

Evaluation and Strengthening of Existing RC Buildings Using FEMA P695 Methodology and Calculation of CMR for Quick Assessment

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ABSTRACT

Investigating the seismic performance of existing structures that have been designed and constructed in accordance with previous codes is one of the existing challenges. Retrofitting techniques must be practical and comprehensive. Reinforced concrete (RC) buildings in Iran comprise a large part of residential buildings. Four editions of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard no. 2800) have been published since 1986. Noticeable changes have been made between the second and third editions regarding the seismic design of RC buildings. Firstly, this study focuses on evaluating the design reliability and seismic performance factors presented in Standard No. 2800-99 (second edition) and Standard No. 2800-05 (third edition). A set of 51 reinforced concrete archetypes (1 to 12 stories) and 2244 incremental dynamic analyses were performed. For the first time, the concept of Adjusted Collapse Margin Ratio or ACMR is used for seismic evaluation of stock buildings. ACMR is obtained from the results of pushover analysis, incremental dynamic analyses, and fragility analysis based on the FEMA P695 methodology. The results reveal that the structural systems designed in accordance with Standard No. 2800-05 fulfilled the requirements for moderately intense earthquakes. In contrast, the vulnerability of low and mid-height building stocks designed based on the second edition are significant and are prone to incur dramatic damage in the event of far-field earthquakes. Secondly, the possibility of adding concentric steel braces with different configurations was investigated to strengthen vulnerable archetypes. In this study, the concept of the collapse probability of a structure due to a set of far-field earthquake records, has been investigated in order to find the optimal design point in strengthening the vulnerable stock RC structures. In addition, appropriate seismic design coefficients have been investigated and introduced using the probability of

collapse. Sixty archetypes were defined in 20 performance groups and 15800 non-linear dynamic analyses were performed. The probability of collapse of all possible total uncertainties (0.3 to 0.9) was investigated. Using the trial and error method, the best seismic design coefficients of steel braces were found. Thirdly, a set of analyses including Pushover Analysis, Incremental Dynamic Analysis (IDA), along with Fragility Analysis, which are usually time-consuming and computationally unaffordable, must be performed to obtain CMR. Artificial Neural Network (ANN), Response Surface Method (RSM), and Adaptive Neuro-Fuzzy Inference System (ANFIS) are also introduced to solve this issue and to achieve a quick and efficient method to estimate the CMR. So, almost 5016 IDA analyses were used based on 114 archetypes. In this regard, five parameters were used as independent and desired entries of the system. In addition, CMR is considered the output of the systems. In the RSM method, ANOVA analysis is also applied to obtain significant parameters. However, ANFIS offered the best efficiency and accuracy with the least number of errors. In comparison, the ANN method was found to be more effective than the RSM and has a higher regression coefficient and fewer statistical errors.

Keywords: Incremental dynamic analysis, Pushover analysis, Steel bracing, Collapse probability, FEMA P695 methodology, Soft computing

ÖZ

Önceki deprem şartnamelerine uygun olarak tasarlanmış ve inşa edilmiş mevcut yapıların sismik performansının araştırılması mevcut zorluklardan biridir. Güçlendirme teknikleri pratik ve kapsamlı olmalıdır. İran'daki betonarme binaların büyük bir kısmı konut türü yapılardır. İranda Binaların Sismik Dirençli Tasarımı için Uygulama Kuralları'nın dört baskısı (Standart no. 2800) 1986'dan beri yayınlanmıştır. Bu deprem şartnamesinin ikinci ve üçüncü baskılarında önemli değişiklikler yapılmıştır. İlk olarak, bu çalışma, Standart No. 2800-99 (ikinci baskı) ve Standart No. 2800-05'te (üçüncü baskı) sağlanan tasarım güvenilirliği ve sismik performans faktörlerinin değerlendirilmesine odaklanmaktadır. 51 adet betonarme örnek yapı (1 ila 12 kat arası) seçilip, 2244 artımsal dinamik analiz yapılmıştır. İlk kez, bina stoğunun sismik değerlendirmesi için ACMR kavramı kullanılmıştır. ACMR, FEMA P695 metodolojisine dayalı itme analizi, artımsal dinamik analizler ve hasargörebilirlik eğrileri sonuçlarından elde edilir. Sonuçlar, 2800-05 No'lu Standart'a göre tasarlanan yapısal sistemlerin orta şiddetli depremler için gereksinimleri karşıladığını ortaya koymaktadır. Buna karşılık, ikinci baskıya göre tasarlanan düşük ve orta yükseklikteki yapı stoklarının güvenlik açığı önemlidir ve uzak mesafe yer hareketi kaydı depremlerinde dramatik hasara yol açmaktadır. İkinci olarak, depreme karşı savunmasız yapıları güçlendirmek ve uygun tekniği saptamak için farklı konfigürasyonlarda merkezi çelik çaprazların betonarme yapılara eklenme olasılığı araştırıldı. Bu çalışmada göçme olasılığı konsepti göz önünde bulundurularak, yapıların uzak mesafe yer hareketi kayıtlarına göre güçlendirilmesinde optimum sonucu bulmak için bina stoğunun güvenilirliği incelenmiştir. Buna ek olarak göçme olasılığına göre uygun sismik tasarım katsayıları araştırılmıştır. 20 performans grubunda altmış örnek bina tanımlanmış ve 15800

doğrusal olmayan dinamik analiz yapılmıştır. Tüm olası toplam belirsizliklerin (0,3 ila 0,9) göçme olasılıkları incelenmiştir. Deneme yanılma yöntemi kullanılarak betonarme binalardaki çelik çaprazların optimum sismik tasarım katsayıları bulunmuştur. Üçüncü olarak, CMR'ı elde etmek için, Doğrusal Olmayan Statik Analiz (NSA), Artımsal Dinamik Analiz (IDA) ile birlikte hasargörebilirlik eğrileri sonuçlarını da içeren, çok zaman alan ve pratik olmayan bir dizi analizin yürütülmesi gerekmektedir. Bu sorunu çözmek, CMR'ı tahmin etmek için hızlı ve verimli bir yöntem elde etmek amacı ile, mevcut araştırmada Yapay Sinir Ağı (ANN), Yanıt Yüzey Yöntemi (RSM) ve Adaptif Ağ Tabanlı Bulanık Çıkarım Sistemi (ANFIS) kullanılmıştır. Bu amaçla, 114 örnek binaya yaklaşık 5016 IDA analizi yapılmıştır. Bu kapsamda hızlı değerlendirme yöntemlerinde 5 parametre kullanılmıştır. Ayrıca, CMR hızlı değerlendirme methodlarının çıktısı olarak seçilmiştir. RSM yönteminde, önemli parametreleri test etmek için Varyans Analizi (ANOVA) Yöntemi de kullanılmıştır. Sonuç olarak ANFIS, en iyi verimliliği ve doğruluk oranını sunmuştur. Öte yandan ANN yönteminin RSM'e göre daha etkili olduğu ve daha yüksek regresyon katsayısına ve daha düşük istatistiksel hatalara sahip olduğu belirlenmiştir.

Anahtar Kelimeler: Artımsal Dinamik Analiz, Statik İtme Analizi, Merkezi Çaprazlı Çerçeve, Göçme Olasılığı, FEMA P695, Hızlı Değerlendirme

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LIST OF SYMBOLS AND ABBREVIATIONS

$\overline{ACMR}_i$	The Mean of the ACMR Values
A	Earthquake Design Acceleration
AAD	Average Absolute Deviation
$ACMR$	Adjusted Collapse Margin Ratio
$ACMR_{10\%}$	Acceptable Value for the Performance Group
$ACMR_{20\%}$	Acceptable Value for a Specified Archetype
$ACMR_i$	Specific Values of the ACMR
$Adjusted R^2$	Adjusted Coefficient of Determination
$ANFIS$	Adaptive Neuro-Fuzzy Inference System
ANN	Artificial Neural Network
$ANOVA$	Analysis of Variance
B	Reflection Factor
C	Base Shear Coefficient
C_d	Deflection Amplification Factor
CMR	Collapse Margin Ratio
DOF	Degree of Freedom
EDP	Engineering Demand Parameter
$LRFD$	Load and Resistance Factor Design
MAE	Mean Absolute Error
MCE	Maximum Considered Event
MLP	Multi-Layer Perceptron
MSE	Mean Square Error
NSA	Nonlinear Static Analysis
PGV	Peak Ground Velocity

R	Correlation Coefficient
$Rfactor$	Response Modification Factor
R^2	Coefficient of Determination
RC	Reinforced Concrete
$RMSE$	Root Mean Square Error
RSM	Response Surface Method
S_{CT}	Median Collapse Capacity
S_{M1}	Spectral Response Acceleration at a Period of 1s
S_{MS}	Spectral Response Acceleration at Short Periods
S_{MT}	Spectral Response Acceleration at the Main Period
SEP	Standard Error of Prediction
T	Fundamental Period of the Structure
V_{max}	Maximum Base Shear
W	Seismic Weight
μ	Ductility Factor
Ω	Overstrength Factor
β_{DR}	Record to Record Uncertainty
β_{MDL}	Nonlinear Modelling Uncertainty
β_{RTR}	Median Collapse Capacity
β_{TD}	Test Data Uncertainty
β_{TOT}	Total Uncertainty
δ_u	Maximum Displacement
$\delta_{y,eff}$	Effective Roof Displacement

Chapter 1

INTRODUCTION

1.1 Problem Statement

Reinforced concrete buildings built in the past may not fully withstand the expected seismic demand for a variety of reasons. Such buildings must be strengthened to be developed in accordance with recent codes. One of the common techniques for remediating such deficiencies is the addition of structural elements such as steel braces. The use of steel braces to strengthen RC buildings is increasing, mainly due to the reduction in building weight, convenience, and rapid construction. Notably, adding concentric steel braces in RC frames increases the stiffness and reduces the lateral displacement. Evaluation of how to strengthen existing structures is done with the help of nonlinear analysis methods. Nonlinear analysis methods are generally expensive and time consuming. On the basis of artificial intelligence and statistical methods can be used to overcome these problems. Soft computing models are able to solve complex and time-consuming problems.

1.2 Aim and Scope

Four editions of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) have been published [1, 2]. Significant changes have been observed between the second and third editions regarding the seismic design of reinforced concrete buildings. 1) This study first evaluates the reliability of seismic resistance of RC stock buildings designed by Iranian Earthquake Code based on FEMA P695 methodology. A set of 51 reinforced concrete archetypes (1, 3, 6, 9, and

12-storey) and 2244 IDA analyses were done. Therefore, the main purpose is to assess the seismic reliability of existing residential RC buildings with low to mid-rise. In the next step, comparison of seismic characteristics of structures designed based on the second and third editions of Iranian seismic resistant design codes. Moreover, the FEMA P695 methodology has been used to calculate potential of structural collapse. For the first time, the concept of ACMR (derived from the concept of the probability of collapse of a structure) is used to assess the seismic safety of stock buildings.

The shaking caused by earthquakes can damage buildings and cause collapse. The seismic assessment of building performance and the establishment of a statistical model of desirable responses has become prominent research areas. 2) In this study the evaluation of performance of RC building stock designed according to the second version of Standard No. 2800 (1999 to 2005) shows that low- and mid-rise RC buildings are prone to deficiencies and might incur dramatic damage in the event of far-field earthquakes. Using the FEMA P695 methodology, it is possible to evaluate safety based on the structural collapse stochastic. Moreover, the seismic behaviour of the building stock of 1, 3 and 6-storey RC frames that need retrofitting is investigated. Concentric steel braces with different configurations such as cross bracing, chevron bracing, and diagonal bracing were considered. Sixty archetypes were studied, and 26400 nonlinear analyses were performed. In addition, the probability of collapse due to total uncertainties (0.3 to 0.9) was calculated to determine the effect of uncertainties on the collapse probability. The response modification factor was also determined for steel-braced RC frames. Seismic parameters such as the ductility, overstrength, and maximum drift between the stories and the collapse margin ratio (CMR) were compared between unbraced and steel-braced RC frames. In this study, the concept of the collapse probability of a structure due to a set of earthquake records

have been examined to find the optimal design point in strengthening the vulnerable stock RC structures. Moreover, appropriate seismic design coefficients have been investigated and introduced using the probability of collapse.

3) The CMR is a notable index used for seismic assessment of the structures. As proposed by FEMA P695, a set of analyses including the Nonlinear Static Analysis (NSA), Incremental Dynamic Analysis (IDA), together with Fragility Analysis, which are typically time-consuming and computationally unaffordable, need to be conducted so that the CMR could be obtained. To address this issue and to achieve a quick and efficient method for estimating CMR, the Artificial Neural Network (ANN), Response Surface Method (RSM, and Adaptive Neuro-Fuzzy Inference System (ANFIS) have been introduced in current research. Accordingly, using the NSA results, an attempt was made to find a fast and efficient way to derive CMR. To this end, nearly 5016 IDA analyses based on FEMA P695 methodology have been performed on 114 various Reinforced Concrete (RC) frames with 1 to 12 stories. In this respect, five parameters have been used as the independent and desired inputs of the systems. CMR, on the other hand, is considered the output of the systems. Accordingly, a double hidden layer neural network with Levenberg-Marquardt training and learning algorithm was taken into account. Moreover, in the RSM approach, a quadratic system incorporating 20 parameters was implemented. Correspondingly, analysis of variance (ANOVA) has been employed to discuss the results obtained from the developed model. Additionally, the basic parameters and interactions are extracted and the input parameters are sorted according to their importance. Moreover, the ANFIS was employed by Takagi-Sugeno fuzzy system. Finally, all methods were compared, and the effective parameters and associated relationships were extracted. In contrast to the other approaches, the ANFIS provided

the best efficiency and high accuracy with the minimum desired errors. Comparatively, it was found that the ANN method is more effective than the RSM and has a higher regression coefficient and lower statistical errors.

A few types of research have been accomplished to predict the CMR or IDA results using the results of pushover analysis. Previous research methods have been based on computer software such as SPO2IDA or computational methods based on tables. Nevertheless, no studies have been performed to estimate the CMR parameter with soft computing techniques. To fill the gap, also, this study also evaluated the efficiency of the ANN, RSM, and ANFIS approaches in accurate CMR prediction, as described by FEMA P695. This part has been done to reduce the computational time and estimate the desired results without the need for IDA analyses. The accuracy and capability of each technique have been compared and examined by statistical indices. Accordingly, the input variables and significant parameters have been extracted and sorted according to their importance.

1.3 Overview of the Study

In order to achieve the objectives of the research, a number of tasks have been accomplished as follows:

1. Development and design of archetypes: In this study, 32 performance groups with different numbers of stories are taken into consideration. A total of 114 archetypes are modelled. The archetypes are designed in compliance with Iran's Seismic Design Code (Standard No. 2800). Two dimensional nonlinear numerical models appropriate for each archetype building are developed using the SeismoStruct computational framework.

2. Nonlinear static pushover analyses and incremental dynamic analyses:

Non-linear static analyses are performed for archetype buildings. The main lateral

resistance and deformation parameters are derived from the results of static pushover analyses for each model. Moreover, nonlinear dynamic response history analyses are performed as incremental dynamic analyses for each.

3. Collapse performance evaluation: The possibility of each archetype to collapse is estimated according to the results of the IDA analysis. The collapse margin ratio and the ACMR must be determined for each archetype in accordance with the methodology of FEMA P695. Collapse fragility curves are developed for each model based on the results of incremental dynamic analysis.

4. A probabilistic analysis of collapse: A probabilistic analytical framework for seismic collapse is developed for models to cover the possible range of total uncertainty.

5. Quick assessment technique to find CMR using soft computing: Three soft computing methods (ANN, RSM method, and ANFIS method) have been used to find a quick way to estimate CMR. In this way, the importance of the seismic parameters is compared and sorted; the computational formula is also presented.

1.4 Organisation of the Thesis

The thesis consists of six chapters. The introduction in Chapter 1 contains general information about the subject matter of the study and provides an overview of the thesis. Chapter 2 provides a review of the literature on relevant research. The FEMA P695 methodology and soft computing descriptions are described in Chapter 3. Chapter 4 details the methodology used to model unbraced and steel braced RC archetypes and soft computing methods, as well as the design procedure and provide key details of the various Archetype designs. Chapter 5 presents the results of the design, modelling and seismic assessment of a set of archetypes. Moreover, it details the results of nonlinear static pushover and incremental dynamic analyses including the collapse assessment outcomes, and the results of the use of artificial neural

networks, response surface method, and adaptive neuro-fuzzy system. Chapter 6 summarises the overall findings and conclusions of the study, including recommendations for future studies.

Chapter 2

LITERATURE REVIEW AND BACKGROUND

2.1 Introduction

This chapter summarises some of the previous research conducted on topics of relevance to this research. The first section examines the last researches on the seismic performance of reinforced concrete structures. In the second section, the retrofitting of RC structures is summarised. In the third section, some previous studies based on FEMA P695 are reviewed. In addition, in the forth section, previous researches related to soft computing consist of ANN, RSM method, and ANFIS system, are reviewed. In the last section, previous researches on the concept of probability of collapse are summarised based on IDA analysis, fragility analysis, and performance levels.

2.2 Previous Work Done

Past research in the present study is continuously classified into five sections, and presented as follows:

2.2.1 Previous Researches on Reinforced Concrete Structures

Over the years, reinforced concrete structures have been considered, due to the combination of economy, the plasticity of form, significant resistance to fire, effectiveness in time, and ease of implementation. But the most damaging natural phenomenon, which affects numerous structures and brings massive damage to buildings, is an earthquake. The design of any structure must effectively consider seismic behaviour. Subsequently, nonlinear analysis methods could be applied to

achieve the overall performance of structures and more accurate responses, such as damages and collapse of structures. An applied nonlinear method for the purpose of seismic demand and capacity of the structure provides a much more reasonable assessment of the seismic behaviour subjected to strong ground motions. Owing to the large randomness and uncertainty that exists in determining both seismic demand and seismic capacity, only approximate estimates of the seismic behaviour of these structures are obtained. But, based on past researches and knowledge of earthquake events, updating and developing of building codes are continuously in progress.

Iran is located in the Alpine Himalayas earthquake zone and it is one of the most earthquake-prone areas on the earth. This region is categorised by repeated moderate-magnitude earthquakes and with less frequent events of damaging earthquakes of a magnitude greater than 6 Richter [3]. Therefore, it is important to pay attention to earthquake science and the construction of earthquake-resistant structures. Already, the Iranian code of practice for the seismic-resistant design of buildings Standard No. 2800 has been published and released by the Road, Housing, and Urban Development Research Center (BHRC) [1, 2]. Recently, the fourth edition (last edition) was published. In this edition, seismic design criteria for non-structural components, geotechnical requirements, nonlinear analysis methods, and soil and structure interaction are also considered. Many studies on seismic evolution of RC structures, related to the assessment of the performance level by using the FEMA 356 have been published. Subsequently, by applying nonlinear dynamic analysis and pushover analysis, the criteria for elements rotation and displacements and inter-storey drifts could be checked as local controls, and by comparing the capacity curve, demand curves, and total drifts, global criteria were determined [4, 5]. Respectively, Mahdi and Darehshiri [6] studied the performance level of RC frames

and reported that the models designed by using the third edition are within the life safety performance level. Asgarian and Jalaeefar [7] studied the influence of the third edition of standard No. 2800 on the steel frames and the structures show vulnerability before achieving the required levels. While, in the third edition, both the ductility criteria and earthquake force requirements were developed.

2.2.2 Previous Researches on the Strengthening of RC Structures Using Steel Braces

Many researchers have applied steel bracing to RC structures. In 1980, Dell Valle [8] reported on strengthening a 12-storey RC building after a mild earthquake in Mexico City. In this project, the building was strengthened by adding steel braces to the perimeter frames. In 1990, Badoux and Jirsa [9] conducted a study on the abovementioned building (Dell Valle reported project) and found that, contrary to its neighbouring structures, this building exhibited proper seismic performance during the Mexico City Earthquake in 1985. Bush, Goel [10] and Lee [11] performed experimental tests on adding steel braces to RC frames and demonstrated that the efficiency of the lateral strength is lost by the buckling of the diagonal members. In 1991, Bush et al. [12] conducted laboratory tests on the cyclic behaviour of braced buildings and reported their adequate performance. In 2007, Youssef et al. [13] and Ghaffarzadeh and Maheri [14] conducted studies on various bracing configurations. Through cyclic loading tests, they showed that bracing systems could be utilised to enhance the performance of existing RC frames. In 2010, Viswanath et al. [15] studied the impact of X-shaped steel bracing buildings with 4, 8, 12, and 16 stories. The results show that the X-shaped braces considerably improved the stiffness and reduced the maximum inter-storey drift. In 2013, Malekpour et al. [16] conducted a study on a direct displacement-based design procedure for X-shaped steel-braced RC frames. They adopted the storey drifts and rotations of elements as the controlling

criteria and showed that the 4, 8, and 12-storey frames are capable of meeting the necessary life safety performance level. In 2013, Akbari and Maheri [17] studied the response modification factor of chevron steel-braced RC frames. The tentative R factor values for steel-braced RC frames were 5 to 11. In 2017, Beheshti Aval et al. [18] performed a comparative investigation of retrofitting solutions for RC buildings. They examined chevron bracing and cylindrical friction dampers and compared their performance with respect to the amount of damage according to FEMA P58 methodology. They found that the life safety performance level was satisfied and the chevron braces were more economical. In 2017, to compare different types of steel bracing systems, Tahamvali Roudsari et al. [19] experimentally investigated the impacts of X-shaped, chevron, and knee braces on the general performance of RC structures. They assumed equal steel weight in all models. The results show that X-shaped braces were more desirable with respect to stiffness, strength, and cracking control. In 2018, Rahimi and Maheri [20] conducted a study on the effects of applying X-shaped braces to retrofit RC structures. They analysed the fatigue, shear, and axial forces in columns and their performance levels. It is concluded that the steel bracing retrofitting technique is considerably more beneficial for low-rise buildings than high-rise ones.

2.2.3 Previous Researches on FEMA P695 Procedure

The application of a more novel and rigorous method necessitates more realistic modelling and calculation of the seismic parameter. In ATC-63, they agreed to present a global method by FEMA P695 to evaluate the seismic responses of structures. This method for identifying the seismic parameters of structural systems were designed to prevent building collapses in the event of strong earthquakes that are equivalent to the maximum considered earthquake (MCE) hazard level, which was prescribed by the NEHRP. The research approach adopted by the NEHRP assesses

the collapse margin ratio and processes the data by considering the uncertainties such that the buildings achieve reliable safety against ground motion events. Consequently, FEMA P695 methodology allows for checking the reliability of the quantification of a building system's performance against seismic design codes [21–28].

The FEMA P695 methodology utilises incremental dynamic analysis (IDA) and pushover analysis in the seismic performance assessment. FEMA P695 was established based on a methodology for dependable qualification of seismic performance features appropriate to each seismic building structure. Hence, the procedure is helpful for estimating the seismic performance of systems that are presented in building codes. AlHamaydeh et al. [29] used FEMA P695 methodology to study seismic design factors such as response modification factor and system overstrength factor for RC framed building in Dubai. Gogus and Wallace [30] studied on the seismic safety assessment of RC wall according to FEMA P695 and suggested a larger R factor for this system. Nobahar et al. [31] evaluated new lateral resisting systems to find required design parameters. Ghateh et al. [32] employed this guideline to evaluate the R factor of RC pedestals in elevated water tanks. Burton et al. [33] used collapse criteria to show, how a rocking spine system may recover the seismic collapse safety of concrete frames. Bolhassani et al. [34] focused on introducing an optimized system to improve the seismic performance and safety index of reinforced masonry shear walls. Arabzadeh and Galal [35] used FEMA P695 methodology and presented the effect of FRP retrofitting in RC couple walls. It was shown that the collapse margin ratio could be increased more than 60 percent after proper retrofitting scheme. Manafpour and Tohidian [36] studied the collapse margin ratio for RC frames designed by the third edition and they considered the effects of soil type classifications, and ductility levels too.

2.2.4 Previous Research on Fast Evaluation Techniques

One of the main reasons for losses life and economic damages during seismic events is structural collapse. Estimating the collapse margin ratio (CMR) in a logical framework could play an essential role in assessing the seismic response of the structural systems. Accordingly, the FEMA P695 methodology utilises the nonlinear static analysis (NSA), IDA, and fragility analyses under specific earthquake records for evaluating the collapse capacity of the structures. However, this method cannot be practically employed for seismic assessment of the building stock or even novel structural systems due to its high computational time and costs. By utilising a convenient and quick method for the process of analysis, it would be possible to more extensively apply the method for seismic assessment of the existing building stocks.

Until now, many studies have been conducted to simplify and expedite the process of developing the IDA results. In this respect, Vamvatsikos and Cornell [37] introduced the SPO2IDA software, which could adapt the pushover curve of the first mode of vibration into the IDA curves. This program employed results of the dynamic analysis taken from 30 earthquake records. Han et al. [38] utilised a three-linear model developed from the pushover analyses to generate the fragility curves. Shafei et al. [39] also suggested a pushover-based technique used for the MDOF systems together with regression analysis to predict the collapse capacity of the buildings. Peruš et al. [40] applied an extended network-based approach to predict the IDA curves for the reinforced concrete (RC) structures. De Luca et al. [41] studied an optimal bilinear adoption of pushover capacity curves to estimate low error approximation of the dynamic response.

Hamidian et al. [42–44] proposed a table-based approach and used the pushover

analysis to estimate the collapse margin ratio (CMR) based on FEMA P695 methodology. His research attempt incorporated both structural systems equipped with viscous damper and unequipped with viscous damper. The method was dependent on changing an MDOF structural model with an inelastic SDOF system. Birzhandi and Halabian [45] proposed an approach for the development of IDA and fragility curves of the structures using the 2DMPA (2-degree of freedom modal pushover analysis). They concluded that this approach could be confidently used for the collapse assessment of the structures. Manafpour and Jalilkhani [46] presented a method based on pushover analysis and numerical calculations to estimate the median collapse capacity of the steel moment-resisting frames. However, the main purpose of the FEMA P695 procedure is to find the median collapse capacity of various seismic-resistant structures. The CMR prediction can play an important role in evaluating the performance of structural systems and calculating the appropriate parameters for designing the buildings as per available codes, such as the deflection amplification factor (C_d) and the response modification factor (R factor). Therefore, using artificial intelligence and statistical methods, it would be possible to analyse the available data based on predicting the desired parameters. The tools based on artificial intelligence are the ANN and ANFIS, and the RSM method is one of the statistical techniques.

ANN, RSM, and ANFIS have been applied separately or in combination to model the respective processes. In this regard, the ANFIS design as a novel predictive analytical tool, along with the RSM method and ANN, is used extensively [47–54]. An ANN approach has been used as a different statistical approach with high predictive power. It can achieve patterns from the complex data and can be employed to extract parameters from the non-linear data. They are applied to resolve various cases of

problems with different subjects [55–61]. On the other hand, the RSM is a mathematical and statistical tool for examining variables, individually and interactively, and is typically read by a polynomial equation [62–68]. ANFIS is also widely taken into account for several uses, such as prediction and estimation with deficient error [69–71].

2.2.5 Previous Research on Collapse Probability Using IDA Analysis

If a structure cannot even tolerate its weight against lateral seismic forces, the structure will be collapsed. Several studies were conducted to predict and evaluates the collapse of structures, from simplified single-degree-of-freedom systems to studies of complex multi-degree-of-freedom systems that can simulate how the collapse of key structural components advances [72–74]. The complexities in the collapse and progress of collapse in a structure and the random nature of earthquakes have resulted in introducing the possible effective methods for different uncertainties [75–78]. For example, it is possible to define the safety of a collapsed surface as a specific probability at a seismic hazard level in performance-based earthquake assessment [79]. Therefore, fragility curves can be used to calculate the probability of collapse.

The engineering demand parameter (EDP) method was utilised to evaluate the probability of collapse at a certain level of risk (IM) in FEMA [76]. In this method, if (EDP_{dmnd}) is higher than the value of EDP_{pcy} , the collapse of the structure will occur.

In general, EDP shows the seismic behaviour of a structure. For instance, Cornell et al. [75] have indicated the relative displacement of a structure as an EDP.

By implementing IDA analysis using a set of earthquake records scaled by IM value, EDP_{dmnd} values can be obtained in specific IM values.. This trend advances by increasing IM values until collapse occurs. It is possible to detect EDP_{dmnd} in structural collapse values using the lognormal distribution [75, 78]. Figure 2.1 indicates the collapse fragility curve utilising IDA results for an 8-story moment frame structure [79]. The data points shown to the right of the IDA results in this figure are the data points of the collapsing fragility in increasing the IM values. This method is called the EDP-based method for estimating the probability of collapse.

Ibarra et al. [80] applied the IM value to estimate the probability of collapse and accordingly, introduced an IM-based approach to evaluate the probability of collapse. The structure collapse capacity can be expressed as a random variable (IM_C) in the IM-based approach. Likewise, the IDA results are used to provide an IM-based

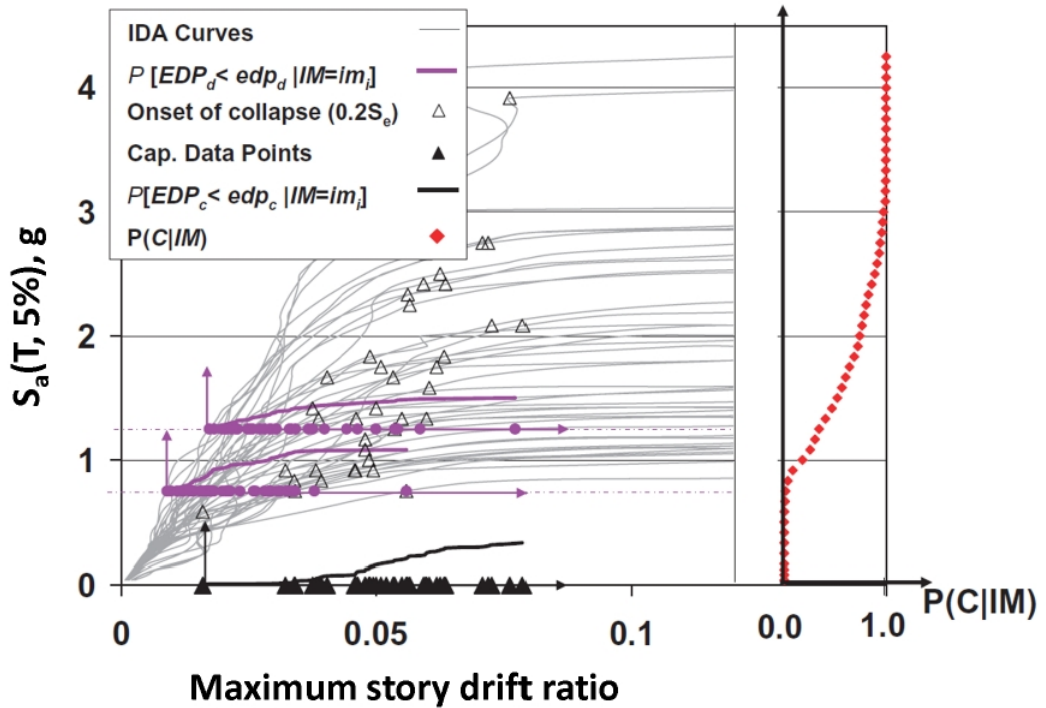


Figure 2.1: Finding collapse probability for an IM, using an EDP (maximum drift ratio) and IDA curves [79].

fragility curve, and the collapse fragility curve includes all IM_C values from the IDA results. IM and IM_C are used to explain the probability of collapse as follows [79]:

$$P(C | IM) = P(IM \geq IM_C) \quad (2.1)$$

Figure 2.2 illustrates the collapse probability assessment process using the IM-based method and equation. The right side of the IDA diagram IM_C shows each earthquake record as a circle. As Figure 2.2 shows, IM_C values often follow the model of a lognormal distribution [78].

Figure 2.3 indicates a comparison of the two fragility curves achieved by EDP-based and IM method. The fragility curve obtained from EDP method indicates that there is a higher probability of collapse compared to IM method. This difference is due to the definition of collapse capability for each [79].

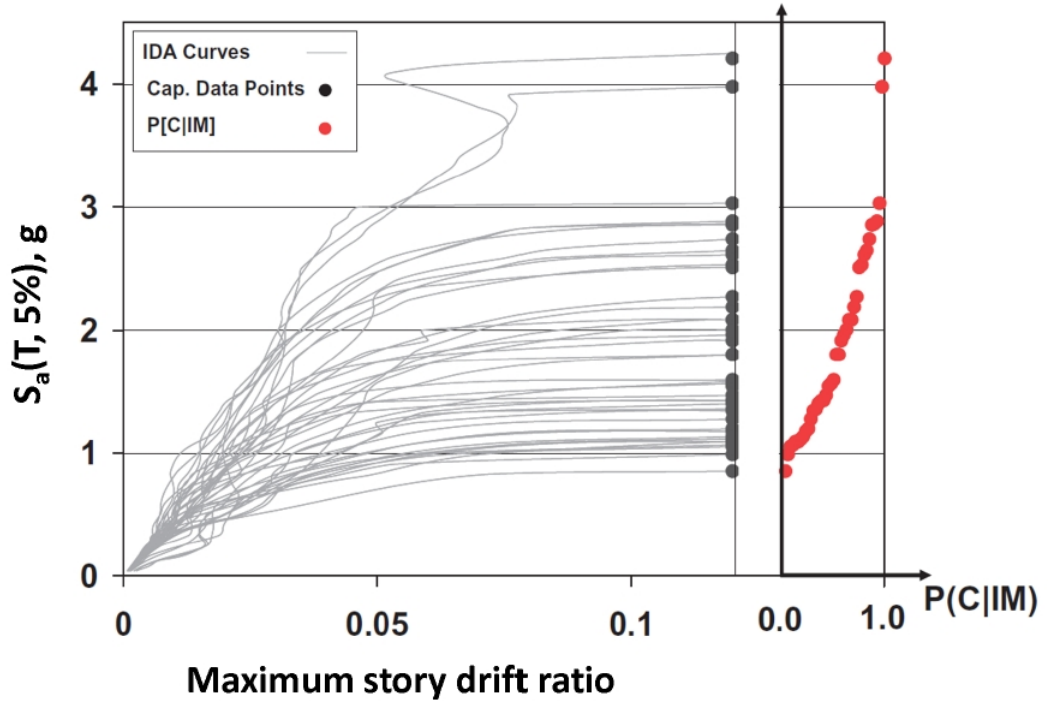


Figure 2.2: Finding collapse capacity data using an IDA curve and IM approach [79].

Various sources cause to form uncertainty to calculate the probability of collapse. Uncertainty in calculating the probability of collapse can be due to three sources: (1) record-to-record (RTR) diversity; (2) the diversity of different structural system parameters; and (3) uncertainty about the style of numerical modelling and designing. RTR diversity is caused by considering various earthquake records in dynamic analysis.

Methods such as Monte Carlo (MC) or the first-order second-moment methods (FOSM) cause to change the uncertainty to numerical modelling considerations and parameters. Therefore, for example, Luco and Cornell [81, 82] and Foutch et al. [83] examined the effect of stochastic analysis on the dynamic response of steel structures and the effect of hysteretic models for moment connections.

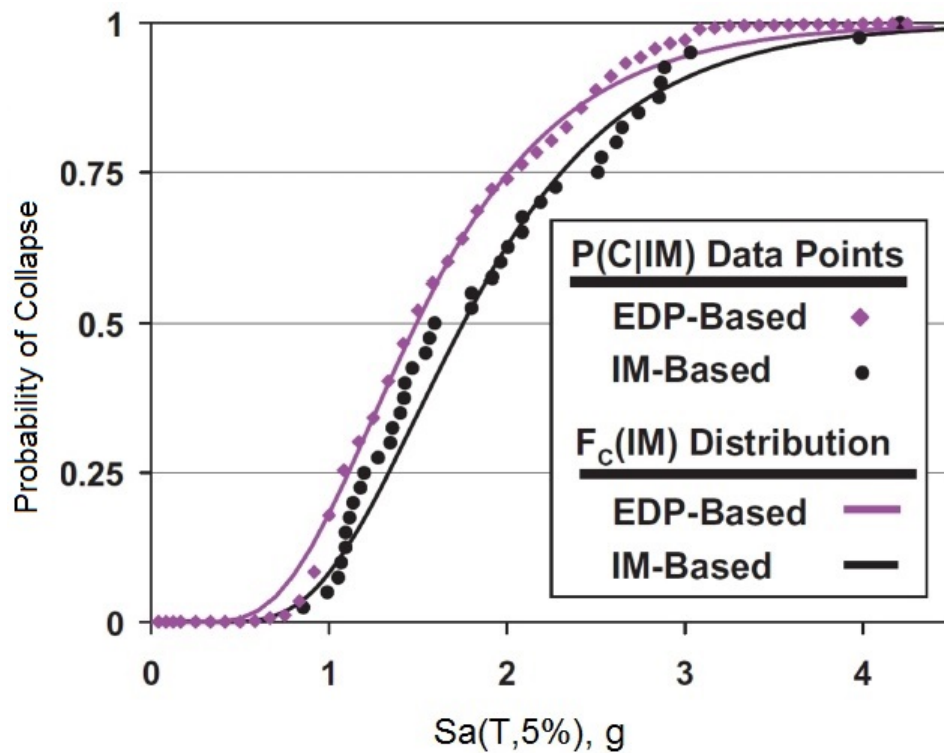


Figure 2.3: Comparison of CDF (collapse fragility curves) based on IM and EDP [79].

Ibarra and Krawinkler [78, 84] used the FOSM method to study the effect of changes in the parameters influencing the collapse of SDOF structures. Ibarra et al. have introduced a peak-oriented deterioration model [73]. As Figure 2.4 indicates, the amount of collapse has been defined as a function of hysteresis energy and a hysteresis energy dissipation capacity based on experimental results [78]. The capacity of ductility, post-capping stiffness ratio and deterioration rate (g) have been defined as random parameters. Ibarra and Krawinkler used the FOSM method to estimate the variance of the collapse capacity of a structure as follows [78, 85, 86]:

$$\sigma_{\ln S_{a,C}(TOT)}^2 \cong \sum_{i=1}^n \sum_{j=1}^n \left(\frac{\partial g(x)}{\partial x_i} \frac{\partial g(x)}{\partial x_j} \right)_{X=\mu_x} \rho_{x_i, x_j} \sigma_{x_i} \sigma_{x_j} + \sigma_{\ln S_{a,C}(RTR)}^2 \quad (2.2)$$

Where X is the lognormal vector of the feasible parameters of the system, μ_x reveals the vector of the mean values of X . g is collapse performance function. ρ_{x_i, x_j} is the correlation coefficients and σ_{x_i} is the standard deviation. The first term to the right of Equation 2.2 is uncertainty contribution to all random system parameters in collapse variance. The second term to the right side is the variance portion of the RTR variety [78].

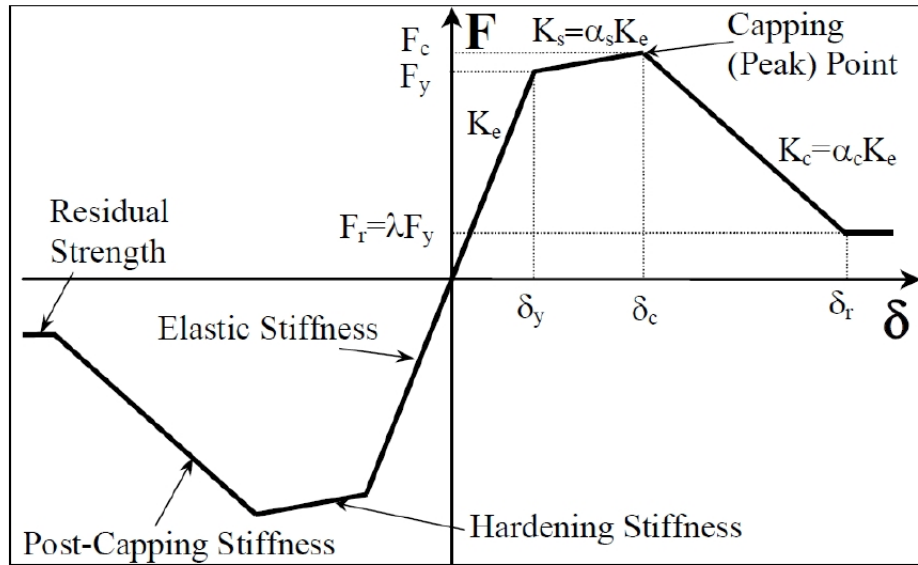


Figure 2.4: Backbone curve [78].

A mathematical expression is determined to describe the collapse performance of a vague structural system. The partial derivatives of g in the first term to the right side of the Equation 2.2 are numerically estimated. The mean collapse capability is calculated for two variations of each system parameter. Figure 2.5 schematically shows this partial numerical determination.

Figure 2.6 presents the contribution of the different parameters of the Equation 2.2 to the collapse capacity of an SDOF during natural periods. It is observed that RTR diversity is the most significant factor in the structural response and the combined correlation of system parameters is the second most significant factor [78].

Liel et al. [78] evaluated the collapse uncertainty of concrete MRFs using the FOSM method and the Monte Carlo simulation approach with a response surface method. Figure 2.7 shows an example of the response surface to evaluate the collapse capacity against beam change and column strength and ductility. Several parameters such as moment strength, initial stiffness, post-yield stiffness, capping point, post-capping deformation capacity, and rotational collapse are considered as random system

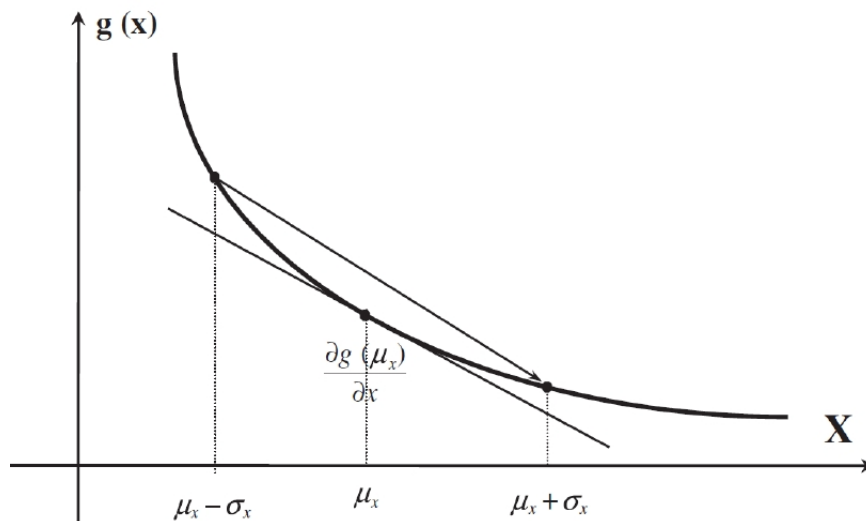


Figure 2.5: Collapse performance function and partial derivative [78].

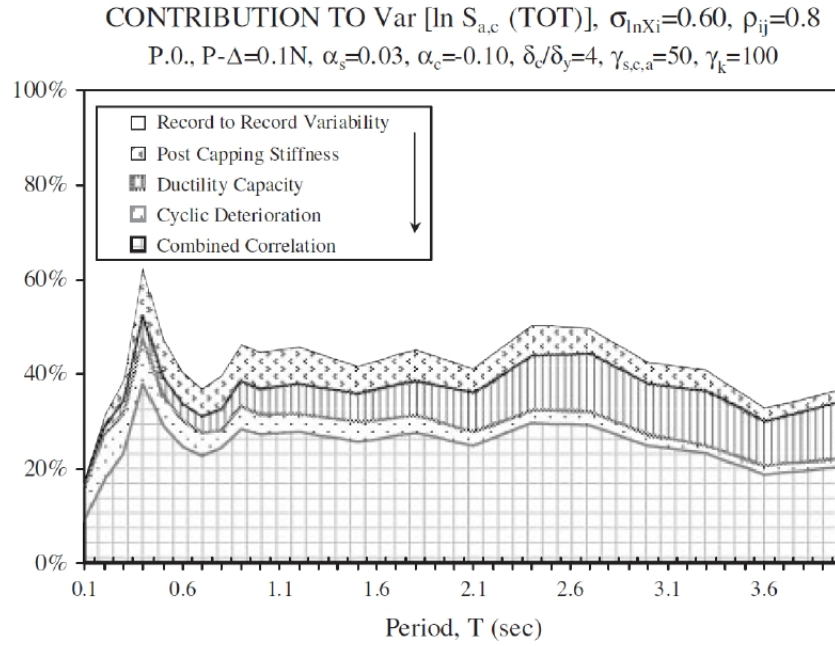


Figure 2.6: The relative contribution of different amounts of uncertainty in collapse capacity [78].

parameters for beams and columns. Figure 2.8 reveals the collapse fragility curve for a 4-story reinforced concrete torque frame developed with and without modelling uncertainty [87]. It may be observed that the effect of system uncertainty increases the scatter of the fragility curve.

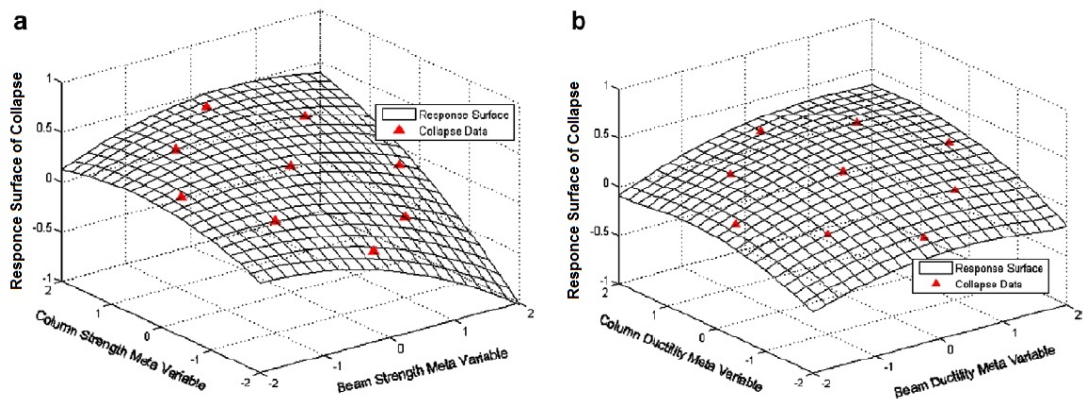


Figure 2.7: Graphical representation of the RSM for the collapse capacity of the 4-story MRB. : (a) The effects of structural strength are indicated, whereas the variables of beam ductility and column ductility are maintained constant (b) deformation effects of beams and columns [87].

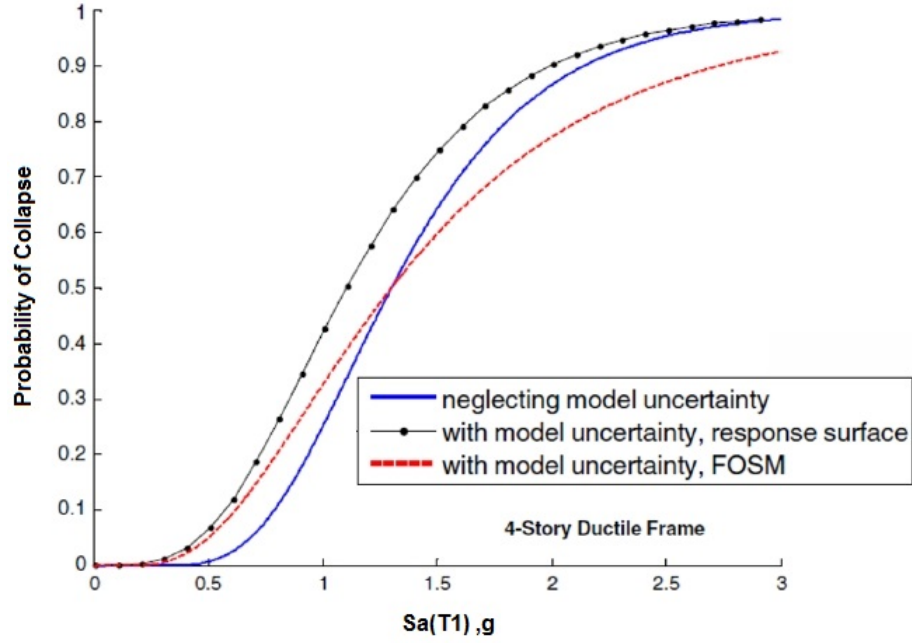


Figure 2.8: Comparison of the collapse fragility curves [87].

Vamvatsikos and Fragiadakis [88] examined the sensitivity of the push-over curve and the average IDA curve and change the ductility of the beam hinges in the moment frame (Figure 2.9). Figure 2.10 shows the effect of uncertainty using Monte Carlo simulation, FOSM method, and point estimation method, where LHS refers to Monte Carlo simulation results, FOSM refers to FOSM results, and PEM refers to the point. Figure 2.11 shows the estimated results of the distribution of S_a values (T_1 ; 5%) up to four values of the maximum drift ratio in the stages, due to the deform-ability of MRF plastic hinges. The ductility of the beam hinge contributes significantly to the overall dispersion of the steel MRF performance.

The four steps of performance-oriented earthquake engineering include hazard analysis, structural analysis, damage analysis, and loss analysis. The analysis in these stages focuses on separate categories of random variables: IM, EDP, damage measurement, and decision variable. Losses are estimated using the results of structural analysis and damage analysis [89,90].

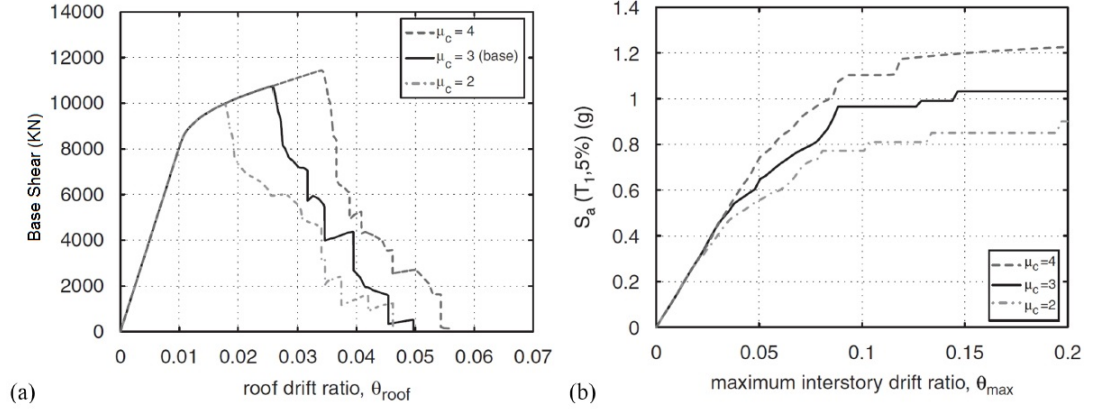


Figure 2.9: Comparison of different ductility: (a) pushover curves, (b) IDA curves [88].

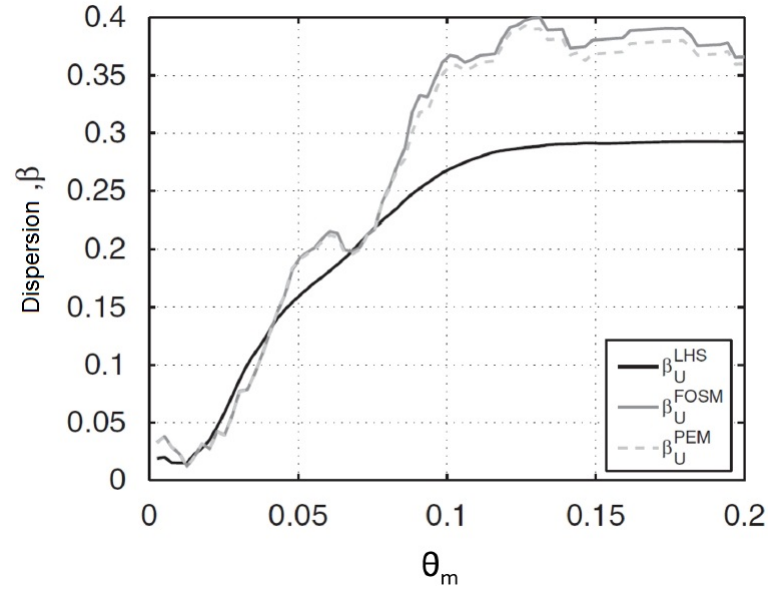


Figure 2.10: Comparison of uncertainty values in IM and drift ratio [88].

As Figure 2.12 shows, the Structural Engineers Association of California (SEAOC) Vision 2000 [91] combines several optimal performance levels with several seismic hazard levels to define the performance design objectives. Uma et al. [92] utilise the class drift distribution to extend the target matrix approach, and then employ it to define performance objectives by increasing the level of risk.

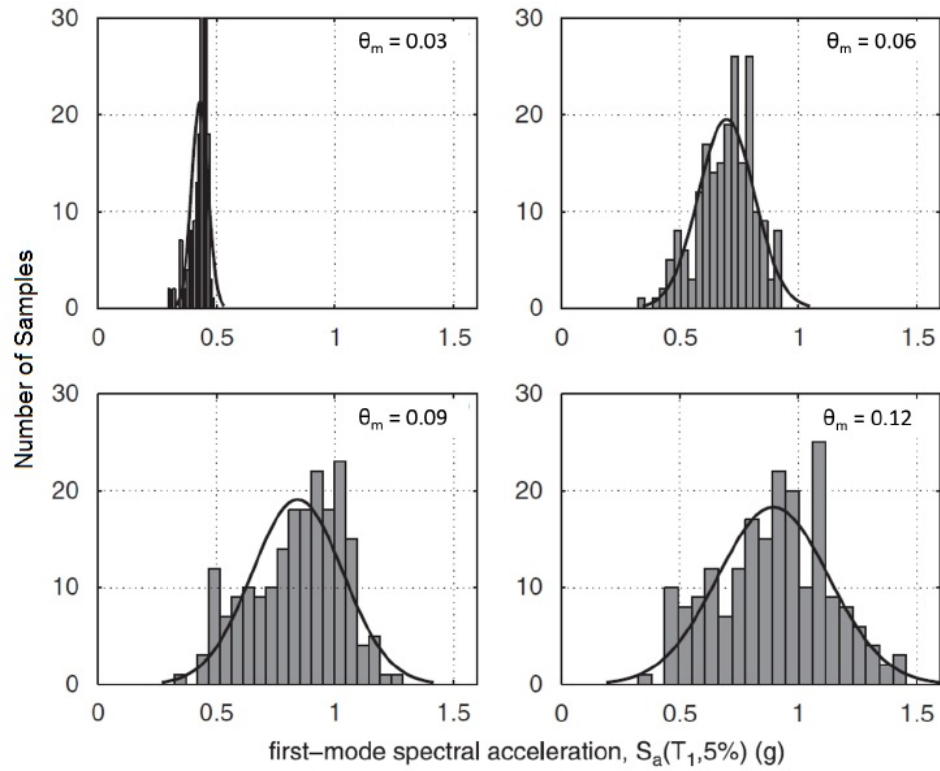


Figure 2.11: Four maximum drift ratio and normal distribution curve of median values of $S_a(T_1, 5\%)$ related to ductility factor [88].

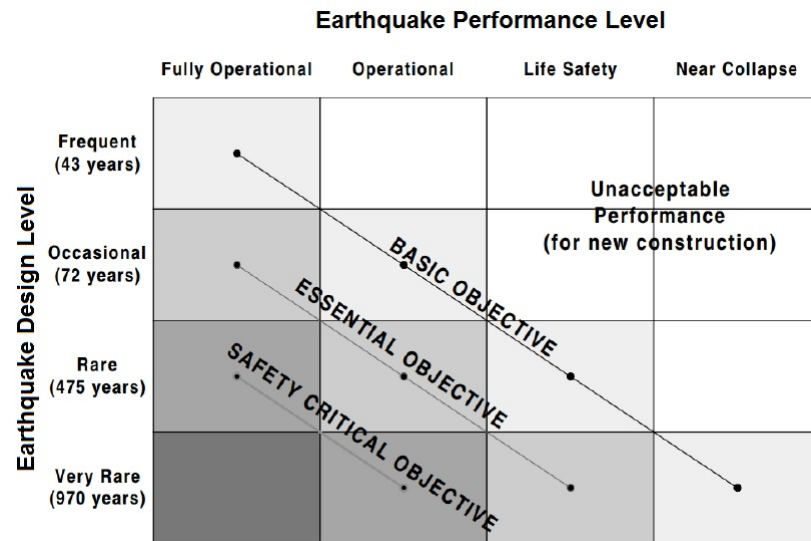


Figure 2.12: Performance-level targets [88].

Chapter 3

FEMA P695 METHODOLOGY AND SOFT COMPUTING METHODS

3.1 Introduction

In this chapter, the method of seismic modelling and evaluation of results in accordance with FEMA P695 is discussed. The procedure is expressed on how to evaluate structures with the concept of ACMR, and collapse probability is explained. The details of the FEMA P695 are discussed in the selection, normalisation, and scaling of the far-field set of ground motion records. Finally, due to the powerful properties of the Artificial Neural Network, Response Surface Method, and ANFIS method in predicting complex data, these methods are explained separately. Furthermore, the concepts of the methods presented in this chapter will be applied in the fourth chapter for modelling and evaluation.

3.2 FEMA P695 Methodology

Developments in seismic performance-based design and access to experimental data on the cyclic behaviour of structural elements have led to the assessment of seismic load-resistant systems based on probabilistic theories. On this basis, FEMA P695 was established in 2009. FEMA P695 presented a novel analytical methodology to quantify the performance levels and structural responses and was developed for seismic design with adequate reliability against structural damages. This analytical method uses a rational framework for determining the seismic parameters, including the response modification factor, overstrength, and deflection amplification factor for

existing or newly built structures. The Incremental Dynamic Analysis (IDA) curve for a specific record can be derived through successive time-history analyses using a structural model. To conduct (IDA), a number of earthquake records are incrementally scaled, and several factors are used to produce a result. Therefore, the structural response includes all behavioural ranges from linear to the moment of lateral instability (i.e., collapse mechanism). This curve can elucidate the actual behaviour, design parameters, and accuracy of previous techniques, such as the lateral load distribution pattern, pushover or spectral methods, for evaluating post-failure behaviour. Accordingly, the potential for structural collapse can be evaluated. This method is a combination of pushover, incremental dynamic analyses, and proposed criteria, according to probabilistic models. The purpose of this method is to determine the life safety performance, by providing the possibility of an accepted collapse in maximum earthquake-ground motion (MCE). In general, the precise calculation of the collapse risk of a system is complicated due to the uncertainties, but methodology provides an almost uniform approach to evaluate the collapse risk of the various structural systems. Consequently, according to FEMA P695 methodology the followings can be achieved:

- 1- The reliability of the quantification of building system performance in seismic design
- 2- Consistency of the life safety performance level and seismic performance factor evaluation in seismic codes and standards
- 3- Definition of the collapse margin ratio to express the limit of safety

The structures could be evaluated by conducting certain incremental dynamic analyses according to a number of specified earthquake records. As a result, the collapse margin ratio should be obtained to evaluate the collapse capacity of the archetypes. In the procedure, the acceptable margin ratios for different indexes are

also provided. In Figure 3.1, the elements of FEMA P695 methodology are illustrated. Table 1 shows 22 pair earthquake events, the 44-individual far-field record set recommended by FEMA P695. IDA analyses are conducted to estimate the median collapse capacity, S_{ct} , which is equivalent to a 50% collapse probability acquired from fragility curves.

For dynamic analysis, 22 pairs of far-field earthquake records that were presented in Table A-4A of FEMA P695 must be scaled. Records are first scaled using the peak ground velocity (PGV) because this method can remove redundant variables in records due to their inherent differences such as the epicentre distance, magnitude of the earthquake and soil type. The records are scaled to the maximum spectral acceleration (S_{MT}) in the MCE spectrum corresponding to the seismic design spectrum in the fundamental period. In another way, the normalised records are scaled based on $S_a(T_1)$. The median spectrum of the 44 normalised records must be anchored to the MCE spectrum based on the fundamental period of the structure. The following curves illustrate the scaled spectra and response spectra of the 44 individual components of the normalised far-field records (Figure 3.2). Each archetype is executed with increasing intensity in each of the 44 individual records with growing intensity, the time-history nonlinear analyses must be continued to reach collapse or maximum story drift ratio that passed 10%, whichever comes first.

3.2.1 Non-Linear Static Analyses

To achieve the model's capacity curve, pushover analyses are performed for all archetypes. The overstrength factor and period-based ductility are calculated from the pushover curve. The gravity load combination for nonlinear static analyses and nonlinear dynamic analyses are defined as follows:

$$1.05D + 0.25L \quad (3.1)$$

Table 1: Summary of 22 earthquake events obtained from far-field record set and specified in the FEMA P695 [70].

No.	M	Year	Name	Station
1	6.7	1994	Northridge, USA	Beverly Hills
2	6.7	1994	Northridge, USA	Canyon Country
3	7.1	1999	Düzce , Turkey	Bolu
4	7.1	1999	Hector Mine, USA	Hector
5	6.5	1979	Imperial Valley, USA	Delta
6	6.5	1979	Imperial Valley, USA	El Centro Array
7	6.9	1995	Kobe, Japan	Nishi-Akashi
8	6.9	1995	Kobe, Japan	Shin-Osaka
9	7.5	1999	Kocaeli, Turkey	Düzce
10	7.5	1999	Kocaeli, Turkey	Arcelik
11	7.3	1992	Landers, USA	Y Fire Station
12	7.3	1992	Landers, USA	Coolwater
13	6.9	1989	Loma Prieta, USA	Capitola
14	6.9	1989	Loma Prieta, USA	Gilroy Array 3
15	7.4	1990	Manjil, Iran	Abbar
16	6.5	1987	Superstition Hills, USA	El Centro
17	6.5	1987	Superstition Hills, USA	Poe Road
18	7.0	1992	Cape Mendocino, USA	Rio Dell
19	7.6	1999	Chi-Chi, Taiwan	CHY101
20	7.6	1999	Chi-Chi, Taiwan	TCU045
21	6.6	1971	San Fernando, USA	LA
22	6.5	1976	Friuli, Italy	Tolmezzo

where D is the dead load and L is the live load.

The lateral distribution force for pushover analysis, F_x , at every story level is given in equation 3.2:

$$F_x \propto m_x \phi_{1,x} \quad (3.2)$$

where m_x is the mass at story x and $\phi_{1,x}$ is the first mode at story x. For this purpose, adaptive pushover analyses were applied to account for the changes of modes in every step, automatically. Maximum base shear could be achieved by capacity curve regarding roof level. Ultimate displacement, δ_u , is equal to the displacement of $0.8V_{max}$. The overstrength factor is most commonly used for archetypes as the ratio of

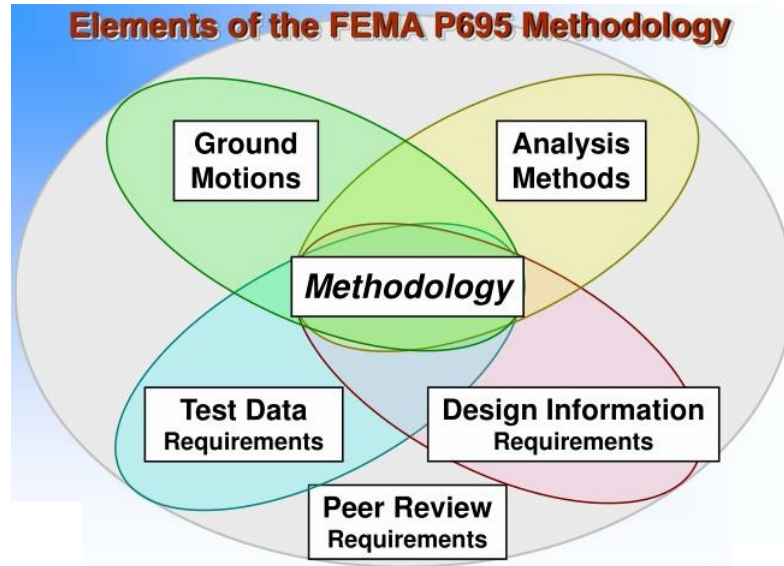


Figure 3.1: FEMA P695 elements [70].

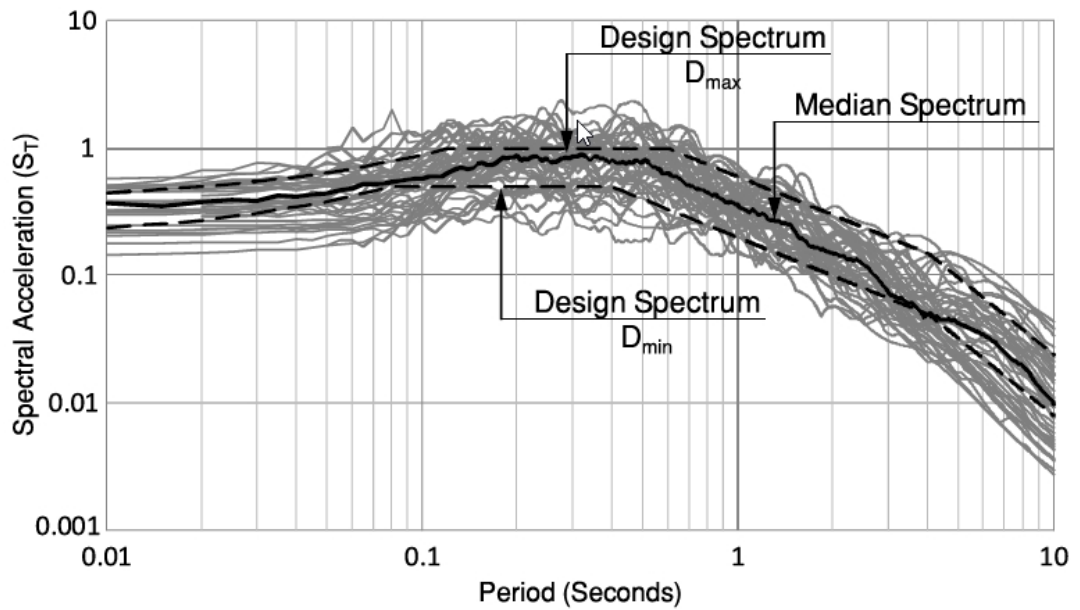


Figure 3.2: Response spectra of the 44 individual components of the normalized far-field record set and median spectrum [100].

maximum base shear strength to base shear design; that is:

$$\Omega = \frac{V_{max}}{V} \quad (3.3)$$

The period base ductility is determined as the ultimate roof drift divided by the effective roof displacement; that is:

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \quad (3.4)$$

The effective yield roof displacement was calculated according to the following formula:

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W} \frac{g}{4\pi^2} [\max(T, T_1)]^2 \quad (3.5)$$

where the coefficient C_0 is related to the first mode displacement to roof displacement of the system intended by ASCE41-06 [93], W is the weight of the structure, and g is the gravity constant. The coefficient C_0 is defined as follows:

$$C_0 = \phi_{1,r} \frac{\sum_1^N m_x \phi_{1,x}}{\sum_1^N m_x \phi_{1,x}^2} \quad (3.6)$$

where N is the number of stories, m_x is the mass of story x , and $\phi_{1,x}$ is the ordinate of the first mode at x and $\phi_{1,r}$ is the ordinate of the first mode at roof. The ordinates are obtained by eigenvalue analysis.

3.2.2 Incremental Dynamic Analyses and Establishing Median Collapse Capacity

3.2.2.1 Collapse Margin Ratio

At the first time, incremental dynamic analysis was presented by Vamvatsikos and Cornell in 2002 [94]. Until now, this method has been used by many researchers for different objectives and purposes (e.g. [95–100]). For calculation collapse margin ratio, the archetypes were considered in incremental dynamic analyses, and the median collapse capacity was calculated at the level of ground motion intensity in which 50% of the records were afforded the collapse of an archetype. The CMR is attained by the following relation:

$$CMR = \frac{S_{CT}}{S_{MT}} \quad (3.7)$$

where S_{CT} is the intensity of median collapse and S_{MT} is the intensity of MCE ground motion S_{MT} is determined as follows:

$$S_{MT} = S_{MS} \quad \text{if } T \leq T_s \quad (3.8)$$

$$S_{MT} = \frac{S_{M1}}{T} \quad \text{if } T > T_s \quad (3.9)$$

where S_{MS} and S_{M1} are the parameters of spectral response acceleration used to location category properties at the short period and 1 second period, respectively, and T is the fundamental period of the model elaborated by ASCE/SEI [101]. Besides, S_{MT} was obtained according to the D_{max} seismic design category at the fundamental period of archetype, mentioned by FEMA P695.

3.2.2.2 Adjusted Collapse Margin Ratio and Spectral Shape Factor

To achieve ACMR, spectral shape factor (SSF) was applied to the collapse margin ratio where the SSF was extracted from FEMA P695 by Table 7-1 [102]:

$$ACMR_i = SSF_i \times CMR_i \quad (3.10)$$

Spectral shape factor is considered as a procedural function of period-based ductility and the fundamental period of the structure and seismic design class. Generally, an increase in the fundamental period of structure or ductility leads to an increase in SSF, and vice versa [103]. SSF has a range between 1 and 1.61 for the $SDCD_{max}$ category.

3.2.3 The Definition of Collapse Uncertainty

In FEMA P695, four sources of uncertainty have expanded to evaluate the total system collapse uncertainty:

1- Record-to-record uncertainty, β_{RTR} which is defined as the variability in the response of archetypes under different ground motions. The amount of Record-To-Record uncertainty variability can be calculated as follows:

$$\beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.4 \quad (3.11)$$

where μ_T is the period-based ductility.

2- Design requirements uncertainty, β_{DR} which clarifies the completeness and robustness of the design requirements, and to what extent they protect unforeseen failure modes.

3- Test data uncertainty, β_{TD} , which demonstrates the exactness and validity of the test data used to specify the system.

4- Modelling uncertainty, β_{MDL} which is related to the collection and the wideness of archetypes has been studied. These variables are assumed to be statistically independent [104]. In FEMA P695, several tables are provided for calculating different uncertainty parameters. The total system collapse uncertainty factor is represented as follows:

$$\beta_{TOT} = \sqrt{(\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2)} \quad (3.12)$$

In this research, the values of β_{RTR} , β_{DR} , β_{TD} and β_{MDL} were set to 0.4, 0.2, 0.2 and 0.2, respectively. Finally, the total collapse uncertainty factor, β_{TOT} , was calculated to 0.53.

3.2.4 Corresponding Acceptable Values of ACMR and Overstrength Factor

The acceptable seismic performance design is defined by two items, first, if the probability of collapse for the MCE ground motion ACMR must not be beyond 20% for each archetype within a performance group, second, the probability of collapse for the MCE ground motion ACMR must not beyond 10%, on average, for a performance group.

$$ACMR_i \geq ACMR_{20\%} \quad (3.13)$$

$$\overline{ACMR_i} \geq ACMR_{10\%} \quad (3.14)$$

The overstrength factor could be obtained from the maximum overstrength average amount calculated for archetype groups, and it must be less than 1.5 times of R factor.

3.2.5 Uncertainties and Collapse Probabilities

The analytical method prescribed by FEMA P695 can be used to investigate the uncertainties in the collapse fragility curve. Systems that take advantage of a more rigorous design, comprehensive experimental data, and nonlinear models with thorough details have less uncertainty and can reach the required life safety (LS) performance level with a lower collapse margin ratio (CMR). Therefore, the computed CMRs for each specimen was compared with the acceptable values from FEMA P695 that incorporate all uncertainties. The total uncertainty is specified by combining the Record-To-Record Uncertainty (RTR), the Design Requirements Uncertainty (DR), the Test Data Uncertainty (TD), and the Nonlinear Modelling Uncertainty (MDL). Despite abundant experiments on RC specimens, there are many ambiguities in the behaviour of RC beams and columns. Research must account for other parameters that adversely affect the construction quality of RC buildings in the rating process. Consequently, the β_{TD} for the data analysis was obtained from Table 3 in FEMA P695. Following the lessons that were learned from past earthquakes and recent developments in the earthquake and structural engineering codes and standards, the highest rate is devoted to the design requirement uncertainties. However, considering the present shortages, the β_{DR} complies with Table 2 in FEMA P695. RC buildings are controlled by design requirements and details that restrict the likely failures. There are many unresolved issues regarding the nonlinear modelling of RC elements, which led to the selection of the β_{DR} in the nonlinear modelling of the initial specimens based on Table 4 in FEMA P695. Hence, considering all uncertainties, the total uncertainty is 0.275 to 0.95.

Table 2: Quality assessment for design requirements [102].

Completeness and Robustness	Confidence in Basis of Design Requirements		
	High	Medium	Low
High. Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	(A) Superior $\beta_{DR} = 0.10$	(B) Good $\beta_{DR} = 0.20$	(C) Fair $\beta_{DR} = 0.35$
Medium. Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	(B) Good $\beta_{DR} = 0.20$	(C) Fair $\beta_{DR} = 0.35$	(D) Poor $\beta_{DR} = 0.50$
Low. Questionable safeguards against unanticipated failure modes. Many important design and quality assurance issues are not addressed.	(C) Fair $\beta_{DR} = 0.35$	(D) Poor $\beta_{DR} = 0.50$	--

Table 3: Quality score for test data [102].

Completeness and Robustness	Confidence in Test Results		
	High	Medium	Low
High. Material, component, connection, assembly, and system behavior well understood and accounted for. All, or nearly all, important testing issues addressed.	(A) Superior $\beta_{TD} = 0.10$	(B) Good $\beta_{TD} = 0.20$	(C) Fair $\beta_{TD} = 0.35$
Medium. Material, component, connection, assembly, and system behavior generally understood and accounted for. Most important testing issues addressed.	(B) Good $\beta_{TD} = 0.20$	(C) Fair $\beta_{TD} = 0.35$	(D) Poor $\beta_{TD} = 0.50$
Low. Material, component, connection, assembly, and system behavior fairly understood and accounted for. Several important testing issues not addressed.	(C) Fair $\beta_{TD} = 0.35$	(D) Poor $\beta_{TD} = 0.50$	--

The calculation of the collapse probability and collapse margin ratio (CMR) can be remarkably affected by the frequency of the ground motion records. To account for the impacts of spectral shape, the collapse margin ratio (CMR) is normalised using the

Table 4: Assessment of the quality of the index archetype models [102].

Representation of Collapse Characteristics	Accuracy and Robustness of Models		
	High	Medium	Low
High. Index models capture the full range of the archetype design space and structural behavioral effects that contribute to collapse.	(A) Superior $\beta_{MDL} = 0.10$	(B) Good $\beta_{MDL} = 0.20$	(C) Fair $\beta_{MDL} = 0.35$
Medium. Index models are generally comprehensive and representative of the design space and behavioral effects that contribute to collapse.	(B) Good $\beta_{MDL} = 0.20$	(C) Fair $\beta_{MDL} = 0.35$	(D) Poor $\beta_{MDL} = 0.50$
Low. Significant aspects of the design space and/or collapse behavior are not captured in the index models.	(C) Fair $\beta_{MDL} = 0.35$	(D) Poor $\beta_{MDL} = 0.50$	--

spectral shape factor (SSF), which depends on the building's period.

$$SSF = \exp[\beta_1(\bar{\epsilon}_0(T) - \bar{\epsilon}(T)_{records})] \quad (3.15)$$

In the above equation, β_1 is a coefficient that depends on the nonlinear ductility capacity obtained from equation 3.16. Accordingly, ϵ is highly dependent on the site seismicity whose $SDC D_{max}$ is 1.5. $\bar{\epsilon}(T)_{records}$ is derived using equation 3.17.

$$\beta_1 = (0.14)(\mu_T - 1)^{0.42} \quad (3.16)$$

In the above equation, μ_T denotes the ductility factor whose value should be less than 8.

$$\bar{\epsilon}(T)_{records} = (0.6)(1.5 - T) \quad (3.17)$$

Figure 3.3 and Figure 3.4 show the mean ϵ values for the 44 records of the far-field site and the relation between the base ductility and β_1 , respectively.

Hence, the spectral shape factors (SSF) are affected by the fundamental period, ductility and seismic design classification, which is denoted as $SDC D_{max}$ herein.

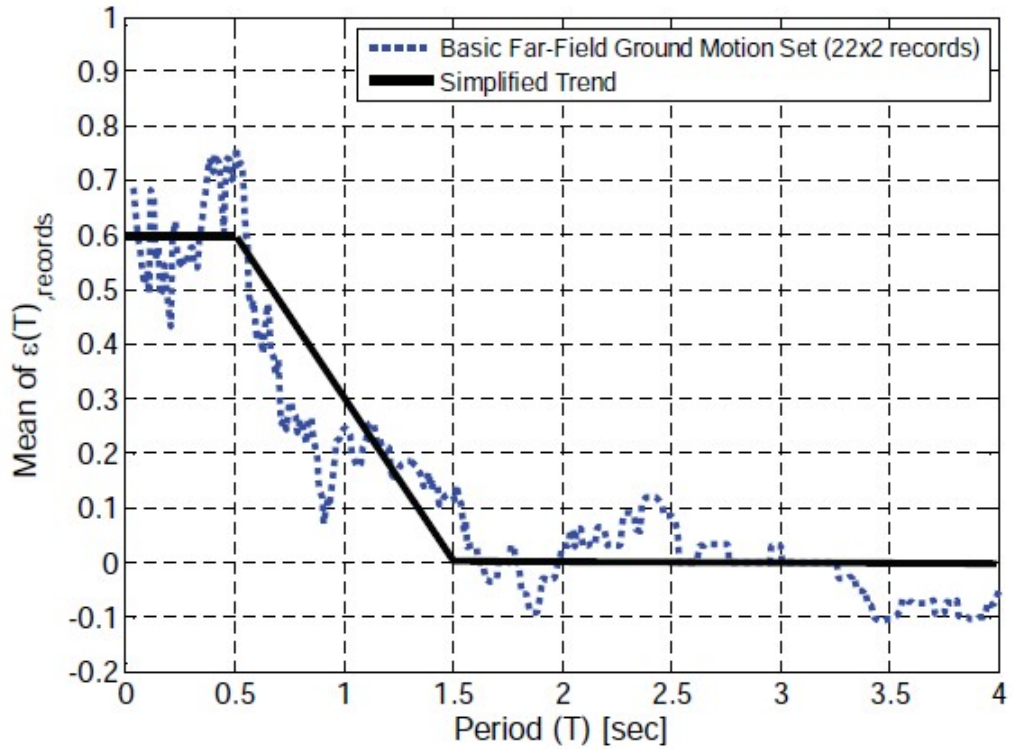


Figure 3.3: Mean ε values of the far-field ground motion set [70].

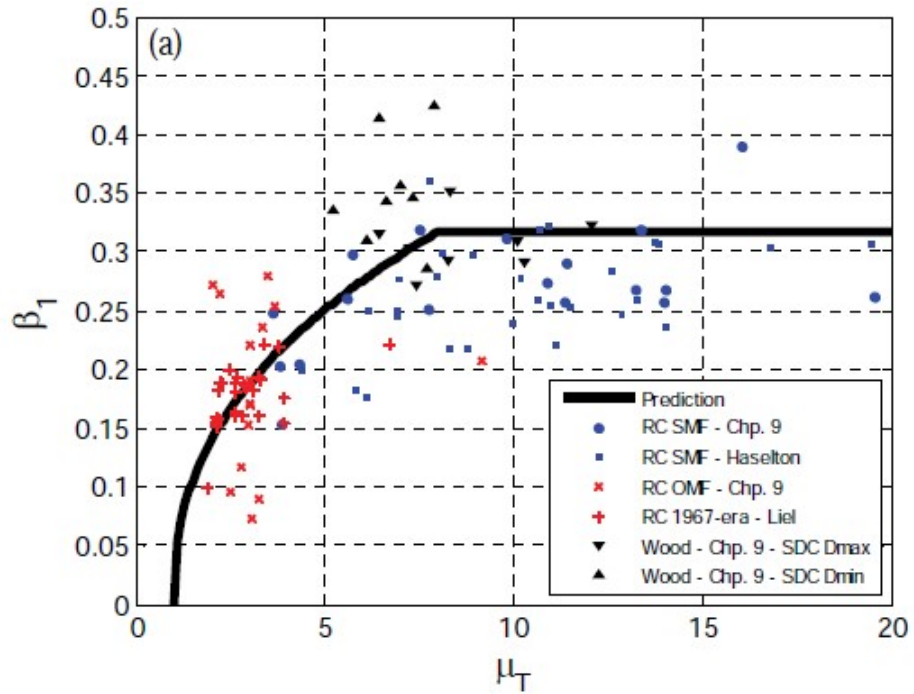


Figure 3.4: Comparison of the predicted and observed values of β_1 [70].

These factors are obtained in accordance with Table 5 in FEMA P695. Table 6 of FEMA P695 is based on the total collapse uncertainty β_{TOT} and the acceptable values of the collapse probability. The collapse probability is obtained based on the following equation according to ATC-63:

$$P(\text{Collapse} | S_{MT}) = \Phi\left(\frac{\ln(S_{MT}) - \ln(S_{CT} \times SSF)}{\beta_{TOT}}\right) \quad (3.18)$$

The acceptable performance of the FEMA P695 analytical method is based on the

Table 5: Spectral shape factor for patterns developed using $SDCD_{max}$ [102].

T (sec.)	Period-Based Ductility, μ_T							
	1.0	1.1	1.5	2	3	4	6	≥ 8
≤ 0.5	1.00	1.05	1.1	1.13	1.18	1.22	1.28	1.33
0.6	1.00	1.05	1.11	1.14	1.2	1.24	1.3	1.36
0.7	1.00	1.06	1.11	1.15	1.21	1.25	1.32	1.38
0.8	1.00	1.06	1.12	1.16	1.22	1.27	1.35	1.41
0.9	1.00	1.06	1.13	1.17	1.24	1.29	1.37	1.44
1.0	1.00	1.07	1.13	1.18	1.25	1.31	1.39	1.46
1.1	1.00	1.07	1.14	1.19	1.27	1.32	1.41	1.49
1.2	1.00	1.07	1.15	1.2	1.28	1.34	1.44	1.52
1.3	1.00	1.08	1.16	1.21	1.29	1.36	1.46	1.55
1.4	1.00	1.08	1.16	1.22	1.31	1.38	1.49	1.58
≥ 1.5	1.00	1.08	1.17	1.23	1.32	1.4	1.51	1.61

following collapse prevention goals: The collapse probability for the MCE hazard level should be 20% or less in one of each preliminary sample.

3.3 Soft Computing Techniques

Soft computing for engineering problems can provide the right solutions. Unlike hard computing, soft computing is particularly effective against predictions, sensitivities, uncertainties, and approximations. The pattern of soft computing is the same as the human mind. Techniques based on soft computing include: expert systems, fuzzy

Table 6: Allowable values of the ACMR [102].

Total System Collapse Uncertainty	Collapse Probability				
	5%	10% (ACMR _{10%})	15%	20% (ACMR _{20%})	25%
0.275	1.57	1.42	1.33	1.26	1.20
0.300	1.64	1.47	1.36	1.29	1.22
0.325	1.71	1.52	1.40	1.31	1.25
0.350	1.78	1.57	1.44	1.34	1.27
0.375	1.85	1.62	1.48	1.37	1.29
0.400	1.93	1.67	1.51	1.40	1.31
0.425	2.01	1.72	1.55	1.43	1.33
0.450	2.10	1.78	1.59	1.46	1.35
0.475	2.18	1.84	1.64	1.49	1.38
0.500	2.28	1.90	1.68	1.52	1.40
0.525	2.37	1.96	1.72	1.56	1.42
0.550	2.47	2.02	1.77	1.59	1.45
0.575	2.57	2.09	1.81	1.62	1.47
0.600	2.68	2.16	1.86	1.66	1.50
0.625	2.80	2.23	1.91	1.69	1.52
0.650	2.91	2.30	1.96	1.73	1.55
0.675	3.04	2.38	2.01	1.76	1.58
0.700	3.16	2.45	2.07	1.80	1.60
0.725	3.30	2.53	2.12	1.84	1.63
0.750	3.43	2.61	2.18	1.88	1.66
0.775	3.58	2.70	2.23	1.92	1.69
0.800	3.73	2.79	2.29	1.96	1.72
0.825	3.88	2.88	2.35	2.00	1.74
0.850	4.05	2.97	2.41	2.04	1.77
0.875	4.22	3.07	2.48	2.09	1.80
0.900	4.39	3.17	2.54	2.13	1.83
0.925	4.58	3.27	2.61	2.18	1.87
0.950	4.77	3.38	2.68	2.22	1.90

logic, artificial neural networks, machine learning, and etc. Theories and techniques of soft computing have existed, since the 1980s, but have become a special area of research and study in engineering. These methods are also used in other issues such as stock exchange, domestic and commercial industrial applications. With the advent of

low-cost and highly efficient digital processors and the reduction in the cost of memory chips, it is clear that techniques and applications of soft computing are still evolving. The following are three methods of soft computing.

3.3.1 Artificial Neural Networks

An artificial neural network (ANN) is part of a computing framework planned to re-enact the way human intelligence analyses and forms data. It is the establishment of artificial intelligence and tackles issues that would demonstrate inconceivable or troublesome by humans. ANN has self-learning abilities that empower them to create superior comes about as more information gets to be accessible.

ANN is a data processing model that is inspired by natural nervous organisms. Neurons work together to process information. Perceptron is the most popular part used in modelling. An input layer, an output layer, and one or more hidden layers form the components of a neural network system. The ANN consists of five parts: inputs (X_i), weights (W_i), sum function, activation function, and outputs. The total weight of the input component is considered with equation 3.19 as follows:

$$net_j = \sum_{i=1}^n w_{ij}X_i + b \quad (3.19)$$

where net_j is the total weight of the j^{th} neuron for the input obtained from the former layer with n neuron; w_{ij} is the weight; x_i is the output of an i^{th} neuron in the former layer, and b is a constant value or bias [105].

The activation function converts the input of the sum function to the output of neurons. The most common activation function is the sigmoid function, and tansig, logsig, and purelin are the most widely used. The output of neuron j is calculated using equation

3.20 as follows:

$$out_j = f(net_j) = \frac{1}{1 - \exp(-Knet_j)} \quad (3.20)$$

where K is the coefficient used to control the gradient of the non-linear section.

3.3.1.1 Training of ANN Model

Adjusting the weights by the perceptron to produce the appropriate output is called network training; this mechanism provides the ability to learn the network [43]. The multilayer perceptron (MLP), among the other learning algorithms of ANN, is the most popular. Moreover, the backpropagation learning algorithm (BP) is the most regularly utilised learning algorithm. The way toward learning algorithm comprises of two primary stages; in the process of moving forward flow from inputs to outputs, the difference between the output and the target value indicates the system error, the overall error is minimised by re-correcting the weights. The subsequent stage includes re-passing judgement on the biases and weights of the hidden and output layers to diminish the contrast between the ideal and determined outputs. Adjusting weights by using the general Delta law in the gradient descent on the error was accomplished. The preparation cycle of training is executed over and over to limit the total squared errors. Figure 3.5 illustrates a typical layout of a double hidden layer ANN. To work on the learning cycle of the BP neural network and to decrease the training time.

3.3.1.2 Validation and Test Data

To prevent over-fitting, if any parameters need to be adjusted, it is necessary to have a validation data set in addition to the training and test data sets. Training data sets are used to train different classifiers, validation data sets are used to compare their performance and decisions about which one to make. Performance characteristics include accuracy, sensitivity, characteristics.

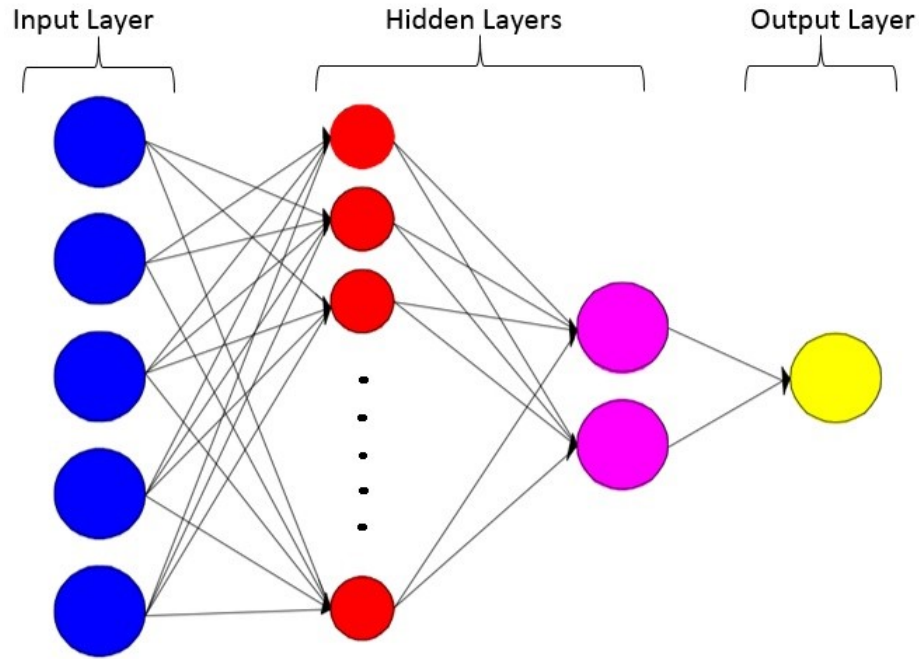


Figure 3.5: Double-hidden-layer perceptron ANN design.

A test set is therefore a set of examples used only to evaluate the performance of a well-defined classifier. So, the final model is used to predict the classification of samples in the test suite. These predictions are matched with the actual classification of the samples to evaluate the accuracy of the model.

3.3.2 Response Surface Method Theory

RSM methodology is a set of mathematical methods that determine the relationship between one or more response variables and several independent variables. This method was introduced by Box and Wilson in 1951, and is still used today as a tool for designing and predicting experiments. Response level methodology, or RSM for short, is a set of statistical techniques and applied mathematics for constructing experimental models. The goal in response level schemes is to optimise the response (output variable) which is affected by several independent variables (input variables). In each experiment, changes are made to the input variables to determine the causes of the changes in the response variable. In response level designs, constructing

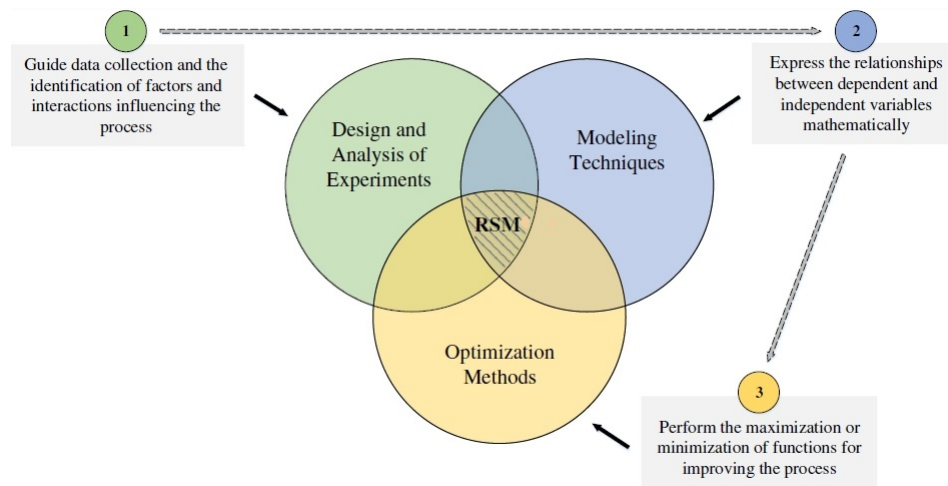


Figure 3.6: Overview of RSM Method [106].

response procedure models is an iterative process. As soon as an approximate model is obtained, it is tested whether the answer is satisfactory or not, if the answer is not confirmed, the estimation process starts again and more experiments are performed. In designing experiments, the goal is to identify and analyse the variables affecting the outputs with the least number of experiments. The methodology is a mathematical-statistical method for optimising the outputs of experiments (Figure 3.6). Basically, the response level methodology consists of the following steps: (a) two-factor experiments to screen for effective input variables; (b) Regression analysis to estimate the output fit function in terms of inputs, and (c) optimisation to determine the optimal levels of input variables.

The RSM follows two fundamental purposes; the first purpose is to optimise responses that are the function of different input parameters; the next purpose is to predict calculated equation between the process factors and estimated reactions. The RSM includes the following stages: identify the effective parameters in the function estimation; consider the acceptable limits of identifying parameters; desirable experimental design; carry out tests under the experimental design; measure the

responses; formation the mathematical model; Accurate control of the model using ANOVA and extracting the validity of constraints in the responses and optimising [107]. Different parameters may affect responses, and it is difficult to identify and control small shares of each. In this way, it is necessary to select these parameters with their main effects and controls. In this regard, it can be done in specific stages. In the first stage, the screening design to determine which factors, their interactions are more effective. In this equation, two-step fractional factorial designs are used. In the second stage, scientific studies in the field related to the identification of efficient parameter research and their limits can be used [108–110].

The regression model of the polynomial with the interaction and the main effects of parameters form the mathematical equations. The response level inputs can be expressed in equation 3.21:

$$f(x) = f(A, B, C, D, E) \quad (3.21)$$

$$f(x) = k_0 + k_1(A) + k_2(B) + k_3(C) + k_{11}(A^2) + k_{22}(B^2) + k_{33}(C^2) + k_{12}(AB) + k_{13}(AC) + k_{23}(BC) \quad (3.22)$$

3.3.2.1 ANOVA Analysis

Analysis of variance (ANOVA) is a set of statistical models and related estimation methods (such as "variation" between groups and between them) that are used to analyse the mean differences. ANOVA is based on the law of total variance, where the variance observed in a particular variable is divided into components attributed to different sources of variation. In its simplest form, ANOVA provides a statistical test of the equality of two or more population averages and thus extends the t-test beyond two means.

3.3.3 Adaptive Neuro-Fuzzy System (ANFIS)

An Adaptive Neuro-fuzzy Inference System or Adaptive Network-based Fuzzy Inference System (ANFIS) is an artificial neural network based on the Takagi-Sugeno fuzzy inference system. This technique was advanced in the early 1990s.

ANFIS is a type of ANN by which neural networks and fuzzy logic principles could be integrated. Actually, ANFIS is able to combine the advantages of fuzzy logic and ANN in the same way. As a result, ANFIS can properly train, construct and classify the data and also benefits from the capability to allow extraction of the fuzzy rules for numerical data or technical information and makes the principles. The ANFIS structure consists of five layers, which include input nodes, 3 intermediate-layer nodes, output nodes, and are directly related to each other. Each node has a function with adjustable or fixed parameters.

For a Sugeno-type fuzzy model of the first order, with x , and y inputs and an output f , the set of laws in the form If-Then are presented as follows:

Rule 1 and 2: if x is A_i and y is B_i , then $f_i = p_i x + q_i y + r_i$, $i=1$, and 2 where A_i and B_i are membership functions (for example, Gaussian, triangle functions), the type of membership function is changeable. The output parameters are p_i , q_i , and r_i . The typical structure of ANFIS is presented in Figure 3.7.

ANFIS performance is as follows:

Layer 1: the outputs of Layer 1 are the fuzzy membership grade of the inputs; they are provided by the following equations:

$$Q_i^1 = \mu_{A_i}(x) = e^{\frac{-(x-c)^2}{2\sigma^2}} \quad (3.23)$$

where x denotes input to the desired node, A_i is the linguistic labels. μ_{A_i} is the

appropriate membership function of the fuzzy set. Q_i stands for the output of each layer. The parameter set is $(x, c, \text{ and } \sigma)$.

Layer 2: In this layer, the fuzzy input operation is performed using the AND operator.

$$Q_i^2 = w_i = \mu_{A_i}(x)\mu_{B_i}(y) \quad i = 1, 2 \quad (3.24)$$

where μ_A and μ_B are the membership functions for linguistic labels. w_i is the output weight of each neuron.

Layer 3: N labelled nodes have a normalisation role to the firing strengths:

$$Q_i^3 = \bar{w}_i = \frac{w_i}{w_1 + w_2} \quad i = 1, 2 \quad (3.25)$$

Layer 4: normalised firing strength and a first-order polynomial are achieved by adaptive nodes.

$$Q_i^4 = \bar{w}_i f_i = \bar{w}_i(p_i x + q_i y + r_i) \quad (3.26)$$

where $\bar{w}$ is the output of the last layer, and (p_i, q_i, r_i) is the parameter set.

Layer 5: this node executes the summation of all received signals as follows:

$$Q_i^5 = \Sigma \bar{w}_i f_i = \frac{\Sigma w_i f_i}{\Sigma w_i} \quad (3.27)$$

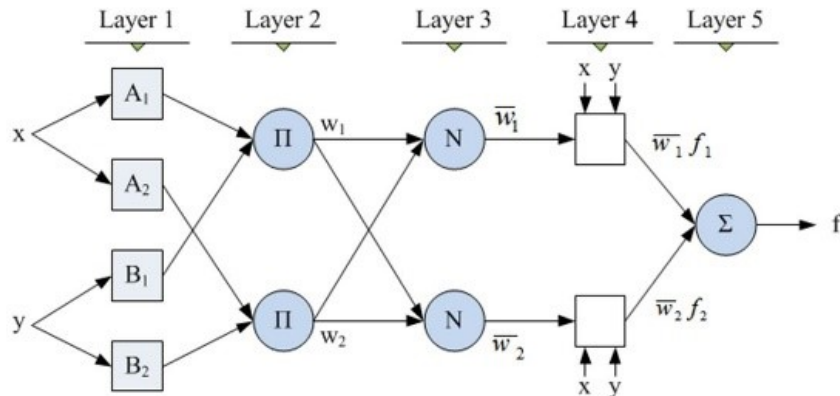


Figure 3.7: The architecture of the ANFIS model [111].

Chapter 4

RESEARCH METHODOLOGY AND FRAMEWORK

4.1 Introduction

In this chapter, the descriptions and characteristics of the studied archetype models for nonlinear analysis are presented. The seismic details of the loads and archetypes geometric properties, space frame, and perimeter frame, and the design assumptions are explained. First, initial models of reinforced concrete structures are introduced based on seismic codes. Seismic requirements control also is explained. Second, the properties used for steel braces in vulnerable reinforced concrete structures are also described. Retrofitting models are developed based on the shape of concentric braces. Finally, how to model the parameters in ANN, RSM method, and ANFIS method are presented to estimate the collapse margin ratio.

4.2 Overall Procedures

The concept of the ACMR and overstrength factor has been used to evaluate the stock reinforced concrete structures. To evaluate the ACMR, the concept of 10% probability of collapse of a model and 20% probability of collapse of a performance group is used. To calculate ACMR, a series of calculations, including adaptive pushover analysis, IDA analyses and fragility analysis are used to find the median collapse capacity. The IDA analyses are achieved using 44 far-field normalised and scaled records. On the other hand, in order to find the appropriate seismic design coefficients of reinforced concrete structures using steel bracing, the trial and error method is used to obtain the most suitable seismic factors. In order for the obtained

coefficients to be general and usable in all possible cases, the probability of collapse is obtained according to different values of uncertainty. In order to calculate the probability of collapse of the structure and also to calculate the ratio of collapse margin, FEMA P695 and ATC 63 instructions have been used by incremental dynamic analysis and pushover analysis. The collapse margin ratio coefficient is calculated using the results of incremental dynamic analysis and fragility analysis. The artificial intelligence models are used to efficiently estimate the findings of incremental dynamic analysis. Sensitisation and sorting of important parameters is also be done based on ANOVA analysis, the final formula is extracted in the calculation of the collapse margin coefficient.

4.3 Overview of Study Procedure

Reinforced concrete (RC) buildings in Iran comprise a large part of residential buildings. Until now, four editions of the Earthquake Resistance Building Design Regulations (Standard No. 2800) have been published. Its use in the design of buildings in Iran is mandatory. Accordingly, designed residential buildings must have a level of life safety. The dimensions and size of the elements in the RC buildings designed according to the third edition were impressive compared to the second edition. In the first section of this study, the seismic evaluation modelling of these buildings has been done. A set of 51 stock RC archetypes in the form of 17 performance groups (1- to 12-story) were evaluated by 2244 incremental dynamic analyses. The concept of ACMR (derived from the concept of the probability of a structure collapsing) is used for seismic assessment of existing structures. Subsequently, 44 far-field records were used in incremental dynamic analysis. Examining the results, it can be seen that structures up to 6 stories designed according to the second edition are vulnerable to earthquakes. Therefore, the second part of this chapter considers their retrofitting by steel braces to solve this problem. The

possibility of adding concentric steel braces was investigated with different configurations. To find the optimal design point in strengthening RC structures, the concept of the probability of collapse of a structure has also been investigated due to a set of far-field records. In addition, appropriate seismic design coefficients have been investigated and introduced. A total of 45 archetypes are defined in the form of 15 performance groups. The numbers of 19,800 nonlinear dynamic analyses were performed. The probability of collapse of all possible uncertainties was investigated based on the collapse margin ratio. In addition, the best seismic coefficients of steel braces were found using trial and error. The probability of collapse is the result of the obtained CMR ratio. In the third part, a set of analyses including nonlinear static analysis (NSA), incremental dynamic analysis (IDA), along fragility analysis, which is usually time-consuming must be done to obtain CMR. To solve this problem and achieve a fast and efficient method for estimating CMR, artificial neural network (ANN), response surface methodology (RSM), and adaptive fuzzy neural inference system (ANFIS) have been introduced in current research. Accordingly, using NSA results, an attempt was made to find a rapid and efficient way to extract CMR. For this purpose, approximately the results of 5016 IDA analyses were applied based on 114 archetypes. In this regard, five parameters were used as independent and desired inputs of the system. In addition, CMR is considered the output of the systems. In the RSM method, ANOVA analysis is also applied to obtain important parameters. Subsequently, the results of the estimator models and formulas are extracted and the important parameters are sorted. The results of the study are provided and discussed in the next chapter.

4.4 Development of Stock RC Archetypes Designed Based on Iranian Seismic Code

Standard No. 2800 has been published in Iran since 1986. It provides the necessary parameters for the seismic design of ordinary structures. Accordingly, respective structures should remain stable in severe ground motions with a probability of less than 10 percent in 50 years; and only reduced structural detriment due to moderate and low-intensity earthquakes should be expected with a probability of 99.5 percent in 50 years. The maximum height of intermediate RC buildings is limited to 50 meters.

To cover the status of structures, seventeen groups of RC frame archetypes have been studied. Different heights consist of 1, 3, 6, 9, and 12 stories which make up about 80-90% of the total buildings stock in Iran [112], are concentrated. Various gravity loads and different span numbers have been considered. Figure 4.1 shows typical of residential user frame where located in symmetric plan (Figure 4.2). The 51 archetype frames were chosen and designed by using ETABS software, to model the different possible structural and geometric features, that could be estimated by the wide range of available RC buildings. Intermediate RC frames with the four different classes of height, three types of length, and location such as space frame and perimeter frame were studied. The important factor for residential buildings is 1, soil type III and R factor for RC buildings with intermediate ductility is equal to 8 and 7 according to second and third editions, respectively. The high-risk seismic zone factor was assumed for all frame design. Table 7 shows the calculation of design base shear coefficients.

4.4.1 Comparison of the Second and Third Editions of the Standard No. 2800

In the equivalent static methods, the base shear coefficient can be calculated using the following formula based on standard 2800:

$$C = \frac{ABI}{R} \quad (4.1)$$

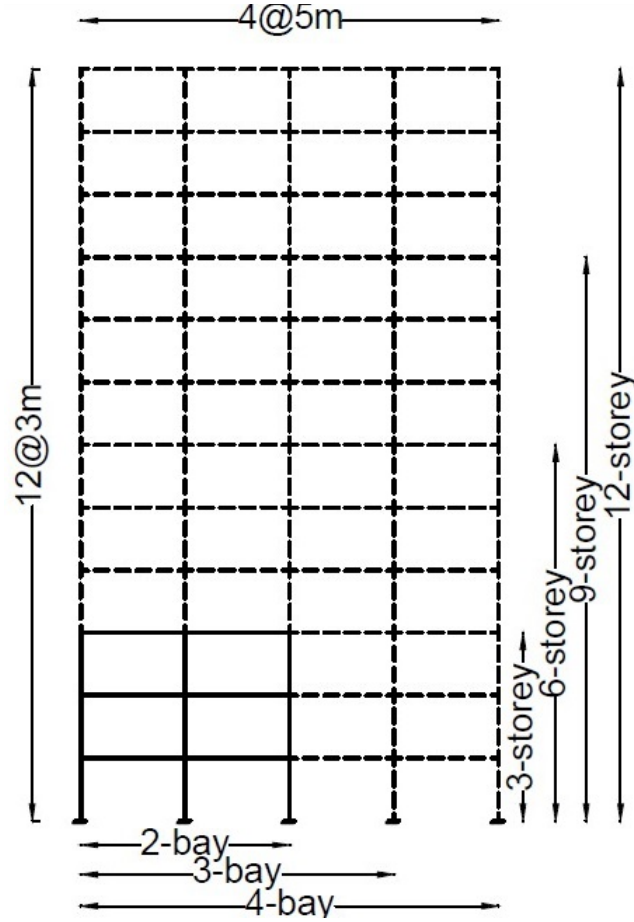


Figure 4.1: Archetypes frame.

where A is earthquake design acceleration, B is the reflection factor, I is the important factor of the building and R is the behaviour factor or response modification factor. According to the second edition, the inter-story drift ratio should be less than $0.03/R$. In the same way, the third edition introduced real lateral story drift, Δ_M , and design lateral story drift, Δ_w :

$$\Delta_M = 0.7R\Delta_w \quad (4.2)$$

The real lateral story drift in the centre of mass of storey by considering the P-Delta effect, Δ_M could be obtained as follows:

$$\Delta_M < 0.025h_i \quad \text{if } T < 0.7s \quad (4.3)$$

$$\Delta_M \leq 0.02h_i \quad \text{if } T \geq 0.7s \quad (4.4)$$

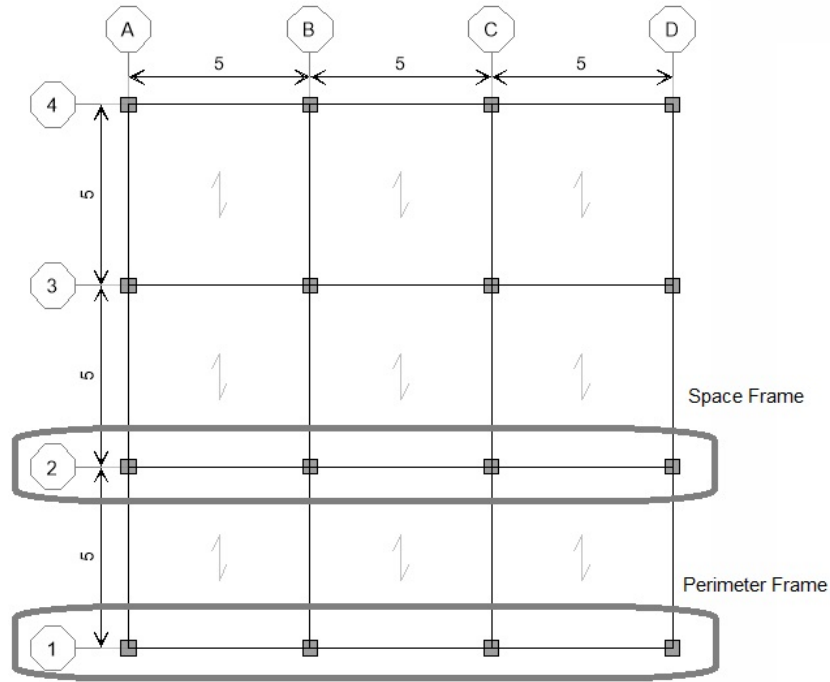


Figure 4.2: Sample plan of 3-bay archetypes with showing space frame and perimeter frame.

where h_i is the height of the story, T is the period of the structure. Another important parameter, considered in the third edition was the correction of inertia moment of the cross-section for each element by the following equation:

$$I_{cracked}^b = 0.35I_g^b \quad (4.5)$$

$$I_{cracked}^c = 0.7I_g^c \quad (4.6)$$

where $I_{cracked}^b$, $I_{cracked}^c$, I_g^b , and I_g^c are the cracked inertia moment of the section of the beams and columns, the gross moment of inertia of the section of the beam and column elements, respectively. This factor can have a large change in the dimensions of the elements and the amount of rebar in the reinforced concrete structure.

4.4.2 Archetypes and Assumptions

The modelling, and how to evaluate seismic performance is given in the flowchart as shown in Figure 4.3. Archetypes were categorised by different performance groups.

Table 7: Base shear design coefficient.

Number of stories	Second edition (Standard 2800-99)	Third edition (Standard 2800-05)
1	0.094	0.1179
3	0.094	0.1179
6	0.094	0.1179
9	0.084	0.1052
12	0.072	0.0912

Tables 10, 11, and 12 illustrate a summary of archetypes in each defined groups. PG symbolises the term “Performance Group” in this table. All nonlinear analyses were performed using the SeismoStruct V7.0.6. SeismoStruct is a finite element program for structural modelling, proficient in calculating the large displacement behaviour of structures under static or dynamic forces. In all analyses, P- Δ effects were considered by including geometric non-linearity. All members have been modelled with inelastic force-based frame elements (infrmFB). Five integration sections for each element have been used as subdivision, and the cross-section of members has been subdivided into 150 fibres, Figure 4.4 shows a schematic division of section and fiber elements in SeismoStruct, and mechanical component of the materials used for the archetype RC frame is shown in Figure 4.5. Also, Figure 4.6 displays the material property of concrete con-ma presented by Mander et al. [113] and rebar type of stl-bl. Moreover, the properties of materials are shown in Tables 8 and 9. The structural properties of archetypes are given in the appendix part.

Archetypes frame in general form is shown in Figure 4.1. Figure 4.2 shows a typical space frame and perimeter frame in the plan.

Table 8: Properties of concrete sections.

Property	Value	Unit
Density	240	Kg/m^3
Modulus of elasticity	253100	Kg/cm^2
Poisson's ratio	0.2	-
Strength class	C20	-
Bending reinf. yield stress, f_y	4000	Kg/cm^2
Shear reinf. yield stress, f_{ys}	4000	Kg/cm^2

Table 9: Properties of steel sections.

Property	Value	Unit
Density	798.1	Kg/m^3
Modulus of elasticity	2039000	Kg/cm^2
Poisson's ratio	0.3	-
Minimum yield stress, f_y	2400	Kg/cm^2
Minimum tensile strength, f_u	3600	Kg/cm^2

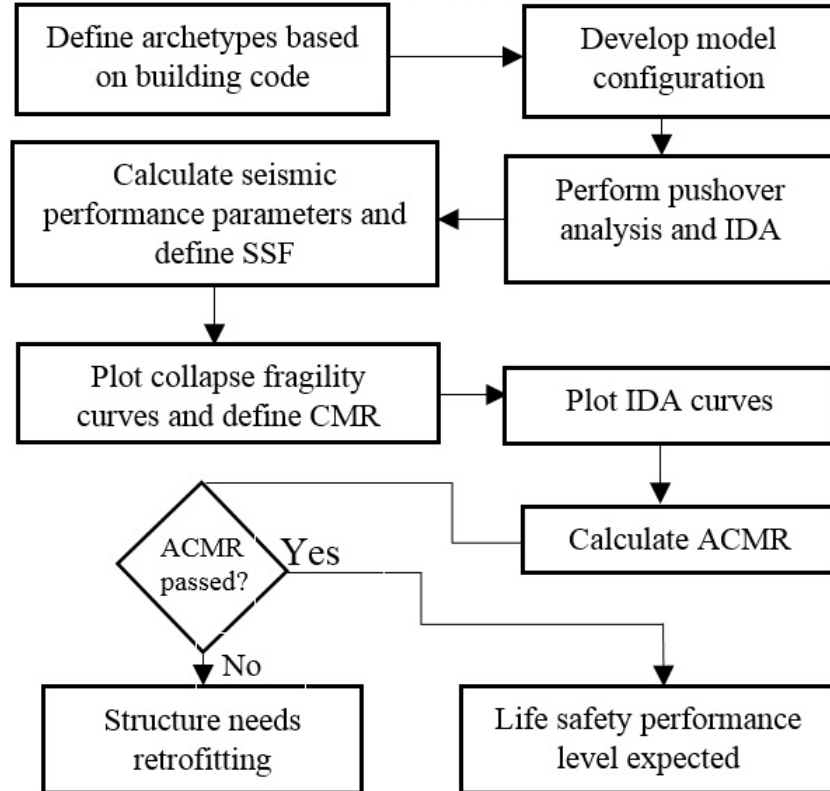


Figure 4.3: Flowchart of methodology [114].

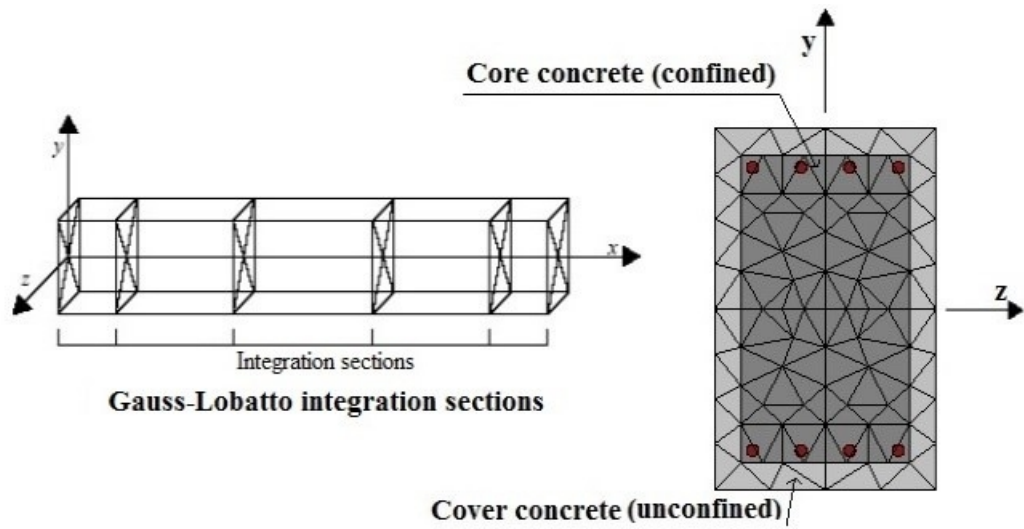


Figure 4.4: Schematic division of section and fiber elements in SeismoStruct.

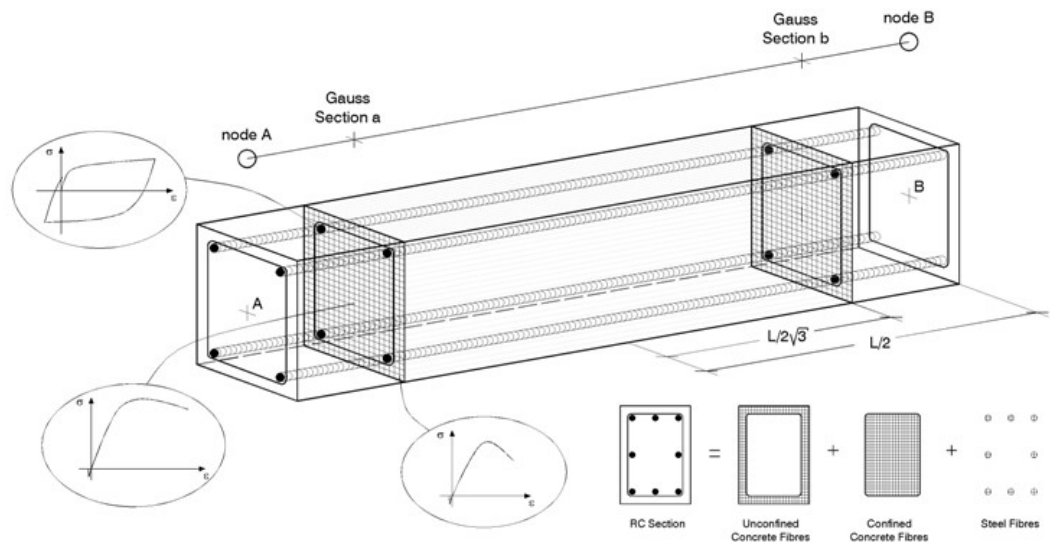
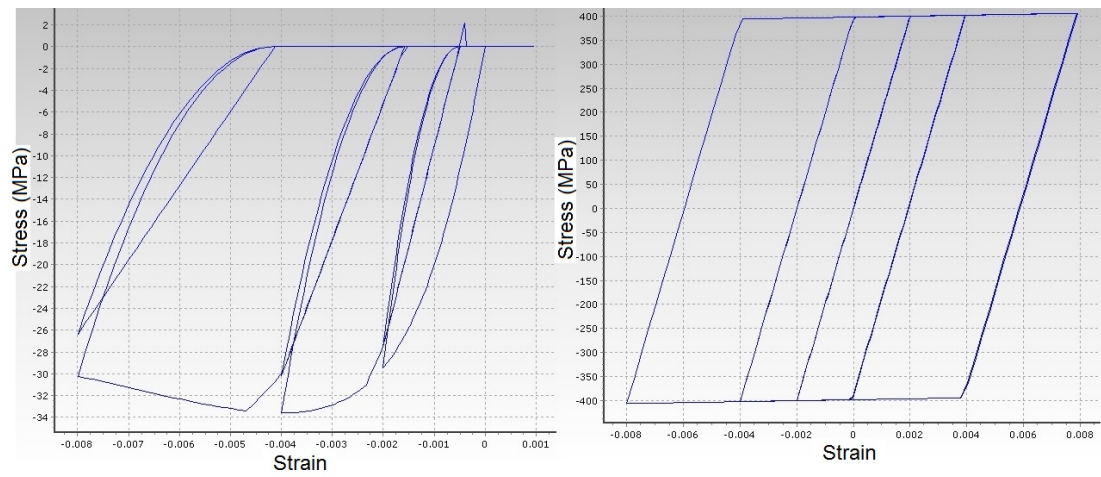


Figure 4.5: Mechanical component of the materials used for the archetype RC frames.



(a) Stress-strain curve for concrete (con_ma)

(b) Stress-strain curve for steel (stl_bl)

Figure 4.6: The behaviour of materials.

Table 10: Properties of archetypes PG 1-6.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Design Standard
PG-1	3st-2b-S-v3	3	9	2	10	High (space frame)	No. 2800-05
	3st-3b-S-v3	3	9	3	15	High (space frame)	No. 2800-05
	3st-4b-S-v3	3	9	4	20	High (space frame)	No. 2800-05
PG-2	6st-2b-S-v3	6	18	2	10	High (space frame)	No. 2800-05
	6st-3b-S-v3	6	18	3	15	High (space frame)	No. 2800-05
	6st-4b-S-v3	6	18	4	20	High (space frame)	No. 2800-05
PG-3	9st-2b-S-v3	9	27	2	10	High (space frame)	No. 2800-05
	9st-3b-S-v3	9	27	3	15	High (space frame)	No. 2800-05
	9st-4b-S-v3	9	27	4	20	High (space frame)	No. 2800-05
PG-4	12st-2b-S-v3	12	36	2	10	High (space frame)	No. 2800-05
	12st-3b-S-v3	12	36	3	15	High (space frame)	No. 2800-05
	12st-4b-S-v3	12	36	4	20	High (space frame)	No. 2800-05
PG-5	3st-2b-P-v3	3	9	2	10	Low (perimeter frame)	No. 2800-05
	3st-3b-P-v3	3	9	3	15	Low (perimeter frame)	No. 2800-05
	3st-4b-P-v3	3	9	4	20	Low (perimeter frame)	No. 2800-05
PG-6	6st-2b-P-v3	6	18	2	10	Low (perimeter frame)	No. 2800-05
	6st-3b-P-v3	6	18	3	15	Low (perimeter frame)	No. 2800-05
	6st-4b-P-v3	6	18	4	20	Low (perimeter frame)	No. 2800-05

Table 11: Properties of archetypes PG 7-12.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Design Standard
PG-7	9st-2b-P-v3	9	27	2	10	Low (perimeter frame)	No. 2800-05
	9st-3b-P-v3	9	27	3	15	Low (perimeter frame)	No. 2800-05
	9st-4b-P-v3	9	27	4	20	Low (perimeter frame)	No. 2800-05
PG-8	12st-2b-P-v3	12	36	2	10	Low (perimeter frame)	No. 2800-05
	12st-3b-P-v3	12	36	3	15	Low (perimeter frame)	No. 2800-05
	12st-4b-P-v3	12	36	4	20	Low (perimeter frame)	No. 2800-05
PG-9	3st-2b-S-v2	3	9	2	10	High (space frame)	No. 2800-99
	3st-3b-S-v2	3	9	3	15	High (space frame)	No. 2800-99
	3st-4b-S-v2	3	9	4	20	High (space frame)	No. 2800-99
PG-10	6st-2b-S-v2	6	18	2	10	High (space frame)	No. 2800-99
	6st-3b-S-v2	6	18	3	15	High (space frame)	No. 2800-99
	6st-4b-S-v2	6	18	4	20	High (space frame)	No. 2800-99
PG-11	9st-2b-S-v2	9	27	2	10	High (space frame)	No. 2800-99
	9st-3b-S-v2	9	27	3	15	High (space frame)	No. 2800-99
	9st-4b-S-v2	9	27	4	20	High (space frame)	No. 2800-99
PG-12	12st-2b-S-v2	12	36	2	10	High (space frame)	No. 2800-99
	12st-3b-S-v2	12	36	3	15	High (space frame)	No. 2800-99
	12st-4b-S-v2	12	36	4	20	High (space frame)	No. 2800-99

Table 12: Properties of archetypes PG 13-17.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Design Standard
PG-13	3st-2b-P-v2	3	9	2	10	Low (perimeter frame)	No. 2800-99
	3st-3b-P-v2	3	9	3	15	Low (perimeter frame)	No. 2800-99
	3st-4b-P-v2	3	9	4	20	Low (perimeter frame)	No. 2800-99
PG-14	6st-2b-P-v2	6	18	2	10	Low (perimeter frame)	No. 2800-99
	6st-3b-P-v2	6	18	3	15	Low (perimeter frame)	No. 2800-99
	6st-4b-P-v2	6	18	4	20	Low (perimeter frame)	No. 2800-99
PG-15	9st-2b-P-v2	9	27	2	10	Low (perimeter frame)	No. 2800-99
	9st-3b-P-v2	9	27	3	15	Low (perimeter frame)	No. 2800-99
	9st-4b-P-v2	9	27	4	20	Low (perimeter frame)	No. 2800-99
PG-16	12st-2b-P-v2	12	36	2	10	Low (perimeter frame)	No. 2800-99
	12st-3b-P-v2	12	36	3	15	Low (perimeter frame)	No. 2800-99
	12st-4b-P-v2	12	36	4	20	Low (perimeter frame)	No. 2800-99
PG-17	1st-2b-S-v2	1	3	2	10	High (space frame)	No. 2800-99
	1st-3b-S-v2	1	3	3	15	High (space frame)	No. 2800-99
	1st-4b-S-v2	1	3	4	20	High (space frame)	No. 2800-99

With the evaluation and identification of the seismic weakness of short and medium height archetypes designed based on the Standard 2800-99, as will be discussed in the next chapter (section 5.2), the modelling of these structures using steel braces is defined in the following section.

4.5 Properties of Steel Braced Archetypes

The basic structural archetypes should provide suitable nonlinear characteristics of the seismic system. The characteristics are determined in this study. Different probable heights of the system and consequently different fundamental periods are considered for 1, 3 and 6-story frames. In addition, different bays are considered from 2 to 4 bays. Due to the significant properties of the seismic and gravity mass, two sets of frames including space frames and perimeter frames are specified. In all the 2-dimensional archetypes, the height of the stories is 3 m, and the length of the bay is supposed to be constant at 5 m. The RC archetypes were designed using ETABS software. Intermediate-sway was assumed for the ductility features, the reinforced concrete design of the structure is based on the Iranian National Building Code, and the seismic parameters were determined based on the second edition of standard No. 2800. The weights of the systems were assumed to consist of a total dead load plus a 20% live load.

4.5.1 Arrangement of the Steel Braced Archetypes

Bracing elements are used to improve the seismic performance of building stocks, and truss elements are applied to RC archetypes. For the strengthening archetypes, three types of concentric braces, cross, chevron, and diagonal, have been used and are labelled X, V, and D, respectively (Figure 4.7). In the braced frames understudy, the R factor is assumed to be 10. In the Seismic Provisions of standard No. 2800, the slenderness ratios of bracing members are defined as follows:

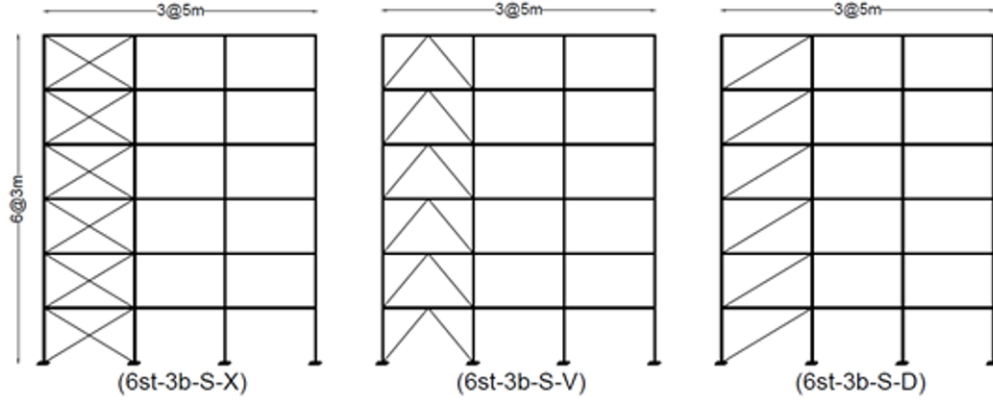


Figure 4.7: Schematic of 6-story and 3-bay space frame archetypes.

$$\frac{kl}{r} \leq \frac{6025}{\sqrt{F_y}} \quad (4.7)$$

where r is the radius of the gyration, KL is the effective length, and F_y is the yield stress of the members. In chevron bracing members, the elements should be designed to withstand 1.5 times the base shear. Also, Tables 13, 14, and 15 show a summary of archetypes in each defined groups for steel braced archetypes. The structural properties of braced archetypes are given in the appendix part.

Table 13: Properties of archetypes PG 18-22.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Bracing Type
PG-18	1st-2b-S-X	1	3	2	10	High (space frame)	Cross
	1st-3b-S-X	1	3	3	15	High (space frame)	Cross
	1st-4b-S-X	1	3	4	20	High (space frame)	Cross
PG-19	3st-2b-S-X	3	9	2	10	High (space frame)	Cross
	3st-3b-S-X	3	9	3	15	High (space frame)	Cross
	3st-4b-S-X	3	9	4	20	High (space frame)	Cross
PG-20	6st-2b-S-X	6	18	2	10	High (space frame)	Cross
	6st-3b-S-X	6	18	3	15	High (space frame)	Cross
	6st-4b-S-X	6	18	4	20	High (space frame)	Cross
PG-21	3st-2b-P-X	3	9	2	10	Low (perimeter frame)	Cross
	3st-3b-P-X	3	9	3	15	Low (perimeter frame)	Cross
	3st-4b-P-X	3	9	4	20	Low (perimeter frame)	Cross
PG-22	6st-2b-P-X	6	18	2	10	Low (perimeter frame)	Cross
	6st-3b-P-X	6	18	3	15	Low (perimeter frame)	Cross
	6st-4b-P-X	6	18	4	20	Low (perimeter frame)	Cross

Table 14: Properties of archetypes PG 23-27.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Bracing Type
PG-23	1st-2b-S-V	1	3	2	10	High (space frame)	Chevron
	1st-3b-S-V	1	3	3	15	High (space frame)	Chevron
	1st-4b-S-V	1	3	4	20	High (space frame)	Chevron
PG-24	3st-2b-S-V	3	9	2	10	High (space frame)	Chevron
	3st-3b-S-V	3	9	3	15	High (space frame)	Chevron
	3st-4b-S-V	3	9	4	20	High (space frame)	Chevron
PG-25	6st-2b-S-V	6	18	2	10	High (space frame)	Chevron
	6st-3b-S-V	6	18	3	15	High (space frame)	Chevron
	6st-4b-S-V	6	18	4	20	High (space frame)	Chevron
PG-26	3st-2b-P-V	3	9	2	10	Low (perimeter frame)	Chevron
	3st-3b-P-V	3	9	3	15	Low (perimeter frame)	Chevron
	3st-4b-P-V	3	9	4	20	Low (perimeter frame)	Chevron
PG-27	6st-2b-P-V	6	18	2	10	Low (perimeter frame)	Chevron
	6st-3b-P-V	6	18	3	15	Low (perimeter frame)	Chevron
	6st-4b-P-V	6	18	4	20	Low (perimeter frame)	Chevron

Table 15: Properties of archetypes PG 28-32.

Performance Group	Model ID	Number of storeys	Height (m)	Number of bays	Width (m)	Gravity load	Bracing Type
PG-28	1st-2b-S-D	1	3	2	10	High (space frame)	Diagonal
	1st-3b-S-D	1	3	3	15	High (space frame)	Diagonal
	1st-4b-S-D	1	3	4	20	High (space frame)	Diagonal
PG-29	3st-2b-S-D	3	9	2	10	High (space frame)	Diagonal
	3st-3b-S-D	3	9	3	15	High (space frame)	Diagonal
	3st-4b-S-D	3	9	4	20	High (space frame)	Diagonal
PG-30	6st-2b-S-D	6	18	2	10	High (space frame)	Diagonal
	6st-3b-S-D	6	18	3	15	High (space frame)	Diagonal
	6st-4b-S-D	6	18	4	20	High (space frame)	Diagonal
PG-31	3st-2b-P-D	3	9	2	10	Low (perimeter frame)	Diagonal
	3st-3b-P-D	3	9	3	15	Low (perimeter frame)	Diagonal
	3st-4b-P-D	3	9	4	20	Low (perimeter frame)	Diagonal
PG-32	6st-2b-P-D	6	18	2	10	Low (perimeter frame)	Diagonal
	6st-3b-P-D	6	18	3	15	Low (perimeter frame)	Diagonal
	6st-4b-P-D	6	18	4	20	Low (perimeter frame)	Diagonal

4.6 Using Soft Computing Techniques to Estimate Collapse Margin Ratio

The main purpose of FEMA P695, and following ATC-63, is to provide a general evaluation of the seismic factors in various lateral load-resisting systems by determining their collapse capacity. Therefore, the seismic R factor, deflection amplification factor (C_d), and over-strength factor (Ω_0) are estimated in compliance with this procedure. This methodology includes a summary of design conditions, component testing information, and specialised design experience. The archetypes are meant to incorporate a thorough range of design parameters, including building height, seismic design classification, seismic weight in the lateral load resistance system, the number of spans, as well as their width and story height.

Based on previous work done, several types of research have been accomplished to predict the CMR or IDA results using pushover analysis results. Nevertheless, no studies have been performed to estimate the CMR parameter using soft computation techniques. To fill the gap, this study was conducted to evaluate the efficiency of the ANN, RSM, and ANFIS approaches in accurately predicting CMR, as described by FEMA P695. It has been performed to reduce the computational time and estimate the desired results without the need for IDA analyses. The accuracy and capability of each technique are compared by statistical indices. Accordingly, the input variables and significant parameters are extracted and sorted according to their importance.

In order to use the CMR parameter more efficiently and more effectively, and with the aim of faster estimation of this parameter using only the pushover analysis, the soft computing methods according to the flowchart shown in Figure 4.8 has been illustrated. Subsequently, 114 various RC structure models were designed to define

the seismic parameters according to FEMA P695 methodology. Archetypes with different heights (1, 3, 6, 9, and 12-storey), different gravity loads (low and high), and the number of spans (2 to 4) have been considered. Archetypes with five height classes, three lengths (2, 3, and 4 spans), and two location groups (space frames and perimeter frames) were considered. The archetypes were designed according to Standard 2800. The models were intended for residential use in high earthquake-prone areas. The IDA and pushover analyses were achieved by SeismoStruct v7.0.6. Totally, 5016 IDA analyses based on FEMA P695 procedure have been performed. The IDA analyses also used 22 pairs of far-field earthquake specified records, taken from the website of the Pacific Engineering Earthquake Research (PEER) Centre, the seismographic base of the NGA, 2006, and the records were both normalised and scaled based on FEMA P695 procedure. SeismoStruct is based on the finite element method for structural modelling, which is appropriate to compute large displacements of models exposed to dynamic and static loads. The effects of P-delta were considered in the analyses of accounting for the geometric nonlinearities. The elements were modelled by inelastic force-based frame elements (infrmFB). Five integration segments were utilised as parts for every element, and the member cross-sections were divided into a hundred and fifty fibres.

In the following, the ANN, RSM, and ANFIS models are built based on computational data, and the efficiency of each model is evaluated to predict the individual CMR features. The input parameters obtained from the pushover analysis and seismic site effect include: 1- The fundamental period of the structure (T_1) 2- Roof yield displacement ($\delta_{y,eff}$) 3- Maximum displacement equivalent with 80% maximum base shear (δ_u) 4- Over strength factor (Ω) 5- S_{MT} , obtained from the spectrum acceleration related to MCE. These factors can cover all the results of the

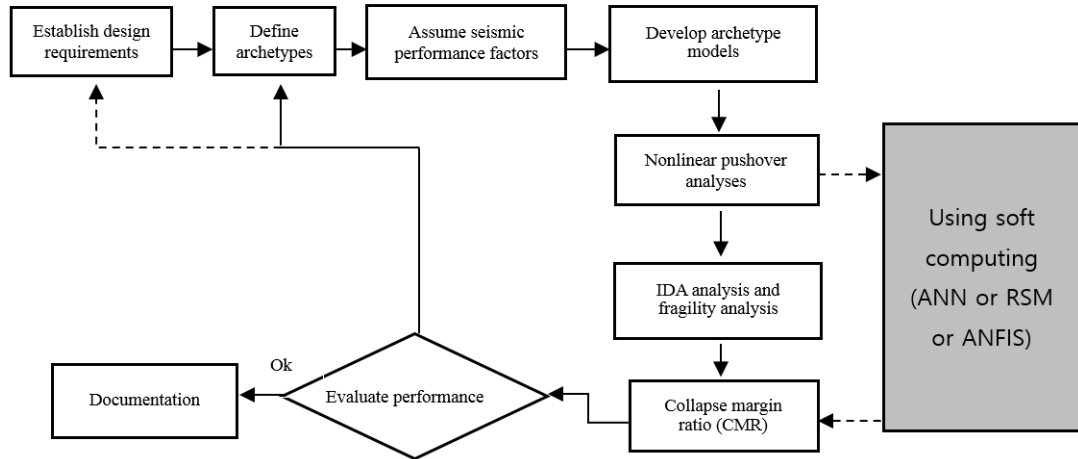


Figure 4.8: Flowchart of FEMA P695 methodology and application of soft computing.

pushover curve because the effective roof yield displacement includes the maximum base shear and structural weight features. Displacement ductility is also acquired by the relation of the ultimate displacement and the yield displacement of the structure. Another parameter in the estimation of CMR is S_{MT} obtained from the MCE spectrum related to the fundamental period of the structure. The S_{CT} is the moderate capacity of the structure collapse, which is derived from the results of IDA results and the fragility analysis.

4.6.1 Artificial Neural Network Structure

The ANN model is separated into three subsets, training, validation, and testing using the available data. A typical issue during training is excessive adoption, and it happens when the network remembers the training category has not been able to summarise the new inputs. This provides a generally little value of error in the training set; while new data is available on the network, a much greater error is created. The early stop feature is widely and efficiently used to overcome the issue of excessive adoption. Moreover, the error in the sets is checked during the training procedure. When the confirmation error increases in the number of repetitions, the training is stopped, and the weights and biases are returned to the minimum

confirmation error. Figure 4.9 shows the optimal point at epoch 14, and the best validation performance is 0.0545. The Levenberg-Marquardt algorithm is used as a learning algorithm. In this research, the Matlab version R2018a was utilised to build the ANN model.

In this research, 70% of the total samples are applied for training, and 30% is equally

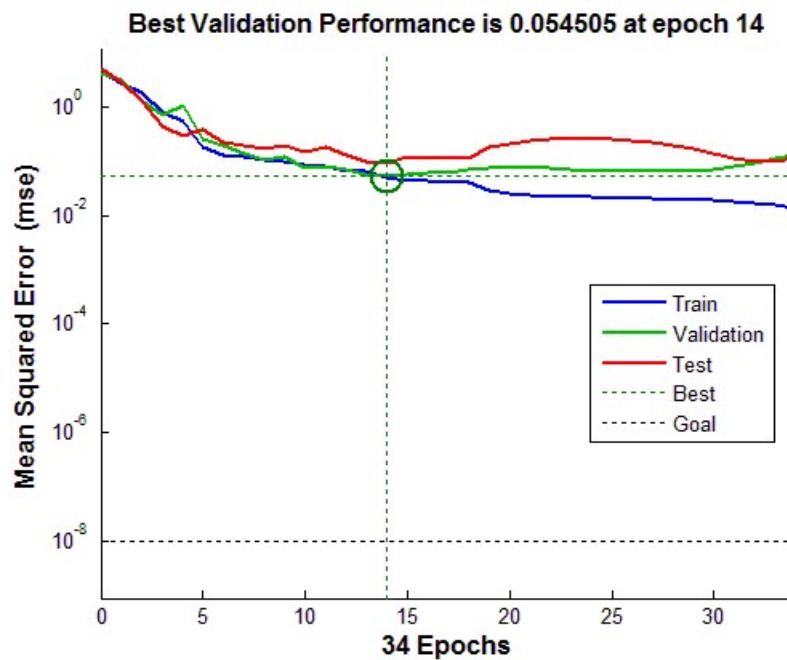


Figure 4.9: Best validation performance.

used for validation and testing ANN efficiency. The data are randomly separated between the sets. The network structure employed in this research includes five variables in the input layer of the two hidden layers, with ten neurons in the first layer and two neurons in the second layer are used. This component has been obtained because of the minimum root mean square (RMSE) value in comparison with the other neuron-number of trying, as shown in Figure 4.10.

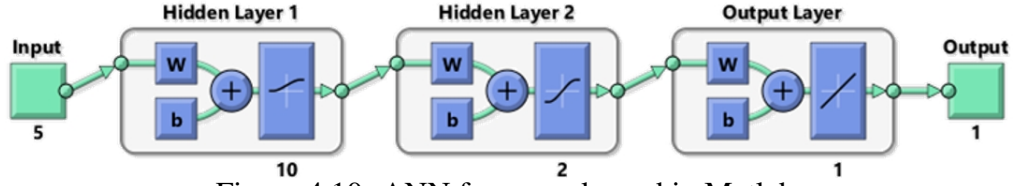


Figure 4.10: ANN framework used in Matlab.

4.6.2 RSM Method and Selection of Parameters

The regression model of the polynomial with the interaction and the main effects of parameters form the mathematical equations showed in equation 3.22. Moreover, in this study, the Design-Expert version 7.0 and NCSS 12 software were used to model RSM. Moreover, if the function is a CMR parameter, the response level inputs can be expressed in equation 4.8:

$$CMR = f(A, B, C, D, E) \quad (4.8)$$

$$CMR = k_0 + k_1(A) + k_2(B) + k_3(C) + k_{11}(A^2) + k_{22}(B^2) + k_{33}(C^2) + k_{12}(AB) + k_{13}(AC) + k_{23}(BC) \quad (4.9)$$

The input parameters of (A, B, C, D, and E) are $(T_1, \delta_{y,eff}, \delta_u, \Omega, \text{ and } S_{MT})$, respectively. Regarding the regression equation, the designated polynomials for the factors (A, B, C, D, and E) are shown in equation 3.22. The k_i is the numerical coefficient for every parameter. In addition, design expert software was employed to determine the coefficients. Subsequently, ANOVA analysis is applied to confirm the capability of the RSM model.

4.6.3 ANFIS Modelling

The Sugeno-type fuzzy inference system is applied to estimate the CMR value. Made ANFIS model has five inputs and one output, according to Figure 4.11. The data in ANFIS have been separated into three sets: training, checking, and testing. The produced model has been trained with 40 epochs. Testing data assessed the model after making the ANFIS model. In this research, the Matlab version R2018a

was applied to build the ANFIS model.

To produce the ANFIS model, the Sub-clustering option has been used, which in this case has the ability to create the most desirable structure of ANFIS, and the best possible membership functions are set up automatically. 100 Epochs were used to find the lowest modelling error. The hybrid optimum method also has been used to train FIS. In this way, the number of rules equal to 10, (And method) = Prob, (Or method) = Probor, Impmethod = Prod, Aggmethode = Max, Defuzz Method = wtaveo, and the number of membership functions for every input and output are 10.

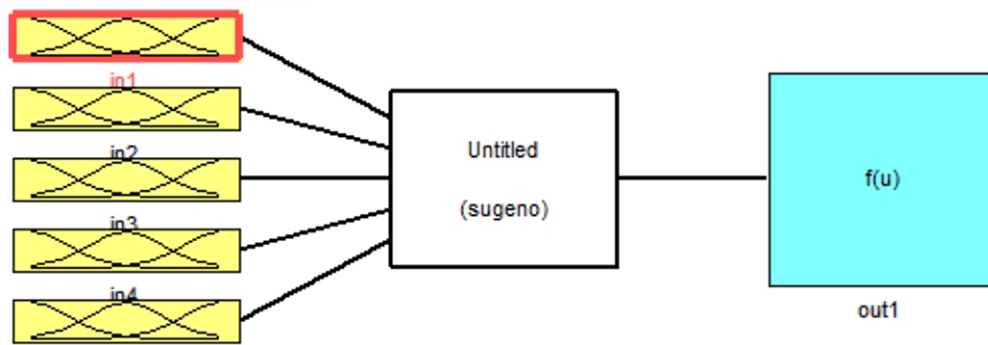


Figure 4.11: System design with five inputs and one output.

Chapter 5

RESULTS AND DISCUSSIONS

5.1 Introduction

This chapter presents and evaluates the results obtained from the proposed study. First, the seismic evaluation results of 19 performance groups of stock RC archetypes that are designed based on the second and third editions of Standard No. 2800 are evaluated using the ACMR concept described in the FEMA P695 methodology. Second, the concept of collapse probability is examined to show the effect of various total uncertainty. Once the vulnerability of five performance groups of stock RC archetypes was determined, the results of their strengthening using steel braces are evaluated. The three types of braces in 15 performance groups are compared and the results are categorised and seismic design factors are introduced. Third, the results of soft computing methods in predicting collapse margin ratio factors are evaluated by 114 full seismic data. Significant parameters are sorted, and the estimator formula is extracted. Eventually, the results obtained from ANN, RSM, and ANFIS methods are described and compared by statistical indices.

5.2 Seismic Assessment of Stock Archetypes

To evaluate the archetypes based on the FEMA P695, the following analyses should be carried out. First, the models are designed linearly according to Standard 2800-99 or Standard 2800-05 using ETABS software. Then, by eigenvalue analysis in SeismoStruct, the fundamental period and the ordinates of the first mode are achieved. Next, by pushover analysis, the capacity of the archetypes is obtained

(Figure 5.1). Accordingly, the results of pushover analyses are shown in Tables 16, and 17, parameters such as overstrength, ductility, maximum base shear, maximum displacement are obtained. In the next step, using incremental dynamic analysis and fragility analysis, median collapse capacity and collapse margin ratio are obtained and their values are summarised in Table 18, and Table 19, respectively. In this study, the 5% damped first mode spectral acceleration ($Sa_{(T1,5\%)}$) and the maximum inter-story drift ratio are selected as the IM and DM parameters, respectively.

The collapse fragility curve is achieved from the incremental dynamic analysis. It is specified as a cumulative distribution function (CDF) related to a lognormal distribution of the ground motion intensity which causes collapse. The collapse margin ratio, CMR, for archetypes was computed from spectral acceleration causing the collapse in a half of 44 analyses. Figure 5.2 shows the collapse fragility curve for sample archetypes.

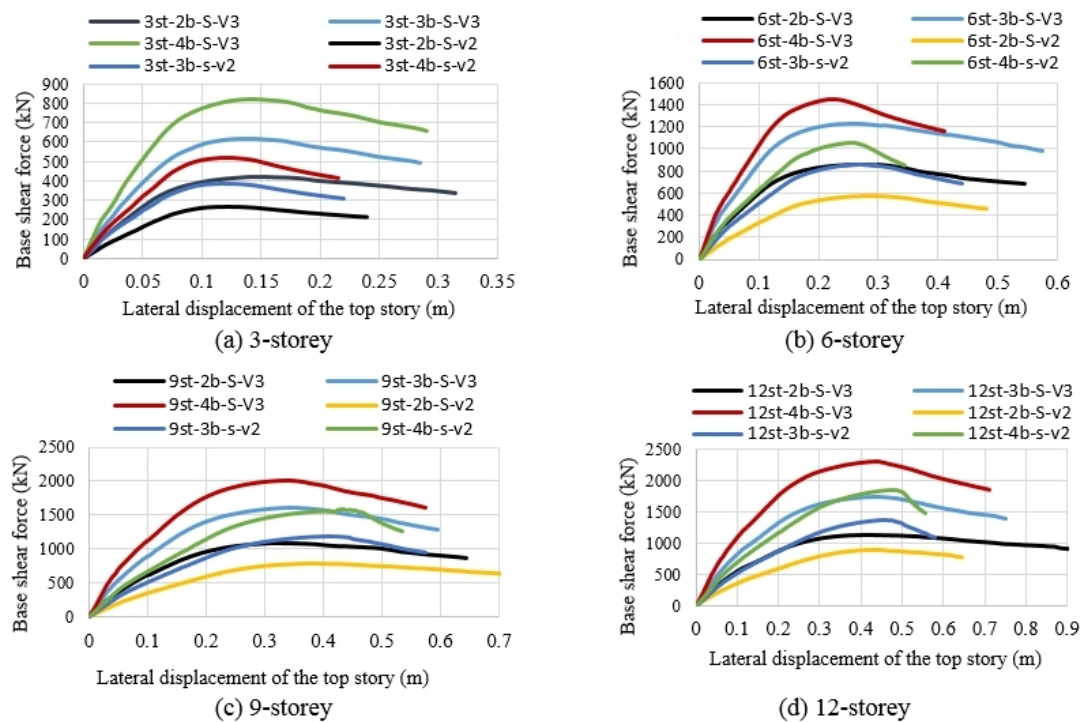


Figure 5.1: Pushover curves of Unbraced Archetypes.

Table 16: Summary of results obtained from nonlinear static (pushover) analysis of archetype RC frames designed based on the Standard No. 2800-05.

PG	Model ID	T (s)	W (kN)	V_{design} (kN)	V_{max} (kN)	$\delta_{y,eff}$	δ_u (m)	μ	Ω
PG1	3st-2b-S-v3	0.58	1234.0	145.4	418.9	0.037	0.315	8.60	2.88
	3st-3b-S-v3	0.63	1835.3	216.2	614.4	0.043	0.285	6.70	2.84
	3st-4b-S-v3	0.60	2436.6	287.0	821.7	0.039	0.290	7.45	2.86
PG2	6st-2b-S-v3	0.75	2667.0	314.2	859.9	0.086	0.545	6.36	2.74
	6st-3b-S-v3	0.76	3949.0	465.2	1222.7	0.082	0.575	6.99	2.63
	6st-4b-S-v3	0.77	5232.0	616.3	1444.2	0.073	0.410	5.59	2.34
PG3	9st-2b-S-v3	0.93	4166.9	438.4	1078.0	0.143	0.642	4.48	2.46
	9st-3b-S-v3	0.94	6158.4	647.9	1602.9	0.144	0.595	4.13	2.47
	9st-4b-S-v3	0.94	8150.0	857.4	2003.7	0.136	0.575	4.23	2.34
PG4	12st-2b-S-v3	1.20	5528.0	504.2	1121.8	0.189	0.915	4.85	2.23
	12st-3b-S-v3	1.19	8181.3	746.1	1732.3	0.196	0.750	3.82	2.32
	12st-4b-S-v3	1.19	10853.2	989.8	2298.8	0.196	0.715	3.64	2.32
PG5	3st-2b-P-v3	0.41	414.6	48.8	222.8	0.037	0.535	14.32	4.56
	3st-3b-P-v3	0.44	609.0	71.7	323.3	0.037	0.445	12.09	4.51
	3st-4b-P-v3	0.44	803.3	94.6	407.2	0.036	0.420	11.70	4.30
PG6	6st-2b-P-v3	0.66	877.0	103.3	315.4	0.092	0.695	7.52	3.05
	6st-3b-P-v3	0.67	1285.6	151.5	432.4	0.086	0.700	8.11	2.86
	6st-4b-P-v3	0.67	1694.2	199.6	555.3	0.084	0.690	8.21	2.78
PG7	9st-2b-P-v3	0.83	1394.2	146.7	360.9	0.144	0.770	5.36	2.46
	9st-3b-P-v3	0.83	2056.1	216.3	515.8	0.139	0.755	5.43	2.38
	9st-4b-P-v3	0.84	2695.3	283.5	669.8	0.138	0.695	5.05	2.36
PG8	12st-2b-P-v3	0.97	1976.0	180.2	468.5	0.223	0.920	4.13	2.60
	12st-3b-P-v3	0.97	2886.5	263.3	681.7	0.221	0.825	3.73	2.59
	12st-4b-P-v3	0.96	3797.0	346.3	891.4	0.219	0.785	3.58	2.57

Tables 20 to 22 show overstrength factor for every archetype and groups. In this research, they were obtained as 2.8, and 2.4 for the third edition and the second edition, respectively. Also, according to the FEMA P695, by assuming 5% damping for the structure, the response modification factor and deflection amplification factor are equal.

Table 17: Summary of results obtained from nonlinear static (pushover) analysis of archetype RC frames designed based on the Standard No. 2800-99.

PG	Model ID	T (s)	W (kN)	V_{design} (kN)	V_{max} (kN)	$\delta_{y,eff}$	δ_u (m)	μ	Ω
PG9	3st-2b-S-v2	0.79	1209.1	113.7	265.2	0.043	0.240	5.57	2.33
	3st-3b-S-v2	0.86	1800.7	169.3	386.3	0.050	0.220	4.42	2.28
	3st-4b-S-v2	0.85	2392.3	224.9	518.6	0.049	0.215	4.38	2.31
PG10	6st-2b-S-v2	1.07	2512.3	236.2	577.8	0.087	0.480	5.49	2.45
	6st-3b-S-v2	1.09	3734.2	351.0	854.0	0.090	0.440	4.88	2.43
	6st-4b-S-v2	1.10	4956.2	465.9	1052.3	0.085	0.345	4.05	2.26
PG11	9st-2b-S-v2	1.33	3874.7	325.5	779.0	0.122	0.720	5.89	2.39
	9st-3b-S-v2	1.34	5763.8	484.2	1180.3	0.126	0.575	4.56	2.44
	9st-4b-S-v2	1.34	7645.8	642.3	1584.2	0.128	0.535	4.19	2.47
PG12	12st-2b-S-v2	1.52	5310.9	382.4	890.0	0.160	0.690	4.31	2.33
	12st-3b-S-v2	1.52	7877.1	567.2	1378.2	0.167	0.580	3.48	2.43
	12st-4b-S-v2	1.52	10443.2	751.9	1851.0	0.165	0.555	3.36	2.46
PG13	3st-2b-P-v2	0.78	372.6	35.0	75.0	0.039	0.255	6.53	2.14
	3st-3b-P-v2	0.84	548.7	51.6	112.0	0.046	0.265	5.79	2.17
	3st-4b-P-v2	0.83	726.1	68.3	149.8	0.045	0.265	5.87	2.20
PG14	6st-2b-P-v2	0.98	584.4	54.9	177.2	0.093	0.485	5.21	3.23
	6st-3b-P-v2	1.00	1163.4	109.4	247.9	0.068	0.375	5.52	2.27
	6st-4b-P-v2	1.01	1537.8	144.6	316.0	0.067	0.340	5.09	2.19
PG15	9st-2b-P-v2	1.21	1246.2	104.7	247.4	0.104	0.605	5.80	2.36
	9st-3b-P-v2	1.23	1834.9	154.1	351.4	0.100	0.580	5.78	2.28
	9st-4b-P-v2	1.24	2423.6	203.6	455.4	0.098	0.560	5.69	2.24
PG16	12st-2b-P-v2	1.53	1683.1	121.2	282.4	0.148	0.830	5.60	2.33
	12st-3b-P-v2	1.54	2477.3	178.4	415.0	0.147	0.755	5.12	2.33
	12st-4b-P-v2	1.54	3271.5	235.6	546.5	0.147	0.705	4.80	2.32
PG17	1st-2b-S-v2	0.39	393.2	37.0	145.1	0.014	0.10	6.80	3.93
	1st-3b-S-v2	0.44	586.0	55.1	169.9	0.014	0.09	6.44	3.09
	1st-4b-S-v2	0.48	778.0	73.1	191.1	0.014	0.09	6.03	2.61

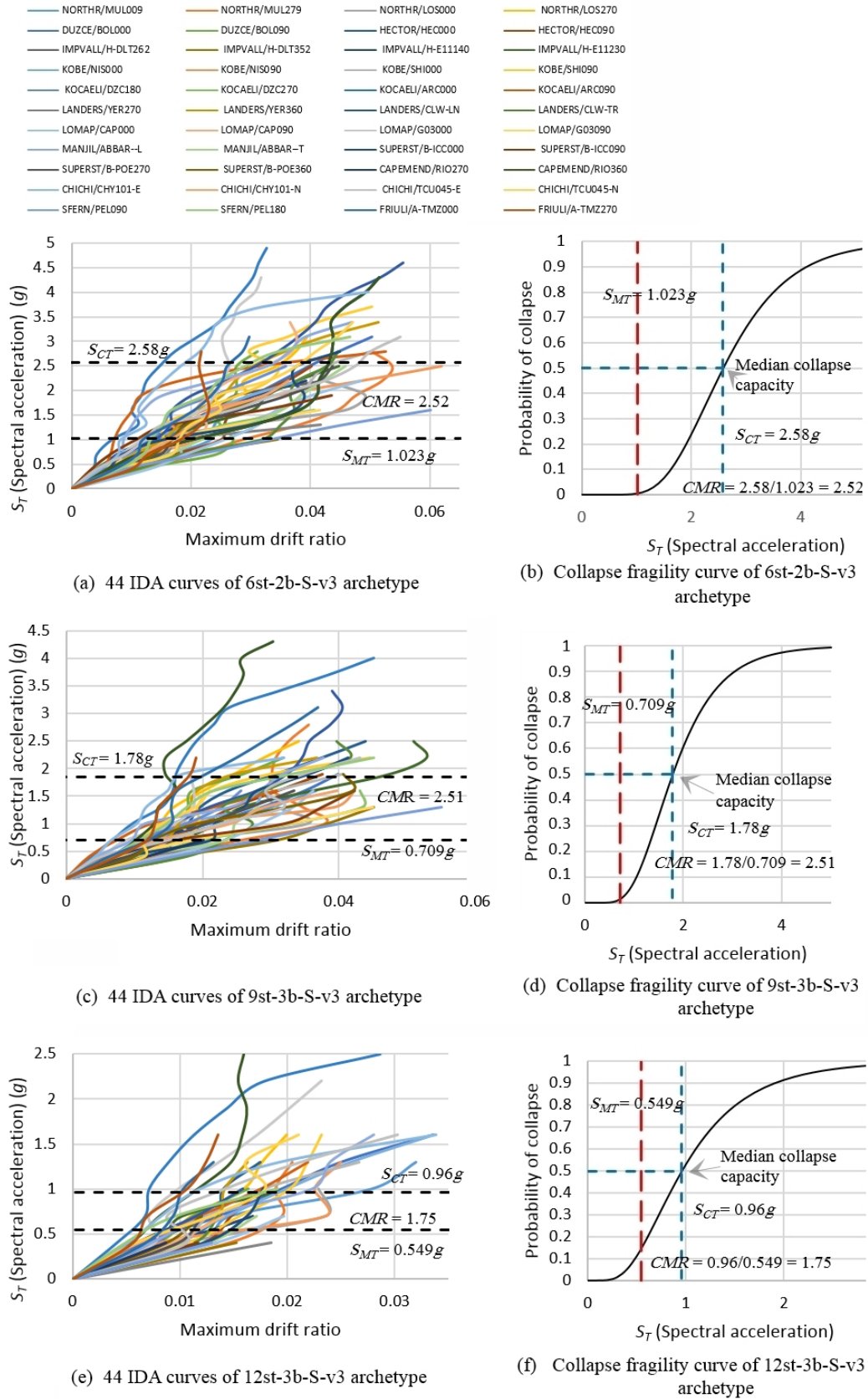


Figure 5.2: Incremental dynamic analyses and fragility curves.

Table 18: Summary of results obtained from nonlinear dynamic analysis (PG1-PG8).

PG	Model ID	S_{CT}	S_{MT}	CMR	SSF	ACMR
PG1	3st-2b-S-v3	2.45	1.500	1.63	1.36	2.22
	3st-3b-S-v3	2.15	1.500	1.43	1.33	1.91
	3st-4b-S-v3	2.51	1.500	1.67	1.34	2.24
PG2	6st-2b-S-v3	2.58	1.023	2.52	1.35	3.40
	6st-3b-S-v3	2.19	1.023	2.14	1.36	2.91
	6st-4b-S-v3	2.25	1.023	2.20	1.34	2.95
PG3	9st-2b-S-v3	2.11	0.709	2.98	1.30	3.87
	9st-3b-S-v3	1.78	0.709	2.51	1.30	3.26
	9st-4b-S-v3	1.34	0.709	1.89	1.30	2.46
PG4	12st-2b-S-v3	1.40	0.549	2.55	1.38	3.52
	12st-3b-S-v3	0.96	0.549	1.75	1.33	2.33
	12st-4b-S-v3	1.05	0.549	1.91	1.33	2.54
PG5	3st-2b-P-v3	4.61	1.500	3.07	1.33	4.09
	3st-3b-P-v3	4.30	1.500	2.87	1.33	3.81
	3st-4b-P-v3	4.22	1.500	2.81	1.33	3.74
PG6	6st-2b-P-v3	5.29	1.023	5.17	1.37	7.08
	6st-3b-P-v3	4.80	1.023	4.69	1.38	6.48
	6st-4b-P-v3	4.53	1.023	4.43	1.38	6.11
PG7	9st-2b-P-v3	3.58	0.709	5.05	1.35	6.82
	9st-3b-P-v3	3.18	0.709	4.49	1.36	6.10
	9st-4b-P-v3	3.00	0.709	4.23	1.29	5.46
PG8	12st-2b-P-v3	3.29	0.549	5.99	1.31	7.85
	12st-3b-P-v3	2.97	0.549	5.41	1.30	7.03
	12st-4b-P-v3	2.66	0.549	4.85	1.28	6.20

Table 19: Summary of results obtained from nonlinear dynamic analysis (PG9-PG17).

PG	Model ID	S_{CT}	S_{MT}	CMR	SSF	ACMR
PG9	3st-2b-S-v2	1.33	1.500	0.89	1.32	1.17
	3st-3b-S-v2	1.15	1.500	0.77	1.29	0.99
	3st-4b-S-v2	1.17	1.500	0.78	1.29	1.01
PG10	6st-2b-S-v2	1.25	1.023	1.22	1.37	1.67
	6st-3b-S-v2	1.25	1.023	1.22	1.36	1.66
	6st-4b-S-v2	1.20	1.023	1.17	1.32	1.55
PG11	9st-2b-S-v2	1.50	0.709	2.12	1.46	3.09
	9st-3b-S-v2	1.10	0.709	1.55	1.38	2.14
	9st-4b-S-v2	1.16	0.709	1.64	1.38	2.26
PG12	12st-2b-S-v2	0.99	0.549	1.80	1.41	2.54
	12st-3b-S-v2	0.85	0.549	1.55	1.36	2.11
	12st-4b-S-v2	0.71	0.549	1.29	1.34	1.73
PG13	3st-2b-P-v2	1.51	1.500	1.01	1.35	1.36
	3st-3b-P-v2	1.26	1.500	0.84	1.35	1.13
	3st-4b-P-v2	1.22	1.500	0.81	1.35	1.10
PG14	6st-2b-P-v2	1.74	1.023	1.70	1.35	2.30
	6st-3b-P-v2	1.43	1.023	1.40	1.37	1.92
	6st-4b-P-v2	1.31	1.023	1.28	1.35	1.73
PG15	9st-2b-P-v2	1.59	0.709	2.24	1.43	3.21
	9st-3b-P-v2	1.30	0.709	1.83	1.44	2.64
	9st-4b-P-v2	1.14	0.709	1.61	1.44	2.32
PG16	12st-2b-P-v2	1.14	0.549	2.08	1.46	3.03
	12st-3b-P-v2	0.90	0.549	1.64	1.46	2.39
	12st-4b-P-v2	0.82	0.549	1.49	1.45	2.17
PG17	1st-2b-S-v2	1.30	1.500	0.8	1.30	1.13
	1st-3b-S-v2	1.00	1.500	0.67	1.29	0.86
	1st-4b-S-v2	1.01	1.500	0.67	1.28	0.86

Table 20: Comparison of the seismic performance factors between performance groups PG-1-PG-8.

Group ID	Ω	ACMR	10%	20%	Result	Group ID	Ω	ACMR	10%	20%	Result
PG-1											
3st-2b-S-v3	2.88	2.22		1.56	Pass	3st-2b-P-v3	4.56	4.09		1.56	Pass
3st-3b-S-v3	2.84	1.91		1.56	Pass	3st-3b-P-v3	4.51	2.87		1.56	Pass
3st-4b-S-v3	2.86	2.24		1.56	Pass	3st-4b-P-v3	4.3	3.74		1.56	Pass
$\overline{ACMR}$		2.12	1.97		Pass	$\overline{ACMR}$		3.57	1.97		Pass
$\overline{\Omega}$	2.86					$\overline{\Omega}$	4.46				
PG-2											
6st-2b-S-v3	2.74	3.4		1.56	Pass	6st-2b-P-v3	3.05	7.08		1.56	Pass
6st-3b-S-v3	2.63	2.91		1.56	Pass	6st-3b-P-v3	2.86	4.69		1.56	Pass
6st-4b-S-v3	2.34	2.95		1.56	Pass	6st-4b-P-v3	2.78	6.11		1.56	Pass
$\overline{ACMR}$		3.09	1.97		Pass	$\overline{ACMR}$		5.96	1.97		Pass
$\overline{\Omega}$	2.57					$\overline{\Omega}$	2.9				
PG-3											
9st-2b-S-v3	2.46	3.87		1.56	Pass	9st-2b-P-v3	2.46	6.82		1.56	Pass
9st-3b-S-v3	2.47	3.26		1.56	Pass	9st-3b-P-v3	2.38	4.49		1.56	Pass
9st-4b-S-v3	2.34	2.46		1.56	Pass	9st-4b-P-v3	2.36	5.46		1.56	Pass
$\overline{ACMR}$		3.2	1.97		Pass	$\overline{ACMR}$		5.59	1.97		Pass
$\overline{\Omega}$	2.42					$\overline{\Omega}$	2.4				
PG-4											
12st-2b-S-v3	2.22	3.52		1.56	Pass	12st-2b-P-v3	2.6	7.85		1.56	Pass
12st-3b-S-v3	2.32	2.33		1.56	Pass	12st-3b-P-v3	2.59	5.41		1.56	Pass
12st-4b-S-v3	2.32	2.54		1.56	Pass	12st-4b-P-v3	2.57	6.2		1.56	Pass
$\overline{ACMR}$		2.8	1.97		Pass	$\overline{ACMR}$		6.49	1.97		Pass
$\overline{\Omega}$	2.29					$\overline{\Omega}$	2.59				

Table 21: Comparison of the seismic performance factors between performance groups PG-9-PG-16.

Group ID	Ω	ACMR	10%	20%	Result	Group ID	Ω	ACMR	10%	20%	Result
PG-9											
3st-2b-S-v2	2.33	1.17		1.56	Fail	PG-13					
3st-3b-S-v2	2.28	0.99		1.56	Fail	3st-2b-P-v2	2.14	1.36		1.56	Fail
3st-4b-S-v2	2.31	1.01		1.56	Fail	3st-3b-P-v2	2.17	1.13		1.56	Fail
$\overline{ACMR}$		1.06	1.97		Fail	3st-4b-P-v2	2.2	1.10		1.56	Fail
$\overline{\Omega}$	2.31					$\overline{ACMR}$		1.20	1.97		Fail
PG-10											
6st-2b-S-v2	2.45	1.67		1.56	Pass	PG-14					
6st-3b-S-v2	2.43	1.66		1.56	Pass	6st-2b-P-v2	3.22	2.30		1.56	Pass
6st-4b-S-v2	2.26	1.55		1.56	Fail	6st-3b-P-v2	2.27	1.92		1.56	Pass
$\overline{ACMR}$		1.63	1.97		Fail	6st-4b-P-v2	2.19	1.73		1.56	Pass
$\overline{\Omega}$	2.38					$\overline{ACMR}$		1.98	1.97		Pass
PG-11											
9st-2b-S-v2	2.39	3.09		1.56	Pass	PG-15					
9st-3b-S-v2	2.44	2.14		1.56	Pass	9st-2b-P-v2	2.36	3.21		1.56	Pass
9st-4b-S-v2	2.47	2.26		1.56	Pass	9st-3b-P-v2	2.28	2.64		1.56	Pass
$\overline{ACMR}$		2.50	1.97		Pass	9st-4b-P-v2	2.24	2.32		1.56	Pass
$\overline{\Omega}$	2.43					$\overline{ACMR}$		2.72	1.97		Pass
PG-12											
12st-2b-S-v2	2.33	2.54		1.56	Pass	PG-16					
12st-3b-S-v2	2.43	2.11		1.56	Pass	12st-2b-P-v2	2.33	3.03		1.56	Pass
12st-4b-S-v2	2.46	1.73		1.56	Pass	12st-3b-P-v2	2.32	2.39		1.56	Pass
$\overline{ACMR}$		2.13	1.97		Pass	12st-4b-P-v2	2.32	2.17		1.56	Pass
$\overline{\Omega}$	2.41					$\overline{ACMR}$		2.53	1.97		Pass
						$\overline{\Omega}$	2.32				

Table 22: Comparison of the seismic performance factors between performance group PG-17.

Group ID	Ω	ACMR	10%	20%	Result
PG-17					
1st-2b-S-v2	3.93	1.13		1.56	Fail
1st-3b-S-v2	3.09	0.86		1.56	Fail
1st-4b-S-v2	2.61	0.86		1.56	Fail
$\overline{ACMR}$		0.95	1.97		Fail
$\overline{\Omega}$	3.21				

As shown in Figure 5.3, the designed models based on the third edition have higher ACMR values, and are able to satisfy the life safety performance level. On the other hand, in models designed on the basis of the second edition, low-rise models have not been able to pass the desired values while models with middle-rise height could succeed. These items may be reviewed as follows: As presented, the capacity curves

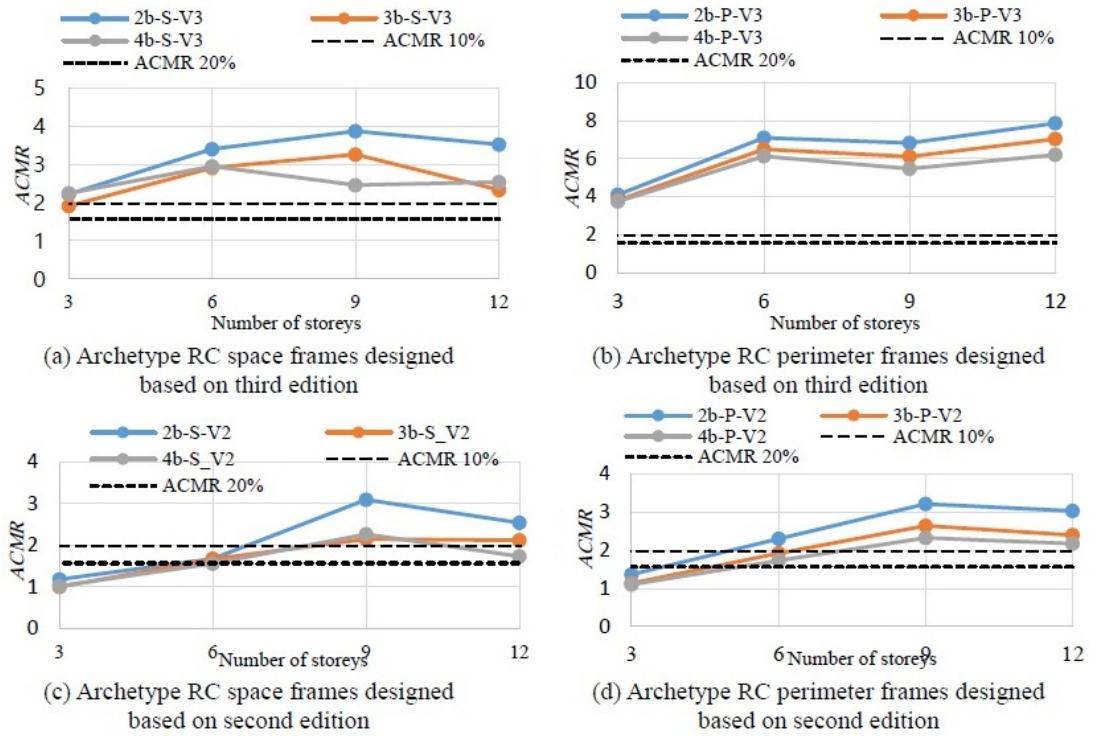


Figure 5.3: Adjusted collapse margin ratio comparison.

of the models between two design codes are compared. By examining them in these forms, models of both design codes are presented together. Capacity curves are close to each other in the 9 and 12-story models. If this item is in the 3 and 6-storey frames, they are far away from each other. It is indicated as the level of the strength of the structure to the lateral forces. According to Standard 2800, it is not required to control the strong-column weak-beam rule for intermediate and ordinary ductility reinforced concrete structures. This factor expresses that the total bending strength of the columns must at least be 20% more than the bending strength of the beams in a joint:

$$\sum M_c \geq 1.2 \sum M_b \quad (5.1)$$

The results of the Table 23 shows that the models designed on the basis of the second edition do not have a favourable status, and in the short structures there are many joints that are faced with this problem. This item can be evaluated as a structural weakness even in this type of RC structures. By drawing the average of the ductility's in Figure 5.4, it was seen that the difference between the ductility's in low-rise frames are more than the ductility average in mid-rise frames. The structures with higher ductility are able to absorb more energy and tolerate more deformation efficiently. As expected, the seismic performance factors (Ω and μ) have bigger value in the perimeter frames compared to values of space frames. Based on the results of the nonlinear static (pushover) analysis, the models designed based on the third edition exhibit higher base shear force and ductility compared to those based on the second edition. The results indicate that the low-rise RC frames designed based on the Standard No. 2800-99 are critically vulnerable and do not fulfil the life safety performance level. However, the RC frames designed based on the third edition are sufficiently strong and fulfil the life safety performance level. The results show that the low-rise RC residential buildings in Iran that were designed and constructed according to the Standard No. 2800-99 need

Table 23: Percentage of failures.

PG	Model ID	Total joints	Failure joints	Fails %
PG1	3st-2b-S-v3	9	1	11.1
	3st-3b-S-v3	12	2	16.7
	3st-4b-S-v3	15	3	20.0
PG2	6st-2b-S-v3	18	1	5.6
	6st-3b-S-v3	24	2	8.3
	6st-4b-S-v3	30	3	10.0
PG3	9st-2b-S-v3	27	3	11.1
	9st-3b-S-v3	36	4	11.1
	9st-4b-S-v3	45	5	11.1
PG4	12st-2b-S-v3	36	3	8.3
	12st-3b-S-v3	48	8	16.7
	12st-4b-S-v3	60	8	13.3
PG9	3st-2b-S-v2	9	8	88.9
	3st-3b-S-v2	12	10	83.3
	3st-4b-S-v2	15	13	86.7
PG10	6st-2b-S-v2	18	3	16.7
	6st-3b-S-v2	24	6	25.0
	6st-4b-S-v2	30	9	30.0
PG11	9st-2b-S-v2	27	5	18.5
	9st-3b-S-v2	36	10	27.8
	9st-4b-S-v2	45	13	28.9
PG12	12st-2b-S-v2	36	13	36.1
	12st-3b-S-v2	48	16	33.3
	12st-4b-S-v2	60	19	31.7

adequate strengthening prior to the occurrence of intense earthquakes.

5.3 Results and Discussion of the Collapse Probability of Steel Braced RC Structures

In this section, first, the probability of collapse of stock vulnerable archetypes is discussed. The collapse probability in relation to median collapse capacity is given in Table 24. Subsequently, the collapse probabilities are separately derived for all

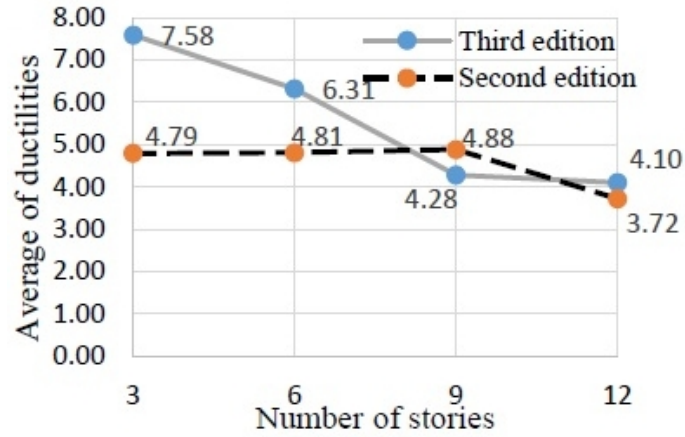


Figure 5.4: Comparison of ductility factor.

possible uncertainties. Compared with the 6-story models, the roles of the different uncertainties in the 1 and 3-story models are insignificant. The probabilities are almost in a fixed range and according to the increase in the general uncertainty of the 6-story models, the probability of collapse also increased. In the case of 1 and 3-story structures, the values obtained for the perimeter and space frames are greater than 20%. Furthermore, in the case of 6-story structures with respect to space and perimeter frames with uncertainties greater than 0.5 and 0.6, respectively, the models designed based on Standard 2800-99 [1] do not satisfy the essential requirements and must be retrofitted. In the following, the results of strengthening using concentric steel braces are discussed.

Table 25 to 27 summarises the calculations and analysis results for the frames retrofitted using X-shaped, chevron, and diagonal braces. The comparisons of the seismic parameters such as the ductility, overstrength, and collapse margin ratio indicate that these parameters dramatically increase compared with those of the unbraced cases. Accordingly, the CMRs in the X-shaped, chevron, and diagonal models increased by 2.30, 2.03, and 2.08 times, respectively. In the case of low-rise structures, the increase in the CMR is more profound, implying a greater impact of

Table 24: Collapse probability of unbraced frames versus various uncertainties.

PG	Model ID	β_{TOT}						
		0.3	0.4	0.5	0.6	0.7	0.8	0.9
PG17	1st-2b-S	34.5	38.3	40.6	42.1	43.2	44.1	44.7
	1st-3b-S	69.2	64.7	61.9	59.9	58.5	57.5	56.7
	1st-4b-S	69.0	64.5	61.7	59.8	58.4	57.4	56.6
PG9	3st-2b-S	30.0	34.7	37.6	39.7	41.1	42.2	43.1
	3st-3b-S	51.5	51.1	50.9	50.7	50.6	50.6	50.5
	3st-4b-S	49.2	49.4	49.5	49.6	49.6	49.7	49.7
PG10	6st-2b-S	4.3	9.9	15.1	19.5	23.1	26.0	28.4
	6st-3b-S	4.5	10.2	15.5	19.9	23.4	26.3	28.6
	6st-4b-S	7.3	13.7	19.1	23.3	26.6	29.2	31.4
PG13	3st-2b-P	15.3	22.2	27.0	30.5	33.1	35.1	36.7
	3st-3b-P	33.8	37.7	40.1	41.7	42.9	43.8	44.4
	3st-4b-P	37.8	40.8	42.6	43.8	44.7	45.3	45.9
PG14	6st-2b-P	0.3	1.9	4.8	8.3	11.8	14.9	17.8
	6st-3b-P	1.5	5.2	9.7	13.9	17.7	20.8	23.5
	6st-4b-P	3.4	8.6	13.7	18.1	21.7	24.7	27.2

the bracing system on such structures. Also, a comparison of the CMR indicates that the archetypes with shorter periods generally had lower collapse margin ratios and the 2-bay archetype models provided greater collapse margins than the archetype models with three and four bays. Moreover, the perimeter archetypes have larger collapse margin ratios than the space frame archetypes.

Compared to the unbraced archetypes, the period of braced models has been reduced by about half, which may indicate an increase in stiffness. Ductility in 3- and 6-story structures has a higher increase than one-story structures; overall, in braced one-story structures, ductility has not changed much. In general, S_{CT} has grown about three times. The ACMR values are compared with the assumed values of Section 4.1 and satisfy the corresponding values.

The collapse probabilities for the structures braced by X-shaped, chevron, and diagonal braces are given in Tables 28 and 29. Unlike the unbraced case, the uncertainty values regularly increase. In general, the probabilities associated with the X-shaped braces are less than those of the chevron and diagonal braces. In the braced structures, the collapse probability is directly related to the number of bays. Moreover, the collapse probability is lower for low-rise buildings. The rate of reduction in the collapse probability of the perimeter frames is considerably greater, reflecting the lighter gravity loads in the perimeter frames. Retrofitting using braces leads to efficient performance.

Table 25: Summary of the CMR results for the RC frames strengthened by various braces.

PG	Model ID	T_1 (s)	W (kN)	V_{design} (kN)	V_{max} (kN)	$\delta_{y,eff}$ (m)	δ_u (m)	μ	S_{CT}	S_{MT}	SSF	CMR	ACMR	Result
PG18	1st-2b-S-X	0.15	394.4	55.6	674.7	0.01	0.1	6.42	4.00	1.5	1.30	2.67	3.47	Pass
	1st-3b-S-X	0.18	586.8	82.7	696.4	0.01	0.1	6.79	3.81	1.5	1.31	2.54	3.33	Pass
	1st-4b-S-X	0.21	779.3	109.8	717.2	0.01	0.1	6.93	2.93	1.5	1.33	1.95	2.59	Pass
PG19	3st-2b-S-X	0.28	1214.2	171.1	1232.1	0.04	0.7	16.50	4.31	1.5	1.33	2.87	3.82	Pass
	3st-3b-S-X	0.35	1805.8	254.4	1342.1	0.03	0.7	23.20	3.38	1.5	1.33	2.25	2.99	Pass
	3st-4b-S-X	0.39	2397.5	337.8	1459.5	0.03	0.7	22.50	2.57	1.5	1.33	1.71	2.27	Pass
PG20	6st-2b-S-X	0.51	2523.4	355.5	1402.6	0.07	0.5	7.13	3.70	1.5	1.33	2.47	3.29	Pass
	6st-3b-S-X	0.59	3746.1	527.8	1664.2	0.05	0.4	8.35	2.70	1.5	1.36	1.80	2.45	Pass
	6st-4b-S-X	0.64	4968.9	700.1	1988.4	0.05	0.5	9.24	2.40	1.5	1.36	1.60	2.18	Pass
PG21	3st-2b-P-X	0.19	376.4	53.0	667.6	0.07	0.6	8.92	4.71	1.5	1.33	3.14	4.18	Pass
	3st-3b-P-X	0.24	552.8	77.89	695.1	0.05	0.3	6.91	3.41	1.5	1.30	2.27	2.95	Pass
	3st-4b-P-X	0.27	729.6	102.8	748.6	0.04	0.4	10.6	2.82	1.5	1.33	1.88	2.50	Pass
PG22	6st-2b-P-X	0.37	796.1	112.2	628.3	0.09	0.3	3.11	4.51	1.5	1.19	3.01	3.58	Pass
	6st-3b-P-X	0.41	1172.0	165.1	727.3	0.08	0.4	5.47	3.62	1.5	1.26	2.41	3.04	Pass
	6st-4b-P-X	0.45	1547.2	218.0	865.9	0.07	0.4	6.37	3.93	1.5	1.33	2.62	3.48	Pass
PG23	1st-2b-S-V	0.13	394.5	55.6	853.3	0.01	0.11	7.62	3.95	1.5	1.33	2.63	3.50	Pass
	1st-3b-S-V	0.16	587.0	82.7	866.6	0.01	0.11	11.69	3.79	1.5	1.33	2.53	3.36	Pass
	1st-4b-S-V	0.18	779.4	109.8	878.8	0.01	0.12	12.64	2.84	1.5	1.33	1.89	2.51	Pass

Table 26: Summary of the CMR results for the RC frames strengthened by various braces.

PG	Model ID	T_1 (s)	W (kN)	V_{design} (kN)	V_{max} (kN)	$\delta_{y,eff}$ (m)	δ_u (m)	μ	S_{CT}	S_{MT}	SSF	CMR	ACMR	Result
PG24	3st-2b-S-V	0.29	1213.1	170.9	1026.0	0.03	0.26	7.88	3.40	1.5	1.33	2.27	3.02	Pass
	3st-3b-S-V	0.34	1805.3	254.4	1273.0	0.03	0.25	9.01	2.81	1.5	1.33	1.87	2.49	Pass
	3st-4b-S-V	0.38	2396.9	337.7	1399.8	0.03	0.25	9.49	2.49	1.5	1.33	1.66	2.21	Pass
PG25	6st-2b-S-V	0.47	2521.7	355.3	1578.2	0.08	0.49	6.44	3.14	1.5	1.27	2.09	2.65	Pass
	6st-3b-S-V	0.55	3744.9	527.7	1844.7	0.06	0.47	7.87	2.78	1.5	1.35	1.85	2.50	Pass
	6st-4b-S-V	0.61	4967.3	699.9	2069.9	0.05	0.54	10.31	2.50	1.5	1.36	1.67	2.27	Pass
PG26	3st-2b-P-V	0.20	374.9	52.8	568.9	0.06	0.66	11.06	4.00	1.5	1.33	2.67	3.55	Pass
	3st-3b-P-V	0.24	552.2	77.8	733.4	0.05	0.22	4.11	3.00	1.5	1.22	2.00	2.44	Pass
	3st-4b-P-V	0.25	729.5	102.8	925.8	0.05	0.56	10.88	2.30	1.5	1.33	1.53	2.03	Pass
PG27	6st-2b-P-V	0.33	796.5	112.2	740.2	0.11	0.39	3.44	3.27	1.5	1.20	2.18	2.62	Pass
	6st-3b-P-V	0.39	1170.8	165.0	836.0	0.09	0.39	4.49	3.40	1.5	1.25	2.27	2.84	Pass
	6st-4b-P-V	0.44	1545.2	217.7	922.0	0.07	0.34	4.70	3.01	1.5	1.33	2.01	2.67	Pass
PG28	1st-2b-S-D	0.16	394.2	55.5	590.0	0.01	0.1	5.76	3.89	1.5	1.27	2.59	3.29	Pass
	1st-3b-S-D	0.20	586.6	82.7	614.7	0.01	0.2	14.9	3.68	1.5	1.33	2.45	3.26	Pass
	1st-4b-S-D	0.22	779.1	109.8	635.8	0.01	0.2	15.8	2.84	1.5	1.33	1.89	2.51	Pass
PG29	3st-2b-S-D	0.33	1212.5	170.9	866.1	0.03	0.2	8.52	3.10	1.5	1.33	2.07	2.75	Pass
	3st-3b-S-D	0.39	1804.8	254.3	1130.0	0.03	0.3	8.19	2.61	1.5	1.33	1.74	2.31	Pass
	3st-4b-S-D	0.41	2396.9	337.7	1343.1	0.03	0.4	14.3	2.22	1.5	1.33	1.48	1.97	Pass

Table 27: Summary of the CMR results for the RC frames strengthened by various braces.

PG	Model ID	T_1 (s)	W (kN)	V_{design} (kN)	V_{max} (kN)	$\delta_{y,eff}$ (m)	δ_u (m)	μ	δ_{CT}	δ_{MT}	SSF	CMR	ACMR	Result
PG30	6st-2b-S-D	0.53	2522.1	355.4	1383.3	0.07	0.4	6.51	3.49	1.5	1.33	2.33	3.10	Pass
	6st-3b-S-D	0.59	3745.6	527.8	1772.7	0.06	0.5	8.17	2.78	1.5	1.36	1.85	2.52	Pass
	6st-4b-S-D	0.63	4969.2	700.2	2140.3	0.06	0.5	9.34	2.40	1.5	1.36	1.60	2.18	Pass
PG31	3st-2b-P-D	0.21	375.1	52.9	576.3	0.06	0.5	7.59	4.00	1.5	1.32	2.67	3.52	Pass
	3st-3b-P-D	0.26	552.3	77.8	613.8	0.04	0.8	19.1	3.40	1.5	1.33	2.27	3.02	Pass
	3st-4b-P-D	0.29	729.1	102.7	652.4	0.04	0.3	8.85	2.90	1.5	1.33	1.93	2.57	Pass
PG32	6st-2b-P-D	0.39	795.0	112.0	614.0	0.09	0.4	4.24	3.72	1.5	1.23	2.48	3.05	Pass
	6st-3b-P-D	0.44	1170.0	164.9	781.7	0.08	0.3	3.98	3.51	1.5	1.22	2.34	2.85	Pass
	6st-4b-P-D	0.48	1544.9	217.7	871.52	0.07	0.3	5.06	3.29	1.5	1.26	2.19	2.76	Pass

The collapse probabilities are shown in Figure 5.5. In the case of (a) for primary archetypes, for 1 and 3-story structures, the collapse probability remains steady. In the case of concentrically braced frames, with increasing β_{TOT} , the probabilities regularly increase. In the case of (b), for the structures that are braced by X-shaped braces, all models experience a collapse probability of less than 20%. In the cases of (c) and (d), only chevron-braced models (3st-4b-P-V) and diagonal-braced models (3st-4b-S-D) have collapse probability the values greater than 20% when β_{TOT} is 0.9.

Table 28: Percentage of collapse probabilities of frames strengthened with x-shape and chevron braces, based on the different total uncertainties.

PG	Model ID	β_{TOT}						
		0.3	0.4	0.5	0.6	0.7	0.8	0.9
PG18	1st-2b-S-X	0.0	0.1	0.6	1.9	3.8	6.0	8.4
	1st-3b-S-X	0.0	0.1	0.8	2.3	4.3	6.6	9.1
	1st-4b-S-X	0.1	0.8	2.8	5.6	8.6	11.6	14.4
PG19	3st-2b-S-X	0.0	0.0	0.4	1.3	2.8	4.7	6.8
	3st-3b-S-X	0.0	0.3	1.4	3.4	5.8	8.5	11.1
	3st-4b-S-X	0.3	2.0	5.0	8.5	12.0	15.2	18.0
PG20	6st-2b-S-X	0.0	0.1	0.9	2.4	4.5	6.9	9.3
	6st-3b-S-X	0.1	1.3	3.7	6.8	10.0	13.2	16.0
	6st-4b-S-X	0.5	2.6	6.0	9.8	13.3	16.6	19.4
PG21	3st-2b-P-X	0.0	0.0	0.2	0.9	2.1	3.7	5.6
	3st-3b-P-X	0.0	0.3	1.5	3.5	6.1	8.8	11.4
	3st-4b-P-X	0.1	1.1	3.3	6.3	9.5	12.6	15.4
PG22	6st-2b-P-X	0.0	0.1	0.5	1.7	3.4	5.6	7.8
	6st-3b-P-X	0.0	0.3	1.3	3.2	5.6	8.2	10.8
	6st-4b-P-X	0.0	0.1	0.6	1.9	3.7	5.9	8.3
PG23	1st-2b-S-V	0.0	0.1	0.6	1.8	3.7	5.9	8.2
	1st-3b-S-V	0.0	0.1	0.8	2.2	4.2	6.5	8.9
	1st-4b-S-V	0.1	1.0	3.2	6.2	9.4	12.4	15.2
PG24	3st-2b-S-V	0.0	0.3	1.4	3.3	5.7	8.4	11.0
	3st-3b-S-V	0.1	1.1	3.4	6.4	9.6	12.7	15.5
	3st-4b-S-V	0.4	2.4	5.7	9.3	12.9	16.1	18.9
PG25	6st-2b-S-V	0.1	0.7	2.5	5.2	8.1	11.1	13.9
	6st-3b-S-V	0.1	1.1	3.3	6.3	9.5	12.6	15.4
	6st-4b-S-V	0.3	2.0	5.1	8.6	12.1	15.3	18.2
PG26	3st-2b-P-V	0.0	0.1	0.6	1.7	3.5	5.7	8.0
	3st-3b-P-V	0.1	1.3	3.7	6.9	10.1	13.2	16.1
	3st-4b-P-V	0.9	3.7	7.7	11.7	15.4	18.7	21.4
PG27	6st-2b-P-V	0.1	0.8	2.7	5.4	8.5	11.5	14.3
	6st-3b-P-V	0.0	0.5	1.9	4.1	6.8	9.6	12.4
	6st-4b-P-V	0.1	0.7	2.5	5.1	8.0	11.0	13.8

Table 29: Percentage of collapse probabilities of frames strengthened with diagonal braces, based on the different uncertainties.

PG	Model	β_{TOT}						
	ID	0.3	0.4	0.5	0.6	0.7	0.8	0.9
PG28	1st-2b-S-D	0.0	0.1	0.9	2.3	4.4	6.8	9.3
	1st-3b-S-D	0.0	0.2	0.9	2.4	4.6	7.0	9.4
	1st-4b-S-D	0.1	1.0	3.2	6.2	9.4	12.4	15.2
PG29	3st-2b-S-D	0.0	0.6	2.2	4.6	7.4	10.3	13.1
	3st-3b-S-D	0.3	1.8	4.7	8.1	11.5	14.7	17.6
	3st-4b-S-D	1.2	4.5	8.8	13.0	16.7	19.9	22.6
PG30	6st-2b-S-D	0.0	0.2	1.2	3.0	5.3	7.9	10.5
	6st-3b-S-D	0.1	1.0	3.2	6.2	9.3	12.4	15.2
	6st-4b-S-D	0.5	2.6	6.0	9.8	13.3	16.6	19.4
PG31	3st-2b-P-D	0.0	0.1	0.6	1.8	3.6	5.8	8.1
	3st-3b-P-D	0.0	0.3	1.4	3.3	5.7	8.4	11.0
	3st-4b-P-D	0.1	0.9	2.9	5.8	8.9	11.9	14.7
PG32	6st-2b-P-D	0.0	0.3	1.3	3.2	5.6	8.2	10.8
	6st-3b-P-D	0.0	0.4	1.8	4.0	6.7	9.5	12.2
	6st-4b-P-D	0.0	0.6	2.1	4.5	7.3	10.2	12.9

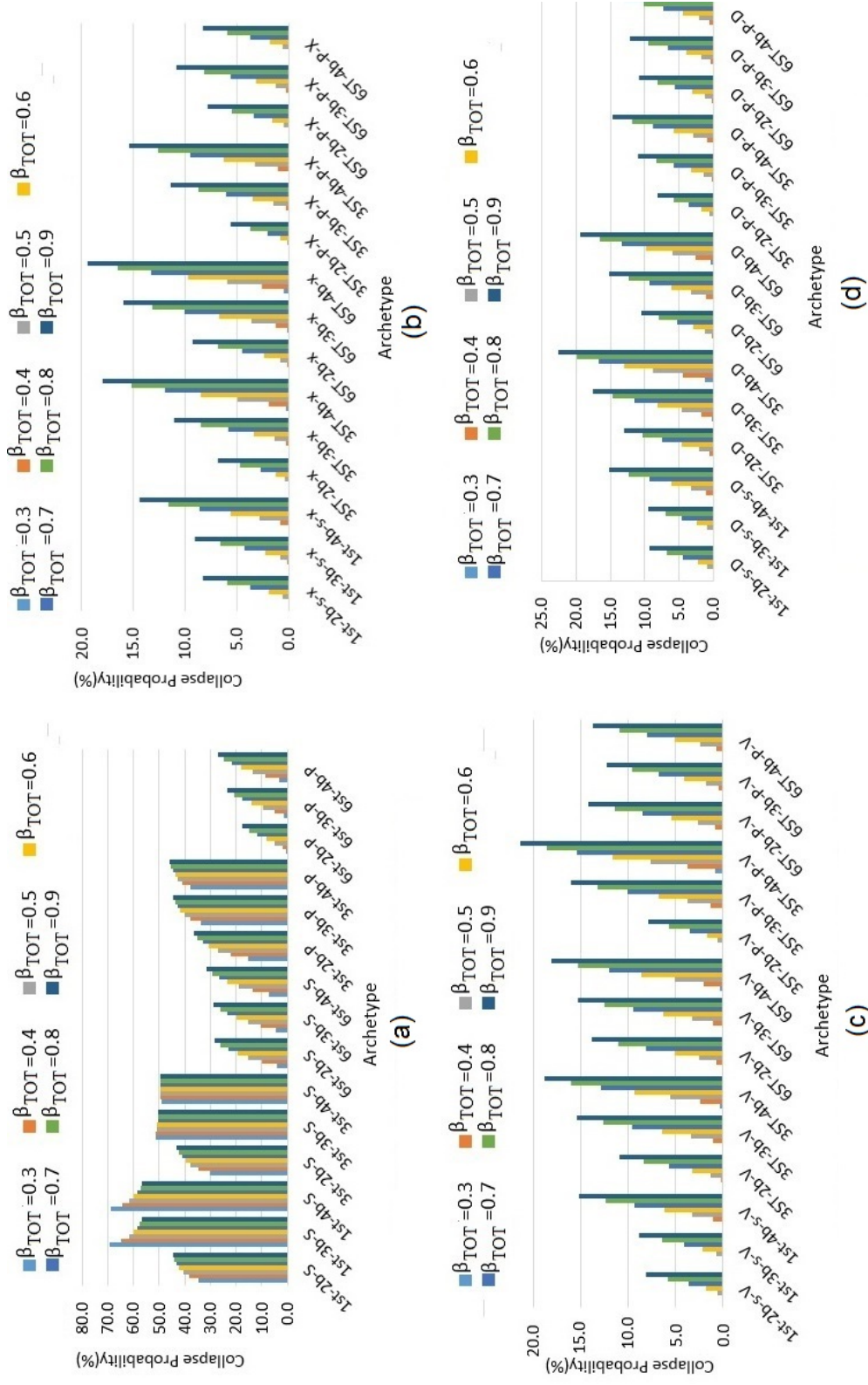


Figure 5.5: Collapse probabilities of the different archetypes (a): primary archetypes (b): X-braced archetypes (c): V-braced archetypes (d): D-braced archetypes.

Figure 5.6 shows the pushover curve of the space frame archetypes. In all braced models, the braced elements reach the yielding point in a short interval of roof drifts. The outcomes of the pushover analyses demonstrate the reasonable influence of braces in the strengthening process. For instance, increased maximum base shear and maximum roof displacement occur in all braced archetypes. By strengthening the RC frames using a common bracing system with X-shaped, chevron, or diagonal braces, the base shear considerably increases by approximately four times in 1-story frames, three times in 3-story frames, and nearly two times in 6-story frames. In all cases, the application of bracing systems enlarges the pushover curves compared with those of the unbraced cases. The same trend of curves is seen for identical braces, and as their shapes differ, the opposite trend is observed. Thus, the braces possess a specific lateral load-resisting capacity. Concerning the improvement in the elastic stiffness, the strengthened archetypes are generally more significant than the primary archetypes. In Figure 5.7, the maximum inter-story drift for the 3- and 6-story frames are depicted. A large difference in the unbraced case signifies that it is inefficient for life safety performance level. In most unbraced frames, the maximum inter-story drifts exceeded 0.11. Conversely, the braced frames demonstrate proper drift values.

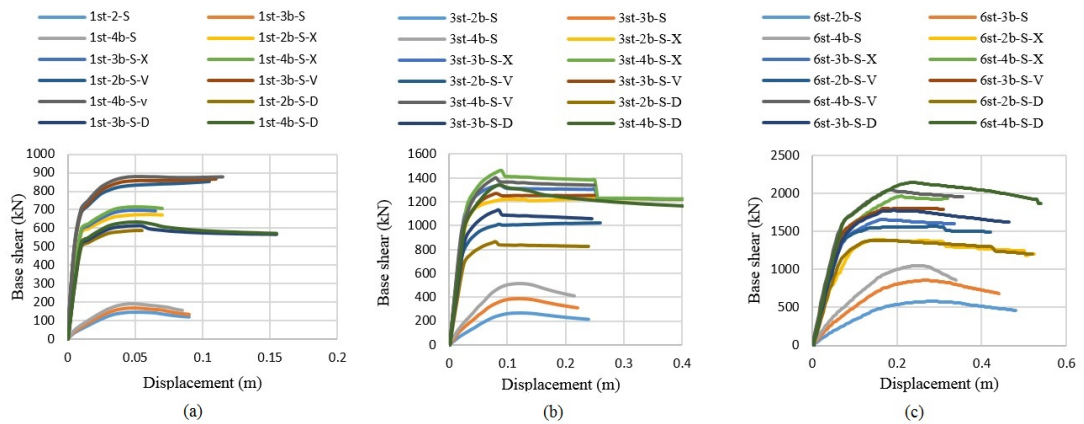


Figure 5.6: Comparison of pushover curves before and after retrofitting (a): Pushover curves of the space frame 1-story archetypes (b): Pushover curves of the space frame 3-story archetypes (c): Pushover curves of the space frame 6-story archetypes.

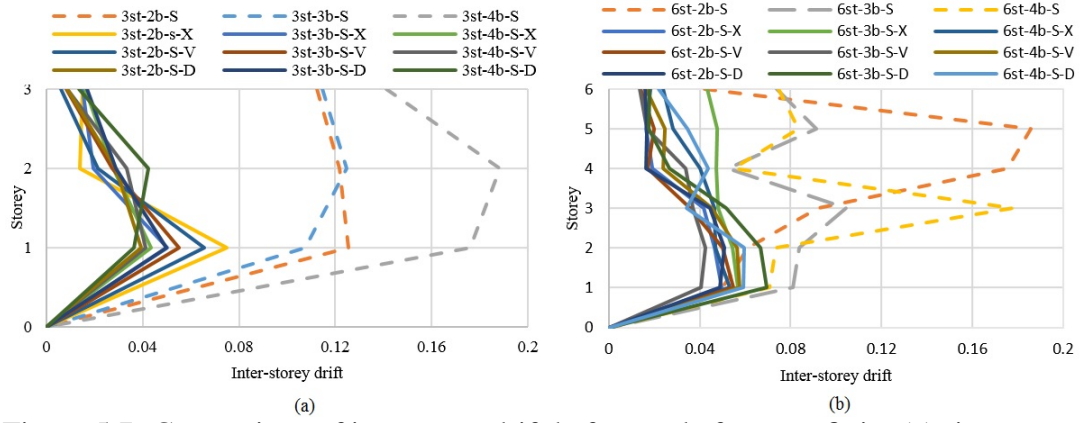


Figure 5.7: Comparison of inter-story drift before and after retrofitting(a): inter-story drift diagram of space frame 3-story archetypes (b): inter-story drift diagram of space frame 6-story archetypes.

In the braced 3-story model, the maximum drift occurs with the first story at 0.075, whereas the maximum drift at 0.069 occurs with the first story in the 6-story building. The application of bracing systems lessened the story drifts by 50%. According to FEMA P695, the largest average of the Ω of the performance group is calculated to compute the overstrength factor. The computed values for the original and strengthened cases show the growth of this parameter for all bracing configurations. However, FEMA P695 limits the values of this factor to those specified in Table 12.2-1 of ASCE/SEI 7-05. Since this factor is three for dual load-resisting systems, this value was adopted in this study.

5.4 The Results of Soft Computing to Find the CMR

5.4.1 ANN Results

The best composition of the network was adopted based on the two major criteria and consists of the least RMSE and the most regression coefficient (R^2). Figures 5.8, 5.9, 5.10, and 5.11 demonstrate the ANN estimates and computational values. The details of the data used for training are presented in Figure 5.8. The value of the coefficient R is 0.979, and the amount of MSE is equal to 0.051. The normal curve has a maximum in the zero zones and is well fitted. The mean and standard deviation of the normal curve

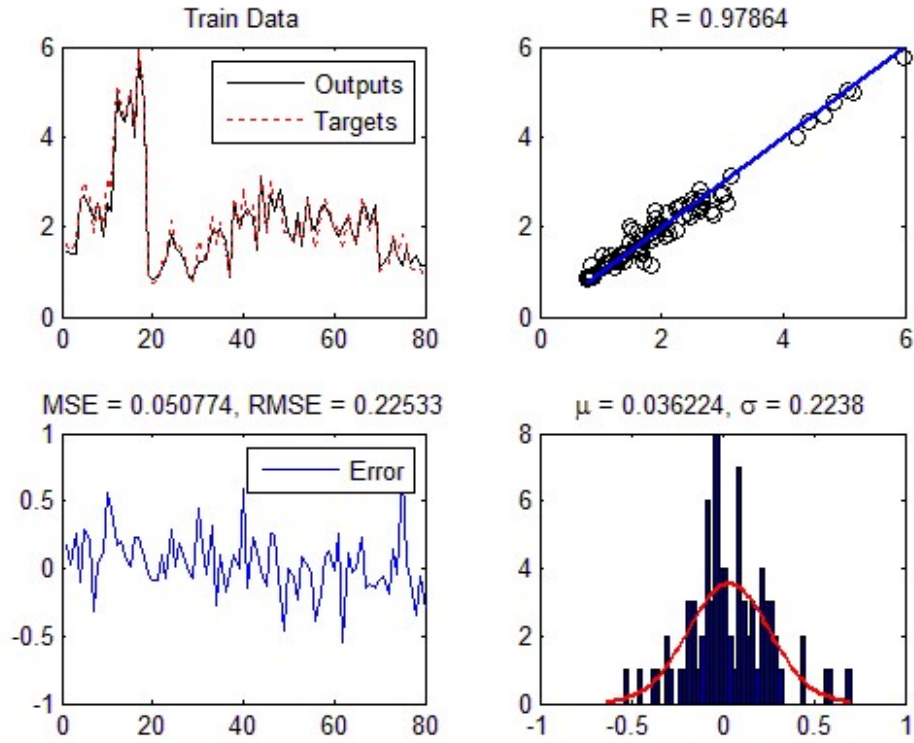


Figure 5.8: Train data analysis.

are equal to 0.0362 and 0.224, respectively. The details of the validation data analysis are given in Figure 5.9. As observed, the coefficient R is 0.975, and the value of MSE is 0.055. The normal curve has a maximum in the zero zones and is well fitted. The mean and standard deviation of the normal curve are 0.0336 and 0.238, respectively. The details of all data analysis are displayed in Figure 5.10, the coefficient R is equal to 0.973, and the value of MSE is 0.057. Similarly, the normal curve has a maximum in the zero zones and is well fitted. The mean and standard deviation of the normal curve are 0.0157 and 0.239, respectively. In Figure 5.11, the correlation coefficients for estimating the CMR in all three sets of training and validation and test data are equal to 0.979, 0.975, and 0.939, respectively. The high values of the R coefficients of the training proficiency of the ANN model are presented in learning the relationship between the factors and the various targets. There is minimal deviation between the computational data and the model, indicating the accuracy of the model used to direct

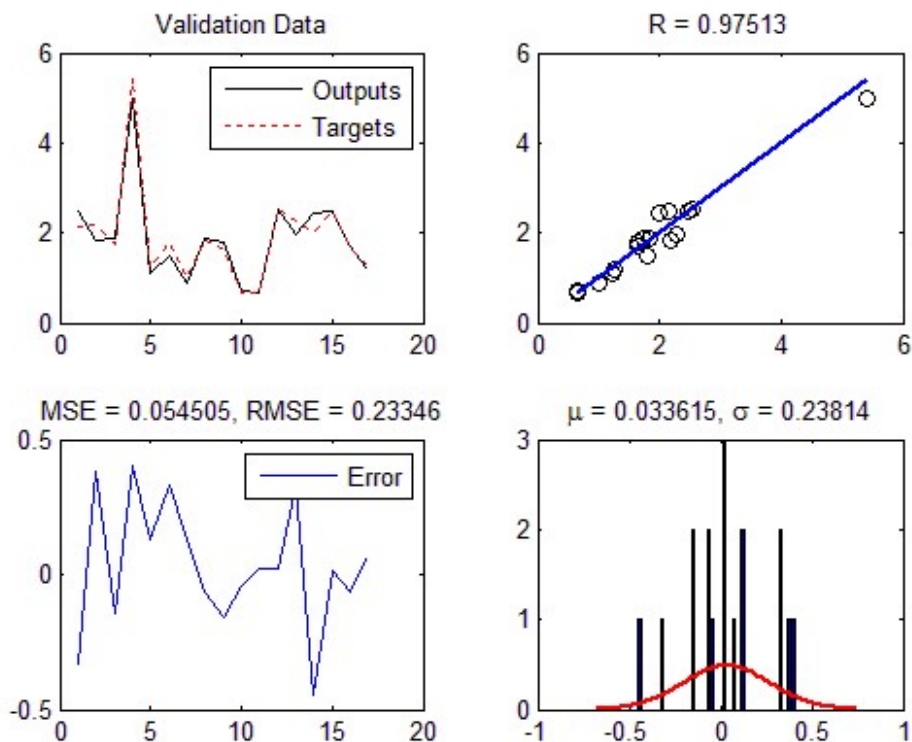


Figure 5.9: Validation data analysis.

the design space for all reactions. In all three parts, the error rate, including the variance between the output and the target data, is in the range of -0.5 to +0.5.

5.4.2 Results and Discussion of Numerical Equations/Relations and ANOVA Analysis and RSM Method

The standard error of the design plot is shown in Figure 5.12. This plot is three-dimensional or as a contour (Figure 5.13) that shows the standard error in the design space. These values represent only the design, not the response data. In general, this plot is better to have a moderately low standard error in the target area of about 1.0 or less.

The ANOVA analysis result for the CMR response is summed up in Table 30. The F-value, P-value, and adjusted R^2 are utilised to determine the efficiency of the model and the proper impact coefficients. While the F-value is high and the adjusted R^2 and

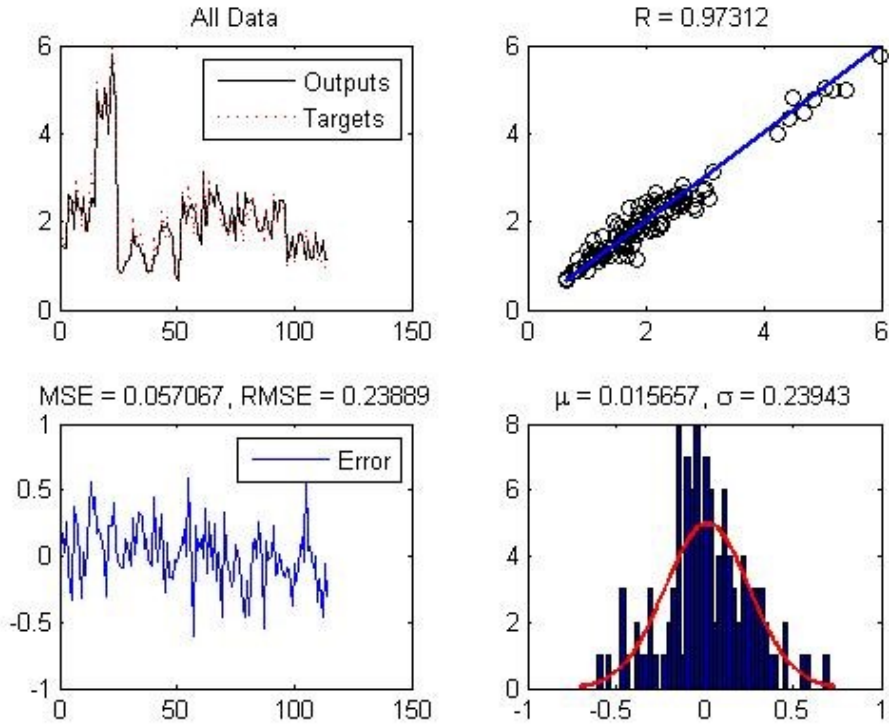


Figure 5.10: All data analysis.

P-value are low shows the ANOVA model is significant. The values of adjusted R^2 , P-value, and F-value for the CMR model are 86.23%, 0.0001, and 29.11, respectively. Therefore, it can be concluded that the model correctly predicts. Thus, P-values less than 0.05 show that the coefficients are considerable, and the P-value greater than 0.1 means that the coefficients are insignificant. Hence, regarding the obtained values, the parameters of AC, BC, CD, A^2 , C^2 , and D^2 have a significant effect on the predicted CMR model.

After taking out the insignificant parameters, the following equation is obtained simply by considering significant terms, and CMR can be computed as:

$$CMR = +9.167 - 11.015AC + 75.865BC - 0.885CD + 12.713A^2 - 4.944C^2 + 0.025D^2 \quad (5.2)$$

In addition, according to the F-value, it shows that the degree of terms with higher

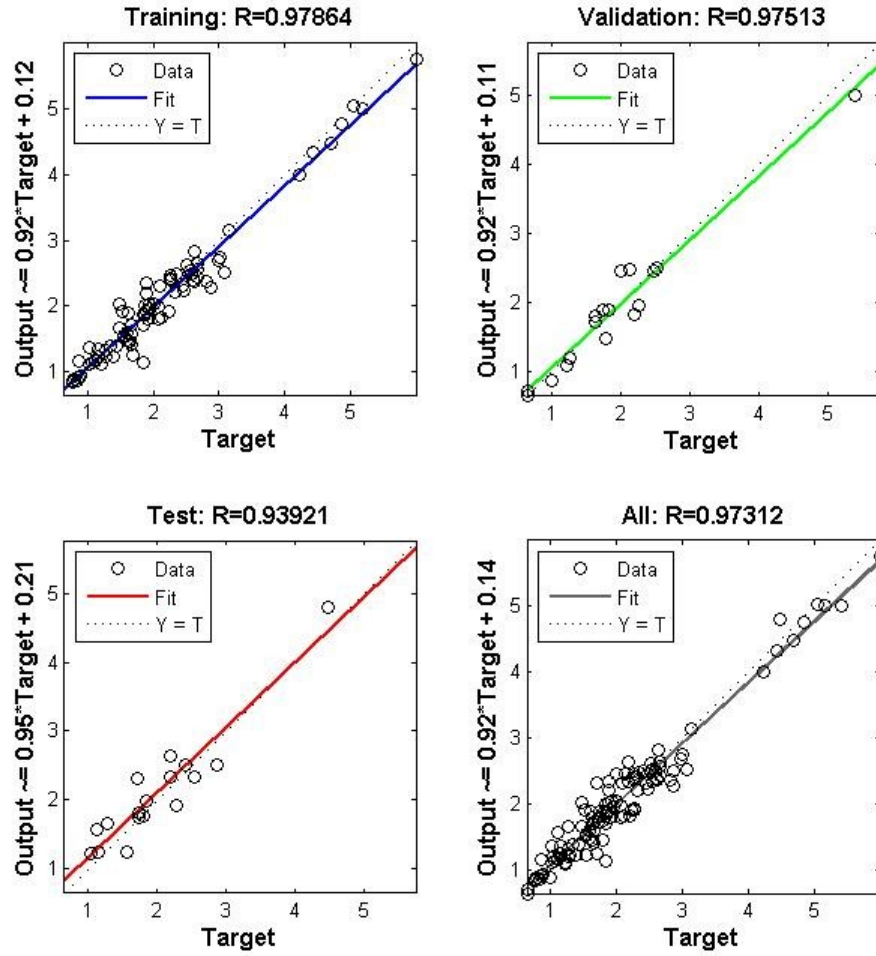


Figure 5.11: All regressions.

impact is as follows:

$A^2 > BC > CD > AC > C^2 > D^2$ The interaction of A^2 had a greater impact on the CMR rate, and the order of higher significance of variables for CMR was found to be A^2 , BC , CD , AC , C^2 , and D^2 , respectively.

The normal plot of residuals, the predicted plot in terms of actual response, residual response plot in terms of predicted response, and residual plot in terms of experimental plot are displayed. The normal probability plot determines whether the residuals follow a normal distribution, in which case the points follow the straight line. Figure 5.14

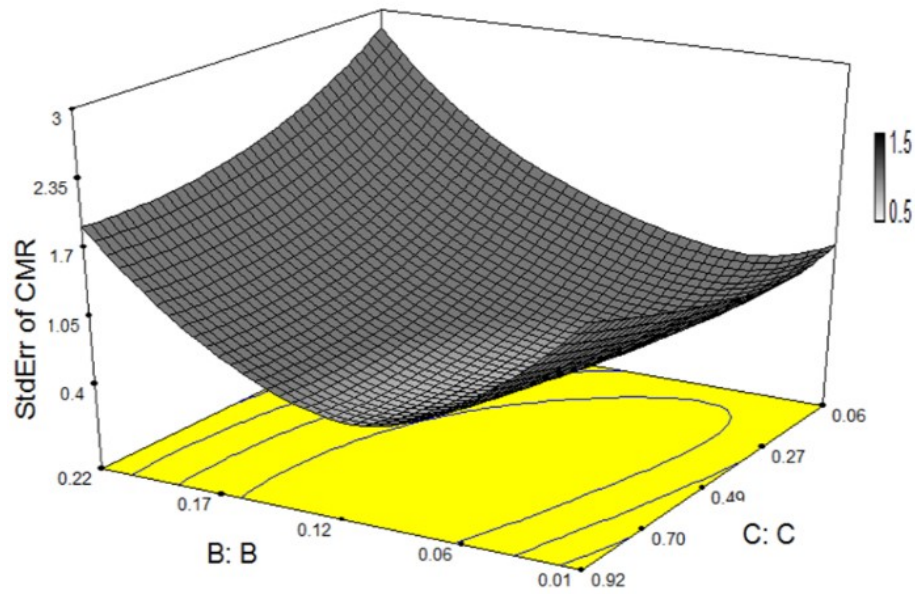


Figure 5.12: The standard error of the design graph (three-dimensional plot).

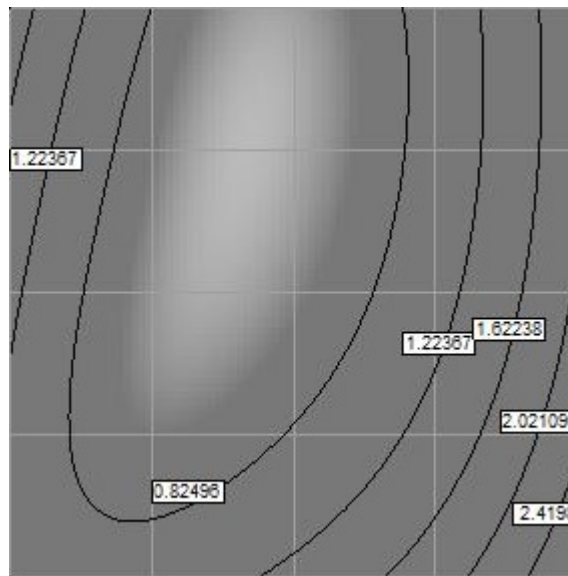


Figure 5.13: The standard error of the design graph (contour plot).

shows that the errors normally extend because the residuals follow the straight line. Figure 5.15 indicates that the predicted response values agree well with the actual values in the intervals of the parameters of the CMR estimation process because the data points are separated in parallel by the 45° line. Figures 5.16 and 5.17 show that the numerical models calculate the responses correctly because of the randomly distributed

Table 30: ANOVA table for response surface quadratic model.

Source	Terms of factors	Sum of squares	DOF	Mean square	F-value	P-value	Result
Model	9.1666	104.280	20	5.210	29.11	< 0.0001	Significant
A	-26.0195	0.120	1	0.120	0.69	0.4099	
B	40.3258	0.003	1	0.003	0.02	0.8892	
C	17.1665	0.011	1	0.011	0.06	0.8022	
D	-0.1565	0.015	1	0.015	0.08	0.7739	
E	-0.4486	0.072	1	0.072	0.40	0.5269	
AB	-38.7931	0.450	1	0.450	2.52	0.1161	
AC	-11.0148	2.250	1	2.250	12.59	0.0006	Significant
AD	0.5982	0.250	1	0.250	1.40	0.2405	
AE	6.6477	0.310	1	0.310	1.73	0.1919	
BC	75.8654	2.640	1	2.640	14.74	0.0002	Significant
BD	-0.6591	0.018	1	0.018	0.10	0.7490	
BE	-16.6187	0.035	1	0.035	0.19	0.6599	
CD	-0.8854	2.480	1	2.480	13.86	0.0003	Significant
CE	-3.3702	0.130	1	0.130	0.74	0.3918	
DE	-0.2130	0.015	1	0.015	0.09	0.7696	
A^2	12.7130	5.010	1	5.010	27.97	< 0.0001	Significant
B^2	-109.7973	0.200	1	0.200	1.14	0.2885	
C^2	-4.9441	0.890	1	0.890	4.95	0.0285	Significant
D^2	0.0247	0.870	1	0.870	4.85	0.0301	Significant
E^2	-1.2824	0.018	1	0.018	0.10	0.7520	
Residual		16.660	93	0.180			

residuals.

By assessing the interaction between the input parameters and the output results, it is possible to determine which items can more or less affect the CMR. As shown in Figure 5.18, most CMRs are predictable for periods shorter than one second with more significant ultimate displacement and offer a more ductile structure with a higher CMR. In Figure 5.19, the increase in CMR is expected directly proportional to the ratios of δ_u , $\delta_{y,eff}$, or ductility along with the rise of the base shear. In Figure 5.20, the increase in maximum displacement and the factor of overstrength lead to a greater CMR. In general, it can be stated that concerning the assessment of the pushover analysis parameters by the RSM, while δ_u , the ductility, overstrength, or the

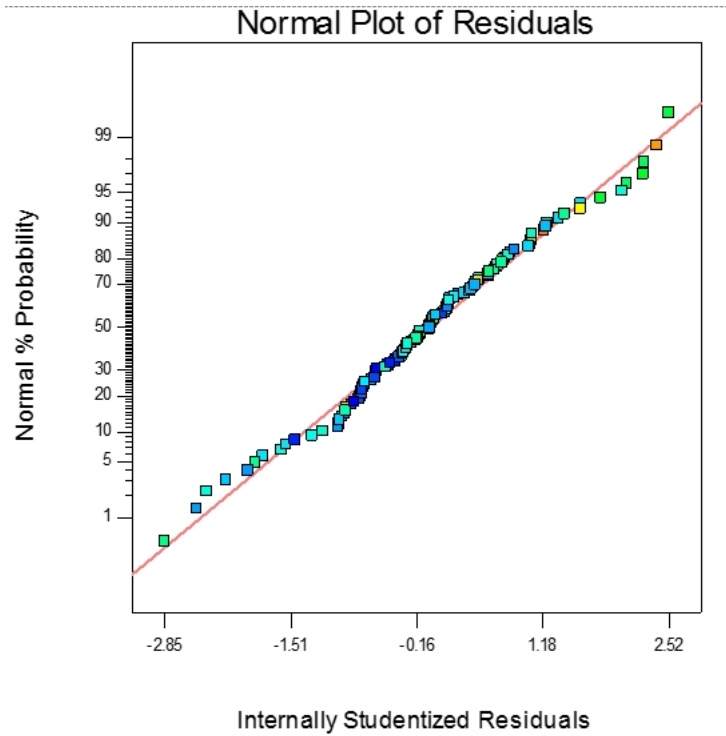


Figure 5.14: Normal probability plot of residuals.

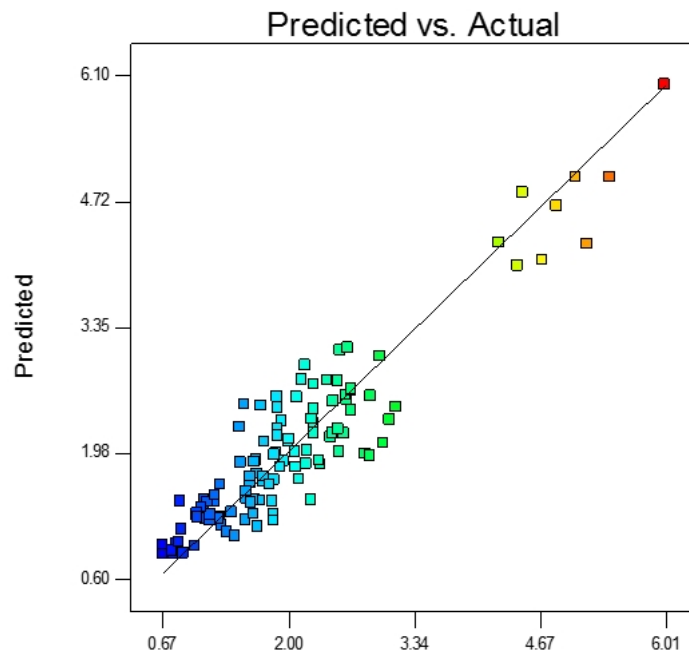


Figure 5.15: Predicted versus actual response plot.

maximum base shear increase, then can be expected a higher CMR. On the other hand, with the increasing weight of the structure and the fundamental period of the structure, it can be called a lower CMR.

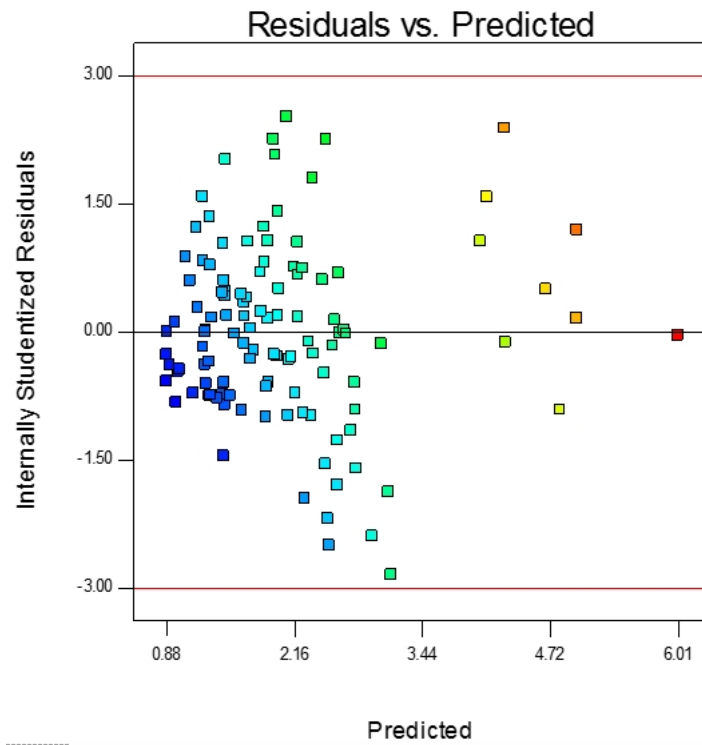


Figure 5.16: Residuals versus predicted plot.

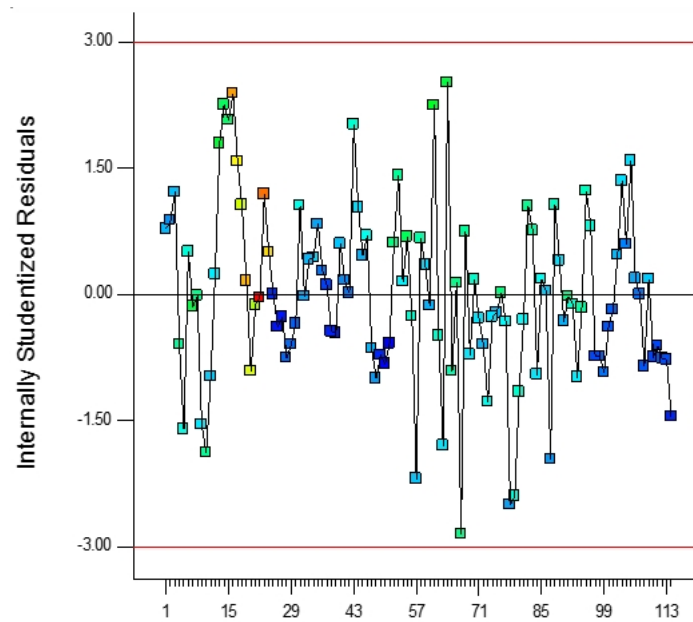


Figure 5.17: Residuals versus run plot.

5.4.3 ANFIS Results

Figure 5.21 shows a schematic of the defined rules. It displays the high capability of the adaptive neural-fuzzy model in predicting the CMR value and its high accuracy and

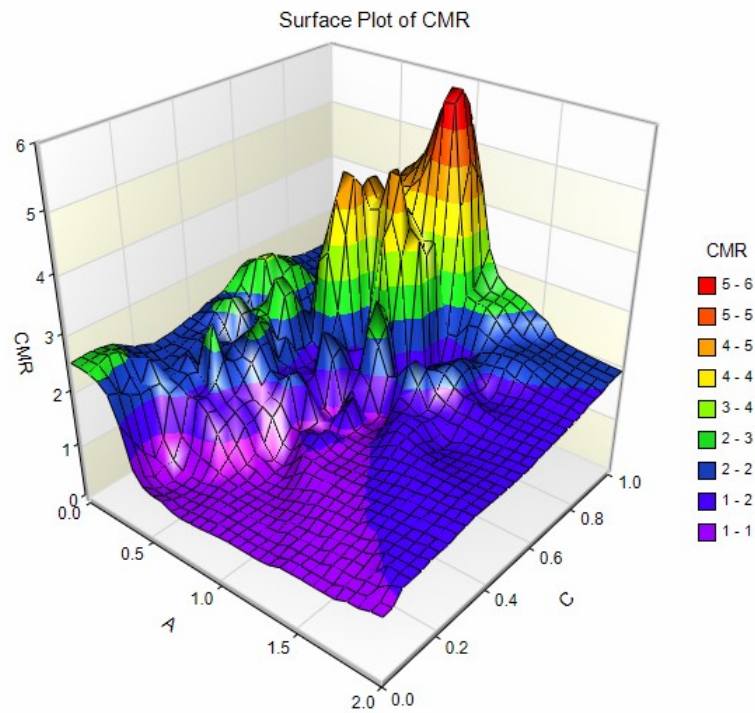


Figure 5.18: The surface plot of CMR vs. A, C.

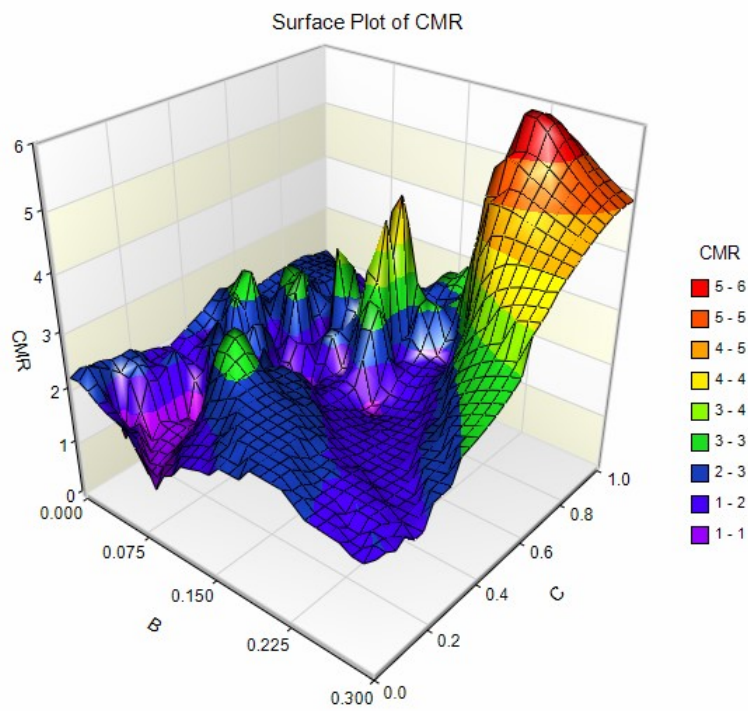


Figure 5.19: The surface plot of CMR vs. B, C.

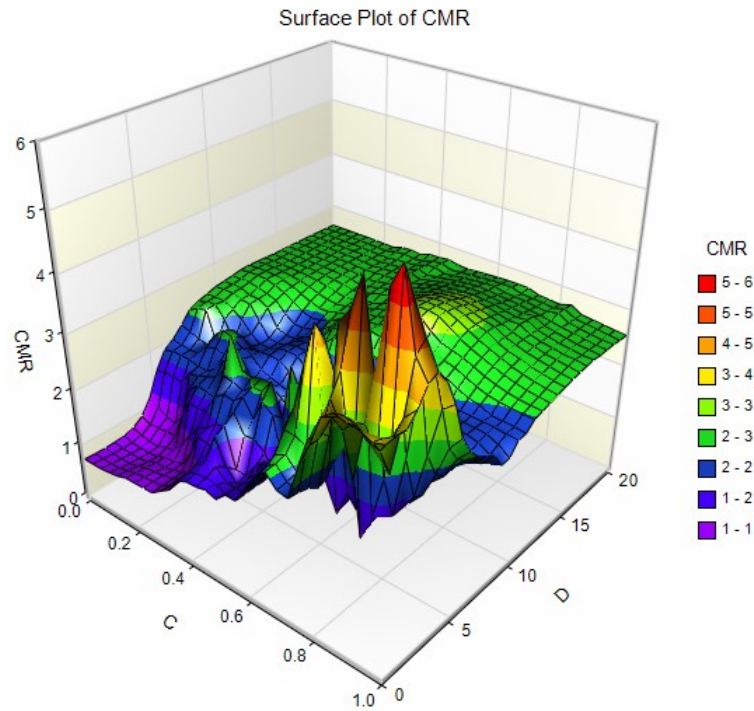


Figure 5.20: The surface plot of CMR vs. C, D.

simplicity. The structural model of ANFIS with five inputs and one output is presented in Figure 5.22.

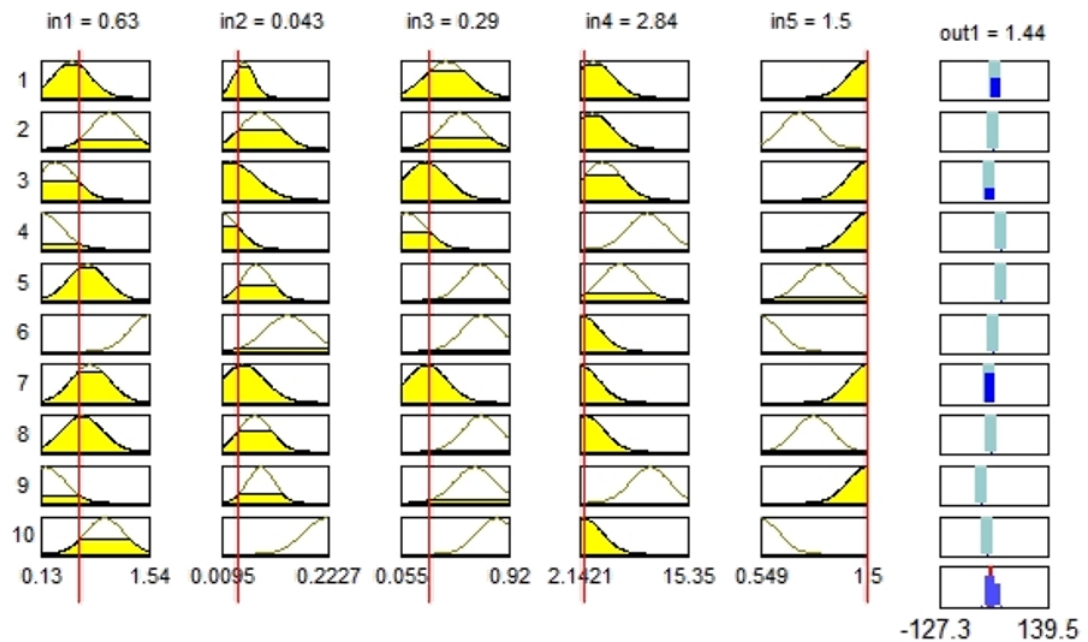


Figure 5.21: The obtained rules.

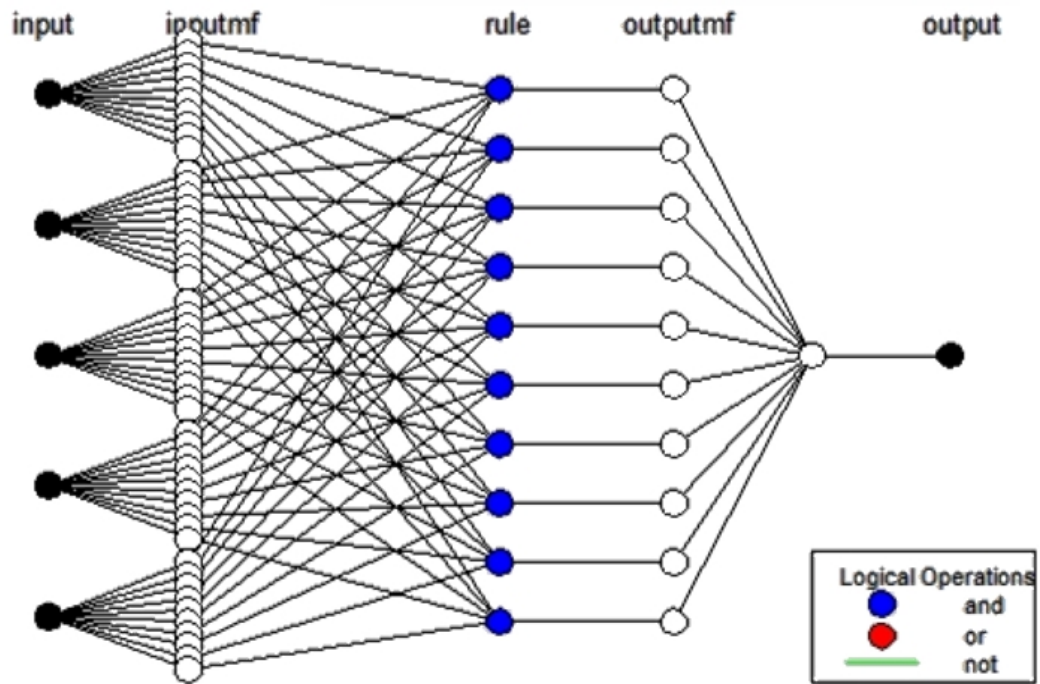


Figure 5.22: Structure of the ANFIS model.

5.4.4 Comparison of ANN, RSM and ANFIS Models for CMR Prediction

Numerous studies suggest that ANFIS and ANN provide reasonable estimates, which may even act better than other available techniques [49, 51]. Wang et al. [50] found that ANFIS and ANN are more accurate in forecast ability. The present study could demonstrate that ANFIS and ANN are superior to the RSM in terms of exactness and accuracy of expectations; at the same time, the results and performance of the ANN and ANFIS models are more in line with each other, with ANFIS being slightly better than the ANN. On the other side, it has been presented that the ANN is marginally better than RSM in forecast ability (e.g. [47]). Researchers have provided blended reports by examining the performance of the ANFIS and ANN. In some works, it has been stated that both ANN and ANFIS yield comparable outcomes. Many researchers have reported that ANFIS outperformed ANN in the process of assessment (e.g. [48, 52]). Nevertheless, RSM uses factors derived from regression models to analyse collaborative (main and interaction) factors. It also helps to identify the main

and reciprocal factors or undesirable terms in the model, which may lessen the complication of the problems.

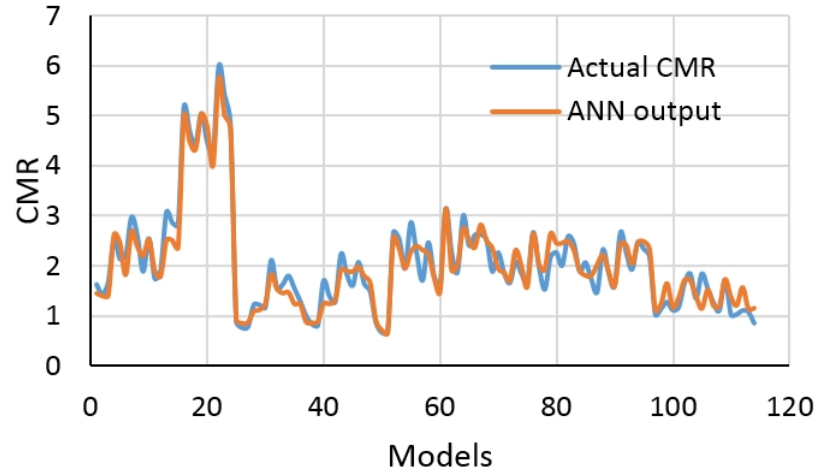


Figure 5.23: Comparison of actual CMR and ANN outputs.

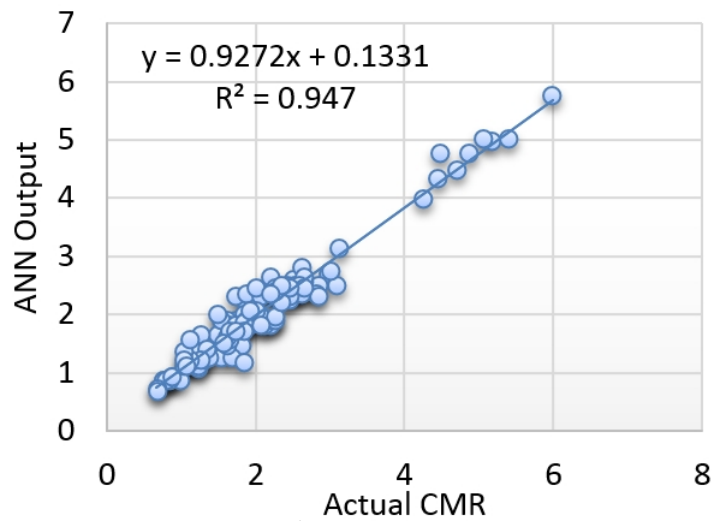


Figure 5.24: R^2 factor for ANN outputs.

Using the capabilities of the ANN, RSM, and ANFIS, to estimate the CMR was considered by determining their statistical indices (R , R^2 , adjusted R^2 , MSE, RMSE, SEP, MAE, and AAD). The outcomes are calculated in Table 31. The schemes of calculated and estimated values are illustrated in Figures 5.23 to 5.28, and all three

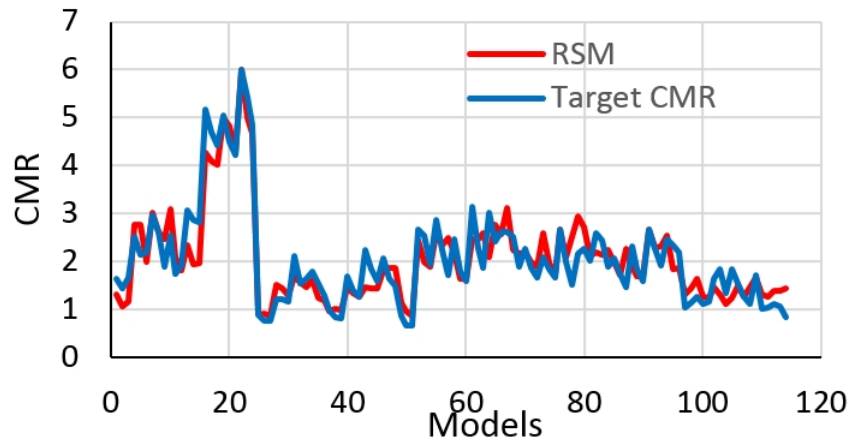


Figure 5.25: Comparison of actual CMR and RSM outputs.

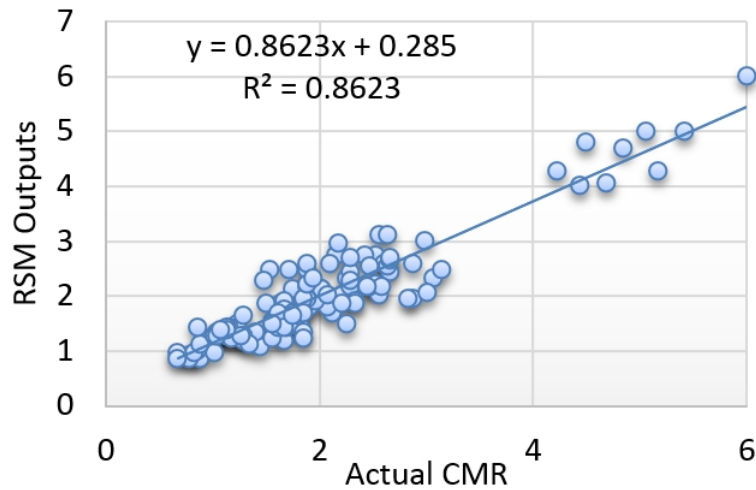


Figure 5.26: R^2 factor in RSM results.

models show relatively high values of R^2 . Between the calculated and predicted values, the closer the value of R to 1 the more efficient the model. The R^2 parameter for ANN, RSM, and ANFIS is equal to 0.947, 0.862, and 0.961. Moreover, great values of R^2 , which were all almost close to 1, specified a good fit for the models. The adjusted R^2 was used in testing the overestimation of R^2 , and they were also high enough for all models. The MSE, a measure of proximity of a fitted line to data points, was determined for the three models.

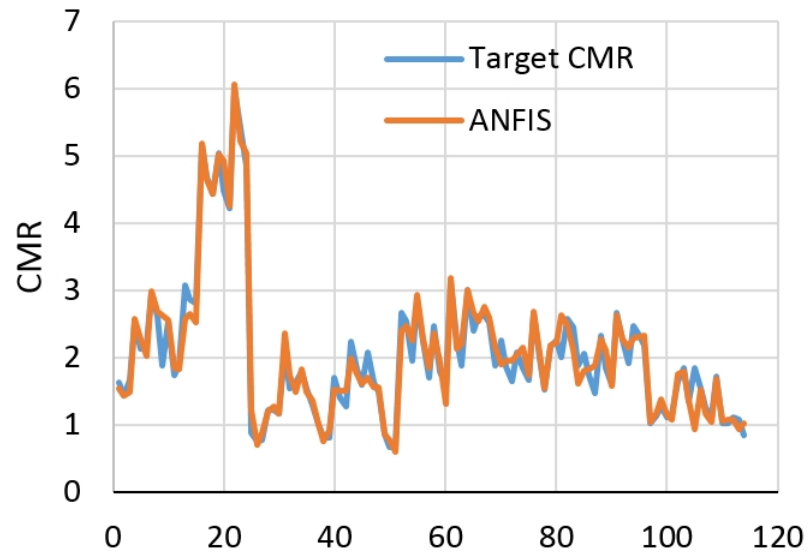


Figure 5.27: Comparison of actual CMR and ANFIS outputs.

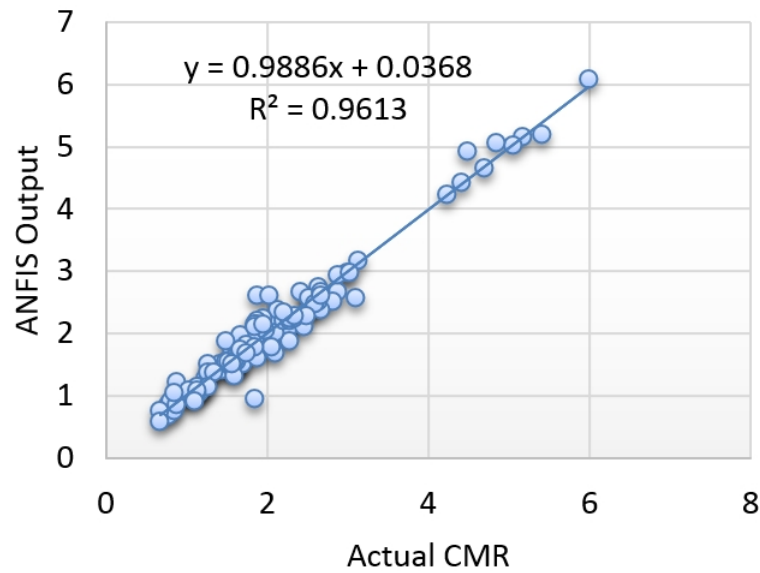


Figure 5.28: R^2 factor in ANFIS results.

Moreover, the square root of the MSE is the RSME, the values obtained for MSE and RMSE show all low, associate with a suitable fit, and show less scatter of errors. SEP, MAE, and AAD measure the accuracy and precision. The lower values of statistical factors show the appropriate performance of the models. The statistical indices outcomes display the ANFIS model has the best performance, in comparison with the ANN model and finally the RSM model. Improper performance of RSM is confirmed

by its high AAD (16.36%). All soft computing results are summarised in Table 32.

Table 31: Statistical indices used for evaluation of models.

Parameter	Equation	ANN	RSM	ANFIS
Correlation coefficient	$R = \frac{\sum_{i=1}^n (y_{p,i} - y_{p,ave})(y_{a,i} - y_{a,ave})}{\sqrt{[\sum_{i=1}^n (y_{p,i} - y_{p,ave})^2][\sum_{i=1}^n (y_{a,i} - y_{a,ave})^2]}}$	0.973	0.929	0.980
Coefficient of determination	$R^2 = \frac{\sum_{i=1}^n (y_{a,i} - y_{p,i})^2}{\sum_{i=1}^n (y_{p,i} - y_{a,ave})^2}$	0.940	0.840	0.961
Adjusted coefficient of determination	$Adjusted R^2 = 1 - [(1 - R^2) \times \frac{(n-1)}{(n-k-1)}]$	0.938	0.833	0.959
Mean square error	$MSE = \frac{1}{n} \sum_{i=1}^n (y_{p,i} - y_{a,i})^2$	0.057	0.146	0.042
Root mean square error	$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n (y_{p,i} - y_{a,i})^2}$	0.239	0.382	0.205
Mean absolute error	$MAE = \frac{1}{n} \sum_{i=1}^n y_{a,i} - y_{p,i} $	0.183	0.298	0.139
Standard error of prediction (%)	$SEP = \frac{RMSE}{y_{a,ave}} \times 100$	11.551	18.474	9.910
Average absolute deviation (%)	$AAD = \frac{1}{n} (\sum_{i=1}^n (\frac{ y_{a,i} - y_{p,i} }{y_{a,i}})) \times 100$	9.863	16.360	7.689

Table 32: Comparison of current CMR and ANN, RSM, and ANFIS outputs (Run 1 to 60).

Run	CMR	ANN	RSM	ANFIS	Run	CMR	ANN	RSM	ANFIS	Run	CMR	ANN	RSM	ANFIS
1	1.63	1.45	1.32	1.55	21	4.23	4.00	4.28	4.26	41	1.40	1.24	1.33	1.51
2	1.43	1.40	1.07	1.44	22	5.99	5.76	6.01	6.07	42	1.28	1.32	1.27	1.52
3	1.67	1.40	1.18	1.50	23	5.41	5.00	4.99	5.23	43	2.24	1.92	1.47	1.99
4	2.52	2.63	2.76	2.58	24	4.85	4.76	4.68	5.05	44	1.83	1.90	1.45	1.77
5	2.14	2.47	2.78	2.27	25	0.89	0.93	0.89	1.22	45	1.61	1.89	1.44	1.61
6	2.20	1.82	2.00	2.04	26	0.77	0.86	0.91	0.71	46	2.08	1.98	1.82	1.71
7	2.98	2.69	3.03	3.00	27	0.78	0.87	0.88	0.89	47	1.64	1.80	1.88	1.57
8	2.61	2.39	2.62	2.69	28	1.22	1.09	1.52	1.22	48	1.49	1.67	1.88	1.57
9	1.89	2.20	2.47	2.63	29	1.22	1.12	1.45	1.27	49	0.87	0.90	1.15	0.87
10	2.55	2.53	3.10	2.56	30	1.17	1.24	1.31	1.17	50	0.67	0.71	0.98	0.78
11	1.75	1.90	2.10	1.86	31	2.12	1.83	1.69	2.37	51	0.67	0.65	0.88	0.60
12	1.91	1.79	1.83	1.84	32	1.55	1.54	1.56	1.74	52	2.67	2.57	2.44	2.41
13	3.07	2.52	2.35	2.58	33	1.64	1.46	1.47	1.49	53	2.54	2.34	2.00	2.48
14	2.87	2.51	1.95	2.66	34	1.80	1.47	1.63	1.83	54	1.95	1.96	1.89	2.26
15	2.81	2.37	1.97	2.53	35	1.55	1.24	1.25	1.49	55	2.87	2.28	2.60	2.95
16	5.17	4.99	4.27	5.18	36	1.29	1.23	1.19	1.36	56	2.25	2.40	2.35	2.28
17	4.69	4.48	4.09	4.65	37	1.01	0.88	0.96	1.02	57	1.71	2.32	2.50	1.85
18	4.43	4.33	4.03	4.43	38	0.84	0.87	1.01	0.76	58	2.47	2.24	2.20	2.36
19	5.05	5.03	4.99	5.03	39	0.81	0.88	0.99	0.93	59	1.80	1.76	1.66	1.93
20	4.49	4.80	4.82	4.94	40	1.70	1.25	1.46	1.53	60	1.60	1.48	1.65	1.31

Table 33: Comparison of current CMR and ANN, RSM, and ANFIS outputs (Run 61 to 114).

Run	CMR	ANN	RSM	ANFIS	Run	CMR	ANN	RSM	ANFIS	Run	CMR	ANN	RSM	ANFIS
61	3.14	3.15	2.48	3.18	81	2.01	2.46	2.13	2.63	101	1.17	1.36	1.24	1.08
62	2.27	1.91	2.46	2.13	82	2.59	2.50	2.19	2.49	102	1.65	1.71	1.47	1.77
63	1.88	2.02	2.59	2.21	83	2.45	2.32	2.16	2.12	103	1.85	1.71	1.31	1.79
64	3.01	2.75	2.08	3.01	84	1.89	1.91	2.25	1.63	104	1.35	1.39	1.12	1.37
65	2.41	2.51	2.77	2.66	85	2.07	1.81	2.00	1.81	105	1.85	1.15	1.24	0.95
66	2.62	2.37	2.57	2.55	86	1.74	1.80	1.72	1.83	106	1.56	1.52	1.48	1.53
67	2.63	2.82	3.13	2.76	87	1.48	2.02	2.26	1.89	107	1.27	1.20	1.27	1.17
68	2.53	2.51	2.24	2.60	88	2.33	2.22	1.90	2.28	108	1.11	1.19	1.46	1.05
69	1.89	2.35	2.17	2.16	89	1.85	1.88	1.69	2.11	109	1.73	1.73	1.66	1.69
70	2.27	1.95	2.20	1.91	90	1.60	1.60	1.72	1.58	110	1.03	1.38	1.31	1.06
71	1.87	1.87	1.99	1.96	91	2.67	2.44	2.68	2.63	111	1.04	1.21	1.28	1.09
72	1.66	1.71	1.90	1.97	92	2.27	2.42	2.30	2.27	112	1.11	1.57	1.39	1.09
73	2.09	2.32	2.59	2.04	93	1.93	2.04	2.33	2.15	113	1.08	1.14	1.39	0.94
74	1.85	1.99	1.96	2.15	94	2.48	2.47	2.54	2.30	114	0.86	1.16	1.45	1.04
75	1.67	1.58	1.75	1.74	95	2.34	2.49	1.85	2.32					
76	2.67	2.64	2.66	2.69	96	2.19	2.34	1.86	2.33					
77	2.00	2.04	2.11	2.07	97	1.03	1.12	1.32	1.04					
78	1.53	1.92	2.51	1.55	98	1.14	1.25	1.44	1.17					
79	2.18	2.64	2.94	2.20	99	1.28	1.65	1.63	1.39					
80	2.27	2.46	2.73	2.22	100	1.12	1.17	1.27	1.15					

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDIES

6.1 Summary

In this study, firstly, the seismic performance of stock RC residential frames designed based on the third and second editions of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800), was studied. The space and perimeter frames were firstly designed based on previous editions of Standard No. 2800 (the third and second editions). Nonlinear static (pushover) analysis and nonlinear incremental dynamic analysis (IDA) were then performed to analyse the seismic performance of the archetype frames subjected to 44 earthquake records, using the FEMA P695 methodology. To evaluate the results, the $ACMR_{10\%}$ value for a performance group and the $ACMR_{20\%}$ value for each archetype were used. Secondly, the vulnerable stock buildings were redesigned and strengthened by adding one of three types of braces: cross, diagonal, or chevron. The probability of collapse of each archetype was used to assess the safety. Also, different values of total uncertainty were considered to cover all possible conditions, and correspondingly, all values of probability of collapse were calculated and compared. Moreover, the response modification factor and overstrength factor of the redesign of vulnerable stock reinforced structures in optimal conditions were presented. Thirdly, soft computing techniques such as ANN, RSM, and ANFIS models were used to estimate the collapse margin ratio (CMR) of the RC structures using the pushover analysis. Five

input parameters from the pushover curve were used as inputs. The computational formula was also extracted, the value of the input parameters was sorted. The statistical indices evaluated, the performance of the models regarding exactness and accuracy was in this specific order: ANFIS, next ANN, and the last RSM method. The conclusion of the study and recommendation for future studies are discussed as follows:

6.2 Conclusions

In the study of seismic evaluation of existing stock archetypes, it is concluded that according to the results of the pushover analysis, it is observed that the models designed according to the third edition have a higher base shear force and ductility in comparison with designed according to the second edition. On the other hand, the overstrength values are approximately at the same level, the difference in base shear and ductility is more evident in models 3 and 6 stories.

By considering the median IDA values, with increasing the height of the structure, they met earlier nonlinear zone. Moreover, the S_T values of archetypes for a constant drift ratio are declined in comparison with growing the height of archetypes.

Collapse margin ratio could be used as a suitable parameter obtained from incremental dynamic analyses and fragility curves results. The accepted collapse criterion is covered by 10% chance of collapse for an individual archetype and 20% chance of collapse for all archetypes in a performance group. Generally, the main results are observed that by comparing values, the third edition designed models have more CMR and ACMR values. Results indicated suitable seismic design parameters in the third edition such as response modification factor, overstrength factor and deformation amplification factor assumed the same as R factor. On the other hand,

according to the FEMA P695 methodology, the archetypes designed based on the second edition, the short-rise RC frames did not succeed in passing the desired parameters, but the mid-rise frames had an acceptable level. It could also be seen that the values of CMR obtained for the perimeter frames are higher than the space frames.

The results show that RC short frames designed according to the second edition are critically vulnerable and cannot meet the life safety performance level. However, RC frames designed according to the third edition have the necessary strength to pass life safety level. The results imply that before an intense earthquake strike, existent short RC buildings designed and built according to the second edition of Standard 2800 are necessary to be considerably retrofitted.

The strengthening of RC buildings using steel concentric braces was considered. Subsequently, 1, 3 and 6-story stock buildings were designed based on the second version of Standard No. 2800, a high collapse probability was found, even with low collapse margin ratios. Hence, these buildings were redesigned and strengthened by adding one of three types of braces: cross, diagonal, or chevron.

The steel braced archetypes were evaluated, and based on the results, all braced archetypes successfully met the provisions. Consequently, the R-factor of 10 and the overstrength factor equal to 3 meet the required life safety performance level. Based on the collapse probabilities of structures against a wide range of global uncertainties, strengthening using cross-braces with an uncertainty of less than 0.8 and diagonal and chevron braces with an uncertainty of less than 0.7 are acceptable.

The results illustrate that the overstrength and ductility factors are considerably enhanced after the strengthening process. However, as the height of the strengthened models increased, these two factors decreased. According to the load-displacement curve, there is a substantial increase in the base shear with braced frames, and the slope of the curve accordingly reflects the stiffness and ductility.

In 1-story models that were strengthened using chevron bracing, the base shear was greater than those of the other models, although diagonal braces provide greater ductility. In the case of 3-story frames that were equipped with cross-braces, the ductility and base shear were at their maximums compared with those of the other models. In the case of 6-story frames, diagonal braces result in the highest ductility, although the base shears in all models are the same.

Based on the inter-story drift values, unbraced frames experience drifts greater than 10%, which decreased by 50% after strengthening.

Comparison of the space and perimeter frames shows that space frames are generally more vulnerable to parameters such as the CMR and overstrength factor, and thus, space frames are more susceptible to collapse than perimeter archetypes. In the case of unbraced frames, increasing the building's height increases the CMR. However, after strengthening, increasing the building's height decreases the CMR. In addition, the frames with more bays possess lower CMRs and have greater collapse probabilities.

The ANN, RSM, and ANFIS models were used to estimate the collapse margin ratio (CMR) of the RC structures proposed by FEMA P695. Based on the statistical indices evaluated, the performance of the models regarding exactness and accuracy was in

this specific order: ANFIS, ANN, and RSM. High values of correlation coefficient (R) show the capability of a polynomial regression model to calculate the CMR performance using the ANFIS (0.98) ANN (0.97) and RSM (0.93) models. The RSM model had a considerably higher average absolute deviation (AAD) (16.36%) compared with ANN (9.86%) and ANFIS (7.69%) models. Additionally, it is essential that these techniques correspond to the RSM to evaluate specific and interactive effects of the model input variables, while others are not able to do it. Nonetheless, the higher accuracy and precision of both ANFIS and ANN in prediction capability ought not to be ignored.

The ANN model was optimised with 10 and 2 neurons in the two hidden layers in which high correlation coefficients with the minimum value of RMSE were obtained. In particular, the neural network is highly capable of modelling the relationship between input and output parameters using the MLP and back-propagation method. It was also achieved that the proposed ANN model could be utilised as an effective method for anticipating the CMR parameter. An ANN method can perform more computational efforts than the usual method.

The ANOVA showed that the RSM method could be successfully used to predict the CMR. In addition, it was found that between the five input factors evaluated, fundamental period and ductility parameters had a more significant role in predicting the CMR. In contrast, the S_{MT} parameter has a minimum effect. Using the results of the RSM, it is concluded that higher CMR can be attained by raising the ductility and the over strength, the maximum base shear, and on the other hand, by decreasing the weight and the fundamental period of the structure.

The subjective judgement of the unwavering quality and heartiness of two determining numerical models (ANFIS and ANN) discovers a persuading guarantee regarding their inherent strength, affecting ability, brilliant speculation, and extraordinarily low errors. Nevertheless, by concentrating on ANFIS and ANN model, it was discernible that the ANFIS predicted information coordinated the computational information with higher in general exactness, which prescribes the inalienable affect-ability and vigour of the ANFIS model.

6.3 Recommendations for Future Research

1- In this research, FEMA P695 methodology was used to evaluate the existing RC structures that were designed and built based on past Iranian seismic standards. However, the use of this seismic assessment method is also recommended for existing buildings designed according to other standards or codes.

2- In this research, stock reinforced concrete structures have been thoroughly evaluated. Further research on the safety of other structural systems is also suggested. In this research, reinforced concrete structures were evaluated based on the far-field earthquake records. Still, it may be necessary to study the near-field earthquake records as a supplement.

3- In this research, steel concentric braces have been used to strengthen vulnerable RC structures. Research on how to strengthen other techniques such as the FRP fibbers, eccentric bracing, and steel jacket may also be carried out.

4- Soft computing techniques were used to properly estimate one of the most time-consuming seismic parameters (CMR). This method may also be used to estimate other seismic parameters such as the probability of collapse.

5- In this research, Artificial Neural network, RSM method, and ANFIS were used for modelling. The use of other methods of artificial intelligence can also be considered.

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Diameters in the Drilling Process." *Artificial Neural Networks—Architectures and Applications* (2013).

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APPENDIX

Table 34: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
3st-2b-S-v3	Beams size	30x40	30x40	30x30									
	Col. size	40x40	40x40	35x35									
	Ext. col. bars	16T18	12T18	8T18									
	Int. col. bars	16T18	12T18	8T18									
3st-3b-S-v3	Beams size	30x40	30x40	30x30									
	Col. size	40x40	40x40	35x35									
	Ext. col. bars	16T18	12T18	8T18									
	Int. col. bars	16T18	12T18	8T18									
3st-4b-S-v3	Beams size	30x40	30x40	30x30									
	Col. size	40x40	40x40	35x35									
	Ext. col. bars	16T18	12T18	8T18									
	Int. col. bars	16T18	12T18	8T18									
6st-2b-S-v3	Beams size	40x50	40x50	40x45	40x45	40x40	30x30						
	Col. size	55x55	55x55	50x50	50x50	45x45	35x35						
	Ext. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						
	Int. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						
6st-3b-S-v3	Beams size	40x50	40x50	40x45	40x45	40x40	30x30						
	Col. size	55x55	55x55	50x50	50x50	45x45	35x35						
	Ext. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						
	Int. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						

Table 35: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
6st-4b-S-v3	Beams size	40x50	40x50	40x45	40x45	40x40	30x30						
	Col. size	55x55	55x55	50x50	50x50	45x45	35x35						
	Ext. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						
	Int. col. bars	16T22	14T18	12T18	12T18	12T18	8T18						
9st-2b-S-v3	Beams size	50x50	50x50	50x50	50x50	40x50	40x50	35x50	35x50	35x35			
	Col. size	65x65	65x65	65x65	65x65	6x6	55x55	50x50	45x45	40x40			
	Ext. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
	Int. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
9st-3b-S-v3	Beams size	50x50	50x50	50x50	50x50	40x50	40x50	35x50	35x50	35x35			
	Col. size	65x65	65x65	65x65	65x65	6x6	55x55	50x50	45x45	40x40			
	Ext. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
	Int. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
9st-4b-S-v3	Beams size	50x50	50x50	50x50	50x50	40x50	40x50	35x50	35x50	35x35			
	Col. size	65x65	65x65	65x65	65x65	6x6	55x55	50x50	45x45	40x40			
	Ext. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
	Int. col. bars	22T22	16T18	12T18	12T18	12T18	12T18	12T18	12T18	8T18			
12st-2b-S-v3	Beams size	45x55	45x55	45x55	45x55	40x55	40x55	40x55	35x55	35x55	35x55	35x55	35x35
	Col. size	65x65	65x65	65x65	65x65	65x65	6x6	6x6	50x50	50x50	45x45	40x40	40x40
	Ext. col. bars	26T22	22T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18
	Int. col. bars	22T22	18T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
12st-3b-S-v3	Beams size	45x55	45x55	45x55	45x55	40x55	40x55	40x55	35x55	35x55	35x55	35x55	35x35
	Col. size	65x65	65x65	65x65	65x65	65x65	6x6	6x6	50x50	50x50	45x45	40x40	40x40
	Ext. col. bars	26T22	22T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18
	Int. col. bars	22T22	18T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18
12st-4b-S-v3	Beams size	45x55	45x55	45x55	45x55	40x55	40x55	40x55	35x55	35x55	35x55	35x55	35x35
	Col. size	65x65	65x65	65x65	65x65	65x65	6x6	6x6	50x50	50x50	45x45	40x40	40x40
	Ext. col. bars	26T22	22T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18
	Int. col. bars	22T22	18T18	16T18	16T18	16T18	14T18	14T18	12T18	12T18	12T18	12T18	8T18
3st-2b-P-v3	Beams size	30x30	30x30	30x30									
	Col. size	35x35	35x35	35x35									
	Ext. col. bars	8T18	8T18	8T18									
	Int. col. bars	8T18	8T18	8T18									
3st-3b-P-v3	Beams size	30x30	30x30	30x30									
	Col. size	35x35	35x35	35x35									
	Ext. col. bars	8T18	8T18	8T18									
	Int. col. bars	8T18	8T18	8T18									
3st-4b-P-v3	Beams size	30x30	30x30	30x30									
	Col. size	35x35	35x35	35x35									
	Ext. col. bars	8T18	8T18	8T18									
	Int. col. bars	8T18	8T18	8T18									

[illegible]

Table 39: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
3st-3b-S-v2	Beams size	30x30	30x30	30x30									
	Col. size	35x35	35x35	35x35									
	Ext. col. bars	12T18	12T18	12T18									
	Int. col. bars	12T18	8T18	8T18									
3st-4b-S-v2	Beams size	30x30	30x30	30x30									
	Col. size	35x35	35x35	35x35									
	Ext. col. bars	12T18	12T18	12T18									
	Int. col. bars	12T18	8T18	8T18									
6st-2b-S-v2	Beams size	35x40	35x40	35x40	35x35	35x35	35x35						
	Col. size	45x45	45x45	40x40	40x40	35x35	35x35						
	Ext. col. bars	18T18	16T18	12T18	12T18	12T18	12T18						
	Int. col. bars	18T18	16T18	14T18	12T18	12T18	8T18						
6st-3b-S-v2	Beams size	35x40	35x40	35x40	35x35	35x35	35x35						
	Col. size	45x45	45x45	40x40	40x40	35x35	35x35						
	Ext. col. bars	18T18	16T18	12T18	12T18	12T18	12T18						
	Int. col. bars	18T18	16T18	14T18	12T18	12T18	8T18						
6st-4b-S-v2	Beams size	35x40	35x40	35x40	35x35	35x35	35x35						
	Col. size	45x45	45x45	40x40	40x40	35x35	35x35						
	Ext. col. bars	18T18	16T18	12T18	12T18	12T18	12T18						
	Int. col. bars	18T18	16T18	14T18	12T18	12T18	8T18						

Table 40: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
9st-2b-S-v2	Beams size	45x45	45x45	45x45	40x45	40x40	40x40	40x40	35x35	35x35			
	Col. size	45x45	45x45	45x45	45x45	45x45	45x45	40x40	35x35	35x35			
	Ext. col. bars	20T22	18T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18			
	Int. col. bars	22T22	20T22	20T22	16T22	16T22	16T18	14T18	12T18	8T18			
9st-3b-S-v2	Beams size	45x45	45x45	45x45	40x45	40x40	40x40	40x40	35x35	35x35			
	Col. size	45x45	45x45	45x45	45x45	45x45	45x45	40x40	35x35	35x35			
	Ext. col. bars	20T22	18T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18			
	Int. col. bars	22T22	20T22	20T22	16T22	16T22	16T18	14T18	12T18	8T18			
9st-4b-S-v2	Beams size	45x45	45x45	45x45	40x45	40x40	40x40	40x40	35x35	35x35			
	Col. size	45x45	45x45	45x45	45x45	45x45	45x45	40x40	35x35	35x35			
	Ext. col. bars	20T22	18T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18			
	Int. col. bars	22T22	20T22	20T22	16T22	16T22	16T18	14T18	12T18	8T18			
12st-2b-S-v2	Beams size	40X55	40X55	40X55	40X50	40X50	40X50	40X45	40X45	40X40	40X40	35X35	35X35
	Col. size	55X55	55X55	50X50	50X50	50X50	45X45	45X45	45X45	40X40	40X40	35X35	35X35
	Ext. col. bars	14T22	14T22	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18
	Int. col. bars	18T22	18T22	18T22	18T22	18T22	18T22	18T22	18T18	18T18	12T18	12T18	8T18
12st-3b-S-v2	Beams size	40X55	40X55	40X55	40X50	40X50	40X50	40X45	40X45	40X40	40X40	35X35	35X35
	Col. size	55X55	55X55	50X50	50X50	50X50	45X45	45X45	45X45	40X40	40X40	35X35	35X35
	Ext. col. bars	14T22	14T22	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18
	Int. col. bars	18T22	18T22	18T22	18T22	18T22	18T22	18T22	18T18	18T18	12T18	12T18	8T18

Table 41: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
12st-4b-S-v2	Beams size	40X55	40X55	40X55	40X50	40X50	40X50	40X45	40X45	40X40	40X40	35X35	35X35
	Col. size	55X55	55X55	50X50	50X50	50X50	45X45	45X45	45X45	40X40	40X40	35X35	35X35
	Ext. col. bars	14T22	14T22	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18	12T18
	Int. col. bars	18T22	18T22	18T22	18T22	18T22	18T22	18T22	18T18	18T18	12T18	12T18	8T18
3st-2b-P-v2	Beams size	20X20	20X20	20X20									
	Col. size	30X30	30X30	30X30									
	Ext. col. bars	8T14	8T14	8T14									
	Int. col. bars	8T14	8T14	8T14									
3st-3b-P-v2	Beams size	20X20	20X20	20X20									
	Col. size	30X30	30X30	30X30									
	Ext. col. bars	8T14	8T14	8T14									
	Int. col. bars	8T14	8T14	8T14									
3st-4b-P-v2	Beams size	20X20	20X20	20X20									
	Col. size	30X30	30X30	30X30									
	Ext. col. bars	8T14	8T14	8T14									
	Int. col. bars	8T14	8T14	8T14									
6st-2b-P-v2	Beams size	25X30	25X30	25X30	25X30	25X30	25X30						
	Col. size	35X35	30X30	30X30	30X30	30X30	30X30						
	Ext. col. bars	10T16	8T16	8T16	8T16	8T16	8T16						
	Int. col. bars	10T16	8T16	8T16	8T16	8T16	8T16						

Table 42: Members size of Archetypes.

Model ID	Element	St-1	St-2	St-3	St-4	St-5	St-6	St-7	St-8	St-9	St-10	St-11	St-12
6st-3b-P-v2	Beams size	25X30	25X30	25X30	25X30	25X30	25X30	25X30					
	Col. size	35X35	30X30	30X30	30X30	30X30	30X30	30X30					
	Ext. col. bars	10T16	8T16	8T16	8T16	8T16	8T16	8T16					
	Int. col. bars	10T16	8T16	8T16	8T16	8T16	8T16	8T16					
6st-4b-P-v2	Beams size	25X30	25X30	25X30	25X30	25X30	25X30	25X30					
	Col. size	35X35	30X30	30X30	30X30	30X30	30X30	30X30					
	Ext. col. bars	10T16	8T16	8T16	8T16	8T16	8T16	8T16					
	Int. col. bars	10T16	8T16	8T16	8T16	8T16	8T16	8T16					
9st-2b-P-v2	Beams size	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35			
	Col. size	35X35	35X35	35X35	35X35	35X35	30X30	30X30	30X30	30X30			
	Ext. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			
	Int. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			
9st-3b-P-v2	Beams size	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35			
	Col. size	35X35	35X35	35X35	35X35	35X35	30X30	30X30	30X30	30X30			
	Ext. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			
	Int. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			
9st-4b-P-v2	Beams size	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35	25X35			
	Col. size	35X35	35X35	35X35	35X35	35X35	30X30	30X30	30X30	30X30			
	Ext. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			
	Int. col. bars	12T18	8T18	8T18	8T18	8T18	8T16	8T16	8T16	8T16			

Table 44: Steel bracing sections.

Model ID	Stock frame	St-1	St-2	St-3	St-4	St-5	St-6
1st-2b-S-X	1st-2b-S-V2	2U6					
1st-3b-S-X	1st-3b-S-V2	2U6					
1st-4b-S-X	1st-4b-S-V2	2U6					
1st-2b-S-V	1st-2b-S-V2	2U8					
1st-3b-S-V	1st-3b-S-V2	2U8					
1st-4b-S-V	1st-4b-S-V2	2U8					
1st-2b-S-D	1st-2b-S-V2	2U8					
1st-3b-S-D	1st-3b-S-V2	2U8					
1st-4b-S-D	1st-4b-S-V2	2U8					
3st-2b-S-X	3st-2b-S-V2	2U8	2U8	2U6			
3st-3b-S-X	3st-3b-S-V2	2U8	2U8	2U6			
3st-4b-S-X	3st-4b-S-V2	2U8	2U8	2U6			
3st-2b-S-V	3st-2b-S-V2	2U8	2U8	2U8			
3st-3b-S-V	3st-3b-S-V2	2U10	2U10	2U8			
3st-4b-S-V	3st-4b-S-V2	2U10	2U10	2U8			
3st-2b-S-D	3st-2b-S-V2	2U10	2U10	2U8			
3st-3b-S-D	3st-3b-S-V2	2U12	2U12	2U8			
3st-4b-S-D	3st-4b-S-V2	2U14	2U12	2U10			
3st-2b-P-X	3st-2b-P-V2	2U8	2U8	2U6			
3st-3b-P-X	3st-3b-P-V2	2U8	2U8	2U8			
3st-4b-P-X	3st-4b-P-V2	2U8	2U8	2U8			
3st-2b-P-V	3st-2b-P-V2	2U8	2U8	2U8			
3st-3b-P-V	3st-3b-P-V2	2U10	2U10	2U8			
3st-4b-P-V	3st-4b-P-V2	2U10	2U10	2U8			
3st-2b-P-D	3st-2b-P-V2	2U8	2U8	2U6			
3st-3b-P-D	3st-3b-P-V2	2U8	2U8	2U8			
3st-4b-P-D	3st-4b-P-V2	2U8	2U8	2U8			
6st-2b-S-X	6st-2b-S-V2	2U8	2U8	2U8	2U8	2U8	2U6
6st-3b-S-X	6st-3b-S-V2	2U8	2U8	2U8	2U8	2U8	2U8
6st-4b-S-X	6st-4b-S-V2	2U10	2U10	2U10	2U8	2U8	2U8

Table 45: Steel bracing sections.

Model ID	Stock frame	St-1	St-2	St-3	St-4	St-5	St-6
6st-2b-S-V	6st-2b-S-V2	2U10	2U10	2U10	2U10	2U10	2U8
6st-3b-S-V	6st-3b-S-V2	2U12	2U12	2U12	2U12	2U10	2U8
6st-4b-S-V	6st-4b-S-V2	2U12	2U12	2U12	2U12	2U12	2U10
6st-2b-S-D	6st-2b-S-V2	2U14	2U14	2U14	2U14	2U12	2U10
6st-3b-S-D	6st-3b-S-V2	2U16	2U16	2U16	2U14	2U14	2U10
6st-4b-S-D	6st-4b-S-V2	2U18	2U18	2U18	2U14	2U12	2U10
6st-2b-P-X	6st-2b-P-V2	2U6	2U6	2U6	2U6	2U6	2U6
6st-3b-P-X	6st-3b-P-V2	2U8	2U8	2U8	2U6	2U6	2U6
6st-4b-P-X	6st-4b-P-V2	2U8	2U8	2U8	2U6	2U6	2U6
6st-2b-P-V	6st-2b-P-V2	2U8	2U8	2U8	2U8	2U8	2U6
6st-3b-P-V	6st-3b-P-V2	2U8	2U8	2U8	2U8	2U8	2U6
6st-4b-P-V	6st-4b-P-V2	2U8	2U8	2U8	2U8	2U8	2U6
6st-2b-P-D	6st-2b-P-V2	2U8	2U8	2U8	2U8	2U8	2U8
6st-3b-P-D	6st-3b-P-V2	2U10	2U10	2U10	2U8	2U8	2U8
6st-4b-P-D	6st-4b-P-V2	2U12	2U12	2U10	2U10	2U8	2U8