(n)$ is the error from the output node k at iteration n and N_0 represents the number of output nodes (Karık 2003).

The error is defined for each output node as (Eq. (5))

$$e_k(n) = d_k(n) - y_k(n) \tag{5}$$

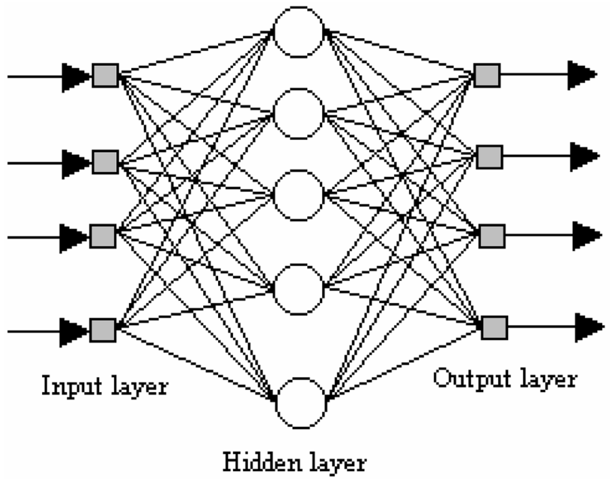


Fig. 4 The general structure of MLP ANNs

where $d_k(n)$ is the desired response of the output node k at iteration n and $y_k(n)$ is the output of the output node k at iteration n.

Haykin (1994) gives a summary of the back-propagation algorithms as follows.

A. Initialization

Set all the weights and threshold levels of the NN to small value that distributed random numbers.

B. Forward Computation

Training example is denoted by $[x(n), d(n)]$, $x[n]$: input vector, $d[n]$: desired response vector. The internal activity level $v_j^{(l)}(n)$ for neuron j in layer l is given by (Eq. (6))

$$v_j^{(l)}(n) = \sum_{i=0}^{p} w_{ji}^{(l)}(n) y_i^{(l-1)}(n) \tag{6}$$

where $y_i^{(l-1)}(n)$ is the signal from neuron i in the previous layer $l-1$ at iteration n and $w_{ji}^{(l)}(n)$ is the weight of neuron j in layer l that is connected to neuron i in layer $l-1$ at iteration n.

Logarithmic sigmoid function is used for threshold function. The output of neuron j in layer l is given as (Eq. (7))

$$y_j^{(l)}(n) = \frac{1}{1 + \exp(-v_j^{(l)}(n))} \tag{7}$$

For the output of neuron j in layer l, the error can be found as (Eq. (8))

$$e_j(n) = d_j(n) - o_j(n) \tag{8}$$

where $d_j(n)$ is the jth element of the desired response vector $d(n)$.

C. Backward Computation

Compute the local gradients (δ) of the NN by progressing backward layer by layer. For neuron j in the output layer L, the local gradient is given by (Eq. (9))

$$\delta_j^{(L)}(n) = e_j^{(L)}(n)o_j(n)[1 - o_j(n)] \tag{9}$$

For neuron j in a hidden layer l, the local gradient is given by (Eq. (10))

$$\delta_j^{(l)}(n) = y_j^{(l)}(n)[1 - y_j^{(l)}(n)]\sum_k \delta_k^{(l+1)}(n)w_{kj}^{(l+1)}(n) \tag{10}$$

The weight of the NN in layer can be adjusted according to the generalized delta rule

$$w_{ji}^{(l)}(n+1) = w_{ji}^{(l)}(n) + \alpha[w_{ji}^{(l)}(n) - w_{ji}^{(l)}(n-1)] + \eta\delta_j^{(l)}(n)y_j^{(l-1)}(n) \tag{11}$$

where η is the learning rate parameter and is the momentum constant (Eq. (11)).

Several different training algorithms were described for feedforward networks. All of these algorithms use the gradient of the performance function to determine how to adjust the weights to minimize performance. The gradient is determined using a technique called back-propagation, which involves performing computations backwards through the network. The back-propagation computation is derived using the chain rule of calculus.

There are many variations of the back-propagation algorithm. The simplest implementation of back-propagation learning updates the network weights and biases in the direction in which the performance function decreases most rapidly, the negative of the gradient.

In this study, 11 back-propagation methods in two main categories were used for prediction of force reduction factor (R) of prefabricated industrial buildings. The first category uses heuristic techniques which were developed from an analysis of the performance of the standard steepest descent algorithm. These heuristic techniques are variable learning rate back-propagation (GDA, GDM, GDX) and resilient back-propagation (RP). The second category of back-propagation methods uses standard numerical optimization techniques. These techniques are conjugate gradient (CGF, CGB, CGP, SCG), quasi-Newton (BFG, OSS) and Levenberg-Marquardt (LM) (Matlab 2006).

Calculation of Training and Test Errors:

The training and test errors given in tables were found according to Eq. (12).

$$\text{Error}(\%) = \left(\frac{\sum_{i=1}^{k} |t(i) - a(i)|}{m*n}\right) \times 100 \tag{12}$$

where $t(i)$ is desired outputs, $a(i)$ is outputs of neural network, k is the number of samples in training or test data, m is the number of segments in training or test data and n is the number of outputs of neural network for training and test procedures (Ozbay 2006).

4. Case study

As a sample solution, a single-floor, single-span prefabricated reinforced concrete frame system shown in Fig. 5 was selected, a mostly preferred structural geometry especially in industrial regions.

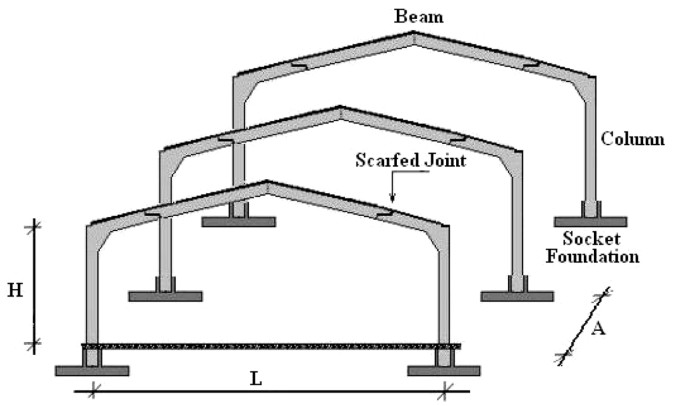


Fig. 5 Isometric (three dimensional) aspect of sample structure type

Table 4 Data range

	H(m) Height of building	L(m) Span length	A(m) Axe length
Minimum	5.00	14.00	5.00
Maximum	9.00	20.00	8.00
Increment	1.00	1	1.00
Number of Data	5	7	4
Total Data		5 × 7 × 4 = 140	

This type of framed system may be designed in various dimensions according to the construction area and utilization purposes. The most preferred dimensions are listed in Table 4, where H is the height of the building (from ground to the roof), L is the span length, A is the distance between two consecutive frames. Sample frame in which seismic loads is fully resisted by single-storey hinged frames with fixed-in bases. It can be seen easily from Table 1 that for this type of frames R factor is recommended as 5 in TEC-98.

4.1 Description of the sample frame and its details

The height, span length and axis interval of the members are given in Fig. 5. The concrete type used in the system was C30 (cylindrical compression strength of concrete is 30 MPa) and reinforcement S420 (yield strength of steel is 420 MPa). The critical sections of the sample frame are given in Fig. 6. This figure also shows the loads that act on the frame as a distributed horizontal dead load (D_L) and lateral pushover joint load (P_L). D_L is calculated by using the roof component's weights. The roof load analysis and the span distributed load (D_L) which is based on axes length is given in Table 5 and Table 6.

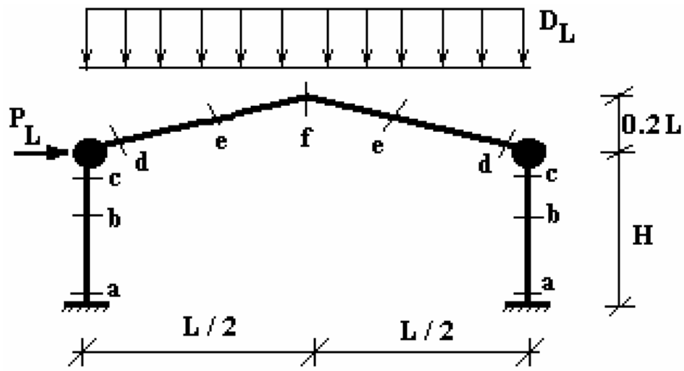


Fig. 6 Critical section of the sample frame and geometrical properties

Table 5 Roof load analysis (kN/m^2)

	Type	kN/m^2	(1.4 G + 1.6Q) kN/m^2	G + Q ± E kN/m^2
Dead Load (G)	Roof Covering	0,20	0,28	0,20
	Purlin	0,18	0,25	0,18
	Beam	0,52	0,73	0,52
	Snow	0,75	1,05	0,75
		1,65	2,31	1,65

Table 6 Span distributed load (kN/m), D_L

	Axe length, A (m)			
	5	6	7	8
	Distributed load on the span (kN/m), D_L			
(1.4 G + 1.6Q)	11,55	13,86	16,17	18,48
G + Q ± E	8,25	9,90	11,55	13,20

Table 7 Section and material properties

Section type / dimensions (mm)	Member type	Reinforcement	Ratio (ρ %)	Longitudinal steel yield strength (MPa)	Concrete compression strength (MPa)	Stirrup steel yield strength (MPa)
a / 350 × 550	Column	10ϕ20 + 2ϕ18	1.89	420	30	220
b / 350 × 550	Column	8ϕ20 + 2ϕ18	1.57	420	30	220
c,d / 350 × 1090	Column-Beam Connection	10ϕ20 + 6ϕ18	1.22	420	30	220
e / 350 × 550	Beam	4ϕ20 + 2ϕ20	0.69*	420	30	220
f / 350 × 550	Beam	4ϕ20 + 4ϕ20	0.69*	420	30	220

*: for only tension region's bars

The reinforcements of the cross-sections were determined at the end of the analyses performed according to the criteria of TEC-98 and TBC-500-2000 (2000). Elastic design earthquake load value was determined for 1st degree earthquake zone and alluvial soil type given in TEC-98. For each mentioned frame, the member types and reinforcement areas are shown one by one in the Table 7. The columns and beams were designed as 350 mm × 550 mm dimensions. The cross section of the columns have been reinforced with $10\phi20 + 2\phi18$ bars, stirrups $\phi8/100$ mm in the critical zones (section a). In the central part (section b), cross section of the columns have been reinforced with $8\phi20 + 2\phi18$ bars, stirrups $\phi8/150$ mm. The longitudinal reinforcement ratio was taken as ρ_l = 0.0189 in a section of the columns and ρ_b = 0.0157 in the *b* sections. According to the TEC-98, longitudinal column reinforcement shall not be less than 1% and more than 4%. The beam was reinforced with $4\phi20 + 2\phi20$ bars in e section and with $4\phi20 + 4\phi20$ in ϕ section. The stirrups were $\phi8/100$ mm in the end zones (section e) and $\phi8/150$ mm in the middle part (section f). The longitudinal reinforcement ratio was taken as ρ_l = 0.0069 in the all sections of the roof beam. In the TEC-98, ratio of tensile reinforcement along beam spans and at supports shall not be more than 2%.

Reinforcement details of the sample frame are given in Fig. 7. This figure also shows the section locations and precast member connection regions. Sample frame's sections and detailing are given in Table 8.

The long spanned beams cast in one length would present problems both in transport and erection phase; therefore, the *scarfed joints* as shown in Fig. 8 are usually provided. The rafters are usually broken at one of the contra flexure points at the span length, and then these two segments are lapped over each other and joined by two bolts to form these connections, conveniently. These bolts provide the shear resistance as well as transferring the axial thrust. The connection zone may be concreted later on to provide monolithic behavior.

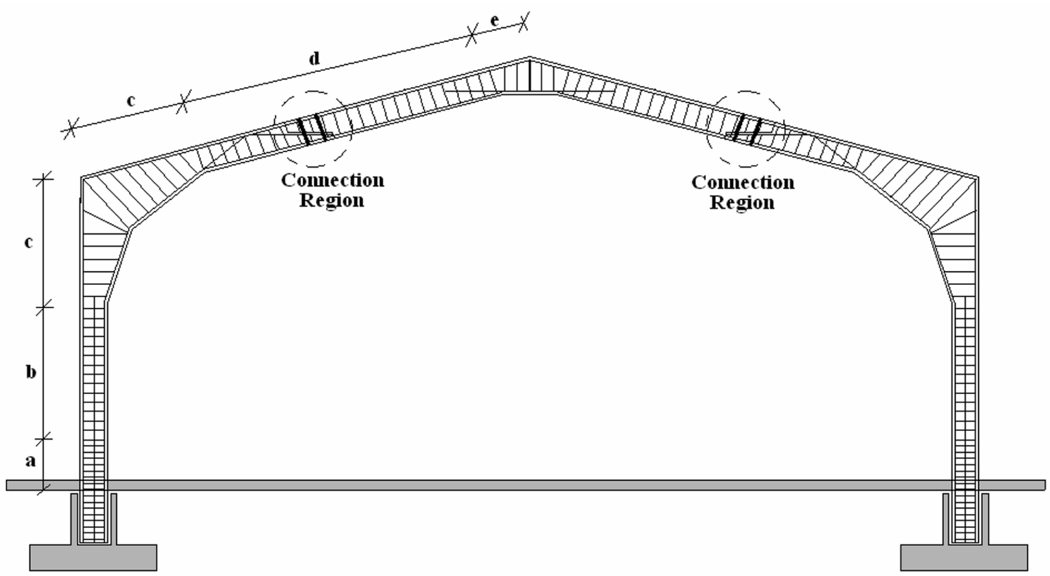
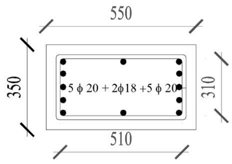
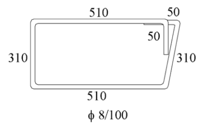
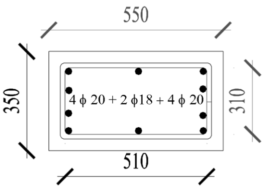
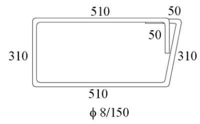
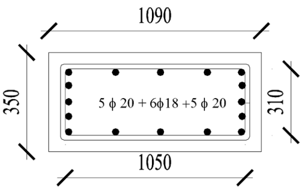
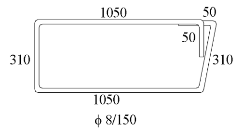
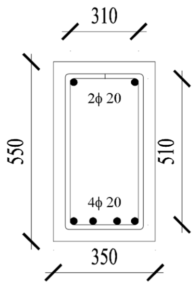
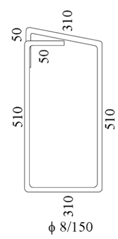
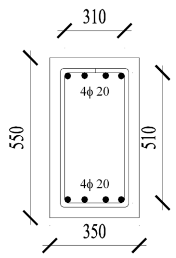
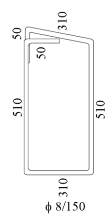


Fig. 7 Reinforcement details of the sample frame and its connection regions

Table 8 Sample Frame's section lengths and detailing

Section	Length (mm)	Section detail	Stirrups detail
a	1500		
b	$H-(a+b)$		
c	1900		
d	$\left(\frac{L}{2\times(\cos 11.30)}-e\right)$		
e	500		

4.2 Model analysis steps

In the analysis stage, the main purpose is to obtain the capacity curve (load-displacement curve) of each frame. In order to determine the capacity curves of these structures beyond the elastic limits, the pushover analysis is required. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. Analyses were preformed using SAP2000 (2000), which is general purpose

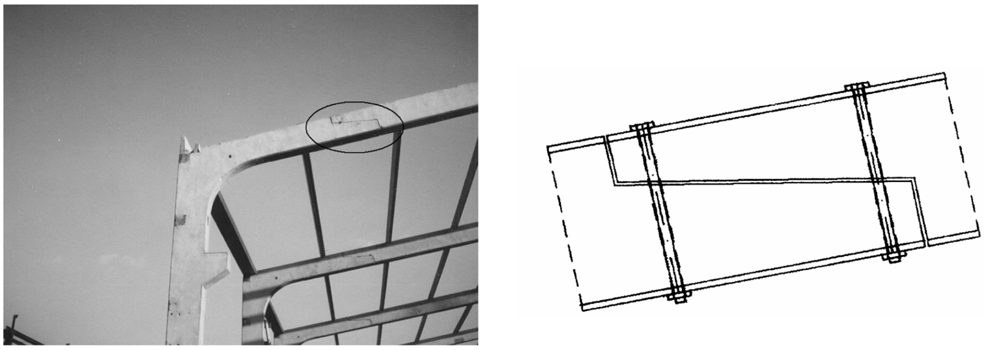


Fig. 8 Scarfed joint of a prefabricated industrial plant

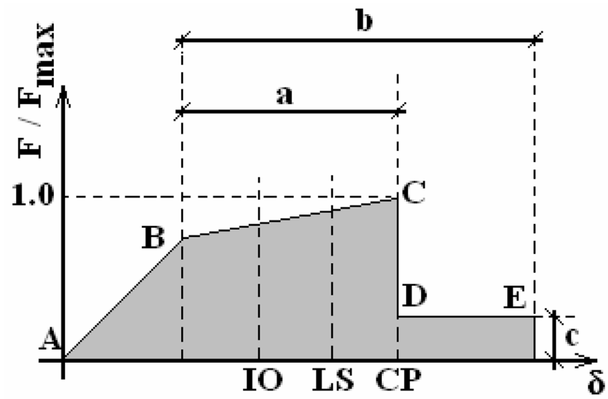


Fig. 9 Idealized force versus deformation curve for the deformation-controlled components from FEMA-356

structural analysis program for pushover analysis of a structure. The pushover analyses were performed as shown in Fig. 1 with gradually increased lateral load for each 140 samples of prefabricated reinforced concrete frame systems. In the study, two dimensional model of each structure is created in the program. The model properties are given in following.

Step-1) After the finite element computer model of the building has been prepared to perform non-linear pushover analysis, hinge properties of the components should be determined. Hinge properties contain the plastic rotation values that a component's end can carry and acceptable plastic rotation values for the performance level. In FEMA-356 and ATC-40 (1996) hinge properties are given according to the component type and failure mechanism. Fig. 9 shows an idealized force versus deformation curve for the deformation-controlled components from FEMA-356.

Step-2) Since the axial load level was very low (about 0.10~0.15 $A_c f_c$), failure mechanisms of all components were assumed as flexural failure, therefore the hinges types were selected as M3 according to the SAP2000. After the horizontal and vertical loads were calculated, internal forces necessary to determine the hinge parameters of the components were determined according to FEMA-356. In this study, five-percent of the component length from the edges was assumed as hinge locations. After all hinges were placed at their components' ends, the pushover cases were defined in the software. The program uses a series of sequential elastic analysis, superimposed to approximate a force displacement capacity diagram of the overall structure.

Step-3) In calculations, the strengths given in Table 7 was used as concrete compression strength and steel yield strength for each member. In the computer program, the loading is performed with a load-controlled normal force and a displacement-controlled horizontal force. The acting loads to the frames are shown in Fig. 6.

Step-4) The control nodes were mass concentrated sections. Applying single lateral story force was at the one of the mass concentrated sections. The lateral force in the selected pattern was applied to the structure in a stepwise manner. The total base shear starts from zero and increases. In each step, the internal member forces were calculated, and the top displacement and the base shear were recorded to plot the capacity curve.

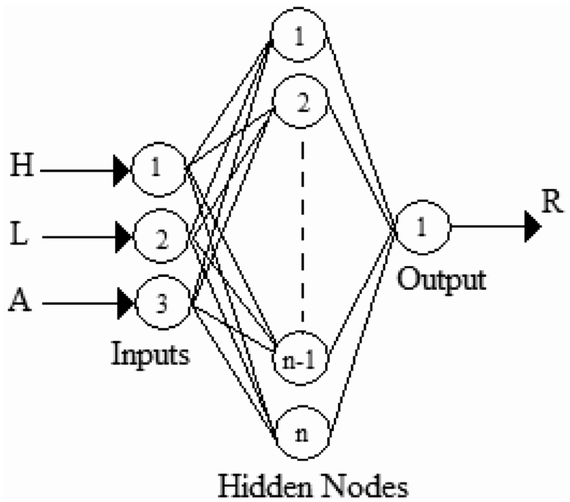


Fig. 10 ANN structure used to compute R factor

Table 9 Performance of back-propagation methods

Back-propagation methods	Optimum number of hidden nodes	Training error (%)	Test error (%)	Iteration number	Training time (second)
BFG	10	4.0436	8.8233	1233	52.64
CGB	6	4.7285	7.6564	4760	31.56
CGF	6	4.9519	8.2960	5000	30.83
CGP	6	5.3367	7.4296	5000	36.86
GDA	6	6.0535	7.7383	5000	12.63
GDM	10	7.1666	7.2863	5000	13.95
GDX	10	5.0924	8.1940	5000	14.36
LM	2	5.9516	8.3327	69	1.36
OSS	6	5.5387	7.9571	5000	42.97
RP	10	5.8718	8.0832	5000	17.36
SCG	6	5.0571	6.9795	5000	23.75

4.3 Calculating R and ANN procedure

After analyzing 140 frames using SAP2000, the force reduction factor R was calculated with equation 3 given above. For the ANN procedure, the values of H, L and A were presented as inputs to ANN and then the various values of R coefficients corresponding to those dimensions were

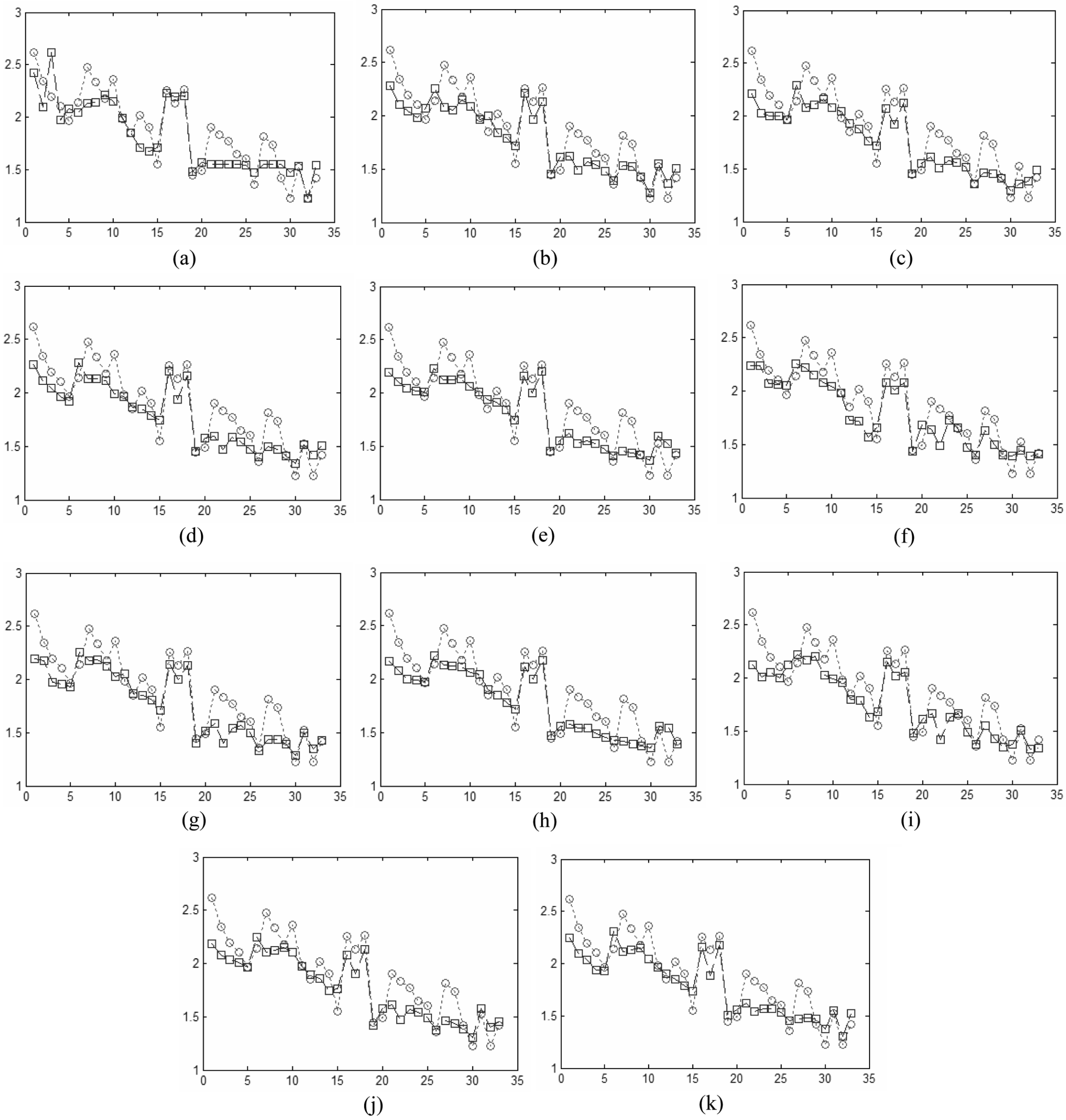


Fig. 11 X axis is number of test samples (33 test points). *Y* axis is R values. Target values are displayed with circles in figure. Network outputs for algorithms are displayed with squares in figure. (a) BFG, (b) CGB, (c) CGF, (d) CGP, (e) GDA, (f) GDM, (g) GDX, (h) LM, (i) OSS, (j) RP, (k) SCG

obtained as outputs, as in Fig. 5. Due to logarithmic sigmoid function (Eq. (9)) was used in this study as activation function, inputs were normalized to [0 1] before presented to ANN.

5. Results and discussion

In this study, an attempt was made to use ANN for computing a reasonable R value for single bay precast gable framed structures. For this aim, 3:HN:1 ANN structure was used (H,L,A are inputs for ANN and R factor is output for ANN). Where, HN is optimum number of hidden nodes. ANN structure is shown at Fig. 10. In this study, the learning rate parameter and momentum constant were chosen as 1.0 and 0.2, respectively.

In this study, for training ANN, GDM, GDA, GDX, RP, CGF, CGP, CGB, SCG, BFG, OSS, LM methods were used. The optimum number of hidden nodes was computed for these back-propagation methods. For training, obtained optimum ANN structure randomly selected 107 sets of 140 sets were used and remained 33 sets were used for test. The obtained, training and test errors, optimum number of hidden nodes, iteration number and training time are shown at Table 9. Moreover, the comparable figures of target values and the obtained network outputs are shown at Fig. 11 for 33 test points as test results.

As seen at Table 9, all of back-propagation methods were obtained about 94% and 92% averaged accuracy rate (100% - error%) for training and test phase of neural network, respectively. In test phase, SCG algorithm was obtained the smallest test error (6.9795%). Other words, SCG algorithm was estimated force reduction factor (R), successfully. However, if CPU times of all algorithms in this study are compared, it is shown that, LM algorithm made classification process in less time than other algorithm. This algorithm required only 1.36 second for training of neural network.

This study also showed that, the force reduction factor R in the calculated prefabricated single storey buildings that have different aspect ratios, ranges from 2.6 to 1.4. However, in the TEC-98, the minimum value of R is 3 and the maximum value is 6 according to the ductility level of the systems.

6. Conclusions

Being the one of the most important parameter in earthquake load calculation, the load reduction factor, R, is defined as independent of the dimensions of the structure but related to the type of the load bearing system in most of the Earthquake Codes.

The concept of load reduction factor is based on the premise that a well-detailed seismic framing system could sustain large inelastic deformations without collapse ductile behavior) and develop lateral strength in excess of the design strength (reserve strength). Thus, the elastic earthquake loads are reduced by dividing them into "R" coefficients.

In this study, in order to obtain exact load reduction coefficient, R, an artificial neural network method (ANN) was used. The paper endeavors to introduce the neural network application on determining load reduction coefficient, R, for single bay prefabricated structures that had met enormous damages and failures in the last big earthquakes.

The R values obtained in this study are more realistic and much lower than those given in TEC –

98. The earthquake design load increases with the decreasing R value. Therefore, it was shown that the existing prefabricated single story structures were designed according to smaller earthquake loads and they have failed or faced with heavy damages.

This study carries a proposal character to revise the R coefficient of prefabricated structures recommend in TEC – 98, besides being the first study in the literature to determine R coefficient by using ANN method. For further studies, the design of different intelligence methods as hybride structures will be performed in order to increase the performance of ANN for the determination of R coefficient.

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